

PRELIMINARY GEOTECHNICAL EVALUATION

WATTS TOWERS CULTURAL CRESCENT

APNs 6065-023-900, 906; 6065-034-900

LOS ANGELES, CALIFORNIA

GEOBASE

SOIL/ROCK MECHANICS AND FOUNDATION ENGINEERING

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Los Angeles, California

Prepared for:

Community Redevelopment Agency

City of Los Angeles, California

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I. INTRODUCTION

1.1 General

The City of Los Angeles is planning construction of the Watts Towers Cultural Crescent, in Los Angeles, California. GEOBASE, Inc. (GEOBASE) was authorized by the City of Los Angeles, Community Redevelopment Agency (LACRA) to carry out a preliminary geotechnical evaluation of the proposed site. The work was coordinated by Ms. Sandra Sunday, P.E., of the Community Redevelopment Agency. The site location is shown on Figure A-1, Appendix A.

For this preliminary geotechnical evaluation, we were provided with the Design and Development Plan, prepared by Takata Associates. The Design and Development Plan is reproduced herein as Figure A-2, Appendix A, Boring and Test Pit Locations Plan.

This report describes the site investigation and summarizes the results of both field and laboratory testing. These results are discussed with reference to the proposed development. General recommendations pertinent to suitable site development and preliminary foundation design recommendations are given. Construction guidelines related to the geotechnical aspects of the project are also addressed.

1.2 Objectives and Scope of Services

The objectives of the geotechnical services were to evaluate the geological and seismological conditions at the site, and obtain preliminary information on the subsurface conditions in order to assist with an evaluation of the proposed site for the planned development. As part of the geotechnical services, six (6) test pits were excavated at the site to determine the thickness of the gravelly fill and locations of the existing railroad ties encountered during a previous investigation by others (reference 1).

The specific services provided during the course of this geotechnical investigation included:

- Review of previous geological, geotechnical and seismological reports, and maps pertinent to the subject site;
- Field exploration program consisting of drilling eleven (11) borings and excavating six (6) test pits. The borings were logged, and samples representative of the materials encountered were selected for laboratory testing;
- Selection of appropriate laboratory tests and laboratory testing;
- Evaluation and engineering analyses of site data, field observations, field test results and laboratory test results; and,
- Preparation of this report describing the field investigation, and summarizing the results of both field and laboratory testing, and engineering analyses. A preliminary geotechnical assessment of the

site, and general recommendations for planing and preliminary design purposes are also provided in this report.

II. SITE AND PROJECT DESCRIPTION

2.1 Site Description

The proposed site for the Watts Towers Cultural Crescent development consists of two (2) parcels. The northern parcel is located between Graham Avenue and Metro Blue Line. It is bounded by Department of Water and Power (DWP) Branch Office to the north and existing MTA Urban Greenways to the south. The site is relatively flat, and currently vacant and unpaved. Scattered gravels and several concrete pieces, were observed at the surface of the site.

The southern parcel is located between the Watts Tower Art Center and Watts Towers, and Santa Ana Boulevard. It is bounded by the intersection of Graham avenue and Santa Ana Boulevard to the north, and 108th Street and Wilmington Avenue to the south. The area is relatively flat except for an earth mound running in a north south direction through the middle of the entire site. Based on the reference 1 report, the site has previously served a railroad track right-of-way. The elevated mound appears to be remnants of the railroad embankment.

2.2 Project Description

Bases on the information provided in the Design Development Plan and discussions with LACRA personnel, the proposed Watts Towers Cultural Crescent development is planned to consist of:

- commercial sites with underground parking approximately twenty-five (25) feet in depth and a market place at the northern parcel.
- landscape areas, Watts Tower Plaza Amphitheater, approximately four (4) to five (5) feet deep, and commercial sites with underground parking based approximately twenty-five (25) feet below grade at the southern parcel.

III. SITE INVESTIGATION

3.1 Field Program

The field work for this geotechnical investigation was carried out on December 12, 1997 and consisted of advancing eleven (11) borings and six (6) test pits at the site, at the approximate locations shown on the Boring and Test Pit Locations Plan, Figure A-2, Appendix A. The borings and test pits were located in the field utilizing cloth tape. Therefore, the boring and test pit locations should be considered accurate only to the degree implied by the method used.

The borings were advanced to a maximum depth of fifty-one and one-half (51.5) feet using a CME-75 drill rig fitted with hollow stem augers. The Log of Borings, together with an Explanation of Terms and Symbols used are given in Appendix B, Figures B-1 thru B-12, inclusive. The test pits were excavated at the northern parcel with a backhoe and advanced to a maximum depth of four (4) feet. The log of test pits are provided on Figures B-13 thru B-18, inclusive, Appendix B.

Field testing consisted of the Standard Penetration Test (SPT). The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, two (2) inches outside diameter and one and three-eighths (1 3/8) inches inside diameter, is driven eighteen (18) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

Sampling consisted of:

- Collection of disturbed samples at selected locations retrieved from the auger and test pits;
- Collection of samples retrieved from the Standard Penetration Test (SPT) split spoon; and,
- Collection of soil samples at selected locations using a California Modified Sampler. The soil samples were retained in a series of brass rings, each having an inside diameter of 2.41 inches and a height of one (1) inch. These ring samples were placed in close-fitting, moisture tight containers for shipment to the laboratory.

3.2 Laboratory Testing

The samples obtained during the field program were returned to the laboratory for visual examination and testing. The soils were classified in accordance with ASTM D 2487 and D 2488.

The laboratory testing program consisted of the following:

- Laboratory determination of water (moisture) content of soils, rock and soil-aggregate mixtures (ASTM D 2216) and dry density;
- Particle size analysis of soils (ASTM D 422);
- Direct shear test of soils (ASTM D 3080);
- Consolidation testing of soils (ASTM D 2435);
- R-Value test of soils (Cal.301);
- Expansion potential of soils (UBC 29-2); and
- Water-soluble sulfates content of soils (CAL. 417A), pH, electrical resistivity and soluble chlorides.

The field and laboratory test results are presented on the Log of Borings and Test Pits, Figures B-2 thru

B-18, inclusive, Appendix B, where applicable, and in Appendix C.

IV. GEOLOGY

4.1 Geological Setting

The project site consists of two (2) parcels and is located in the northwest central portion of the Los Angeles Basin. The Los Angeles Basin is a broad alluviated, northwest trending structural trough which is filled to capacity with a thick sequence of sedimentary bedrock and overlying alluvium ranging from late Cretaceous to Holocene age. In the site region, the sediment cover overlying the basement rock is approximately 30,000 feet thick. Structurally, the basin has slowly subsided as marine and non-marine sediments accumulated for many millions of years. Mountain ranges and series of low hills bound the basin on all sides with the exception of the westerly margin which is formed by the coast line. The closest uplands to the site are the Rosecrans and Dominguez Hills which lie approximately 1.9 miles to the southwest of the site.

The central portion of the basin is characterized as having very low relief with subdued drainages. Regionally the land surface grades down to the south-southeast at approximately fifteen (15) feet per mile and towards the Los Angeles River Channel. The improved flood control river channel lies approximately 3.8 miles to the east of the site. Compton Wash, a secondary flood control channel, drains south and lies 0.8 miles to the west/southwest of the site. Active oil fields lie approximately two (2) to three (3) miles to the southwest of the project site within the Rosecrans and Dominguez Hills. These hills consist of local faulted anticlines (petroleum reservoirs) which are associated with the Newport-Inglewood structural zone of deformation.

Tectonically, the Los Angeles Basin is an active region for faulting and earthquakes. Mapped active faults project along the base of many of the basin bounding uplands. Even at great depth, the basin is underlain by suspected low angle blind thrust faults which rupture infrequently (e.g. Whittier Narrows [1987] and Northridge [1994] earthquakes). This faulting and earthquake activity are in direct response to continued north-south compression across the basin. The compression is caused by movement along the active San Andreas Fault System which forms the boundary between the North American and Pacific tectonic plates.

4.2 Site Geologic Conditions

The project site is situated in an area which is totally urbanized by commercial and residential development. The land surface prior to development was one of very low and well rounded relief. Recent river alluvium mantles the area and was deposited by the Los Angeles River prior to its historic improvements. The project site lies between 100 and 105 feet above sea level.

The Holocene (recent) age alluvium beneath the ground surface is approximately fifty (50) feet thick based on projected water well data (DWR, 1961). Exploratory borings, drilled at both sites to depths of fifty (50) feet, indicate that the recent alluvium consists mostly of unconsolidated, medium dense, thin layers and

lenses of silty sands and sand sized materials. At a depth of approximately fifteen (15) feet interbedded clays and sands were encountered. Pleistocene age older river alluvium underlies the recent deposits to a depth of approximately 1,200 feet and consists of sands and gravel layers which form the basin groundwater aquifers. Bedrock assigned to the marine Pliocene age Pico Formation (siltstone) forms a thick section beneath the alluvial sequence.

The project site lies within the central basin groundwater pressure area (Downey Plain). This is a zone where groundwater has been extracted from the confined aquifers within the Pleistocene age (older alluvium) deposits. Six (6) aquifers under hydrostatic conditions have been identified within the site region. The depth to groundwater for the deep aquifers averages about fifty (50) feet below sea level or one-hundred and fifty (150) feet of depth below the ground surface (Los Angeles Co. Flood Control District, 1982). Prior to pumping in the basin, pressurized groundwater was within about fifty (50) feet of the surface.

4.3 Subsoil Conditions

The generalized stratigraphic profile consists of up to five (5) feet of fill soils (gravelly sands and silty sands) underlain by eleven (11) to eighteen feet of native sands and silty sands which are in turn underlain by interbedded silts, silty sands and clays.

Based on SPT test results at the boring locations, the underlying native soils are considered to be medium dense to very dense, and very stiff to hard for sands and clays, respectively.

4.4 Groundwater Conditions

Groundwater was encountered at approximately twenty-five (25) feet below existing grade at boring B-10 and B-11 locations (northern parcel) at the time of the site investigation; however, groundwater conditions may be altered by geologic detail away from the boring location, by seasonal and meteorological variations, and by construction activity.

As stated in subsection 4.2, the depth to groundwater is approximately 150 feet deep in the vicinity of the site. The indurated silts and clays encountered at boring B-10 and B-11 locations most likely form a relatively impermeable layer which causes groundwater to perch.

4.5 Test Pits

A total of six (6) test pits up to a depth of approximately four (4) feet were excavated at the northern parcel. The objectives of these test pits were to determine the locations and extent of existing railroad ties in this area.

Two and one-half (2.5) to four (4) feet of fill soils (gravelly sands) were encountered at all six (6) test pits locations at the northern parcel. The gravelly sands contain occasional cobbles, boulders and

concrete/asphaltic concrete pieces up to eighteen (18) inches in largest dimension. No railroad ties were encountered at the test pit locations.

V. SEISMOLOGICAL CONDITIONS

5.1 Faulting

There are no known or mapped faults that if projected would trend directly towards or through the subject property; however, due to the thick sequence of Pleistocene age sediments in the central portion of the basin, faults if present go undetected. Current thinking among seismologists is that most of the Los Angeles Basin is underlain by more than one blind thrust fault. These are deeply buried, very low angle, probably north dipping faults that do not project to the ground surface. These thrust faults can produce major earthquakes anywhere beneath the basin surface but direct surface fault rupture is unlikely. The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California division of Mines and Geology. For these reasons, the possibility of direct surface fault rupture in proximity to the property is considered unlikely.

Damaging earthquakes of Magnitude M5.5 or greater occur on an average every two (2) years in Southern California. Most of these are associated with faults that have known Holocene (last 10,000 years) displacements or have produced historic earthquakes. Three (3) near field faults, namely the Newport-Inglewood (Avalon/Compton segment), Palos Verdes and Santa Monica-Hollywood will have the greatest effect on the site with regard to strong earthquake ground motions should they rupture opposite the property. At this time very little is known about the inferred blind thrust fault(s) that might underlie the basin. Their slip surfaces (hypocenter locations) are at an assumed depth of greater than fifteen (15) miles and the ground motions created by such a blind thrust fault rupture, in theory, could be excessively high. If such an event was to occur directly below the site, ground motions may exceed those produced by the 1994, Magnitude M6.7 Northridge Earthquake.

Seismic hazards within the site can be attributed to ground shaking resulting from events on distant active faults. Listed on Table I are the active and potentially active faults which can significantly affect the site. Figure A-3, Appendix A, shows the geographic relationship of these faults to the site.

Historic earthquake epicenters from 1800 to 1997 within a sixty-five (65) mile radius of the site and exceeding a Magnitude of M4.0 on the Richter Scale are shown on Figure A-4, Appendix A. Historic earthquakes exceeding a Magnitude of M6.0 are tabulated on Table II. The seismic recurrence curve based on historical earthquakes is provided in Figure A-5, Appendix A.

5.2 Earthquake Effects

5.2.1 Site Accelerations

The Newport Inglewood and Palos Verdes faults, due to their proximity to the site, will have the greatest

**TABLE I
EARTHQUAKE PARAMETERS**

Fault	Approximate Distance from Site (Mile)	Maximum Magnitude Richter Scale		Peak Horizontal Acceleration (g)		Repeatable Acceleration (g)		Duration of Strong Ground Shaking (Sec)	
		CRED.	PROB.	CRED.	PROB.	CRED.	PROB.	CRED.	PROB.
Newport-Inglewood-Offshore	4	7.0	6.5	0.42	0.32	0.27	0.21	26	21
Elysian Park Seismic Zone	7	6.75	6.0	0.26	0.18	0.17	0.12	23	15
Whittier-North Elsinore	10	7.0	6.5	0.23	0.18	0.15	0.12	26	21
Santa Monica - Hollywood	11	7.5	7.0	0.26	0.20	0.17	0.13	32	26
Raymond	12	6.75	6.25	0.17	0.13	0.11	0.08	23	18
Palos Verdes-Coron. B. - A. Blan	12	7.5	7.0	0.26	0.20	0.17	0.13	32	26
Verdugo	13	6.7	4.5	0.15	0.05	0.10	0.03	23	4
Sierra Madre-San Fernando	18	7.5	7.0	0.17	0.13	0.11	0.08	32	26
San Gabriel	19	7.0	5.75	0.12	0.06	0.08	0.04	26	12
Malibu Coast	20	7.5	6.5	0.15	0.09	0.10	0.06	32	21
Northridge Hills	24	6.5	5.0	0.07	0.03	0.05	0.02	21	4
Cucamonga	27	7.0	6.5	0.08	0.06	0.05	0.04	26	21
Chino	29	7.0	5.5	0.07	0.03	0.05	0.02	26	10
Santa Susana	29	7.0	6.0	0.07	0.04	0.05	0.03	26	15
Anacapa	30	7.0	5.0	0.07	0.02	0.05	0.01	26	4
Simi-Santa Rosa	35	7.0	5.25	0.06	0.02	0.04	0.01	26	7
Holser	37	6.6	5.75	0.04	0.03	0.03	0.02	22	12
San Andreas (Mojave)	41	8.3	8.0	0.09	0.08	0.06	0.04	40	37
Oak Ridge (On shore)	42	7.2	6.5	0.05	0.03	0.03	0.02	28	21
Glen. Helen - Lytle Cr.-Claremont	43	7.0	6.5	0.04	0.03	0.03	0.02	26	21
Elsinore	43	7.5	6.75	0.06	0.04	0.04	0.03	32	23
Catalina Escarpment	44	7.0	6.25	0.04	0.03	0.03	0.02	26	18
San Cayetano	45	7.5	6.25	0.05	0.03	0.03	0.02	32	18
Clearwater	45	7.0	3.0	0.04	0.01	0.03	0.01	26	4
San Andreas (S.Bern. Mtn.)	50	8.0	6.75	0.06	0.03	0.04	0.03	37	23
San Clemente - San Isidro	50	8.0	6.5	0.06	0.03	0.04	0.03	37	21
North Frontal Zone	51	7.7	5.75	0.05	0.02	0.03	0.01	34	12
Oak Ridge (Offshore)	56	7.2	5.5	0.03	0.01	0.02	0.01	28	10
Pine Mountain	56	7.0	4.25	0.03	0.01	0.02	0.01	26	4
Santa Ynez (East)	58	7.5	5.25	0.04	0.01	0.03	0.01	32	7
San Geronio - Banning	58	7.5	7.0	0.04	0.03	0.03	0.02	32	26
Ventura - Pitas Point	59	7.2	7.75	0.03	0.01	0.02	0.01	28	12

TABLE II
HISTORIC EARTHQUAKE EPICENTERS – 1800 to 1997
(exceeding six [6] on the Richter Scale of Magnitude within a sixty-five [65] mile radius of the project site)

DATE (mm/dd/yy)	RICHTER MAGNITUDE (M)	APPROXIMATE DISTANCE SITE TO EPICENTER (miles)	SITE ACCELERATION (g)
12/08/1812	7.0	45	0.04
09/24/1827	7.0	44	0.04
07/11/1855	6.3	14	0.12
12/16/1858	7.0	43	0.04
04/04/1893	6.0	32	0.04
07/30/1894	6.0	44	0.02
07/22/1899	6.5	49	0.03
09/03/1905	5.3*	5	0.15
05/15/1910	6.0	51	0.02
07/23/1923	6.25	57	0.02
03/11/1933	6.3	27	0.05
02/09/1971	6.4	34	0.04
10/01/1987	5.9	13	0.10
01/17/1994	6.7	19	0.10

* Largest Site Acceleration During Time Period

effect on the project site relative to strong ground shaking. Other more distant active faults may also produce notable ground motions but not to the degree as the Newport-Inglewood fault. Table I was prepared to show the anticipated ground motions at the site from the various regional active faults. The maximum probable event is the highest magnitude based on a 100 year return period. The maximum credible is the highest possible magnitude based on the known tectonic framework of any one particular fault.

Based on calculated empirical deterministic data, the Newport-Inglewood fault could produce a Magnitude M6.5 (maximum probable) earthquake at a distance of approximately four (4) miles from the site. Peak horizontal ground accelerations could range up to 0.32g with repeatable accelerations of 0.21g lasting up to twenty-one (21) to twenty-six (26) seconds.

5.2.2 Tsunamis/Seiche

The property is far enough from the coast or large inland body of water to preclude damage from a tsunami or seiche wave, or inundation caused by the rupture of an up-gradient reservoir.

5.2.3 Settlement

A very strong earthquake conceivably could cause minor regional subsidence within the basin; however, such subsidence would not be expected to be differential across the site.

5.2.4 Liquefaction

Liquefaction occurs when the pore pressures generated within a soil mass equals the overburden pressure. This results in a loss of strength and the soil then possesses a certain degree of mobility.

Factors considered to evaluate liquefaction potential include groundwater conditions, soil type, particle size distribution, earthquake magnitude and acceleration, and soil density obtained through the Standard Penetration Test (SPT). Soils subject to liquefaction comprise saturated fine grained sands to coarse silts. Coarser-grained soils are considered free-draining and therefore dissipate excess pore pressures, while fine-grained soils possess undrained shear strength.

Liquefaction at this site is considered unlikely due to the clayey layers and the medium dense to dense character of the granular soils below proposed foundation level (twenty-five [25] feet).

5.2.5 Surface Rupture

There are no known or mapped capable faults trending toward or through the site area. Therefore, the potential for direct surface rupture is considered low.

5.2.6 Landsliding

The site is relatively flat and not near any upland slopes. This precludes the potential for seismically induced or static landsliding.

5.3 Seismic Risk Analysis

The risk analysis is based on the premise that moderate to large earthquakes occur on mappable Quaternary faults and that the occurrence rate of earthquakes on each fault is proportional to the Quaternary fault-slip-rate. This analysis assumes that earthquakes are distributed uniformly and therefore does not consider when the last earthquake occurred on the fault. The length of rupture of the fault as a function of earthquake magnitude is accounted for, and ground motion estimates at a site are made using the magnitude of the earthquake and the closest distance from the site to the rupture zone. The risk analysis summarized on Figure A-6, Appendix A, has explicitly taken into account uncertainties associated with:

- The earthquake magnitude;
- The rupture length given magnitude;
- The location of the rupture zone on the fault;
- The maximum possible magnitude of earthquakes; and,
- The acceleration at the site given magnitude of earthquake and distance from the rupture zone to the site.

It can be noted from Figure A-6, Appendix A, that the ten (10) percent probability of exceedance in fifty (50) years corresponds to the maximum probable earthquake acceleration. The ten (10) percent probability of exceedance is the acceleration that is recommended by the Los Angeles County Fire Department in their "RMPP Guide". This acceleration of 0.32g, as noted in Figure A-7, Appendix A, has an estimated average return period of 500 years.

5.4 Design Response Spectra

The seismic response of a structure or element is dependent upon its strength, damping characteristics, and the stress-strain relationship for the structure considered. Response design spectra for maximum probable (equal to ten [10] percent probability of exceedance in fifty [50] years) and repeatable accelerations are provided as Figures A-8 and A-9, Appendix A, respectively. The response spectrum is defined as a graphical relationship of maximum response of a single-degree-of-freedom elastic system with damping to dynamic motion or forces. The most usual measures of response are maximum displacement, D , which is a measure of the strain in the spring element of the system; maximum pseudo relative velocity, V , which is a measure of the energy absorption in the spring of the system; and maximum pseudo acceleration, A , which is a measure of the maximum force in the spring of the system.

To account for large period motions and low frequencies (Newmark and Hall, 1982), the spectra provided in Figures A-8, A-9, and A-10, Appendix A, have been modified by taking the velocity spectral response value to vary as the period to the one-third (1/3) power. In addition, the spectrum was considered to correspond to a constant displacement equal to amplified ground displacements for periods longer than approximately six (6) seconds.

It is suggested that the design spectrum for vertical response be considered equal to two-thirds (2/3) that for horizontal response. The responses for motions in the various directions (horizontal and vertical) may not occur at the same time. It is reasonable to combine the effects of the several components of motion in a probabilistic manner, by taking the maximum stress, deflection, or other specific response as the square root of the sums of the squares of the corresponding responses to the individual components of motion.

5.5 Site Coefficient

Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile S2. Therefore, a site coefficient of 1.2 may be used for the Watts Towers Crescent project.

5.6 Soil Profile Type and Seismic Source Type

Based on the 1997 Uniform Building Code the project falls into the following categories:

- Soil Profile Type: S_D
- Seismic Source Type: B

VI. CONCLUSIONS

Based on the results of our site investigation, and geologic and seismicity study, it is our opinion that the site is suitable for the proposed development. From a geotechnical engineering standpoint, the following observations which may influence design and construction decisions were noted.

- The subgrade soils at both the northern and southern parcels are saturated at proposed subgrade elevations. Further, groundwater was measured at the northern parcel at subgrade elevation, at the boring locations, immediately subsequent to drilling.
- Layers of silt and clay soils, above proposed subgrade elevation, with moisture contents above optimum were also noted at both the northern and southern parcels.

VII. SITE DEVELOPMENT RECOMMENDATIONS

7.1 General

The proposed Watts Towers Cultural Crescent will consist of commercial developments with underground parking, Watts Tower Plaza amphitheater and landscape areas. The underground parking will be based

approximately twenty-five (25) feet below existing grade and the amphitheater up to approximately four (4) to five (5) feet deep. Structural loads are not available at this time.

The proposed development is feasible from a geotechnical engineering standpoint. Project plans and specifications should take into account the appropriate geotechnical features of the site and conform to the geotechnical recommendations.

7.2 Clearing

All surface vegetation, asphaltic concrete, trash and debris should be cleared and removed from the proposed site. Soils with organic inclusions and particles over six (6) inches in largest dimension should be removed from the construction areas. Topsoil, if any, may be stockpiled for future use.

Underground facilities such as utilities, pipes or underground storage tanks may exist at the site. Removal of underground tanks is subject to state law as regulated by County or City Health and/or Fire Department agencies. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects.

Septic tanks should be removed in their entirety. Cesspools or seepage pits should be pumped of their contents and removed in their entirety.

Any water wells, if encountered during construction, should be exposed and capped in accordance with the requirements of the regulating agencies.

Depressions resulting from the removal of buried obstructions should be backfilled with properly compacted material.

7.3 Excavatability

All types of deposits encountered in the borings are considered economically rippable in open excavations with conventional grading and excavation equipment.

7.4 Fill Placement

7.4.1 Preparation of Surface Soils

Prior to placing any fill, the exposed surface soils should be scarified to a minimum depth of six (6) to eight (8) inches, moisture conditioned or dried to near optimum moisture content and compacted to a minimum of ninety (90) percent relative compaction, based on ASTM D 1557.

7.4.2 Compaction

Cohesive soils should be placed in loose lifts not exceeding six (6) inches, moisture-conditioned or dried to

approximately two (2) to four (4) percentage points above optimum, and compacted to a minimum of ninety (90) percent relative compaction (ASTM D 1557).

Granular fill materials should be placed in loose lifts of six (6) to eight (8) inches, moisture-conditioned or dried to near optimum, and compacted to a minimum of ninety (90) percent relative compaction (ASTM D 1557).

7.4.3 Fill Material

Based on observations at the boring locations, the upper sixteen (16) to twenty-four (24) feet of subsoils at the site consist of primarily granular soils. These on-site granular soils possess a "very low" expansion potential. These soils may be reused as compacted fill provided they are free of organics, deleterious materials, debris and particles over six (6) inches in largest dimension.

Any soils imported to the site for use as fill for subgrade materials should be predominantly granular and non-expansive (Expansion Index less than twenty [20]), and approved by the Geotechnical Engineer prior to importing.

7.5 Subgrade Preparation

The areas within the underground parking should be taken down to the subgrade elevation. The exposed subgrade should be observed by GEOBASE to verify the removal of all unsuitable soils. Areas where the subgrade soils have been disturbed should be reworked and the materials compacted as described in subsection 7.4. Due to the wet nature of the subgrade soils, stabilization of the subgrade may be required prior to recompaction. Stabilization may involve overexcavation and placement of a rock layer and filter fabric.

All existing fills within the construction areas should be removed and replaced as properly compacted fill, unless certified. The lateral extent of removal beyond the construction areas should be at least equal to the depth of fill. Undocumented fills varying from one (1) to four (4) feet in thickness were encountered at various boring locations. If undocumented fills are observed to extend deeper or at other locations during construction, they should be removed and replaced as properly compacted fill. The exposed subgrade should be observed to verify the removal of all unsuitable materials.

The amphitheater, walkways and flatwork areas and within two (2) feet of their proposed limits should be underlain by a uniformly compacted fill blanket a minimum of two (2) feet in thickness. Based on observations at the boring locations, the thickness of undocumented fill soils may vary from one (1) to four (4) feet. Therefore, overexcavation of the natural soils will be required in the amphitheater area.

7.6 Drainage

To enhance future site performance, it is recommended that all pad drainage be collected and directed away from proposed structures and temporary cut slopes to disposal areas off site. For soils areas, we recommend that a minimum of two (2) percent gradient away from foundation elements be maintained. It is important that

drainage be directed away from foundations and that proper drainage patterns be established at the time of construction and maintained throughout the life of the structures.

Care should be exercised in controlling surface runoff into the temporary cut slopes, if used for the basement excavation, to prevent erosion of the slope surfaces.

7.7 Temporary Excavations

Temporary construction excavations are anticipated for construction of utility trenches, amphitheater and the underground parking areas. The following recommendations are provided for planning purposes only. Detailed investigations will be required to establish soil parameters for final design.

7.7.1 Unsupported Excavations

Temporary construction excavations in soils may be made vertically without shoring to a depth of approximately three (3) to four (4) feet below adjacent surrounding grade. For deeper cuts in soils, the slopes should be properly shored as discussed in subsection 7.7.2 or sloped back at least 1H:1V (Horizontal:Vertical) or flatter. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of excavation unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at forty-five (45) degrees below the edge of any nearby adjacent existing site facilities should be properly shored to maintain foundation support of the adjacent structures.

All excavations and shoring systems should meet, as a minimum, the requirements given in the State of California Occupational Safety and Health Standards. Stability of temporary slopes are the responsibility of the contractor.

7.7.2 Shored Excavations

In areas where space does not permit sloped excavations, temporary shoring may be used to support vertically cut excavations. In the following paragraphs, recommendations are provided for both cantilevered and braced/tied back shoring.

All shoring systems should meet minimal requirements given in the State of California Occupational Safety and Health Standards.

For excavation depths of ten (10) to fifteen (15) feet a cantilevered shoring system may be used only in areas where lateral movement of soils behind the wall can be tolerated. A braced or tieback shoring system should be used in areas where the performance of the adjacent structures are susceptible to movements and for excavation depths in excess of fifteen (15) feet.

Temporary shoring walls supporting a horizontal surface should be designed to resist earth pressures, excluding surcharge loads, as follows:

- For cantilever walls, use an equivalent fluid pressure of thirty-five (35) pcf (triangular pressure distribution).
- For braced/tieback wall, use a rectangular pressure distribution of twenty-five (25) H psf; where H is height of wall above base of excavation in feet.

Figure A-11, Appendix A, shows the active pressure distribution for a shoring wall supporting a horizontal surface. Surcharge effects for uniform surcharge loads may be computed using the pressure distribution provided in Figure A-12, Appendix A. It is recommended that a uniform surcharge pressure be included in the design where the localized effect of heavy equipment such as cranes or concrete trucks are anticipated within approximately fifteen (15) feet of the top of the shoring.

Lateral resistance for soldier piles may be assumed to be provided by passive pressure below the bottom of excavation. Allowable passive pressure equivalent to a fluid pressure of 300 pcf may be used for soldier piles embedded in the dense to very dense sandy soils or very stiff clays. The allowable passive resistance is for soldier piles spaced not less than three (3) diameters center-to-center and includes the doubling effect for isolated piles.

Tie-backs may be designed for an allowable unit skin friction of 300 psf. A bond length sufficient to support the anticipated earth and surcharge loads should be installed behind a line rising at fifty-five (55) degrees from the horizontal starting at the base of the wall, as shown on Figure A-11. The anchors may be installed at angles between fifteen (15) degrees to forty-five (45) degrees below the horizontal. If caving occurs in the drilled shafts, casing should be used prior to concrete pour, but casing must be pulled as the shaft is poured. Structural concrete should be placed in the lower portion of the drilled shaft up to the assumed failure plane. Pouring of the anchors should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the failure plane and the face of the shoring may be backfilled with sand after concrete placement. All of the anchors should be pre-tested for a minimum of fifteen (15) minutes and to at least 150 percent of the design load.

To limit local sloughing, we recommend that exposed soils between the soldier piles be supported by timber lagging. All lumber to be left in the ground should be pressure-treated in accordance with the specifications of the American Wood Preservers Association (AWPA).

Adequate bearing capacity should be provided for the soldier piles. The design vertical load will be a function of the anchor loads and their inclination. These piles may be designed for vertical loads using an allowable unit skin friction of 600 psf. The unit skin friction may be applied to one-quarter of the soldier pile surface or one-half of the sheet pile surface above the base of the excavation, and to the full pile surface area below the base of excavation.

It is recommended that a licensed surveyor be retained to establish monuments on the shoring, the surrounding ground and adjacent structures prior to excavations. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project structural (shoring) engineer and GEOBASE for review and evaluation.

It is recommended that the plans and specifications for the proposed shoring system be reviewed by GEOBASE. The installation and monitoring of shoring should also be observed.

7.8 Trench Backfill

It is our opinion that utility trench backfill could be placed and compacted by mechanical means. Basement wall backfill is discussed in subsection 8.3.2.

If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, other methods of utility trench compaction may also be appropriate as approved by GEOBASE at the time of construction. Jetting or flooding of backfill material is not recommended.

VIII. PRELIMINARY FOUNDATION RECOMMENDATIONS

8.1 General

The following recommendations are provided for preliminary design and planing purposes, and are not intended for use for final design or construction. Detailed investigations with additional borings within each of the proposed building areas will be required to establish soils parameters for final design. All proposed structures should be designed for the seismic factors discussed in Section V. Foundations and slabs reinforcement configurations should meet, as a minimum, the requirements of the governing agencies and the Uniform Building Code for low expansive soils. In this respect, additional expansion index tests should be performed on representative samples obtained from the detailed investigations.

8.2 Foundations

The following recommendations have been formulated from visual, physical and analytical considerations of existing site conditions and are believed to be applicable for the proposed development.

Spread or continuous footings may be used for support of the proposed subterranean parking structure. Footings should be based a minimum of three (3) feet, below lowest adjoining grade. Surface structures may also be supported on footings based eighteen (18) inches below lowest adjoining grade. Fill below footing bases should be compacted to ninety-five (95) percent relative compaction based on ASTM D 1557. In this respect, fills are not anticipated beneath the underground parking.

For the underground parking and shallow footings, allowable dead-plus-live load bearing pressures in the order of 4,000 psf and 1,500 psf, respectively, may be used, plus a one-third (1/3) increase for short-term wind or seismic loads; however, these parameters should be confirmed based on more detailed investigations.

Lateral loads against structures may be resisted by friction between the bottom of foundations and the supporting soils. An allowable friction coefficient of 0.3 is recommended. Alternatively, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot acting against the

foundations may be used, provided the foundations are poured tight against undisturbed or compacted soil. The aforementioned equivalent fluid weight should be reduced by one-third (1/3) if lateral bearing and friction are combined.

8.3 Retaining/Basement Walls

8.3.1 Earth Pressures

All parameters outlined in the following should be confirmed based on detailed investigations.

Walls which are fixed at the top should be designed to resist lateral earth pressures imposed by the surrounding soils and surcharge loads such as soil cover and traffic loads. For static loading conditions, a lateral pressure equivalent to that imposed by a fluid weighing sixty (60) pcf may be used. In addition, a uniform surcharge pressure of one-half (1/2) any vertical pressure adjacent to the structure should be assumed to act on the walls. These aforementioned pressures assume positive drainage such as placement of a backdrain and crushed rock at the lower third of the wall height. Alternatively, hydrostatic pressures should be added.

For static loading conditions, walls which are free to rotate at the top may be designed to resist a lateral pressure imposed by an equivalent fluid weighing thirty-five (35) pcf. In addition, a uniform pressure on one-third (1/3) of any vertical pressure adjacent to the structure should be assumed to act on the walls. Drainage recommendations outlined in subsection 8.3.2 should be used.

Wall pressures resulting from earthquakes may be taken as twenty-one (21) percent of static loading conditions, including surcharge. It is emphasized that for triangular pressure distribution the triangular pressure resulting from earthquake loading should be inverted when computing loads on the wall.

8.3.2 Wall Backfill

Placement of wall backfill should conform to the recommendations presented in subsection 7.4. The wall backfill should be well-drained to relieve possible hydrostatic pressures on the wall. Prior to backfill placement, a backdrain consisting of a four (4) inch diameter perforated PVC Schedule 40 pipe should be installed. This drainage pipe may discharge by gravity. Wall backfill may then be placed. At least the lower third (1/3) of wall should consist of crushed rock.

A filter fabric such as "Mirafi 140N", "Supac 4N" or approved equivalent should be placed at the interface between the backfill and finer grained soils. The wall backfill should be sealed at the surface from water infiltration with at least one (1) foot of relatively impermeable soils. A typical wall backfill detail is provided as Figure A-13, Appendix A.

Alternatively to the above, hydrostatic pressures may be incorporated into the wall design.

The basement wall should be waterproofed to prevent moisture build-up on the interior sides of the walls as

a result of water mitigation from the soils in contact with the walls. The waterproofing should be applied for the full height of the basement walls.

8.4 Floor Slabs

Slabs-on-grade may be used for supporting floor loads at the site. The subgrade should be prepared in accordance with the recommendations provided in subsection 7.5.

Based on the results from the present investigation, the subgrade soils have a low expansion potential. In areas with low expansive soils, no special measures, other than maintaining uniform moisture conditions in the subgrade are required for favorable performance.

Additional expansion index tests should be carried out on representative samples obtained from the detailed investigations. Appropriate recommendations for preparing the slab subgrade and for the design of the floor slabs should be based on the results of the additional tests.

The slab should be underlain by minimum of four (4) to six (6) inches of sand. In moisture sensitive areas, a ten (10) mil polyethylene moisture barrier should be placed beneath the slab, with a minimum of two (2) inches of sand between the slab and the moisture membrane. A subgrade modulus of 100 pounds per square inch per inch may be used for slab design. The slab should be reinforced to meet the requirements of the regulating agencies as well as those of UBC for low expansive soils, as appropriate. In addition, structural loading, shrinkage and temperature stresses should be considered.

The recommendations presented above are intended to reduce the potential for cracking of slabs and foundations; however, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade may still exhibit some cracking. The occurrence of concrete shrinkage cracks are independent of the supporting soil characteristics.

8.5 Pavement Areas

The upper four (4) feet of subsoils at the site are predominately granular, with R-values of forty-six (46) and seventy-two (72). Based on the anticipated light traffic conditions (surface parking), a minimum pavement section consisting of three (3) inches of asphaltic concrete and four (4) inches of aggregate base materials may be considered. This should be confirmed with R-value testing of the subsoils.

IX. SOIL CORROSIVITY -- IMPLICATIONS

Electrical resistivities measured on two (2) soil samples from the site indicated a "moderate" potential for attack on steel.

Water-soluble sulfates determination conducted for this site indicated a "low" potential for attack on concrete. therefore, Type I Portland cement may be used for the construction of concrete structures in contact with the subgrade soils.

X. RECOMMENDATION FOR ADDITIONAL WORK

The preliminary recommendations presented in the previous sections were intended for use for preliminary design and planning of the Watts Towers Cultural Crescent and not intended for final design or construction purposes. Detailed investigations should be carried out after the proposed structure locations have been finalized. This work may include:

- Additional, closer spaced borings within the proposed building footprints.
- Additional Expansion Index Tests and R-Value tests should be carried out within pre-determined building and pavement areas, respectively.

XI. LIMITATIONS

The preliminary geotechnical evaluation presented herein was performed in accordance with generally accepted geotechnical engineering principles and practice. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is intended for use with regard to the specific project discussed herein. The conclusions and recommendations contained in this report are based on the data relating only to the project and location discussed herein.

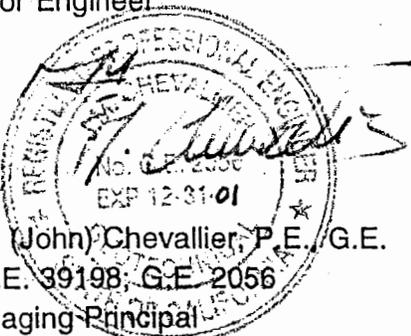
The analyses and recommendations submitted in this report are based upon the observations noted during drilling of the borings shown on the site plans, interpretation of laboratory test results and geological evidence. This report does not reflect any variations which may occur between the borings.

Respectfully submitted
GEOBASE, INC.


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Senior Engineer



C.M. Payne
R.G. 933; C.E.G. 367
Associate Geologist


J-M. (John) Chevallier, P.E., G.E.
P.C.E. 39198; G.E. 2056
Managing Principal

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IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

As the client of a consulting geotechnical engineer, you should know that site subsurface conditions cause more construction problems than any other factor. ASFE/The Association of Engineering Firms Practicing in the Geosciences offers the following suggestions and observations to help you manage your risks.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

Your geotechnical engineering report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. These factors typically include: the general nature of the structure involved, its size, and configuration; the location of the structure on the site; other improvements, such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask your geotechnical engineer to evaluate how factors that change subsequent to the date of the report may affect the report's recommendations.

Unless your geotechnical engineer indicates otherwise, do not use your geotechnical engineering report:

- when the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size, elevation, or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership; or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems that may occur if they are not consulted after factors considered in their report's development have changed.

SUBSURFACE CONDITIONS CAN CHANGE

A geotechnical engineering report is based on conditions that existed at the time of subsurface exploration. Do not base construction decisions on a geotechnical engineering report whose adequacy may have been affected by time. Speak with your geotechnical consultant to learn if additional tests are advisable before construction starts. Note, too, that additional tests may be required when subsurface conditions are affected by construction operations at or adjacent to the site, or by natural events such as floods, earthquakes, or ground water fluctuations. Keep your geotechnical consultant apprised of any such events.

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL JUDGMENTS

Site exploration identifies actual subsurface conditions only at those points where samples are taken. The data were extrapolated by your geotechnical engineer who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your geotechnical engineer can work together to help minimize their impact. Retaining your geotechnical engineer to observe construction can be particularly beneficial in this respect.

A REPORT'S RECOMMENDATIONS CAN ONLY BE PRELIMINARY

The construction recommendations included in your geotechnical engineer's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Because actual subsurface conditions can be discerned only during earthwork, you should retain your geotechnical engineer to observe actual conditions and to finalize recommendations. Only the geotechnical engineer who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

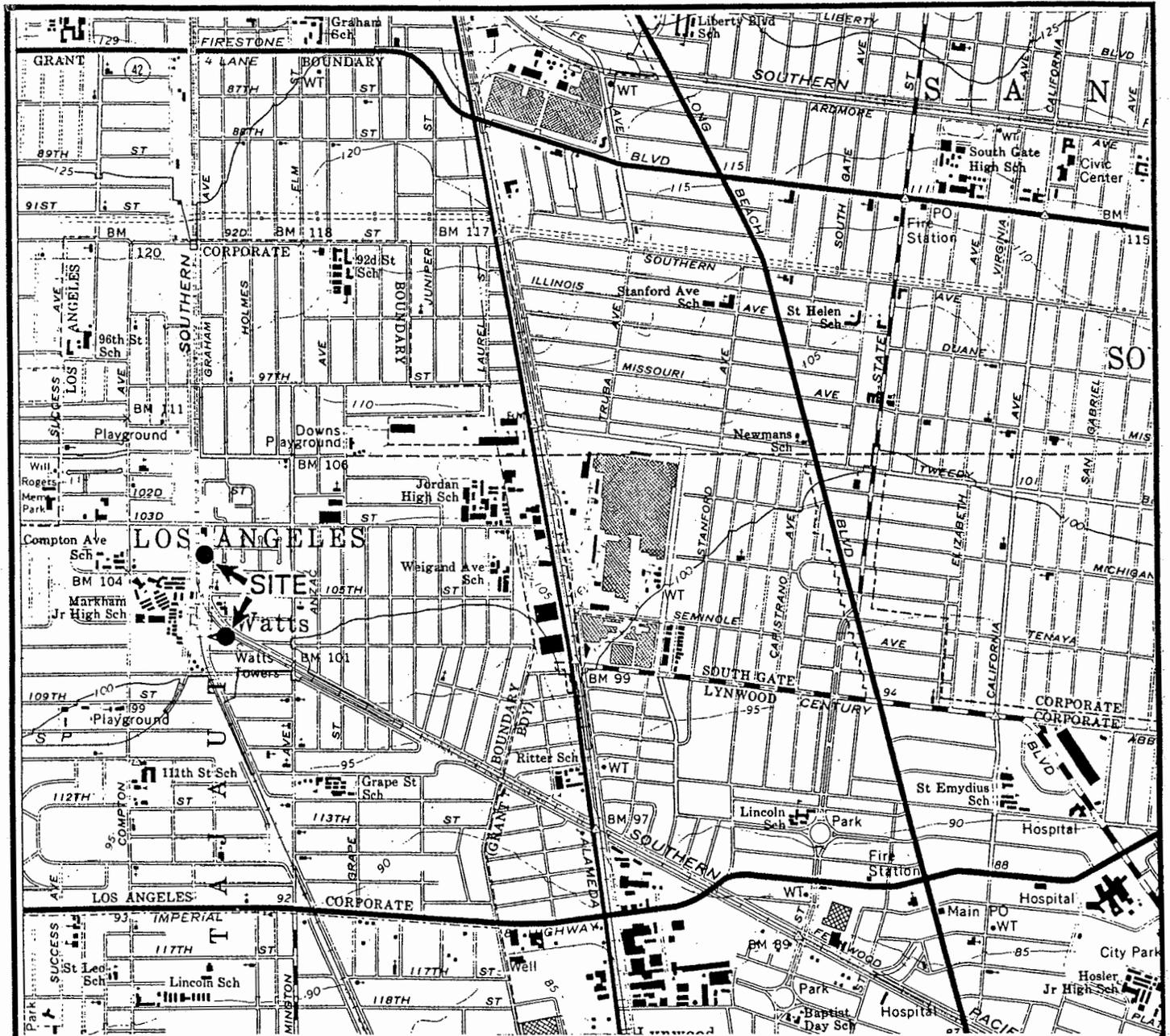
Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your geotechnical engineer prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the geotechnical engineer. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

GEOENVIRONMENTAL CONCERNS ARE NOT AT ISSUE

Your geotechnical engineering report is not likely to relate any findings, conclusions, or recommendations

APPENDIX A

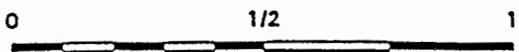
- Figure A-1 Location Map
- Figure A-2 Boring and Test Pit Locations Plan
- Figure A-3 California Fault Map
- Figure A-4 Historic Earthquakes -- 1800 to 1997
- Figure A-5 Seismic Recurrence Curve
- Figure A-6 Seismic Risk Analysis Results -- Probability of Exceedance
- Figure A-7 Seismic Risk Analysis Results -- Average Return Period
- Figure A-8 Horizontal Design Response Spectra -- Acceleration 0.32g
- Figure A-9 Horizontal Design Response Spectra -- Acceleration 0.21g
- Figure A-10 Earth Pressures and Tieback Geometry for Shoring
- Figure A-11 Additional Lateral Earth Pressures on Shoring
- Figure A-12 Typical Wall Backfill Detail



NOTE : Base Map from USGS 7.5 minute series South Gate (1964, Photo Revised 1981), California Topographic Quadrangle.



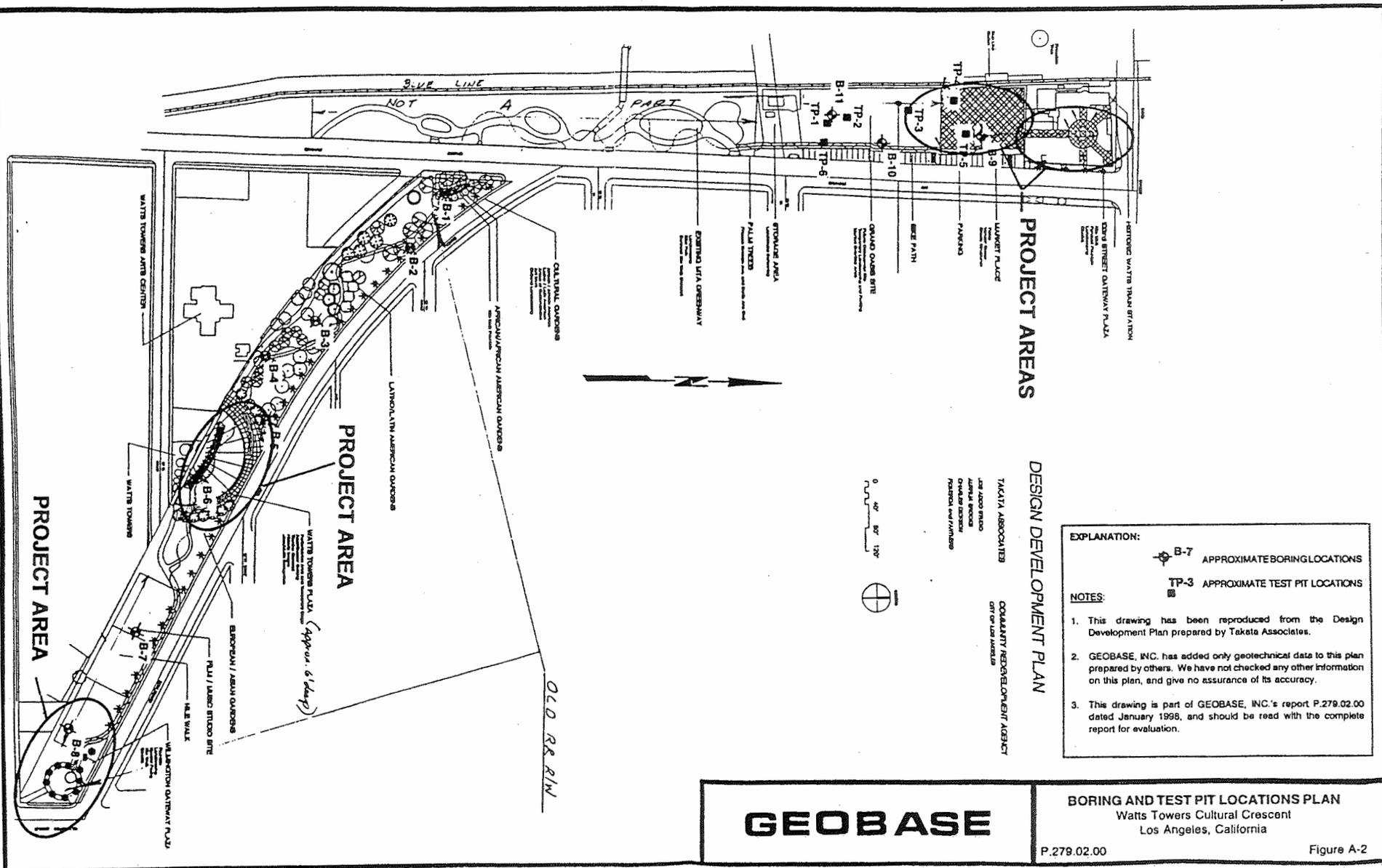
NORTH



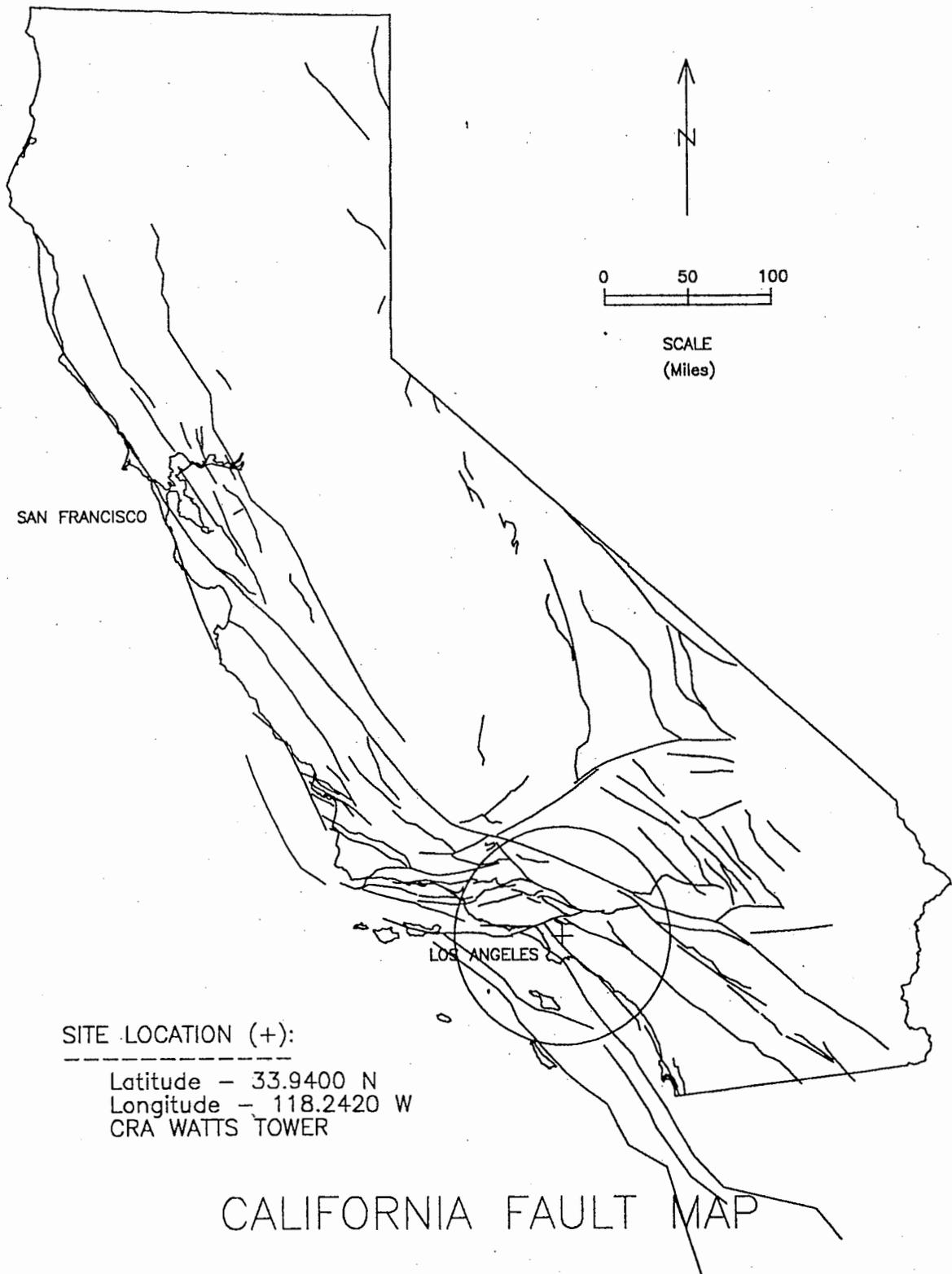
SCALE IN MILES

GEOBASE

LOCATION MAP
Watts Towers Cultural Crescent
Los Angeles, California



GEOBASE



SAN FRANCISCO

LOS ANGELES

SITE LOCATION (+):

Latitude - 33.9400 N
 Longitude - 118.2420 W
 CRA WATTS TOWER

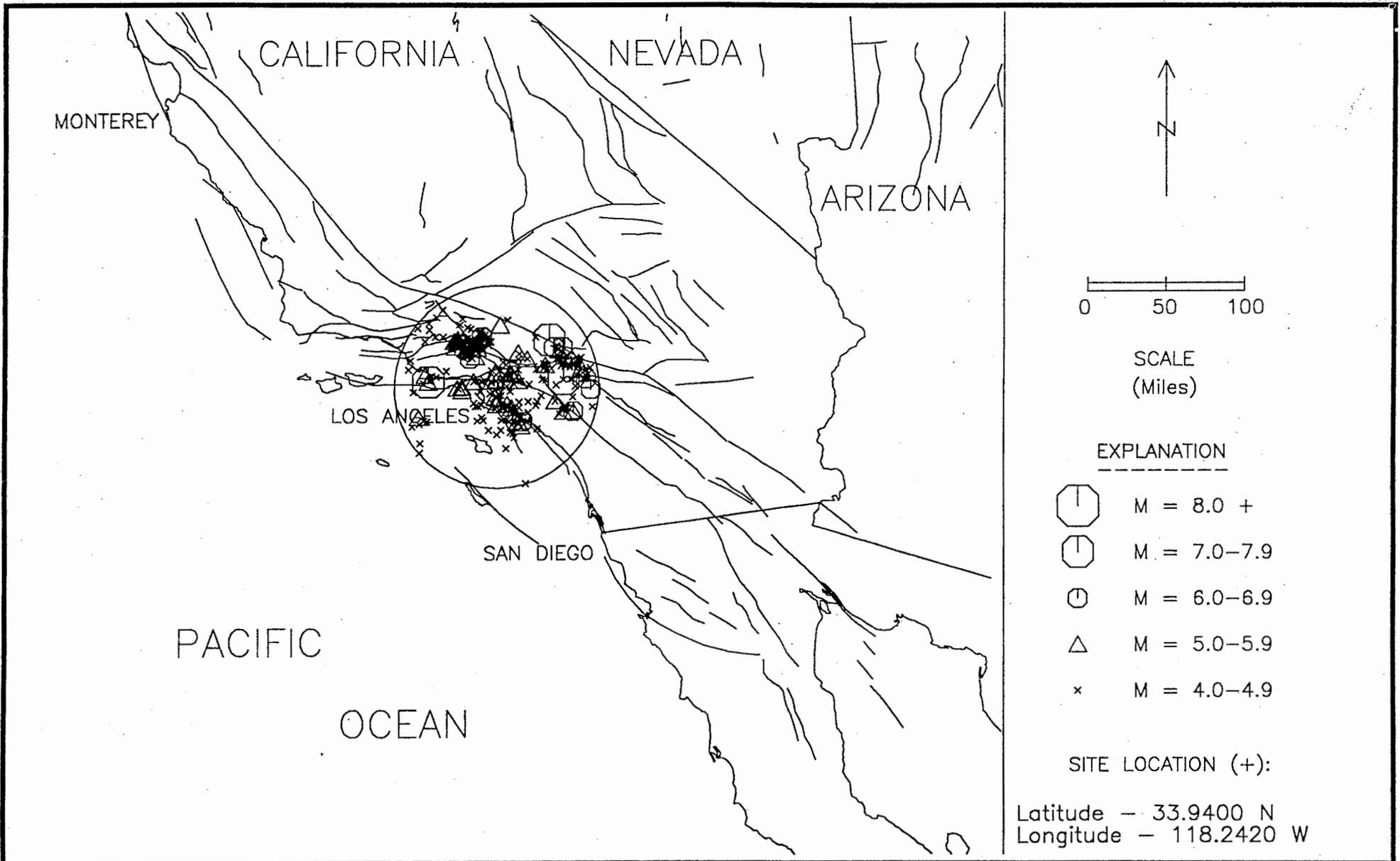
CALIFORNIA FAULT MAP

GEOBASE

CALIFORNIA FAULT MAP
 Watts Towers Cultural Crescent
 Los Angeles, California

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Figure A-3



GEOBASE

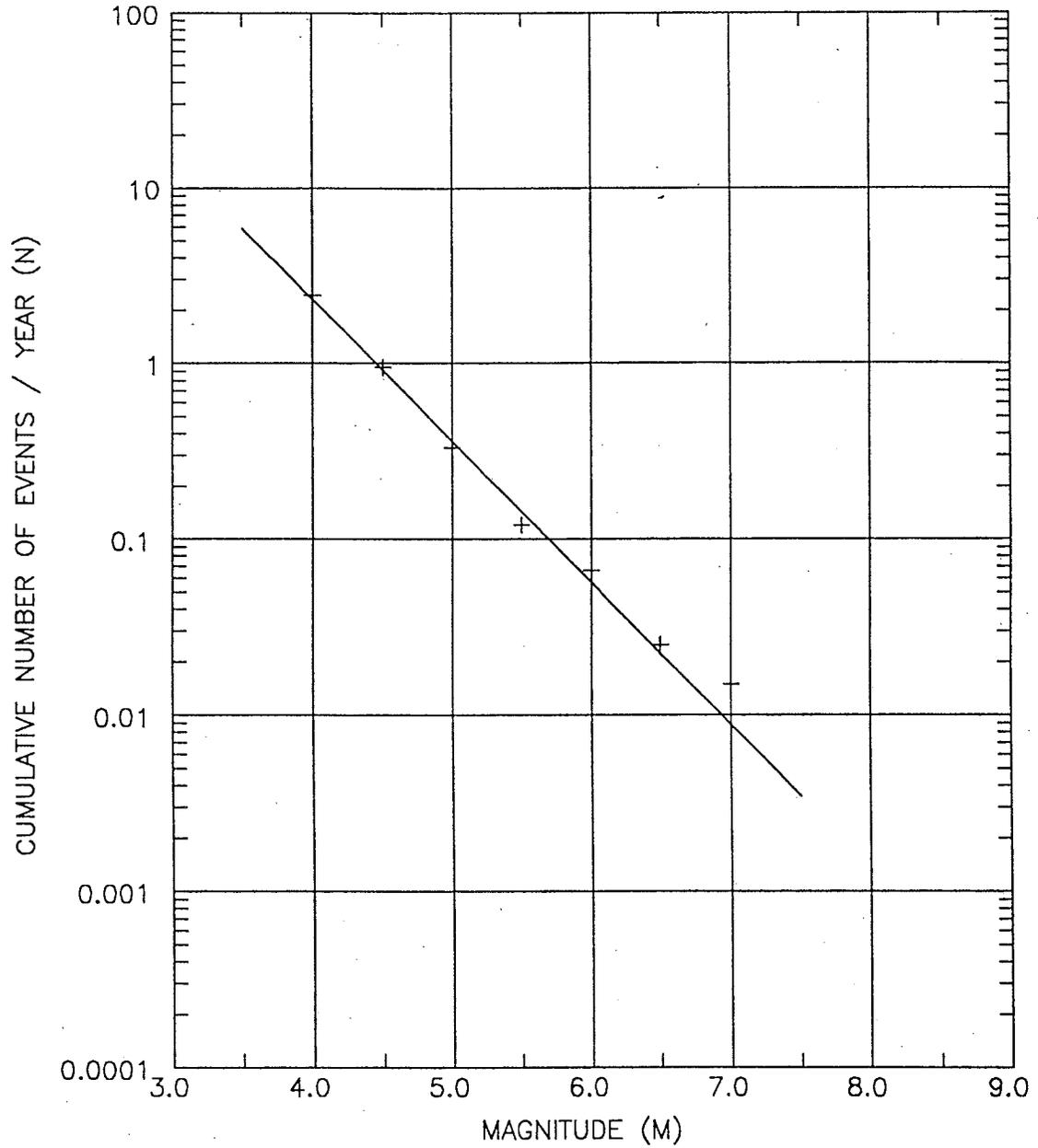
HISTORIC EARTHQUAKES--1800 TO 1997

Watts Towers Cultural Crescent
Los Angeles, California

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Figure A-4

$$\text{LOG } N = 3.594 - 0.807M$$



SEISMIC RECURRENCE CURVE

HISTORICAL EARTHQUAKES FROM 1800 TO 1997

GEOBASE

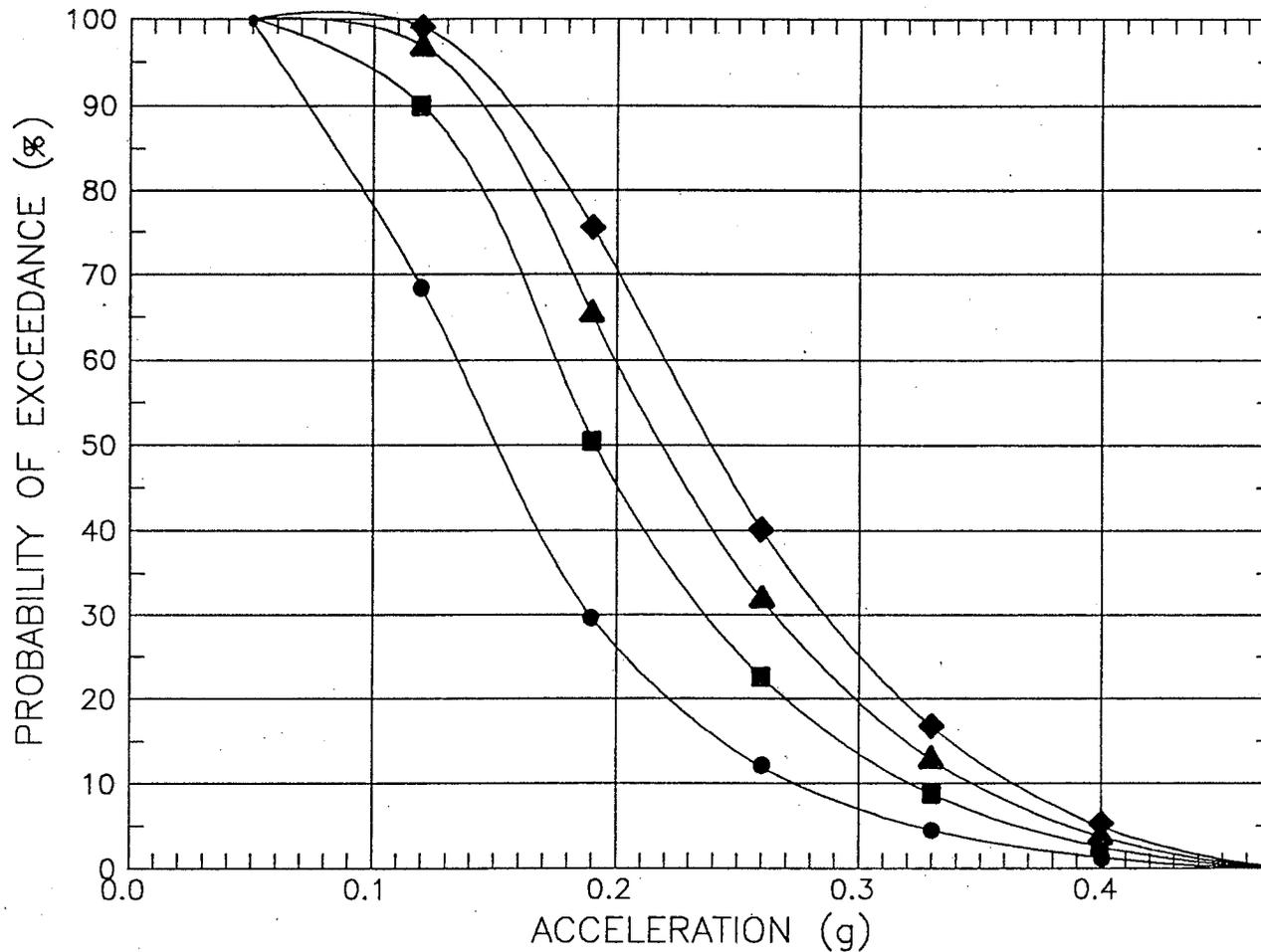
SEISMIC RECURRENCE CURVE

Watts Towers Cultural Crescent
Los Angeles, California

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Figure A-5

PROBABILITY OF EXCEEDANCE vs. ACCELERATION



EXPOSURE PERIODS:
 ● 25 years ▲ 75 years
 ■ 50 years ◆ 100 years

CAMPBELL (1993) HORIZ. MEAN

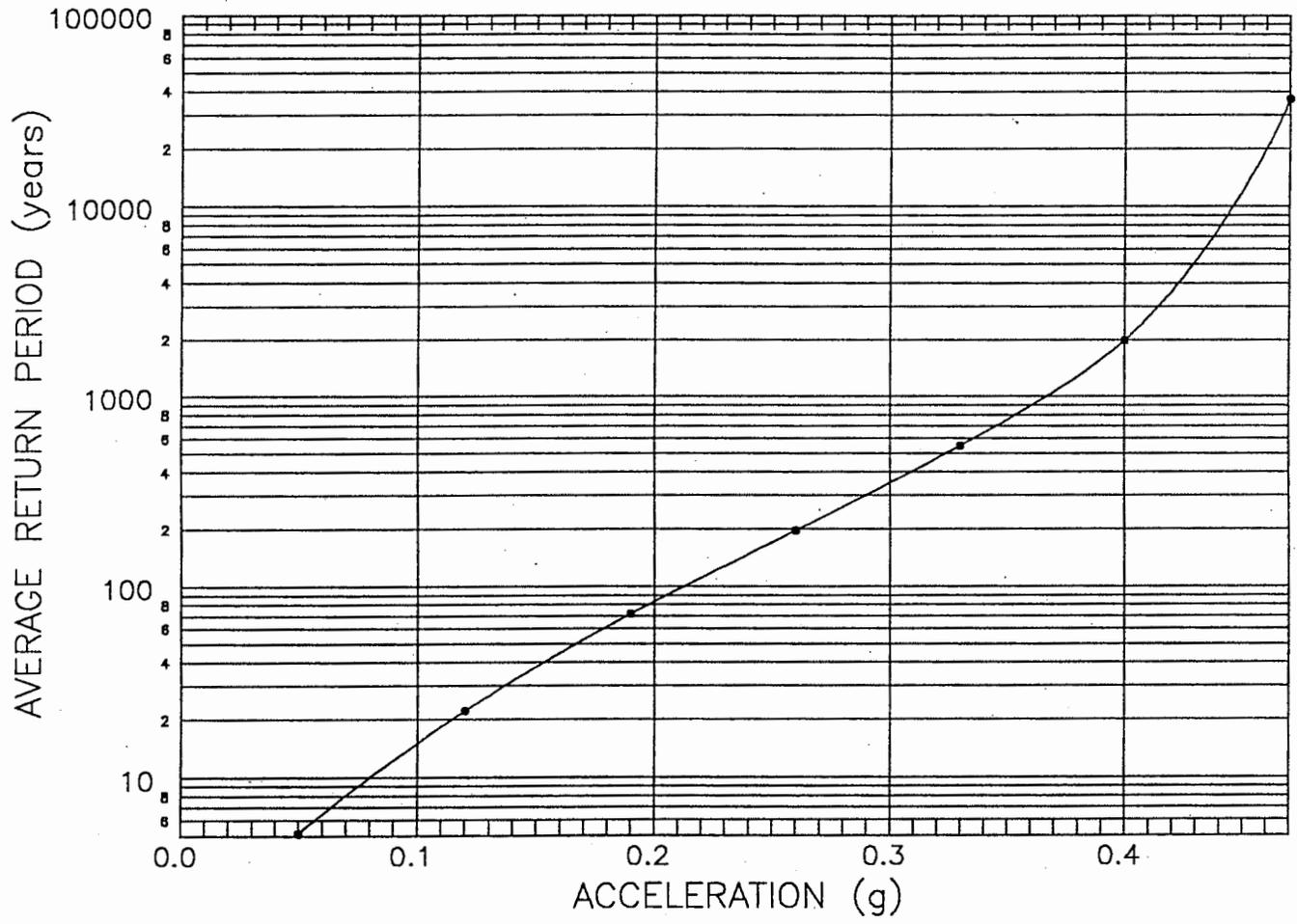
GEOBASE

SEISMIC RISK ANALYSIS RESULTS
-PROBABILITY OF EXCEEDANCE-
 Watts Towers Cultural Crescent
 Los Angeles, California

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Figure A-6

AVERAGE RETURN PERIOD vs. ACCELERATION

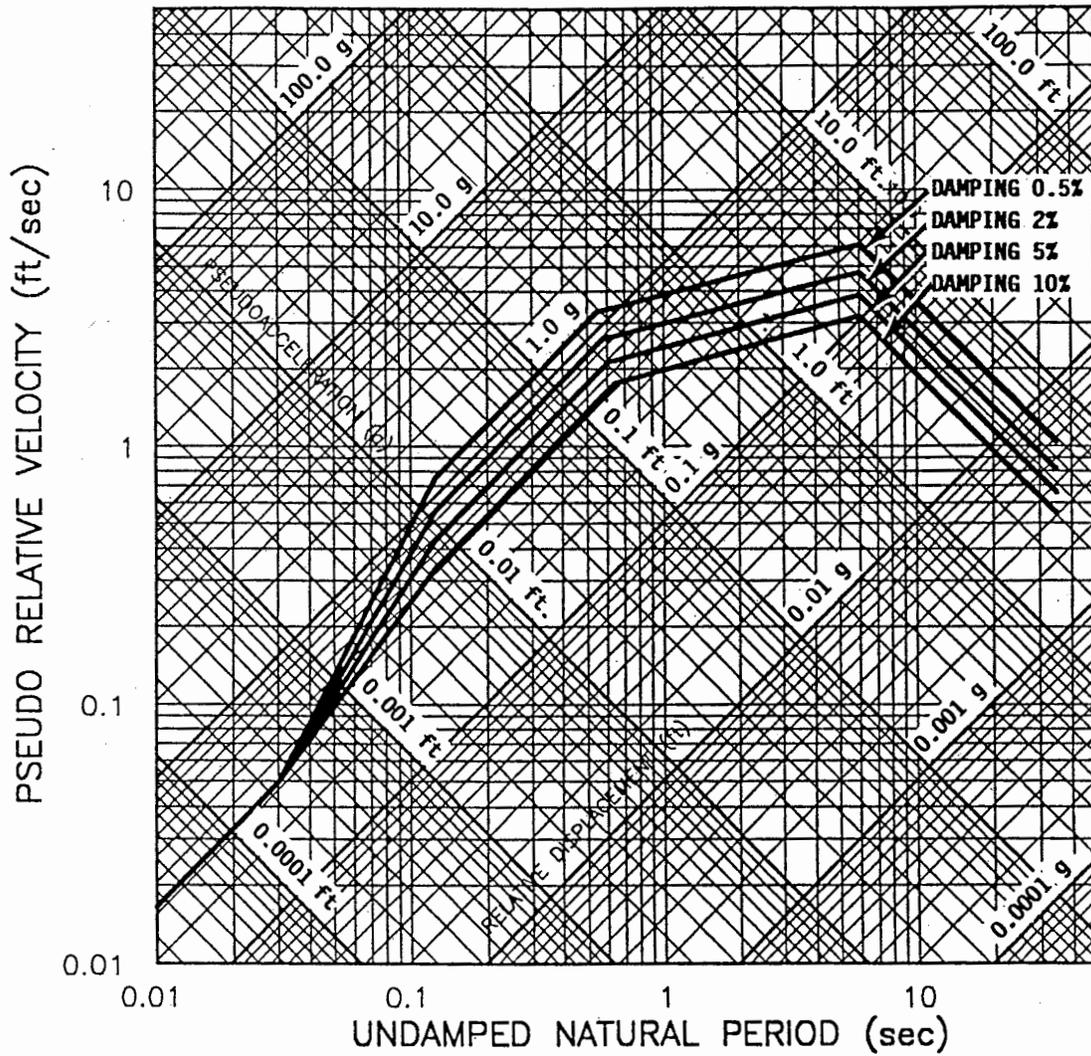


GEOBASE

SEISMIC RISK ANALYSIS RESULTS
-AVERAGE RETURN PERIOD-
Watts Towers Cultural Crescent
Los Angeles, California

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Figure A-7



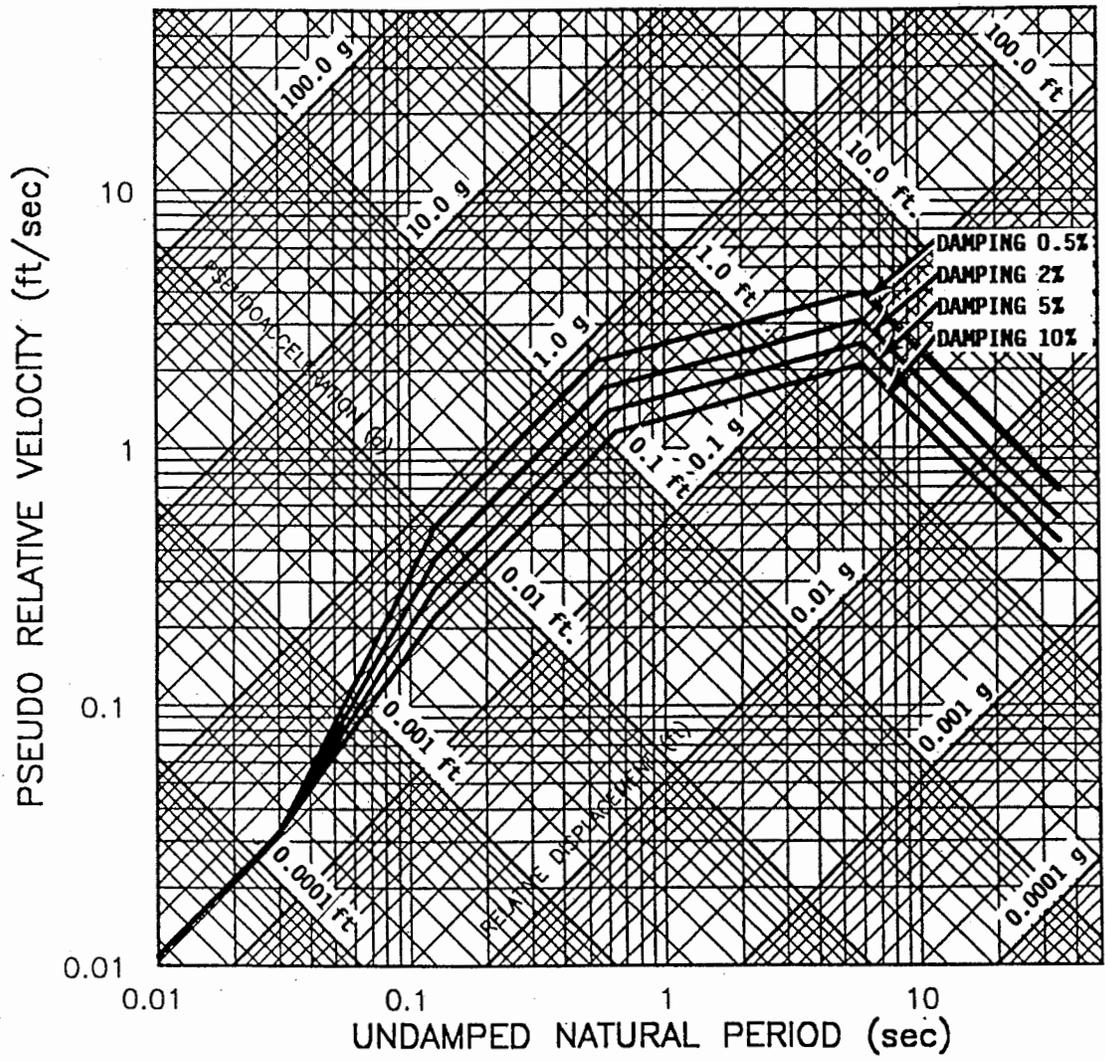
METHOD: NEWMARK & HALL (1982) - MEDIAN
 DAMPING: 0.5 2.0 5.0 10.0 0.0 0.0
 PEAK ACCELERATION: 0.3200

GEOBASE

HORIZONTAL DESIGN RESPONSE SPECTRA
 -ACCELERATION 0.32g-
 Watts Towers Cultural Crescent
 Los Angeles, California

P.279.02.00

Figure A-8



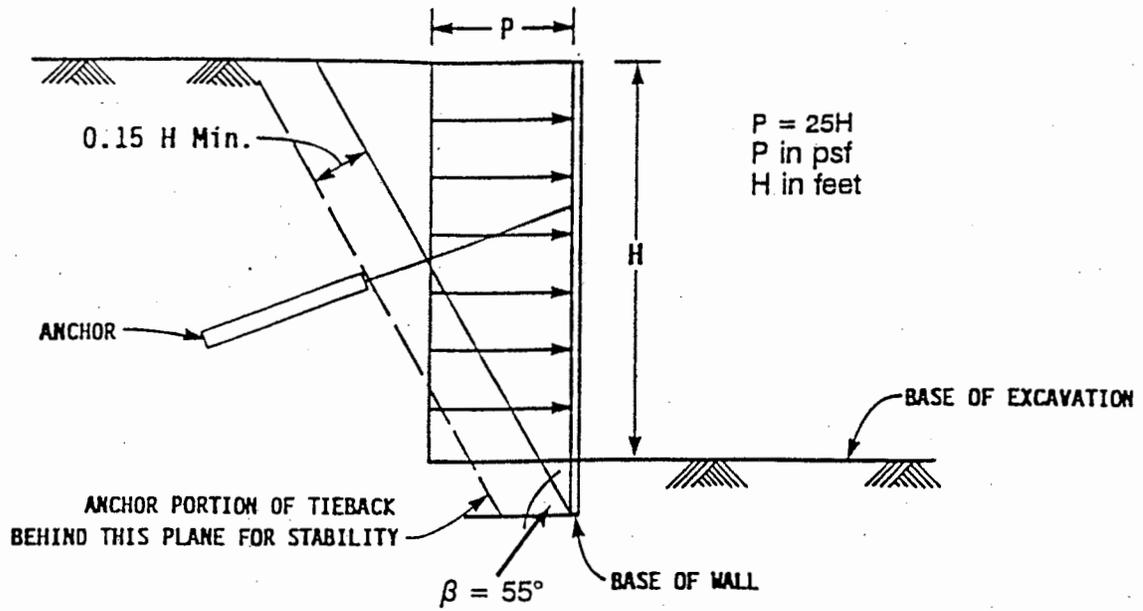
METHOD: NEWMARK & HALL (1982) - MEDIAN
 DAMPING: 0.5 2.0 5.0 10.0 0.0 0.0
 PEAK ACCELERATION: 0.2100

GEOBASE

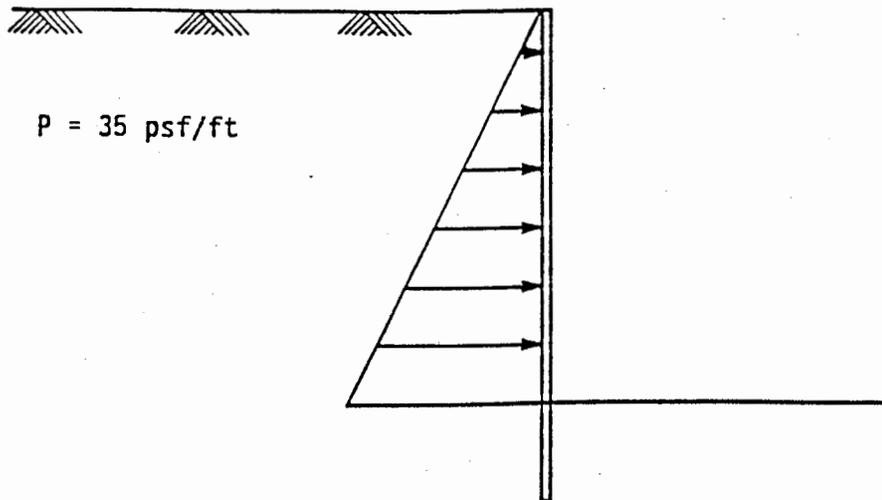
HORIZONTAL DESIGN RESPONSE SPECTRA
 -ACCELERATION 0.21g-
 Watts Towers Cultural Crescent
 Los Angeles, California

P.279.02.00

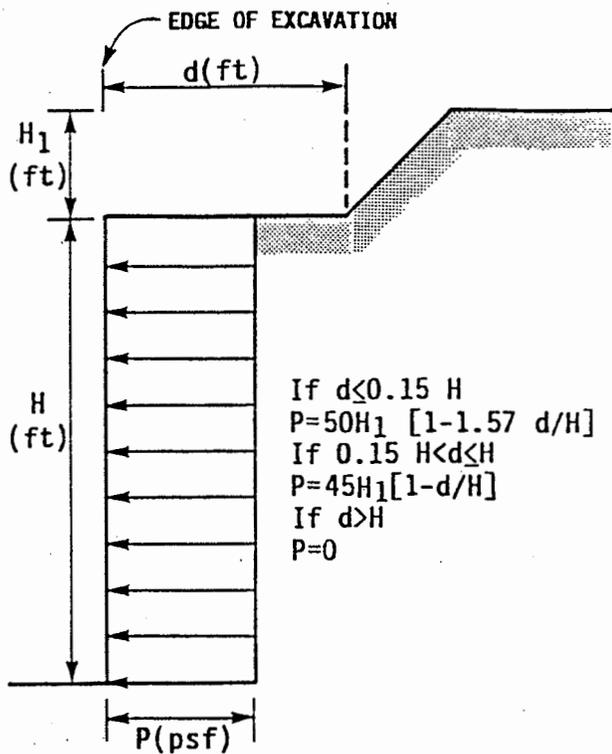
Figure A-9



TIEBACK SHORING



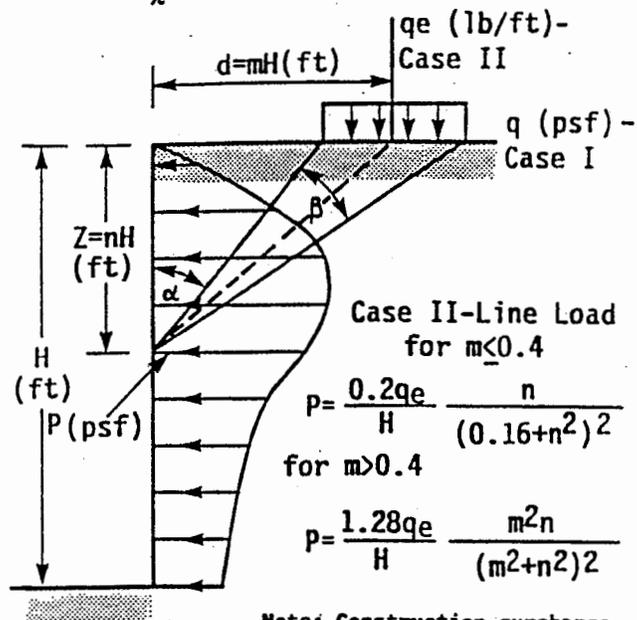
CANTILEVERED SHORING



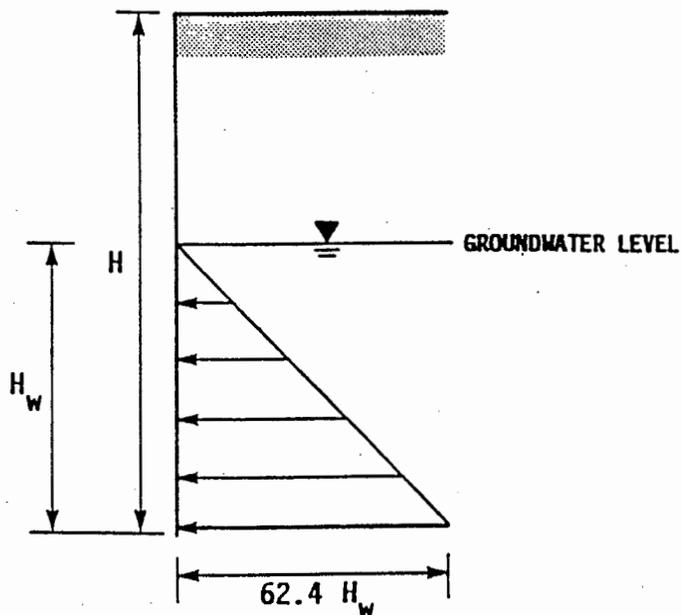
(a) SLOPED EXCAVATION SURCHARGE

Case I - Strip Load

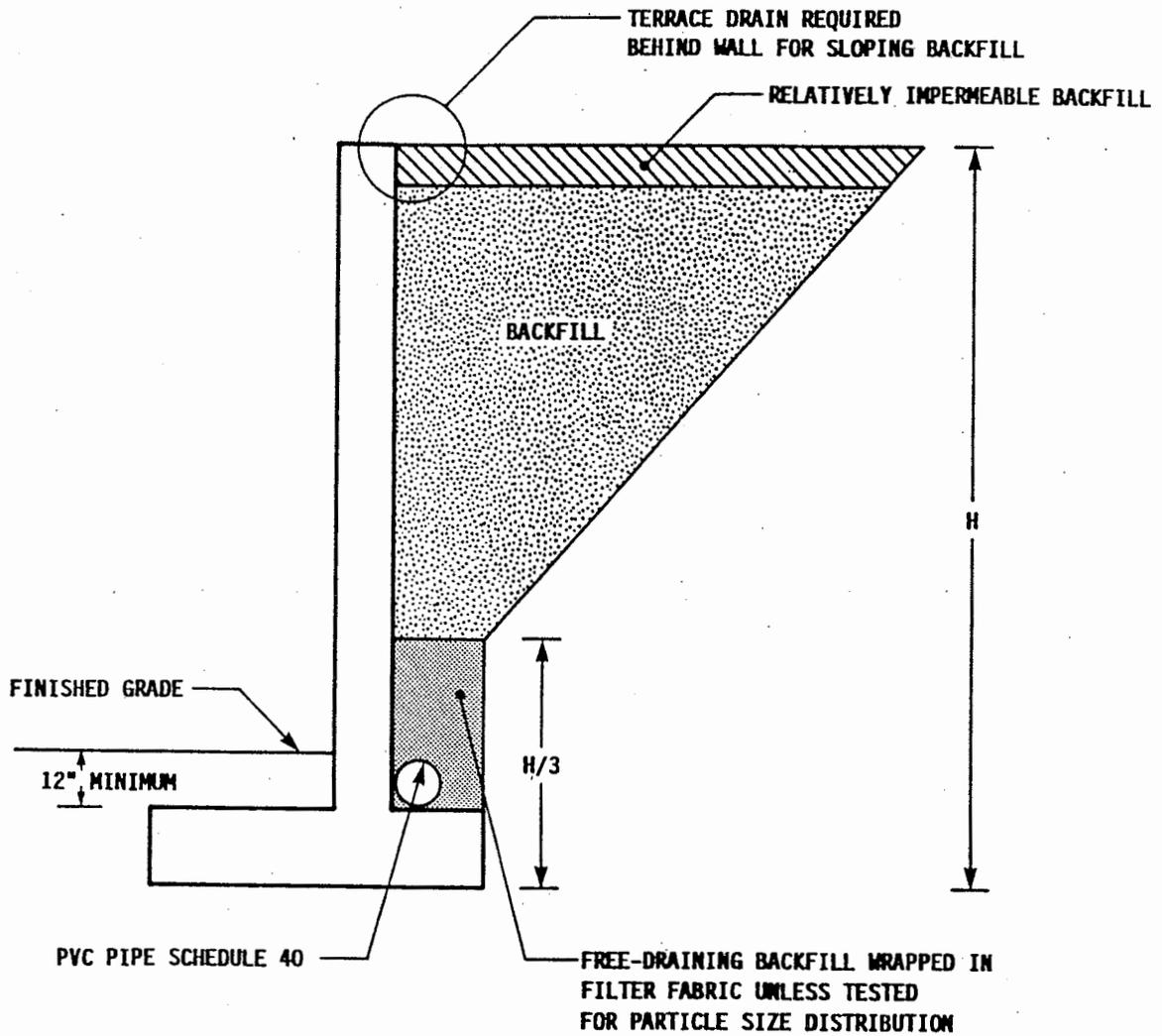
$$p = \frac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$$



(b) CONSTRUCTION AND TRAFFIC SURCHARGE



(c) HYDROSTATIC PRESSURE



GEOBASE

TYPICAL WALL BACKFILL DETAIL
 Watts Towers Cultural Crescent
 Los Angeles, California

P.279.02.00

Figure A-12

APPENDIX B

Figure B-1	Explanation of Terms and Symbols Used
Figure B-2	Log of Boring B-1
Figure B-3	Log of Boring B-2
Figure B-4	Log of Boring B-3
Figure B-5	Log of Boring B-4
Figure B-6	Log of Boring B-5
Figure B-7	Log of Boring B-6
Figure B-8	Log of Boring B-7
Figure B-9	Log of Boring B-8
Figure B-10	Log of Boring B-9
Figure B-11	Log of Boring B-10
Figure B-12	Log of Boring B-11
Figure B-13	Log of Boring TP-1
Figure B-14	Log of Boring TP-2
Figure B-15	Log of Boring TP-3
Figure B-16	Log of Boring TP-4
Figure B-17	Log of Boring TP-5
Figure B-18	Log of Boring TP-6

The terms and symbols used on the Log of Borings to summarize the results of the field investigation and subsequent laboratory testing are described in the following:

It should be noted that materials, boundaries, and conditions have been established only at the boring locations, and are not necessarily representative of subsurface conditions elsewhere across the site.

A. PARTICLE SIZE DEFINITION (ASTM D2487 and D422)

Boulder	-- larger than 12-inches	Sand, medium	-- No. 40 to No. 10 sieves
Cobble	-- 3-inches to 12-inches	Sand, fine	-- No. 200 to No. 40 sieves
Gravel, coarse	-- 3/4-inch to 3-inches	Silt	-- 5 μm to No. 200 sieves
Gravel, fine	-- No. 4 sieve to 3/4-inch	Clay	-- smaller than 5 μm
Sand, coarse	-- No. 10 to No. 4 sieve		

B. SOIL CLASSIFICATION

Soils and bedrock are classified and described according to their engineering properties and behavioral characteristics. The soil of each stratum is described using ASTM D2487 and D2488.

The following adjectives may be employed to define percentage ranges by weight of minor components:

trace	-- 1-10%
little	-- 10-20%
some	-- 20-35%
"and" or "y"	-- 35-50%

The following descriptive terms may be used for stratified soils:

parting	-- 0 to 1/16-in. thickness;
seam	-- 1/16 to 1/2-in. thickness;
layer	-- 1/2-in. to 12-in. thickness;
stratum	-- greater than 12-in. thickness.

C. SOIL DENSITY AND CONSISTENCY

The density of coarse grained soils and the consistency of fine grained soils are described on the basis of the Standard Penetration Test:

<u>COARSE GRAINED SOILS</u>		<u>FINE GRAINED SOILS</u>		
<u>Density</u>	<u>SPT Blows per foot</u>	<u>Estimated Consistency</u>	<u>SPT Blows per foot</u>	<u>Estimated Range of Unconfined Compressive Strength (tsf)</u>
very loose	less than 4	very soft	less than 2	less than 0.25
loose	4 to 10	soft	2 to 4	0.25 to 0.50
medium	10 to 30	firm (medium)	4 to 8	0.50 to 1.0
dense	30 to 50	stiff	8 to 15	1.0 to 2.0
very dense	over 50	very stiff	15 to 30	2.0 to 4.0
		hard	over 30	over 4.0

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EXPLANATION OF TERMS AND SYMBOLS

Figure B1

D: STANDARD PENETRATION TEST (SPT) -- D1586

The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, 2-inches outside diameter and 1 3/8-inches inside diameter, is driven eighteen (18) inches. The sampler is seated in the first six (6) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

E. SAMPLE TYPE

Thin walled tube SPT split spoon California modified sampler Disturbed No recovery Core

F. ABBREVIATION OF LABORATORY TEST DESIGNATIONS

C	Consolidation	PP	Pocket Penetrometer
CBR	California Bearing Ratio	PS	Particle Size
Ch	Water Soluble Chlorides	RV	R-Value
DS	Direct Shear	SE	Sand Equivalent
EI	Expansion Index	SG	Specific Gravity
ER	Electrical Resistivity	SO ₄	Water Soluble Sulfates
k	Permeability	TX	Triaxial Compression
MD	Moisture/Density Relationship	TV	Torvane Shear
O	Organic Content	U	Unconfined Compression
pH	pH		

G. STRATIFICATION LINES

The stratification lines indicated on the boring logs and profiles represent the approximate boundary between material types and the transition may be gradual.

GEOBASE, INC.

**EXPLANATION OF TERMS
AND SYMBOLS**

Figure B1

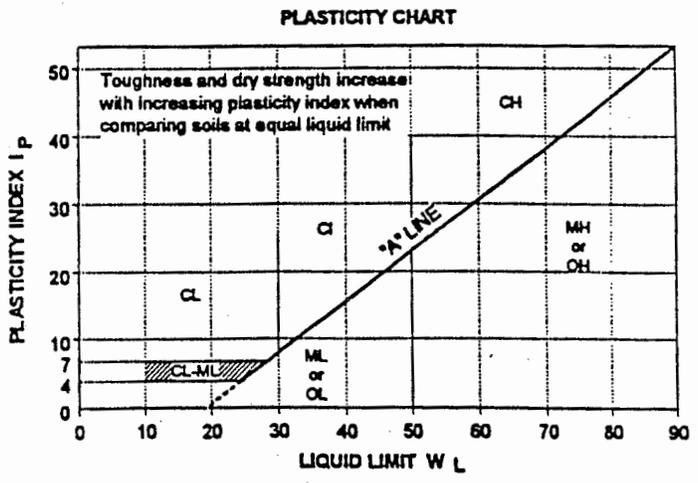
SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISION	GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA		
HIGHLY ORGANIC SOILS	PI		Peat and other highly organic soils	Strong color or odor and often fibrous texture		
COARSE-GRAINED SOILS (More than half by weight larger than No. 200 sieve size)	GRAVELS (More than half coarse fraction larger than No. 4 sieve size)	CLEAN GRAVELS	GW		Well-graded Gravels, Gravel-Sand mixtures (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
		DIRTY GRAVELS	GP		Poorly-graded Gravels and Gravel-Sand mixtures (<5% fines)	Not meeting all above requirements
			GM		Silty Gravels, Gravel-Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$
		GC		Clayey Gravels, Gravel-Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$	
	SANDS (More than half coarse fraction smaller than No. 4 sieve size)	CLEAN SANDS	SW		Well-graded Sands, Gravelly Sands (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
		DIRTY SANDS	SP		Poorly-graded Sands or Gravelly Sands (<5% fines)	Not meeting all above requirements
			SM		Silty Sands, Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$
			SC		Clayey Sands, Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$
FINE-GRAINED SOILS (More than half by weight passes No. 200 sieve size)	SILTS Below "A" line on plasticity chart: negligible organic content		ML		Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	$W_L < 50$
			MH		Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	$W_L > 50$
	CLAYS Above "A" line on plasticity chart: negligible organic content		CL		Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	$W_L < 30$
			CI		Inorganic Clays of medium plasticity, Silty Clays	$W_L > 30, < 50$
			CH		Inorganic Clays of high plasticity, fat Clays	$W_L > 50$
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL		Organic Silts and organic Silty Clays of low plasticity	$W_L < 50$
		OH		Organic Clays of high plasticity	$W_L > 50$	

The soil of each stratum is described using ASTM D2487 and D2488 modified slightly so that an inorganic clay of "medium plasticity" is recognized.

ADDITIONAL SOIL CLASSIFICATION

- Fill Soil
- Ss Sandstone
- Cs Claystone
- Ms Siltstone



GEOBASE, INC.

EXPLANATION OF TERMS AND SYMBOLS

Figure B1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	SAND (FILL) light brown, fine grained, silty, trace gravel, moist.	SM			Top of mound
	[Dotted pattern]	SAND light brown, fine grained, trace silt, moist.	SP			
5		SILT light brown, sandy, moist.	ML	[X]	● ▲	
10		... very stiff.			■ ●	
15		<ul style="list-style-type: none"> * End of boring at 11.5 feet. * Boring dry at completion of drilling. 				
20						
25						
30						
35						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-1
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-2

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

page 1 of 1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	<u>SAND (FILL)</u> dark brown, silty, moist.	SM	[Horizontal lines]		2 inches grass and top soil
5	[Dotted pattern]	<u>SAND</u> light brown, fine grained, iron stained, damp. ... medium dense.	SP	[Horizontal lines]		Bulk sample 0.5 - 5 feet
10	[Diagonal lines]	<u>CLAY</u> grayish brown, trace fine grained sand, moist.	CL	[Vertical lines]	●	
11.5	[Dotted pattern]	<u>SAND</u> light brown, fine grained, very silty, moist.	SM	[X pattern]	● ▲	
15		<ul style="list-style-type: none"> * End of boring at 11.5 feet. * Boring dry at completion of drilling. 				
20						
25						
30						
35						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-2
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-3

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	SAND (FILL) brown, fine grained, silty, trace gravel, moist.	SM	[Horizontal lines]		Top of mound
5	[Dotted pattern]	SAND light brown, fine grained, trace silt, damp to moist. ... medium dense.	SP	[Horizontal lines]		Bulk sample 0 - 5 feet
10	[Diagonal lines]	CLAY grayish brown, trace fine sand, moist. * End of boring at 11.5 feet. * Boring dry at completion of drilling.	CI	[Cross-hatch pattern]		
15						
20						
25						
30						
35						

GEOBASE, INC.	PROJECT	Watts Towers Cultural Crescent Los Angeles, California		BORING NO.	B- 3
	DEPTH TO WATER	▽	SURFACE ELEV. ± feet	LOGGED BY	WYY
	DEPTH TO SLOUGH	▲	DRILL CME-75	DATE LOGGED	12/12/97
Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.					FIGURE NO. B-4
					page 1 of 1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) — Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	SAND (FILL) dark brown, silty, trace gravel, moist.	SM	[Horizontal lines]		2 inches grass and top soil
	[Dotted pattern]	SAND light brown, fine to medium grained, damp to moist.	SP	[Vertical lines]		Bulk sample 0.5 - 5 feet
5	[Dotted pattern]	SAND brown, fine grained, very silty, moist.	SM	[Cross-hatch pattern]	● (at 5 ft)	
10	[Dotted pattern]	... light brown, silty, medium dense.		[Vertical lines]	● (at 10 ft), ■ (at 10 ft)	
11.5		<ul style="list-style-type: none"> • End of boring at 11.5 feet. • Boring dry at completion of drilling. 				
15						
20						
25						
30						
35						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-4
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-5

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated. page 1 of 1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
		SAND (FILL) brown, silty, trace gravel, moist.	SM		2 inches grass and top soil
		SAND light brown, fine grained, silty, damp to moist.	SM		
5					DS
10		... medium dense.		 	
15		<ul style="list-style-type: none"> • End of boring at 11.5 feet. • Boring dry at completion of drilling. 			
20					
25					
30					
35					

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-5
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-6

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

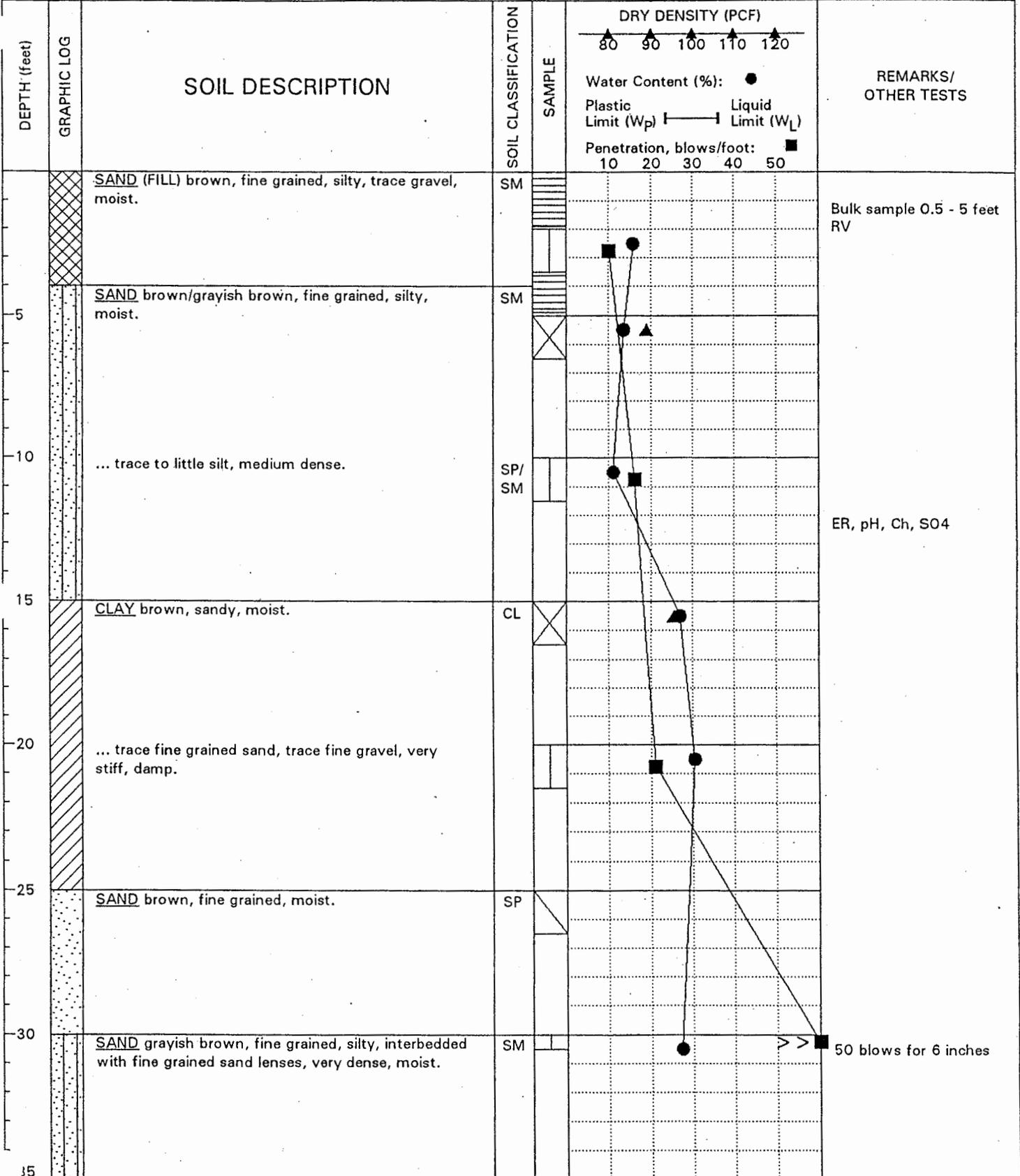
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	X	SAND (FILL) brown, silty, trace gravel, moist.	SM			2 inches grass and top soil
	.	SAND light brown, fine grained, trace to little silt, damp.	SM			Bulk sample 0.5 - 5 feet
5 medium dense.				
10	.			X	● ■ ● ▲	C
15		<ul style="list-style-type: none"> • End of boring at 11.5 feet. • Boring dry at completion of drilling. 				
20						
25						
30						
35						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B- 6
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-7

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE



GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-7
	DEPTH TO WATER	↓ SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	↑ DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02
			FIGURE NO. B-8

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	●	<u>SAND</u> light brown/gray, fine grained, moist.	SP	X	● ▲	
40	●	... gray, trace to little silt, medium dense.	SP/ SM	■	● ■	
45	■	<u>CLAY/SILT</u> , gray, interbedded with fine grained sand lenses, hard, damp.	CL/ ML	■	● ■	
50	●	<u>SAND</u> light/yellowish brown, fine to medium grained, dense, damp to moist.	SP	■	● ■	
55		* End of boring at 51.5 feet. * Trace of water at bottom of hole.				
60						
65						
70						

GEOBASE, INC.	PROJECT		Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-7
	DEPTH TO WATER	▽	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02
	DEPTH TO SLOUGH	▲	DRILL CME-75	DATE LOGGED 12/12/97	FIGURE NO. B-8

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) — Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	<u>SAND (FILL)</u> brown, silty, trace gravel, moist.	SM			2 inches grass and top soil
	[Dotted pattern]	<u>SAND</u> light brown, fine grained, trace silt, damp to moist. ... interbedded with dark brown clay lenses, medium dense.	SP			
5						
10	[Dotted pattern]	<u>SAND</u> light brown, fine grained, silty, moist. ... brown, medium dense.	SM	<input checked="" type="checkbox"/>		DS
15						
20	[Diagonal lines]	<u>CLAY</u> grayish brown, trace to little fine grained sand, damp to moist. ... gray/grayish green, trace fine grained sand, iron stained, very stiff, moist.	CL	<input checked="" type="checkbox"/>		
25						
30	[Diagonal lines]	... moist to very moist.		<input checked="" type="checkbox"/>		El (Mix of 25' and 30')
		* End of boring at 31.5 feet. * Trace of water at bottom of hole.				C pp = 1.5 TSF
35						

GEOBASE, INC.	PROJECT	Watts Towers Cultural Crescent Los Angeles, California		BORING NO.	B-8
	DEPTH TO WATER	▽	SURFACE ELEV. ± feet	LOGGED BY	WYY
	DEPTH TO SLOUGH	▲	DRILL CME-75	DATE LOGGED	12/12/97
				PROJECT NO.	P.279.02
				FIGURE NO.	B-9

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		<u>SAND (FILL)</u> brown, gravelly, little to some silt, red brick pieces, moist.	GM			Bulk sample 0 - 5 feet RV
5		<u>SAND</u> light brown, fine grained, trace silt. ... medium dense.	SP		● ■	
5		* End of boring at 6.5 feet. * Boring dry at completion of drilling.				
10						
15						
20						
25						
30						
5						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-9
	DEPTH TO WATER ∇	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH ∇	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02
			FIGURE NO. B-10

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	[Cross-hatch pattern]	<u>SAND (FILL)</u> brown, gravelly, some silt, damp to moist.	SM			
5	[Dotted pattern]	<u>SAND</u> light brown, fine grained, trace silt, damp. ... medium dense.	SP			
10	[Dotted pattern]	<u>SAND</u> brown, fine grained, silty, moist.	SM	[X]		ER, pH, Ch, SO4
15	[Diagonal lines /]	<u>CLAY</u> gray, some fine grained sand, very stiff, moist.	CL			
20	[Diagonal lines /]	... trace fine gravel.		[X]		PP=0.5 to 0.75 TSF
25	[Diagonal lines /]	... trace fine grained sand.				
30	[Diagonal lines /]	... grayish brown, trace fine grained sand, moist.	ML/CL	[X]		PP=2.25 to 2.5 TSF C
31.5		<ul style="list-style-type: none"> * End of boring at 31.5 feet. * Groundwater measured at 25.0 feet at completion of drilling. 				

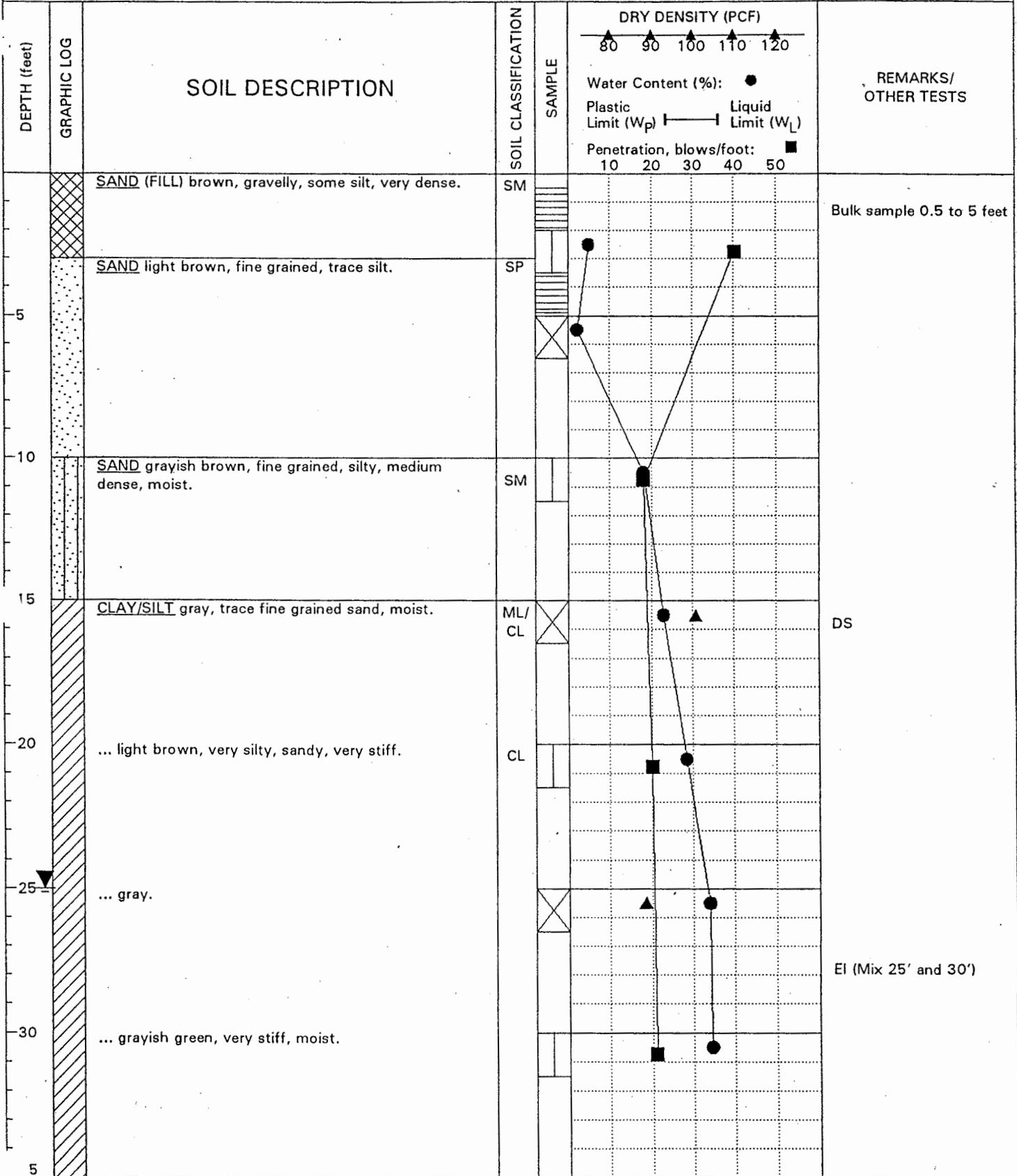
GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California			BORING NO. B-10	
	DEPTH TO WATER 25.0 ▼	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02	
	DEPTH TO SLOUGH ▲	DRILL CME-75	DATE LOGGED 12/12/97	FIGURE NO. B-11	

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

page 1 of 1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE



GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California			BORING NO. B-11	
	DEPTH TO WATER 25.0	▽	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02
	DEPTH TO SLOUGH	▲	DRILL CME-75	DATE LOGGED 12/12/97	FIGURE NO. B-12

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

page 1 of 2

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

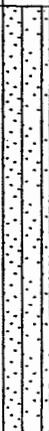
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
 trace fine grained sand, damp.	CL	X	▲	PP = 1.5 to 2.0 TSF
40 very stiff.	ML/CL	□	●	
45	...	<u>SAND</u> gray, fine grained, dense.	SP	□	●	
0 fine to medium grained, very dense, moist.		□	●	
		<ul style="list-style-type: none"> * End of boring at 51.0 feet. * Groundwater measured at 25.0 feet at completion of drilling. 			■	50 blows for 4 inches
55						
60						
65						

GEOBASE, INC.	PROJECT Watts Towers Cultural Crescent Los Angeles, California		BORING NO. B-11
	DEPTH TO WATER 25.0	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/12/97
			PROJECT NO. P.279.02 FIGURE NO. B-12

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF TEST PIT

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W _p) ——— Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
5		<p><u>SAND</u>(Fill) brown, gravelly with some silt, slightly moist.</p> <p>... boulders and concrete pieces between 6 inches and 18 inches.</p>	SM			
0		<ul style="list-style-type: none"> * Dimensions: 21' long (east-west direction) by 7' wide. * End of test pit at 3 feet. * Test pit dry at completion of excavation. 				

GEOBASE, INC.	PROJECT	Watts Towers Cultural Crescent Los Angeles, California		TEST PIT NO. TP- 1
	DEPTH TO WATER	∇	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	↑	DRILL BACKHOE	DATE LOGGED 12/12/97
Note: This log of TEST PIT should be evaluated in conjunction with the complete geotechnical report. This log of TEST PIT represents conditions observed at the specific TEST PIT location and at the date indicated.				page 1 of 1

LOG OF TEST PIT

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120	REMARKS/ OTHER TESTS
		<p><u>SAND (Fill)</u> brown, gravelly with some silt, slightly moist.</p>	SM			
		<p><u>SAND</u> brown, silty, fine grained.</p>	SM			
5		<ul style="list-style-type: none"> * Dimensions: 26' long (east-west direction) by 5' wide. * End of test pit at 4 feet. * Test pit dry at completion of excavation. 				

GEOBASE, INC.	PROJECT	Watts Towers Cultural Crescent Los Angeles, California		TEST PIT NO. TP- 2
	DEPTH TO WATER	▽ SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02
	DEPTH TO SLOUGH	▲ DRILL BACKHOE	DATE LOGGED 12/12/97	FIGURE NO. B-14

LOG OF TEST PIT

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) ▲ 80 ▲ 90 ▲ 100 ▲ 110 ▲ 120 Water Content (%): ● Plastic Limit (W _p) ────┬─── Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND(Fill) brown, gravelly with some silt, slightly moist. ... several boulders. ... Asphalt pieces	SM			
		SAND brown, silty, fine grained.	SM			
5		<ul style="list-style-type: none"> * Dimensions: 25' long (east-west direction) by 5' wide. * End of test pit at 4 feet. * Test pit dry at completion of excavation. 				

GEOBASE, INC.	PROJECT	Watts Towers Cultural Crescent Los Angeles, California		TEST PIT NO. TP- 3
	DEPTH TO WATER	▼	SURFACE ELEV. ± feet	LOGGED BY WYY
	DEPTH TO SLOUGH	▲	DRILL BACKHOE	DATE LOGGED 12/12/97
				PROJECT NO. P.279.02
				FIGURE NO. B-15

Note: This log of TEST PIT should be evaluated in conjunction with the complete geotechnical report. This log of TEST PIT represents conditions observed at the specific TEST PIT location and at the date indicated.

LOG OF TEST PIT

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120	REMARKS/ OTHER TESTS
	[Cross-hatched pattern]	GRAVEL(Fill) brown, some silt.	GM		Water Content (%): ● Plastic Limit (W _p) ----- Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	
	[Dotted pattern]	... abandoned pipe.				
	[Horizontal line pattern]	SAND brown, silty, fine grained.	SM	●		
5		* Dimensions: 20' long (east-west direction) by 7' wide. * End of test pit at 4 feet. * Test pit dry at completion of excavation.				
0						

GEOBASE, INC.	PROJECT		Watts Towers Cultural Crescent Los Angeles, California		TEST PIT NO. TP- 5
	DEPTH TO WATER	▽	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02
	DEPTH TO SLOUGH	↑	DRILL BACKHOE	DATE LOGGED 12/12/97	FIGURE NO. B-17

Note: This log of TEST PIT should be evaluated in conjunction with the complete geotechnical report. This log of TEST PIT represents conditions observed at the specific TEST PIT location and at the date indicated.

page 1 of 1

LOG OF TEST PIT

SAMPLE TYPE: THIN WALLED TUBE SPT SPLIT SPOON CALIFORNIA MODIFIED SAMPLER DISTURBED NO RECOVERY CORE

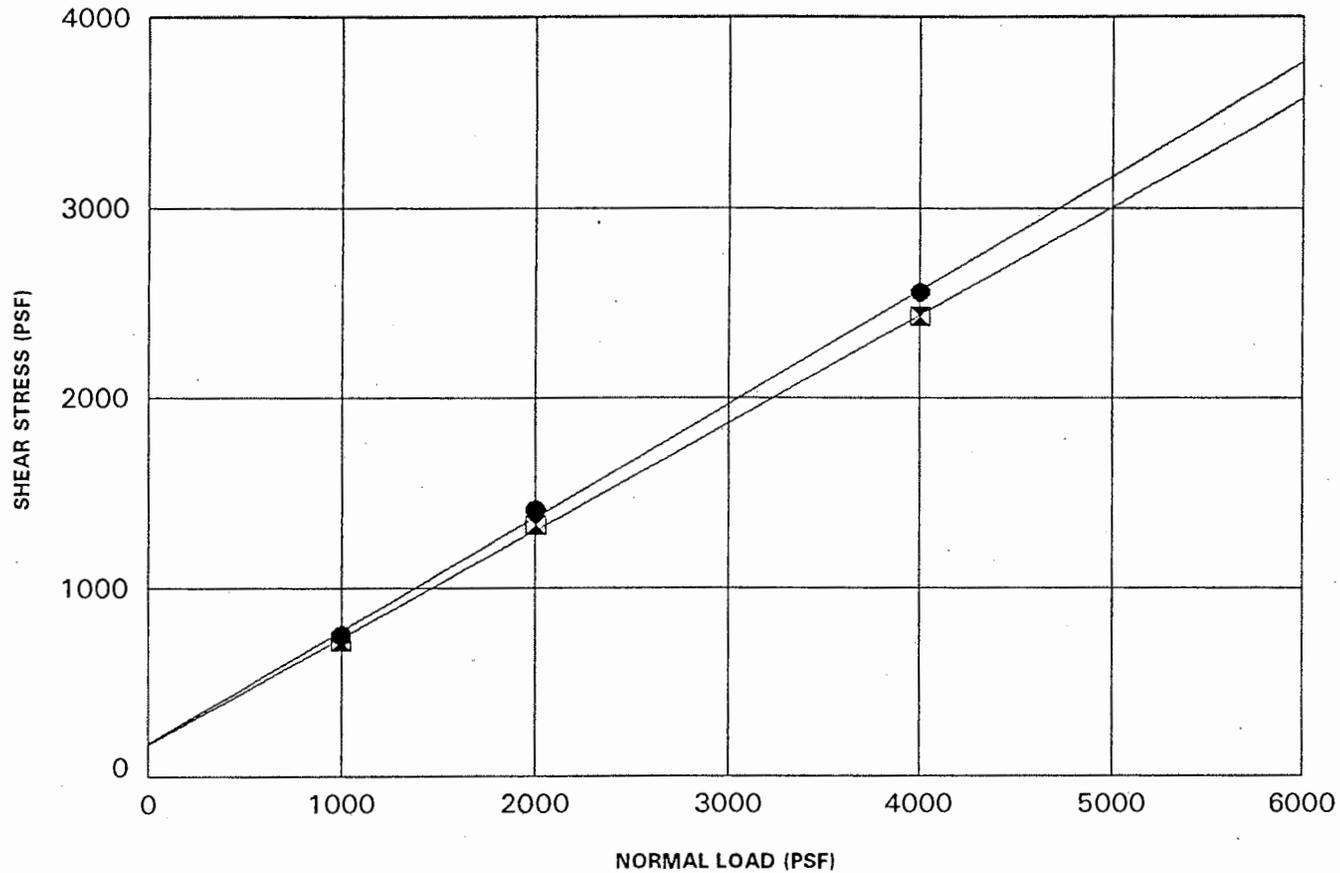
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120	REMARKS/ OTHER TESTS
	▨	GRAVEL(Fill) brown, some sand and silt.	GM		Water Content (%): ● Plastic Limit (W _p) ----- Liquid Limit (W _L) Penetration, blows/foot: ■ 10 20 30 40 50	
	▩	SAND brown, silty, slightly moist.	SM			
	▬	... PVC pipe (active line).		▬	●	
5		<ul style="list-style-type: none"> * Dimensions: 20' long (east-west direction) by 8' wide. * End of test pit at 4 feet. * Test pit dry at completion of excavation. 				

GEOBASE, INC.	PROJECT		Watts Towers Cultural Crescent Los Angeles, California		TEST PIT NO. TP- 6
	DEPTH TO WATER	▼	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.279.02
	DEPTH TO SLOUGH	▲	DRILL BACKHOE	DATE LOGGED 12/12/97	FIGURE NO. B-18

Note: This log of TEST PIT should be evaluated in conjunction with the complete geotechnical report. This log of TEST PIT represents conditions observed at the specific TEST PIT location and at the date indicated.

APPENDIX C

- Figure C-1 Summary of Laboratory Test Results
- Figure C-2 Direct Shear Test Results
- Figure C-3 Direct Shear Test Results
- Figure C-4 Direct Shear Test Results
- Figure C-5 Consolidation Test Results
- Figure C-6 Consolidation Test Results
- Figure C-7 Consolidation Test Results
- Figure C-8 Expansion Potential, Corrosivity Series and Resistance ("R") Value Test Results



SAMPLE DESCRIPTION: SAND, light brown, silty

BORING NO.: B- 5

DEPTH INTERVAL (ft): 5.0 - 6.5

NOTES:

HEIGHT (in): 1.0

AREA (sq in): 4.58

STRAIN RATE (in/min): 0.005

DRY DENSITY (pcf): 88.5

INITIAL MOISTURE (%): 17

FINAL MOISTURE (%): 36

COHESION (psf)

FRICTION ANGLE (deg)

PEAK

●

174

31

ULTIMATE

⊠

166

30

GEOBASE

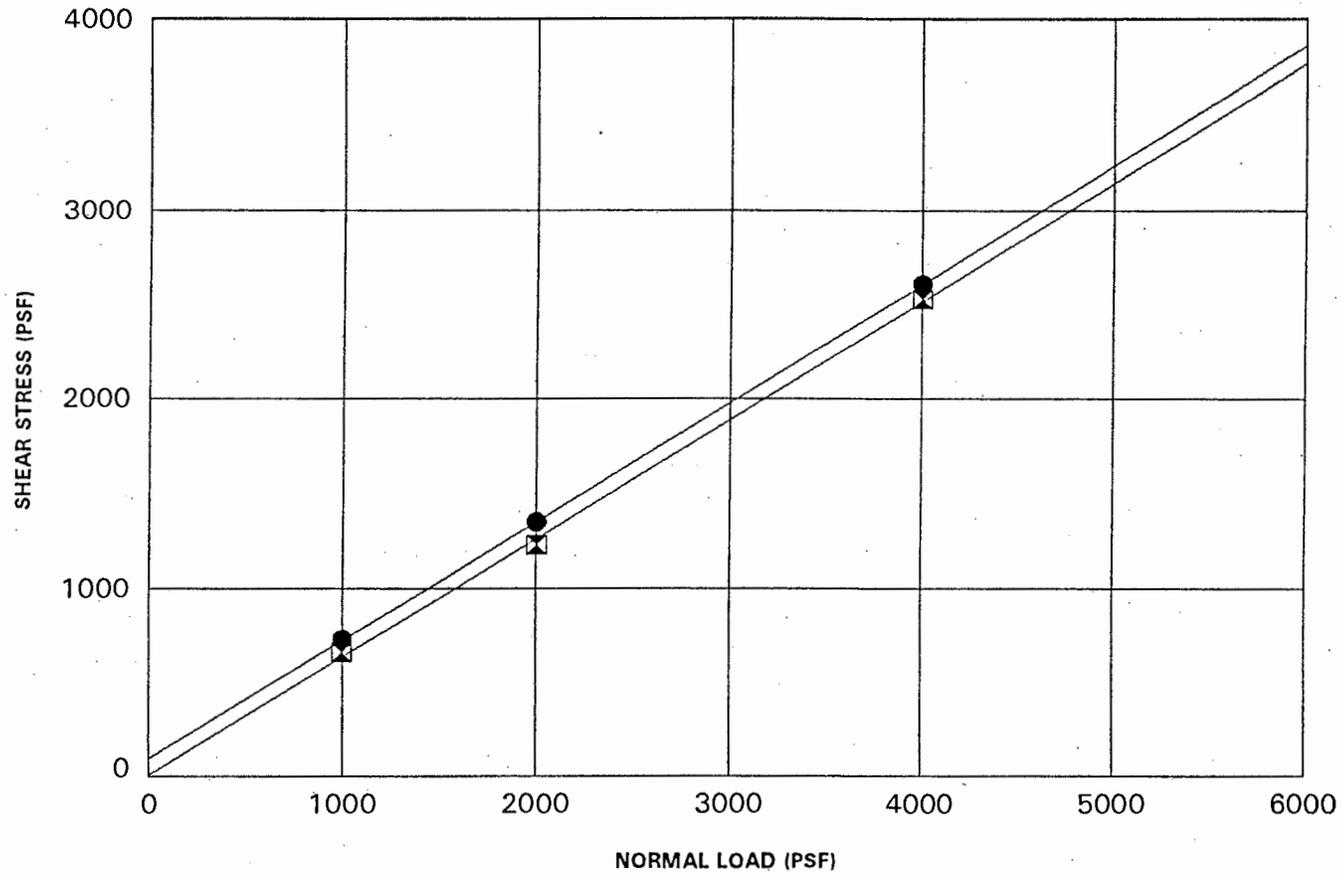
Direct Shear Test Results

Watts Towers Cultural Crescent

Los Angeles, California

P.279.02.00

Figure C-2



SAMPLE DESCRIPTION: SAND, brown, silty

BORING NO.: B- 8

DEPTH INTERVAL (ft): 10.0- 11.0

NOTES:

HEIGHT (in): 1.0

AREA (sq in): 4.58

STRAIN RATE (in/min): 0.005

DRY DENSITY (pcf): 89.7

INITIAL MOISTURE (%): 18

FINAL MOISTURE (%): 30

COHESION (psf)

FRICTION ANGLE (deg)

PEAK

●

97

32

ULTIMATE

⊠

6

32

GEOBASE

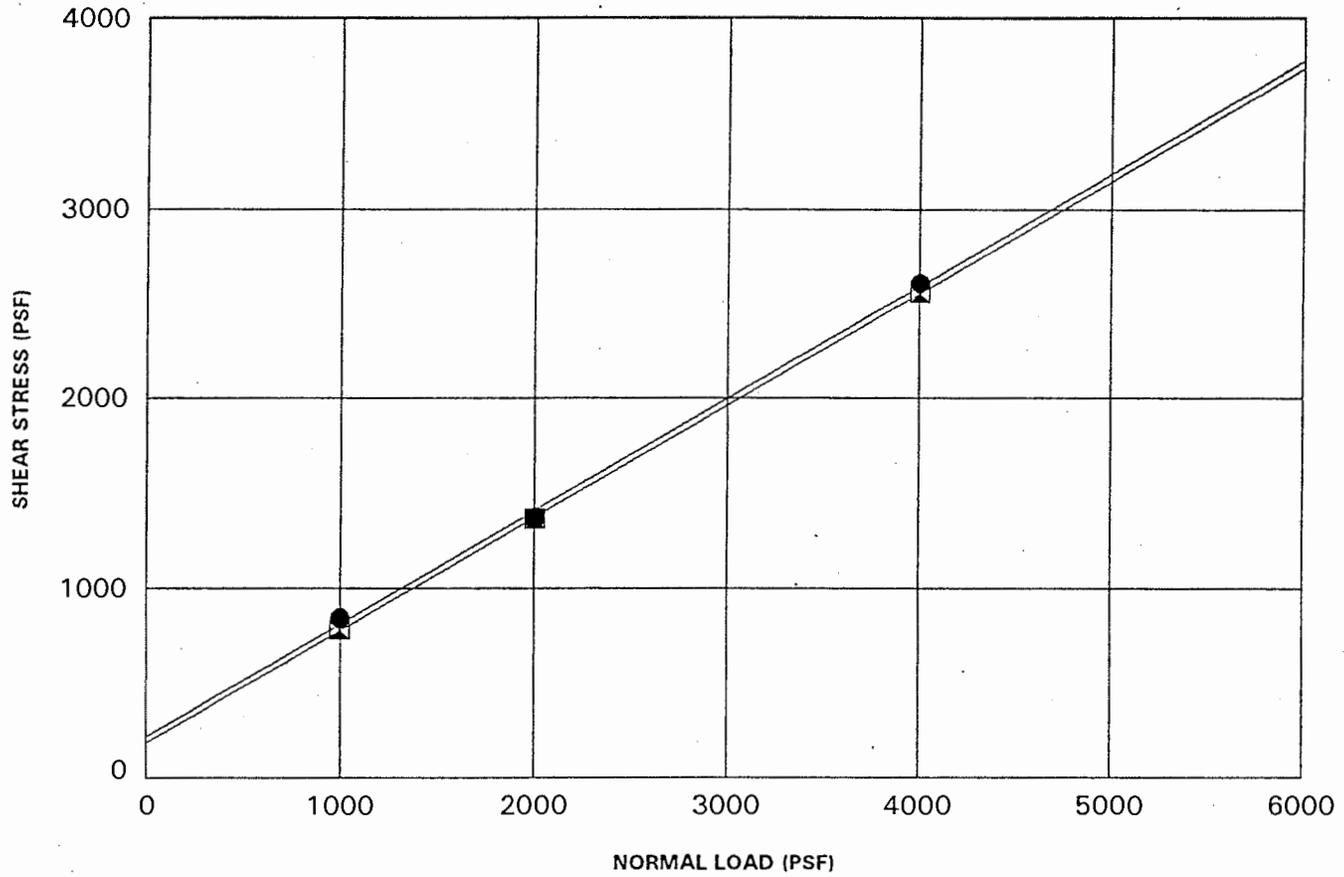
Direct Shear Test Results

Watts Towers Cultural Crescent

Los Angeles, California

P.279.02.00

Figure C-3



SAMPLE DESCRIPTION: CLAY/SILT, grey, little sand

HEIGHT (in): 1.0

DRY DENSITY (pcf): 99.9

PEAK ●

ULTIMATE ☒

BORING NO.: B-11

AREA (sq in): 4.58

INITIAL MOISTURE (%): 25

COHESION (psf)

223

188

DEPTH INTERVAL (ft): 15.0- 16.0

STRAIN RATE (in/min): 0.002

FINAL MOISTURE (%): 25

FRICTION ANGLE (deg)

31

31

NOTES:

GEOBASE

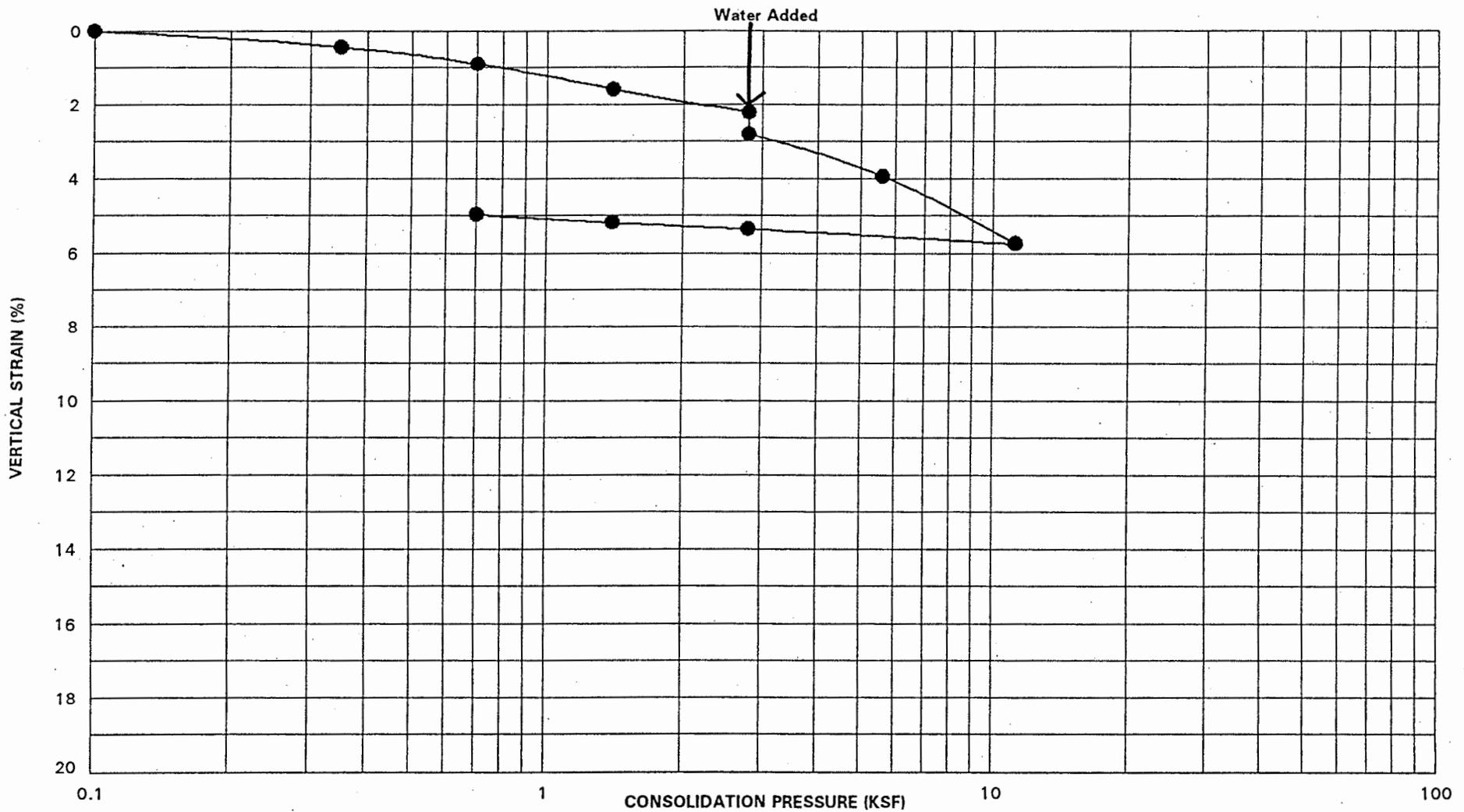
Direct Shear Test Results

Watts Towers Cultural Crescent

Los Angeles, California

P.279.02.00

Figure C-4



BORING NO.: B- 6
 DEPTH (ft): 10.0
 SAMPLE DESCRIPTION: SAND, brown, little silt

GEOBASE

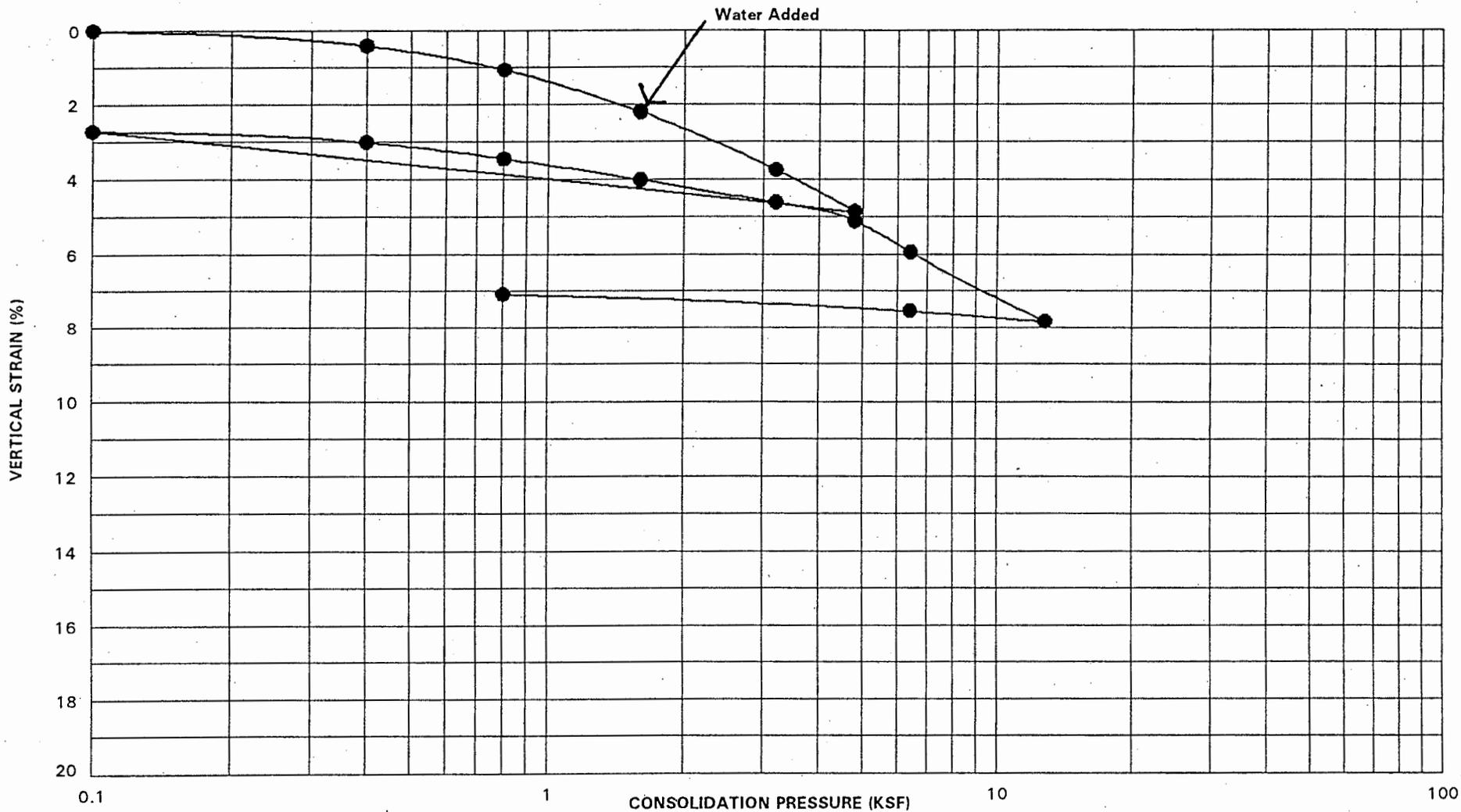
Consolidation Test Results

Watts Towers Cultural Crescent

Los Angeles, California

P.279.02.00

Figure C-5



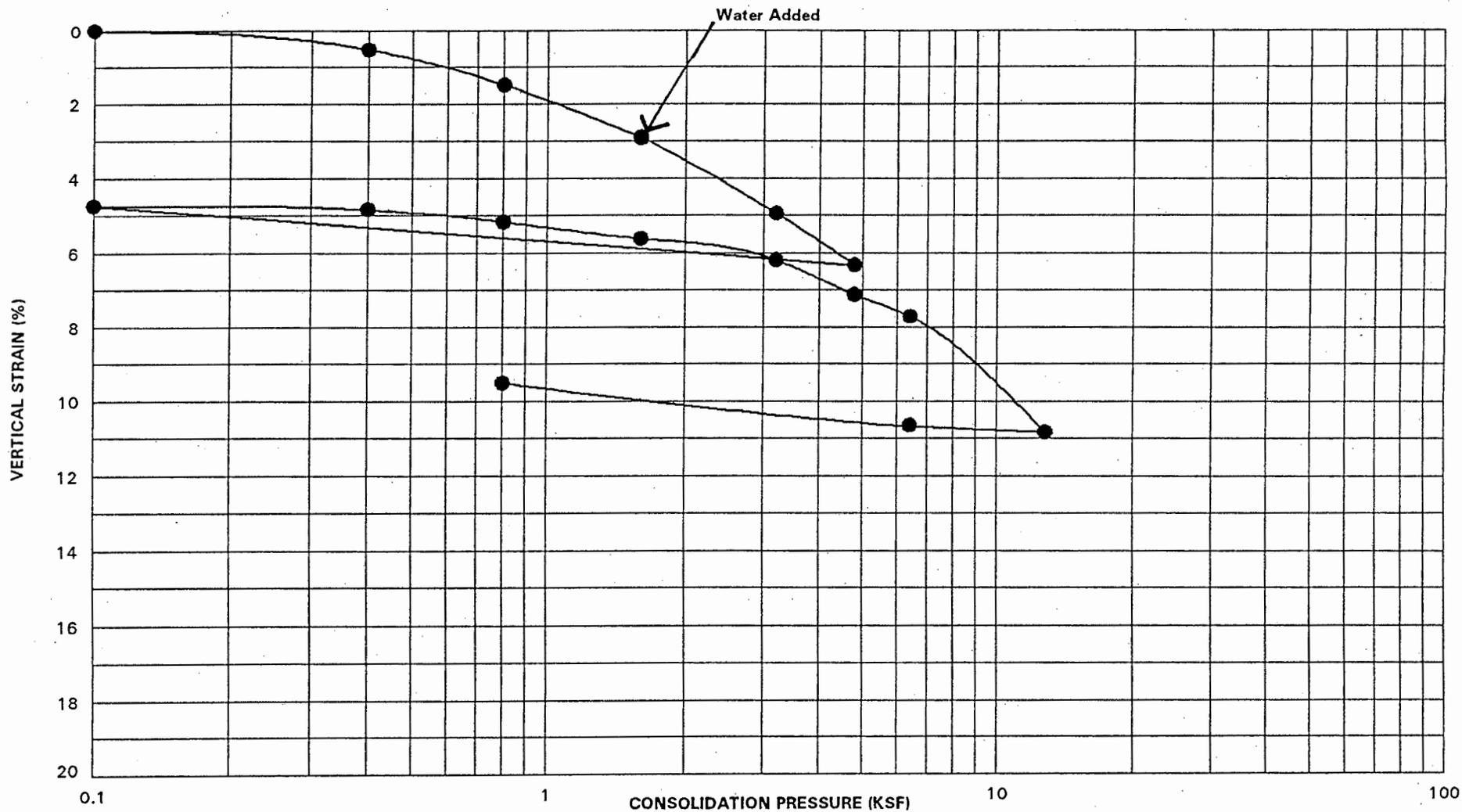
BORING NO.: B- 8
 DEPTH (ft): 30.0
 SAMPLE DESCRIPTION: CLAY, greyish brown, very silty

GEOBASE

Consolidation Test Results
 Watts Towers Cultural Crescent
 Los Angeles, California

P.279.02.00

Figure C-6



BORING NO.: B-10

DEPTH (ft): 30.0

SAMPLE DESCRIPTION: CLAY/SILT, brown

GEOBASE

Consolidation Test Results

Watts Towers Cultural Crescent

Los Angeles, California

P.279.02.00

Figure C-7

**EXPANSION POTENTIAL
U.B.C. No. 29-2**

SOIL SAMPLE LOCATION (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-8 at 25.0 to 30.0	42	Low
B-11 at 25.0-30.0	42	Low

**WATER SOLUBLE SULFATES
CAL. 417-A**

SOIL SAMPLE LOCATION (FEET)	SOLUBLE SULFATES PPM	POTENTIAL FOR ATTACK ON CONCRETE
B-7 AT 15.0	144	Low
B-10 AT 10.0 TO 15.0	132	Low

CORROSIVITY SERIES TEST

SAMPLE LOCATION (feet)	pH (Cal. 747)	SOLUBLE CHLORIDES PPM	ELEC. RESISTIVITY (CAL.643) (OHM-CM)	POTENTIAL FOR ATTACK ON STEEL (SENATOROFF)
B-7 at 15.0	9.3	74	2336	Moderate
B-10 at 10.0 to 15.0	9.9	80	3066	Moderate

**RESISTANCE ("R") VALUE
CAL. 301**

SAMPLE LOCATION (feet)	R-VALUE BY EXUDATION
B-7 at 0 to 5.0	46
B-9 at 0.5 to 5.0	72