Appendix E

Geotechnical Engineering Investigation

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File No. 18568-S

Howard Poyourow Development & Construction
1772 Palisades Drive
Pacific Palisades, California 90272

Attention: Howard Poyourow

Subject: Geotechnical Engineering Investigation
Proposed Mixed Use Development
201 - 224 South San Gabriel Boulevard, San Gabriel, California

Dear Mr. Poyourow:

This letter transmits the Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, and foundations. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependant upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED USE DEVELOPMENT
201 - 224 SOUTH SAN GABRIEL BOULEVARD
SAN GABRIEL, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to evaluate the nature of the soils underlying the site, to ascertain their engineering properties, and to provide recommendations for site preparation, grading, foundation design, retaining wall design, expansive soils, resistance to lateral loading, floor slabs, temporary excavations, and shoring.

This investigation included excavation of eleven exploratory excavations, obtaining representative samples, laboratory testing, engineering analysis, review of pertinent geotechnical literature and the preparation of this report. The exploration locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory tests are shown in the Appendix of this report.

INTENT

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The proposed development will consist of a four story, mixed use structure over a single level subterranean parking garage. It is the understanding of this firm that the garage finished floor level will be approximately 12 to 14 feet deep.

Typical column loads are expected to be approximately 500 to 800 kips. Wall loads are expected to be approximately 5 to 8 kips per foot. Grading on the site will consist of excavations for the proposed subterranean garage, and wall backfilling.

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

SITE CONDITIONS

The subject property is located at 201 - 224 South San Gabriel Boulevard in the City of San Gabriel, California. At the time of exploration, the site was occupied by miscellaneous commercial structures, paved parking, and the Rubio Wash. The wash is a roughly "U" shaped drainage channel which traverses the site from the northwest to the southeast. The wash is approximately 7 to 10 feet in depth and is constructed of concrete.

The subject site is bounded to the north by Live Oak Street, to the south by residential and commercial structures, to the east by South San Gabriel Boulevard, and to the west by Pine Street. No basements are known to exist adjacent to the proposed structure.

At the time this report was prepared, topographic information was not available. In general, the site slopes very gently down from the northwest to the southeast, with an estimated total relief on the order of 10 feet. Drainage is by sheetflow along the existing contours. Vegetation consists of trees and shrubs located in planters.

EXPLORATION AND TESTING

The site was explored on February 12, 13, and 19, 2004, by excavating eight exploratory borings and three exploratory test pits. The borings were excavated to depths between 20 and 80 feet below the existing site grade. The borings were excavated with the aid of a truck-mounted, hollow-stem auger drill rig, and were approximately 8 inches in diameter. The test pits were excavated with hand labor, and were approximately 2 feet square. The boring and test pit locations are shown on the Plot Plan, and the soils encountered are logged on the enclosed Plates A-1 through A-11 Samples of the soils encountered in the excavations were obtained and transported to the laboratory. The results of the laboratory tests, along with a description of the exploration and laboratory test procedures used are given in the Appendix.

EARTH MATERIALS

Fill Material

Between 10 and 15 feet of fill was encountered in the majority of the exploratory excavations. Shallower fill depths of 1, 5, and 8 feet were encountered in Test Pit TP1, Boring B5, and Boring B2, respectively. The fill material consists of silty sands and sands which are light brown to dark brown in color, moist, medium dense to dense, fine to coarse grained, and contain varying amounts of gravel and debris.

Native Soils

The underlying native soils predominantly consist of dense to very dense silty sands and sands, with occasional lenses of sandy silts. The native soils were found to be light brown to dark brown in color, moist, fine to coarse grained, and contain varying amounts of gravel and cobbles.

The native soils consist predominantly of detrital sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual boring and test pit logs.

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

GROUNDWATER AND CAVING

Groundwater was not encountered during exploration, which was conducted to a maximum depth of 80 feet. The historic high groundwater level at the subject site was greater than 100 feet below the ground surface, according to the Seismic Hazard Evaluation of the El Monte 7.5 Minute Quadrangle, Los Angeles County, California, by the California Division of Mines and Geology, Open File Report 98-15 (Loyd, 1998).

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

Caving could not be observed in the borings conducted with the drilling machine, because the boreholes were cased with hollow-stem augers, and caving was not possible. Some caving was observed in the medium dense sands encountered between depths of five and eight feet in the test pits.

REGIONAL GEOLOGY AND SEISMICITY

REGIONAL SETTING

The subject site is located north of the Peninsular Ranges Geomorphic Province and within the Transverse Ranges Geomorphic Province. The Peninsular Ranges are dominated by northwest-trending, strike-slip faults. The Transverse Ranges are dominated by east-west trending, reverse and thrust faults.

Tectonics of this region are controlled by the relative motion of the Pacific and North American crustal plates. The east-west trending structure of the Transverse Ranges province is believed to be a consequence of compression between the Pacific and North American plates and rotation of the province around the "Big Bend" in the San Andreas fault system which is north of the Tejon Pass.

FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years

(Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Seismic sources other than faults with known surface expression are known as "buried thrust faults". These faults are not exposed at the surface of the earth. They are typically broadly defined based on the analysis of seismic wave recordings of several hundreds of small earthquakes in the southern California area.

Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude, is not yet well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be totally precluded.

Fault Rupture-Earthquake Fault Zones

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

The subject site is not located within a State of California Earthquake Fault Zone, and no other known surface fault traces cross the subject site. Therefore, in the opinion of this firm, the possibility of surface fault rupture affecting the subject site should be considered remote.

Fault Locations

The computer program EQFAULT (Blake, 2000), was utilized to determine the location of faults within 60 miles of the subject site. This program utilizes the digitized CDMG fault location database. This data is presented on the attached Table I. The closest fault to the site according to EQFAULT (Blake, 2000), is the Raymond Fault, which is located 3.4 miles from the subject site. The distances calculated by this program come from digitized fault location data and may be slightly different from those measured from the referenced geologic maps. Other faults within 10 miles of the site according to EQFAULT (Blake, 2000) include the Verdugo Fault, located approximately 4.8 miles from the site, the Sierra Madre Fault, located approximately 7.5 miles from the site, the Clamshell-Sawpit Fault, located approximately 8.3 miles from the site, the Hollywood Fault, located approximately 8.5

miles from the site, the Elysian Park Thrust Fault, located approximately 9.0 miles from the subject site, and the Whitter Fault, located approximately 9.2 miles from the subject site.

HISTORIC SEISMICITY

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 within a radius of 60 miles of the site are shown on the attached Table II and Figure II. The computer program EQSEARCH (Blake, 2000) was used to compile this data. The historic seismic record indicates that 66 earthquakes of magnitude 5.0 and greater have occurred within 60 miles of the site between the years 1800 and 2004. Larger, more distant earthquakes, such as the 1857 Fort Tejon earthquake on the San Andreas Fault may have also affected the site.

GROUND MOTION

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

Deterministic Method

Table I in the Appendix presents the deterministic site parameters, and was obtained utilizing the computer program EQFAULT (Blake, 2000). This program utilizes a "maximum" earthquake for each fault or fault segment. This "maximum" earthquake assigned to a fault is derived from formulas which correlate the length of the fault trace to the theoretical maximum magnitude earthquake. The "maximum" earthquake is the theoretical maximum event which could occur along a particular fault or fault segment. This "maximum" earthquake is also sometimes referred to as the "maximum credible" earthquake.

The ground motions resulting from this "maximum" earthquake were attenuated to the site utilizing the attenuation relation of Campbell and Bozorgnia (1997 Rev.) - Alluvium. The resulting peak horizontal accelerations are shown on Table I in the Appendix. These values are the mean plus one standard deviation.

Using the deterministic analysis, the "maximum" earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 6.5 event on the Raymond Fault. Such an event would be expected to generate a peak horizontal acceleration at the site on the order of 0.85g. Again, this value is the mean plus one standard deviation.

Probabilistic Method

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

For this study, 38 regional faults and fault segments were used. These 38 are within the specified search radius of 60 miles from the site. The typical ground motions used for design are those with 10 and 50 percent probability of exceedence in a 50 year period.

The computer program FRISKSP (Blake, 2000) was utilized to make these calculations, utilizing the attenuation relation of Campbell and Bozorgnia (1997 Rev.) - Alluvium. The ground motions with 10 and 50 percent probability of exceedence in a 50 year period at the subject site are 0.56g and 0.27g respectively. These values are one standard deviation above the mean.

SECONDARY SEISMIC EFFECTS

In addition to possible strong ground motions at the site, other secondary effects of a strong nearby earthquake were considered. These include liquefaction, landsliding, flooding and earthquake-induced settlement.

Liquefaction

Liquefaction involves a sudden loss in strength of a saturated, cohesionless soil which is caused by shock or strain and results in temporary transformation of the soil to a fluid mass. The surface effects of liquefaction typically take the form of sand boils, differential ground settlement, or lateral spreading.

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

Groundwater was not encountered to the total depth of exploration, 80 feet below the existing site grade. In addition, the historic high groundwater level for the vicinity of the subject site was greater than 100 feet below the ground surface, according to the Seismic Hazard Evaluation of the El Monte 7.5-Minute Quadrangle, Los Angeles County, California, by the California Division of Mines and Geology, Open File Report 98-15 (Loyd, 1998). The site is not located in a State Seismic Hazard Zone for liquefaction. In addition, the soils underlying the site were found to be of a dense and consolidated nature. Therefore, the subject site would not be considered prone to liquefaction.

Landsliding

The probability of seismically-induced landslides affecting the subject development is considered to be remote, due to the relatively flat nature of the site and surrounding areas.

Earthquake-Induced Flooding

The subject site is high enough and far enough from the ocean and any lakes to preclude potential flooding from a tsunami or seiche. In addition, review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton 1990), indicates that the site does not lie within the potential inundation boundaries of any dam or reservoir that may fail during a seismic event.

Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. The dense to very dense native soils underlying the proposed subterranean garage level would not be considered prone to significant dynamic settlement.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration and laboratory testing, it is the finding of this firm that construction of the proposed mixed use development is feasible from a geotechnical engineering standpoint, provided the advice and recommendations contained in this report are made a part of the development plans and are implemented during construction.

Conventional foundations bearing in the dense to very dense native soils found near the proposed subterranean garage level may be utilized for foundation support. It is anticipated that excavations for the proposed subterranean level will be on the order of 12 to 14 feet below the existing site grade and will remove most of the existing fill materials. In areas where fill material is not completely removed by the proposed subterranean garage excavation, footings should be deepend to bear in dense native soils. Any existing fill soils not removed during excavation of the subterranean level, shall be removed and recompacted for slab support.

Due to the depth of the proposed excavations, the presence of the Rubio Wash structure, and the proximity of the property lines, it is anticipated that excavations for the proposed subterranean garage will require shoring to provide a stable excavation. Drilled, cast-in-place soldier piles are recommended for shoring.

It is the understanding of this firm that the proposed structure will bridge over the existing Rubio Wash structure, which traverses the site. Foundations for the proposed structure should be designed to bear at or below the bottom elevation of the Rubio Wash. It is anticipated that excavations for the proposed subterranean level will extend below the bottom of the wash. Proper support for the wash should be maintained at all times. Shoring may be required for excavations immediately adjacent to the wash.

The validity of the conclusions and design recommendations presented herein is dependant upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings and test pits on the site as indicated, and should not be construed to reflect any variations which may occur between these excavations, or which may result from changes in subsurface conditions.

SEISMIC DESIGN CONSIDERATIONS

The site is not located within a State of California Earthquake Fault Zone, and no known surface fault traces cross the site. In addition, the soils underlying the site are not prone to liquefaction or other potential secondary seismic hazards. Therefore, the major seismic hazard to the site, as with all of Southern California, is moderate to strong ground shaking which would result from a moderate to large earthquake on one of the local or regional faults.

Based on the seismic design provisions of the Uniform Building Code, a Soil Profile Type S_D and near-source factors of N_a=1.12 and N_v =1.4 should be utilized in the design. The closest fault is the Raymond Fault, which is a Seismic Source Type B, located 3.8 kilometers from the subject site.

FILL SOILS

The maximum depth of fill encountered on the site was 15 feet in Borings B1 and B6. It is anticipated that most of the fill will be removed during the excavation of the proposed subterranean garage. Any existing fill soils not removed during excavation of the subterranean level, shall be removed and recompacted.

EXPANSIVE SOILS

Representative samples of the onsite soils remolded to 90 percent of the maximum density were found to be very low in expansion potential, with Expansion Indexes ranging from 1 to 2. Additional reinforcing is required, as noted in the Foundation Design and Floor Slabs sections of this report. Expansion test data is presented on Plate D in the Appendix.

WATER-SOLUBLE SULFATES

The water-soluble sulfate content of the onsite soils was determined to be less than 0.10 percent by weight, for the soils encountered near the proposed foundation level. Based on the Uniform Building Code, Table 19-A-4, the sulfate exposure is considered to be negligible, and Type I cement may be utilized for concrete foundations in contact with the site soils.

SITE DRAINAGE

Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Irrigation in the planter areas around the proposed development should be properly controlled. Excessive irrigation may saturate the underlying soils and adversely affect the proposed structure.

Landscape and Irrigation

It is recommended that a landscape architect be consulted regarding planting adjacent to the proposed development. Planters placed adjacent to the proposed development shall be designed to drain away from the structure. Plants surrounding the development shall be of a variety that requires a minimum of watering. Plants with extensive root structures could have adverse effects on the proposed development. A landscape architect should be consulted in regards to root control and/or mitigation methods.

An adequate irrigation system will be required to sustain landscaping. Care should be taken to not saturate the soils (i.e. leaking irrigation lines or excessive landscape watering). Any leaks or defective sprinklers shall be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems shall be adjusted for rainy seasons. A landscape irrigation specialist should be consulted to determine the best times for landscape watering and the maximum amount of water usage.

GRADING

The following guidelines may be used in preparation of the grading plan and job specifications for any areas where fill or recompaction may be required, such as the lower floor subgrade area prior to pouring the floor slab, or driveway and sidewalk areas. The opportunity of reviewing the contract documents prior to the solicitation of bids to see that the intent of these recommendations is conveyed to the contractor is appreciated.

A. The areas to receive compacted fill shall be stripped of all vegetation, existing loose fill, and soft or disturbed soils. The excavated area shall be observed by a representative of the geotechnical engineer prior to placing controlled compacted fill.

B. The exposed grade shall then be scarified to a depth of 6 inches, moistened to optimum moisture content, and recompacted to a minimum of 90 percent of the maximum density.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in compacted layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills, but may require moisture adjustment prior to placing as fill. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Imported fill material should have an expansion index less 50. Rocks larger than 6 inches in diameter shall not be used in the fill.

D. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by ASTM D 1557-00.

E. Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until the required compaction is obtained.

F. Utility trenches should be properly backfilled in accordance with the following: The pipe should be bedded with clean sands to a depth of at least 1 foot over the pipe. The remainder of the backfill may be onsite soil compacted to 90 percent of the maximum density.

G. Any vegetation or associated root system located within the footprint of the proposed structure should be removed during grading. Any existing or abandoned utilities located within the footprint of the proposed structure should be removed or relocated. All fill materials and disturbed soils resulting from grading operations should be removed and properly recompacted prior to foundation excavation.

FOUNDATION DESIGN

Conventional foundations bearing in the dense to very dense native soils found near the proposed subterranean garage level may be utilized for foundation support. Any fill that is not removed during excavation of the proposed subterranean level should be penetrated so that footings bear in the dense to very dense native soils. Footings adjacent to the Rubio Wash should be deepend to bear at or below the bottom elevation of the wash.

Continuous footings a minimum of 1 foot wide, and 1.5 feet in depth below the lowest adjacent grade may be designed for an initial allowable bearing pressure of 3000 pounds per square foot. Column footings a minimum of 2 feet wide and 1.5 feet in depth below the lowest adjacent grade may be designed for an initial allowable bearing pressure of 3500 pounds per square foot.

The bearing values provided above may be increased by 300 pounds per square foot for each additional foot of width, and 600 pounds per square foot for each additional foot of depth. The maximum allowable bearing pressure is 5000 pounds per square foot.

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. The weight of concrete in the footings may be taken as 50 pounds per cubic foot, and the weight of the soil backfill may be neglected when determining the downward load on the footings.

All footing excavations should be observed by personnel of this firm to verify penetration into competent, undisturbed natural soils. Footings should be deepened if necessary to extend into satisfactory soils. Footing excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required footing backfill should be mechanically compacted. Flooding of backfill is not permitted.

Lateral Foundation Design

Resistance to lateral loading may be provided by friction acting at the base of foundations, and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces.

Passive earth pressure may be computed as an equivalent fluid having a density of 300 pounds per cubic foot, with a maximum earth pressure of 3000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

Foundation Settlement

The majority of foundation settlement is expected to occur on initial application of loading. The maximum settlement is expected to be 3/4 inch, and will occur below the heaviest loaded columns. Differential settlement is not expected to exceed 1/4 inch.

RETAINING WALL DESIGN

It is anticipated that the proposed subterranean garage retaining walls will retain up to approximately 14 feet. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

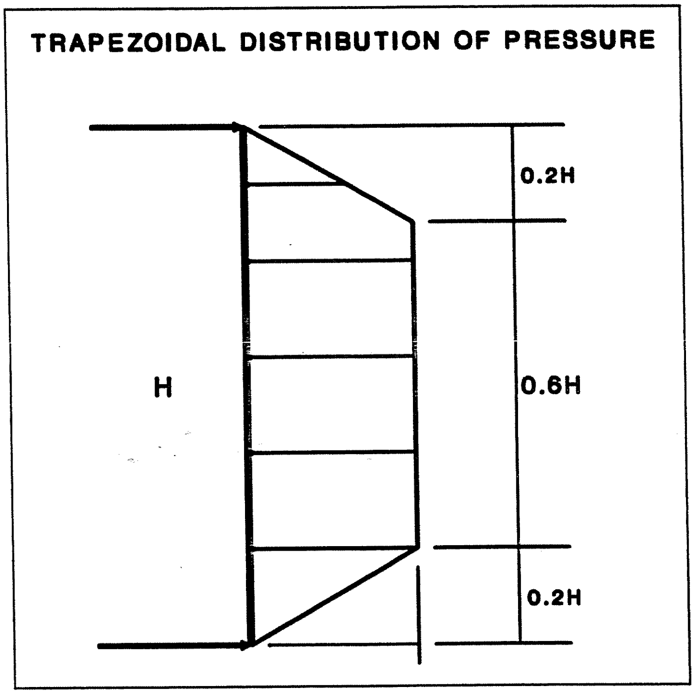
For traffic surcharge, the upper 10 feet any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Cantilever Walls

Free-standing, cantilevered retaining walls up to 14 feet in height supporting a level backslope may be designed utilizing a triangular distribution of pressure, and an equivalent fluid pressure of 35 pounds per square foot per foot of depth. Surcharge from any adjacent traffic, sloping ground or adjacent structures should be added as described above.

Restrained Retaining Walls

Restrained retaining walls retaining up to 14 feet supporting a level backslope may be designed to resist a trapezoidal pressure distribution of lateral earth pressure as indicated in the diagram below. Surcharge from any adjacent traffic, sloping ground or adjacent structures should be added as described above.



Design walls for 24H psf
Where H is the height of the walls in feet

Retaining Wall Drainage

Retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. Certain types of subdrain pipe are not acceptable to the various municipal agencies. It is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the laboratory maximum density obtainable by the ASTM Designation D 1557-00 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

Waterproofing

Moisture affecting retaining walls is one of the most common post- construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence deposits or water seepage through the wall. Efflorescence is a process in which a white, powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide appropriate protection to retaining walls.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered to the total depth explored, 80 feet. In addition, the historic high groundwater level is reportedly greater than 100 feet below the existing site grade. Therefore, the only water which could affect the proposed retaining walls would be irrigation and precipitation. Additionally, the site grading will be such that all drainage will be directed to the street and the structure will be designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

EXCAVATIONS

Excavations on the order of 14 to 16 feet in vertical height will be required for the subterranean levels. Due to the depth of the excavations, and the proximity to property lines and the Rubio wash, it is anticipated that excavations for the basement level will require shoring.

The onsite soils are suitable for vertical excavations up to 3 feet where not surcharged by adjacent traffic or structures. Where sufficient space is available, temporary unsurcharged embankments over three feet in height could be sloped back at a uniform 1:1 slope gradient. A uniform sloped excavation does not have a vertical component.

If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff from entering the excavation and eroding the slope faces. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation nor to flow towards it.

SHORING

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

The recommended method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 ½ diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wide flange section.

The slurry must have sufficient strength to impart the lateral bearing pressure developed by the wide flange section to the soil.

For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 5000 pounds per square foot. These values include the 100 percent increase allowed for isolated piles spaced at least 2 ½ diameters on center. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

Groundwater was not encountered during exploration and, therefore, it is not expected that water will be encountered in the pile excavations.

The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam, lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Casing may be required should excessive caving be experienced in the granular soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Difficult drilling conditions should be expected within the sandier materials, due to the presence of cobbles and possibly boulder-sized material.

Lagging

If the clear spacing between soldier piles does not exceed 4 feet, lagging between soldier piles could be omitted within the more cohesive soils. In the less cohesive soils, such as the sandier materials, lagging would be necessary. At this time, it is anticipated that most or all of the excavation will require continuous lagging. It is recommended that the exposed soils be observed by a representative of the geotechnical engineer to verify the cohesive nature of the soils, and determine whether any lagging may be omitted.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure, but be limited to a maximum of 500 pounds per square foot.

Tie-back Anchors

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

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Drilled friction anchors constructed without utilizing post-grouting techniques may be designed for a skin friction of 500 pounds per square foot. Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2500 pounds per square foot could be utilized for post-grouted anchors, provided the system does not rely on end-bearing plates to develop the necessary resistance. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Anchor Installation

Tie-back anchors may be installed between 20 and 40 degrees below the horizontal. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Anchor Testing

At least ten percent of the anchors should be selected for "quick", 200 percent tests and three additional anchors be selected for 24-hour, 200 percent tests. It is recommended that the 24-hour tests be performed prior to installation of additional tiebacks. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory test results are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

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

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

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

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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of a cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. Equivalent fluid pressures for the design of cantilevered shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
16 feet	30 pcf	20H psf

*Where H is the height of the shoring in feet.

In addition, surcharge loads occurring as a result of traffic in the streets and any surcharge loading imposed by any adjacent traffic or structures should be added to the design of the proposed shoring system. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent areas, including buildings and utilities in adjacent streets. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

Monitoring

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

SLABS ON GRADE

The lower garage floor slab should be cast over undisturbed natural earth materials or properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density, as described in the 'Grading' section of this report.

The lower floor slab may be supported directly on the exposed grade, provided the floor would not be adversely affected by moisture. In any areas where dampness would be objectionable, it is recommended that the floor slab be supported on an impermeable moisture barrier, such as 10-mil visqueen. If the membrane is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier should be covered with a thin layer of sand, approximately 2 inches, to prevent punctures and aid in the concrete cure.

For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

The concrete subterranean garage floor slab-on grade should be a minimum of 5 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. Proper spacing and jointing should be utilized.

Outdoor Flatwork

Concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. Proper spacing and jointing should be utilized.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

CONSTRUCTION COMPLIANCE

Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. It is critical that all foundations be observed by a representative of this office prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

The entire length of subdrain behind retaining walls should be observed by a representative of this firm. All gravel backfill above the subdrain must be observed by a representative of this firm prior to placing a minimum of 2 feet of controlled fill as a cap. Any gravel backfill must be densified by vibrating or wheel-rolling.

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

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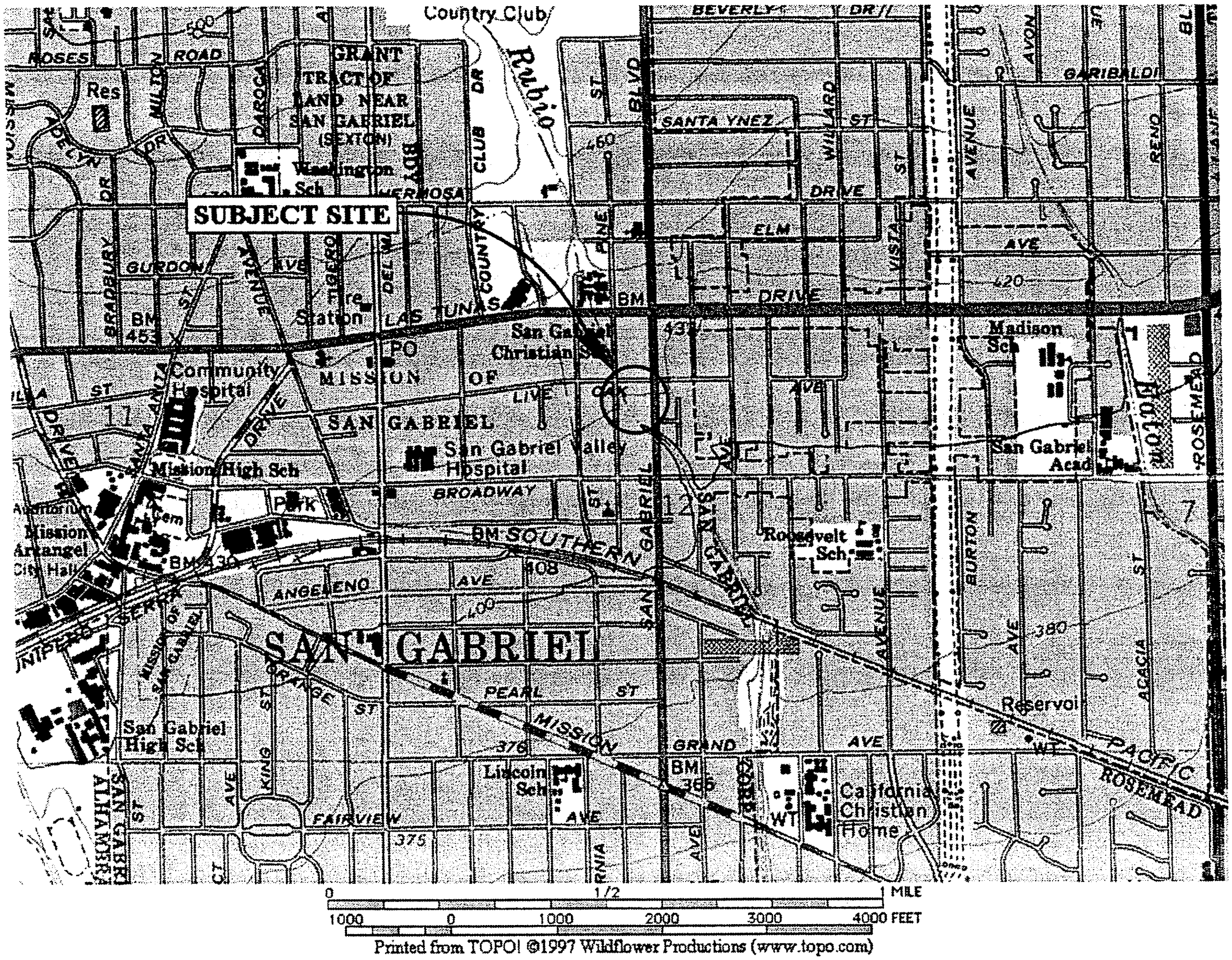

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
EL MONTE, CA QUADRANGLE

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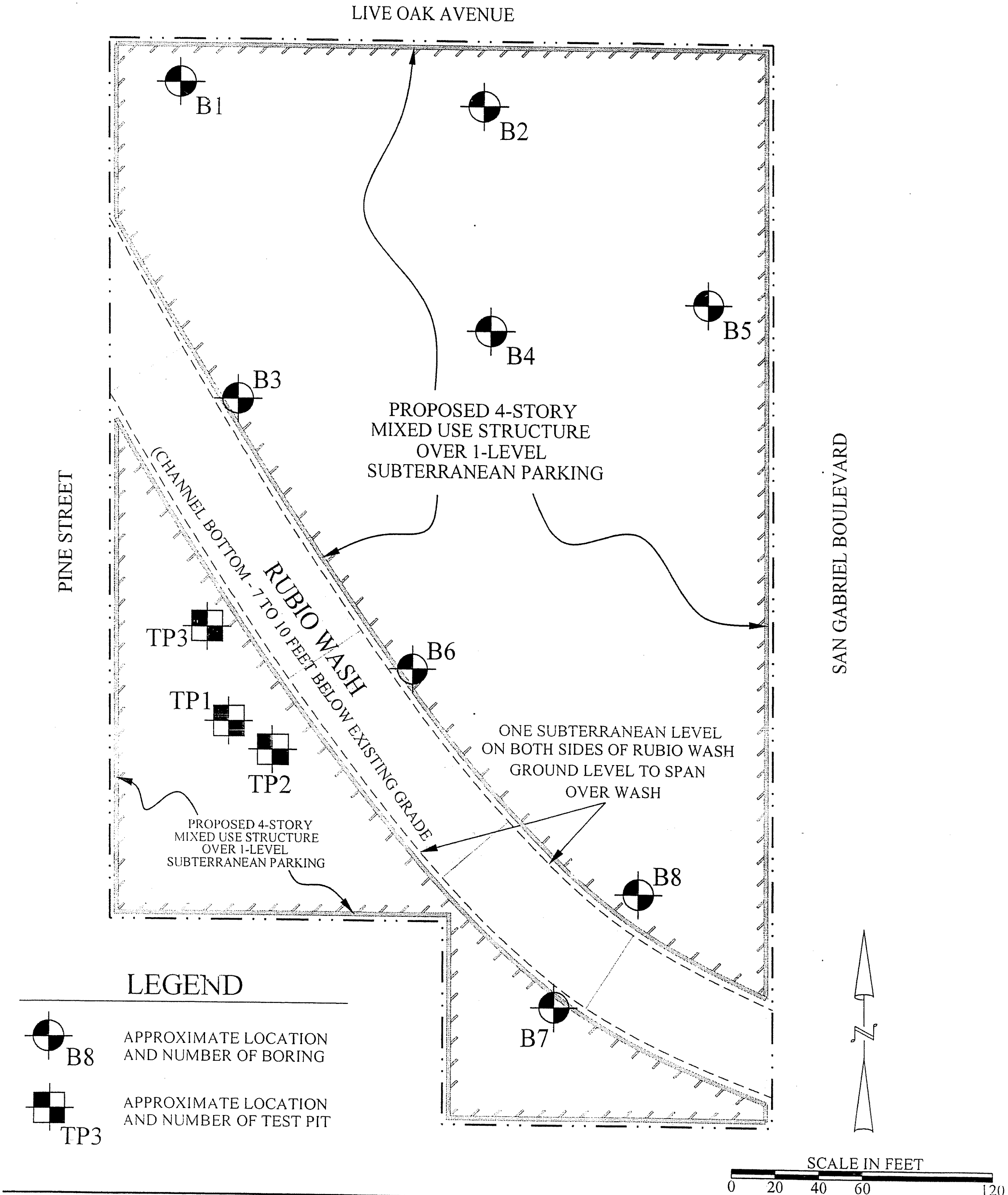
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S



PLOT PLAN

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HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S

APPENDIX - EXPLORATION AND LABORATORY TESTING - DRILL RIG

Exploration

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

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

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

Classification

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

Moisture-Density

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

APPENDIX - EXPLORATION AND LABORATORY TESTING - DRILL RIG - continued

Shear Tests

Shear tests are performed with a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

Consolidation

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

Expansion Tests

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

Remolded Tests

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

BORING LOG NUMBER 1

Drilling Date: 02/12/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 6-inch Asphalt - Poor Condition, No Base
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine to medium grained, some gravel
1	9	9.7	118.0	1		
				2		
3	17	5.3	Disturbed	3		
						Sand, yellowish-brown, moist, medium dense, fine to coarse grained, some gravel and asphalt fragments
				4		
5	18	No Recovery		5		Silty Sand, dark brown, moist, medium dense, fine to medium grained, some gravel and asphalt fragments
				6		
7	14	7.2	111.1	7		
				8		
				9		
10	19	2.5	SPT	10		
						Sand, light brown, moist, medium dense, fine to coarse grained, some gravel
				11		
				12		
12½	24	7.9	116.6			
				13		Silty Sand, medium brown, moist, medium dense, fine grained
				14		
15	34	3.6	118.6	15		
					SW	Sand, light brown, moist, dense, fine to coarse grained, some gravel
				16		
				17		
17½	60	2.3	123.2			
				18		
				19		
20	52	5.4	114.3	20		
				21		
				22		
22½	53	7.3	106.1			
				23		
				24		
25	40	11.3	98.2	25		
				26		
				27		
				28		
				29		
30	48	18.8	112.2	30		
					ML	Sandy Silt, dark to medium brown, moist, firm

BORING LOG NUMBER 1

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
				33		
				34		
35	53	6.6	113.4	35		
				36	SM	Silty Sand, light brown to brown, moist, dense, fine to medium grained, some gravel
				37		
				38		
				39		
40	54	16.7	109.3	40		
				41	ML	Sandy Silt, light brown to brown, moist, firm to stiff
				42		
				43		
				44		
45	79	15.7	99.0	45		
				46		
				47	SM	Sandy Silt, light brown to brown, moist, dense to very dense, fine grained
				48		
				49		
50	30 50/5"	14.7	100.6	50		
				51		Total depth: 50 feet No Water Fill to 15 feet
				52		
				53		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				54		
				55		Used 8-inch diameter Hollow-Stem Auger 140-lb. Slide Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				56		SPT=Standard Penetration Test
				57		
				58		
				59		
				60		

BORING LOG NUMBER 2

Drilling Date: 02/12/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 6-inch Asphalt -Fair Condition, No Base
				0 --		FILL: Silty Sand, dark brown, slightly moist, medium dense, fine to medium grained, some gravel
				-		
				1 --		
				-		
2	24	6.6	121.6	2 --		
				-		
				3 --		
				-		
4	21	6.4	110.1	4 --		
				-		fine to coarse grained, some asphalt fragments
				5 --		
				-		
				6 --		
				-		
7	11	5.3	105.4	7 --		
				-		Sand, light brown to brown, moist, medium dense, fine grained
				8 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine to coarse grained, some gravel
				9 --		
				-		
10	12	5.7	114.7	10 --		
				-		
				11 --		
				-		
				12 --		
12½	15	6.1	117.7	-		
				13 --		brown to dark brown
				-		
				14 --		
				-		
15	46	5.8	120.9	15 --		
				-	SW	Sand, light brown to brown, moist, dense, fine to coarse grained, some gravel
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		slightly more moist
20	88	9.9	104.5	20 --		
				-		Total depth: 20 feet No Water Fill to 8 feet
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		

BORING LOG NUMBER 3

Drilling Date: 02/13/04

Project: File No. 18568-S — **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 3-inch Asphalt - Poor Condition, 2-inch Base
				0 --		FILL: Silty Sand, dark brown, moist, dense, fine to coarse grained, some gravel, trace debris
1	33	11.0	113.3	1 --		some brick fragments, concrete fragments and asphalt fragments
				2 --		
3	33	3.7	113.8	3 --		
				4 --		
5	8	2.8	Disturbed	5 --		Sand, light brown, slightly moist, medium dense, fine to coarse grained, some gravel and cobble
				6 --		
7	32	1.2	Disturbed	7 --		
				8 --		
				9 --		
10	37	3.9	122.7	10 --	SM	Silty Sand, brown, moist, dense, fine to coarse grained, some gravel
				11 --		
				12 --		
				13 --		
				14 --		
15	24 50/5"	4.0	128.3	15 --	SW	Sand, light brown, moist, very dense, fine to coarse grained, some gravel
				16 --		
				17 --		
				18 --		
				19 --		
20	40	17.8	103.3	20 --	ML	Sandy Silt, brown, moist, firm
				21 --		
				22 --		
22½	19 50/6"	9.3	103.4	23 --	SW	Sand, light brown, moist, very dense, fine to coarse grained
				24 --		
25	56	2.3	127.0	25 --	ML	Sandy Silt, medium brown, moist, firm to stiff
				26 --		
				27 --		
27½	38	18.0	113.8	28 --		
				29 --	SM	Silty Sand, brown, moist, very dense, fine to coarse grained, trace cobbles
30	81	8.4	123.2	30 --		
						Total depth: 30 feet; No Water; Fill to 10 feet

BORING LOG NUMBER 4

Drilling Date: 02/12/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 2-inch Asphalt, No Base
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine to medium grained, trace asphalt fragments
				1 --		
2	16	5.8	Disturbed	2 --		
				3 --		
4	58	9.4	101.3	4 --		dense, fine grained, minor asphalt fragments and gravel
				5 --		
				6 --		
7	15	9.7	113.6	7 --		fine to coarse grained
				8 --		
				9 --		
10	17	6.9	121.5	10 --	SM	Silty Sand, medium brown, moist, dense, fine to medium grained, minor gravel
				11 --		
				12 --		
				13 --		
				14 --		
15	50	4.9	117.8	15 --	SW	Sand, light brown to brown, moist, dense, fine to coarse grained, minor cobbles
				16 --		
				17 --		
				18 --		
				19 --		
20	59	4.4	SPT	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	66	13.9	119.8	25 --	SM	Silty Sand, medium brown, moist, dense, fine grained
				26 --		
				27 --		
27½	72	17.0	117.6	28 --	ML	Sandy Silt, medium brown, moist, stiff
				29 --		
30	60	15.0	SPT	30 --	SM	Silty Sand, medium brown, moist, dense, fine grained

BORING LOG NUMBER 4

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
32½	40 50/4"	9.3	121.6	33		very dense, fine to coarse grained, some gravel
				34		
35	55	11.1	123.1	35		light brown to brown
				36		
				37		
37½	50	10.5	109.0	38		fine grained
				39		
40	42	5.8	SPT	40		
				41		
				42		
				43		
				44		
45	30 50/4"	2.7	121.4	45		fine to coarse grained
				46		
				47		
				48		
				49		
50	44	2.3	SPT	50	ML	Sandy Silt, brown, moist, firm to stiff
				51		
				52		
				53		
				54		
55	46	13.6	102.0	55	SM	Silty sand, brown, moist, dense, fine grained
				56		
				57		
				58		
				59	SP	Sand, light brown, slightly moist, dense, fine grained
60	32	13.1	SPT	60	SM	Silty Sand, brown, moist, dense, fine grained

BORING LOG NUMBER 4

Project: File No. **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				61		
				62		
				63		
				64		
65	61	4.1	108.7	65		
				66	SM/SP	Silty Sand to Sand, light brown, moist, dense, fine to medium grained
				67		
				68		
				69		
70	54	8.9	SPT	70		
				71		fine grained
				72		
				73		
				74		
75	80	3.2	101.7	75		
				76	SP	Sand, light brown, moist, dense, fine grained
				77		
				78		
				79		
80	65	13.8	SPT	80	SM	Silty Sand, medium brown, moist, dense, fine to coarse grained
				81		Total depth: 80 feet No Water Fill to 10 feet
				82		
				83		
				84		
				85		
				86		
				87		
				88		
				89		
				90		

BORING LOG NUMBER 5

Drilling Date: 02/12/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 4-inch Asphalt - Poor Condition, No Base
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine to medium grained
1	41	9.3	114.4	1		
3	31	8.8	112.1	3		light brown and dark brown mottling
5	23	9.7	118.7	5	SP	Sand, light brown, moist to slightly moist, dense, fine grained, minor gravel
7	11	7.7	111.4	7	SM	Silty Sand, dark brown, moist, medium dense, fine to coarse grained, minor gravel
10	23	6.6	116.2	10		dense
15	58	3.8	120.3	15	SW	Sand, light brown, slightly moist, dense, fine to coarse grained, some gravel
20	40 50/5"	3.9	115.9	20		
22½	88	4.1	109.0	22		
25	76	4.6	105.7	25	SP	Sand, light brown, slightly moist, dense, fine grained
27½	70	18.7	109.7	28	ML	Sandy Silt, medium brown, moist, stiff
30	50	18.0	115.1	30		Total depth: 30 feet; No Water; Fill to 5 feet

BORING LOG NUMBER 6

Drilling Date: 02/13/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 4-inch Asphalt - Poor Condition, 2-inch Base
				0		FILL: Silty Sand, dark brown, moist, dense, fine to coarse grained, trace concrete and asphalt fragments
				1		
2	45	5.8	Disturbed	2		
				3		
				4		
5	21	1.6	Disturbed	5		
				6		Sand, light brown to brown, slightly moist, medium dense, fine to coarse grained, some gravel
7	34	2.6	112.3	7		
				8		light brown
				9		
10	38	2.7	126.1	10		
				11		Silty Sand, dark brown, moist, dense, fine to coarse grained, trace gravel and cobbles
				12		
12½	65/7"	6.8	117.3	13		occasional Silty Sand inclusions
				14		
15	45	14.5	112.9	15		
				16	SM	Silty Sand, light brown to brown, moist, dense, fine grained
				17		
				18		
				19		grayish-brown and orange brown, fine to coarse grained, some gravel
20	45	17.2	111.1	20		
				21		Total depth: 20 feet No Water Fill to 15 feet
				22		
				23		
				24		
				25		
				26		
				27		
				28		
				29		
				30		

BORING LOG NUMBER 7

Drilling Date: 02/13/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 3-inch Asphalt - Good Condition, 2½-inch Base
				0 --		FILL: Silty Sand, brown, moist, dense, fine to coarse grained
1	20	No Recovery		1 --		dark brown, medium dense, some gravel
				2 --		
3	14	13.7	108.4	3 --		some glass and asphalt debris
				4 --		
5	10	5.0	Disturbed	5 --		brown to dark brown
				6 --		
7	14 50/6"	4.8	Disturbed	7 --		trace cobbles
				8 --		
				9 --		
10	21	2.5	116.8	10 --	SW	Sand, light brown, slightly moist, dense, fine to coarse grained, trace cobbles and gravel
				11 --		
				12 --		
12½	60	4.0	124.1	13 --		light brown to brown
				14 --		
15	27 50/4"	5.1	113.9	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	41 50/5"	4.4	118.0	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	66	4.2	105.6	25 --	SP	Sand, light brown, moist, dense, fine grained
				26 --	SW	Sand, light brown and orange brown, slightly moist, dense, fine to coarse grained, trace gravel and cobbles
				27 --		
				28 --		
				29 --		
30	38	13.3	124.7	30 --	ML	Sandy Silt, dark brown, moist, firm to stiff

BORING LOG NUMBER 7

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
32½	20 50/6"	10.7	123.9	33	SM	Silty Sand, brown to dark brown, moist, very dense, fine to medium grained
				34		
35	30 50/6"	11.2	112.6	35		
				36		
				37		
37½	39 50/4"	7.5	118.5	38		light brown to brown, fine to coarse grained, trace gravel
				39		
40	64	13.9	117.3	40		fine grained
				41		
				42		
				43		
				44		
45	69	3.5	101.4	45	SW	Sand, light brown, slightly moist, dense, fine to coarse grained
				46		
				47		
				48		
				49		
50	40 50/5"	7.2	105.3	50	SP	Sand, light brown, slightly moist, very dense, fine grained
				51		Total depth: 50 feet No Water Fill to 10 feet
				52		
				53		
				54		
				55		
				56		
				57		
				58		
				59		
				60		

BORING LOG NUMBER 8

Drilling Date: 02/13/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 4-inch Asphalt - Poor Condition, 4-inch Base
				0 --		FILL: Silty Sand, brown to medium brown, moist, dense, fine to coarse grained
				1 --		
2	30	4.7	120.1	2 --		Silty Sand, brown, moist, dense, fine to coarse grained, trace gravel
				3 --		
				4 --		
5	15	5.5	118.2	5 --		
				6 --		
7	41	3.0	Disturbed	7 --		Sand, orange brown, moist, dense, fine to coarse grained
				8 --		
				9 --		
10	27	9.2	122.6	10 --	SM	Silty sand, dark brown, moist, very dense, fine to coarse grained, trace gravel
				11 --		
				12 --		
				13 --		
				14 --		
15	15 50/6"	5.2	120.0	15 --	SW	Sand, orange brown, moist, very dense, fine to coarse grained, trace gravel
				16 --		
				17 --		
17½	22 50/6"	5.0	114.4	18 --		
				19 --		
20	79	5.3	117.7	20 --		Total depth: 20 feet No Water Fill to 10 feet
				21 --		
				22 --		
				23 --		
				24 --		
				25 --		
				26 --		
				27 --		
				28 --		
				29 --		
				30 --		

LOG OF TEST PIT NUMBER 1

Drilling Date: 02/19/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground with Gravel
			0		FILL: Silty Sand, brown, moist, medium dense, fine to coarse grained with cobbles
1	2.9	107.3	1	SW	Sand, light brown, slightly moist, medium dense, fine to coarse grained, some cobbles
			2		
3	1.9	122.0	3	SW/SM	Sand to Silty Sand, light to medium brown, slightly moist, dense, fine to coarse grained
			4	SW	Sand, light brown, slightly moist, medium dense, fine to coarse grained, some cobbles
5	1.2	Disturbed	5		
			6		Total depth: 6 feet No Water Caving at 5 feet Fill to 1 foot
			7		
			8		
			9		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		
			26		
			27		
			28		
			29		
			30		

LOG OF TEST PIT NUMBER 2

Drilling Date: 02/19/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		FILL: Silty Sand to Sand, brown, moist, medium dense, fine to medium grained, some roots and cobbles
			-		
			1 --		
			-	SW	Sand, light brown, slightly moist, dense, fine to coarse grained, some cobbles
			2 --		
			-		
3	3.6	119.5	3 --		
			-		
			4 --		
			-		
5	2.3	Disturbed	5 --		medium dense to dense
			-		
			6 --		
			-		Total depth: 6 feet
			7 --		No Water
			-		Caving at 5 feet
			8 --		Fill to 1 foot
			-		
			9 --		
			-		
			10 --		
			-		
			11 --		
			-		
			12 --		
			-		
			13 --		
			-		
			14 --		
			-		
			15 --		
			-		
			16 --		
			-		
			17 --		
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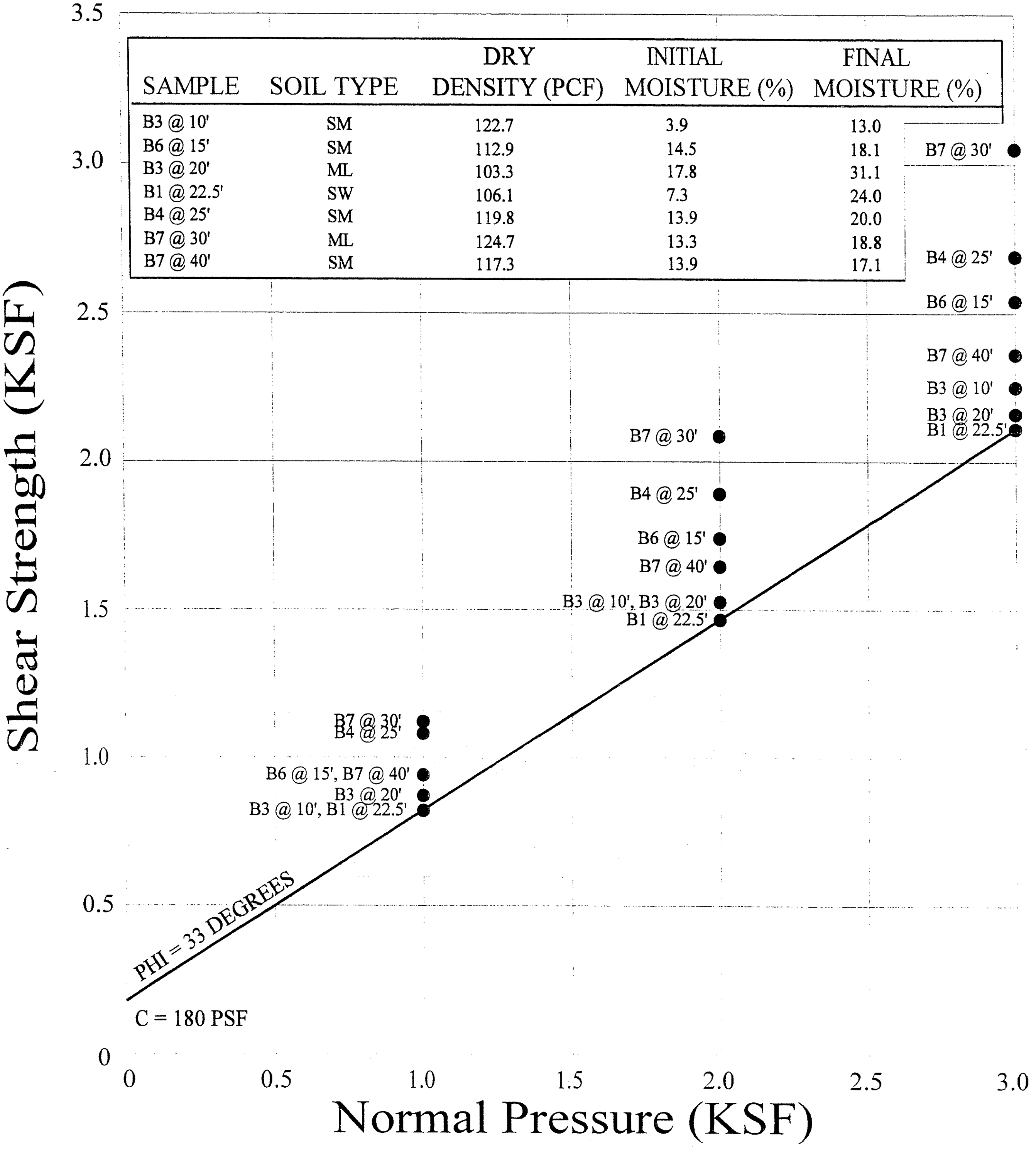
			21 --		
			-		
			22 --		
			-		
			23 --		
			-		
			24 --		
			-		
			25 --		
			-		
			26 --		
			-		
			27 --		
			-		
			28 --		
			-		
			29 --		
			-		
			30 --		
			-		

LOG OF TEST PIT NUMBER 3

Drilling Date: 02/19/04

Project: File No. 18568-S **Howard Poyourow Development and Construction**

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		FILL: Silty Sand to Sand, brown, moist, medium dense, fine to medium grained
			1 --		
			2 --		
3	4.0	105.8	3 --		
			-	SW	Sand, light brown, slightly moist to moist, medium dense, fine to coarse grained, some cobbles
4	4.2	115.5	4 --		
			5 --	SM/SW	Silty Sand to Sand, light brown, slightly moist, dense, fine to coarse grained, some cobbles
			6 --		
			-	SW	Sand, brown, slightly moist, medium dense to dense, fine to coarse grained, some cobbles
			7 --		
			8 --		
9	2.0	Disturbed	9 --		
			-		Total depth: 9 feet No Water Caving at 8 feet Fill to 3 feet
			10 --		
			11 --		
			12 --		
			13 --		
			14 --		
			15 --		
			16 --		
			17 --		
			18 --		
			19 --		
			20 --		
			21 --		
			22 --		
			23 --		
			24 --		
			25 --		
			26 --		
			27 --		
			28 --		
			29 --		
			30 --		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 10'	SM	122.7	3.9	13.0
B6 @ 15'	SM	112.9	14.5	18.1
B3 @ 20'	ML	103.3	17.8	31.1
B1 @ 22.5'	SW	106.1	7.3	24.0
B4 @ 25'	SM	119.8	13.9	20.0
B7 @ 30'	ML	124.7	13.3	18.8
B7 @ 40'	SM	117.3	13.9	17.1

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

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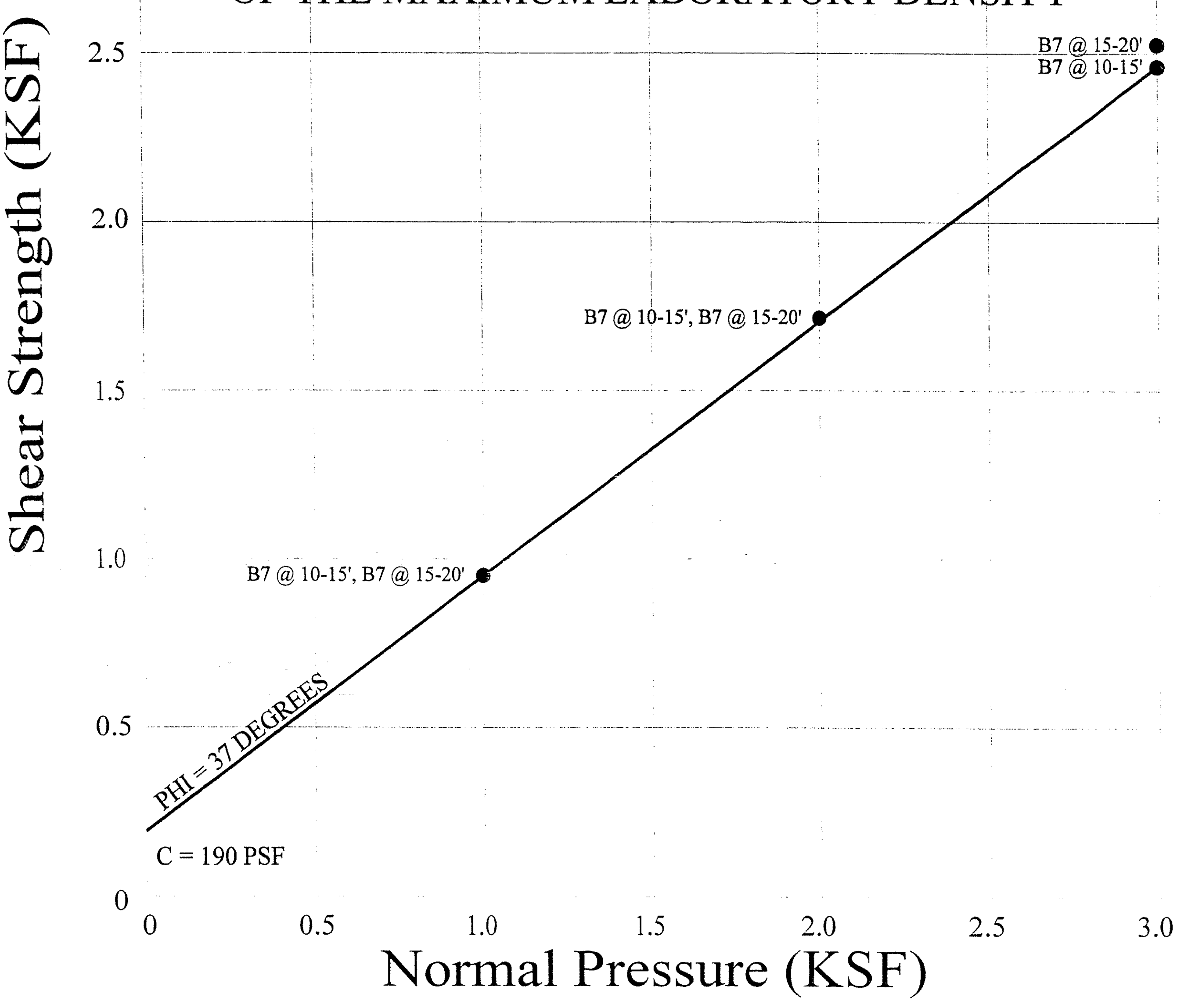
Consulting Geotechnical Engineers

HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7 @ 10-15'	SW	124.2	6.0	9.8
B7 @ 15-20'	SW	124.2	7.0	11.5

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

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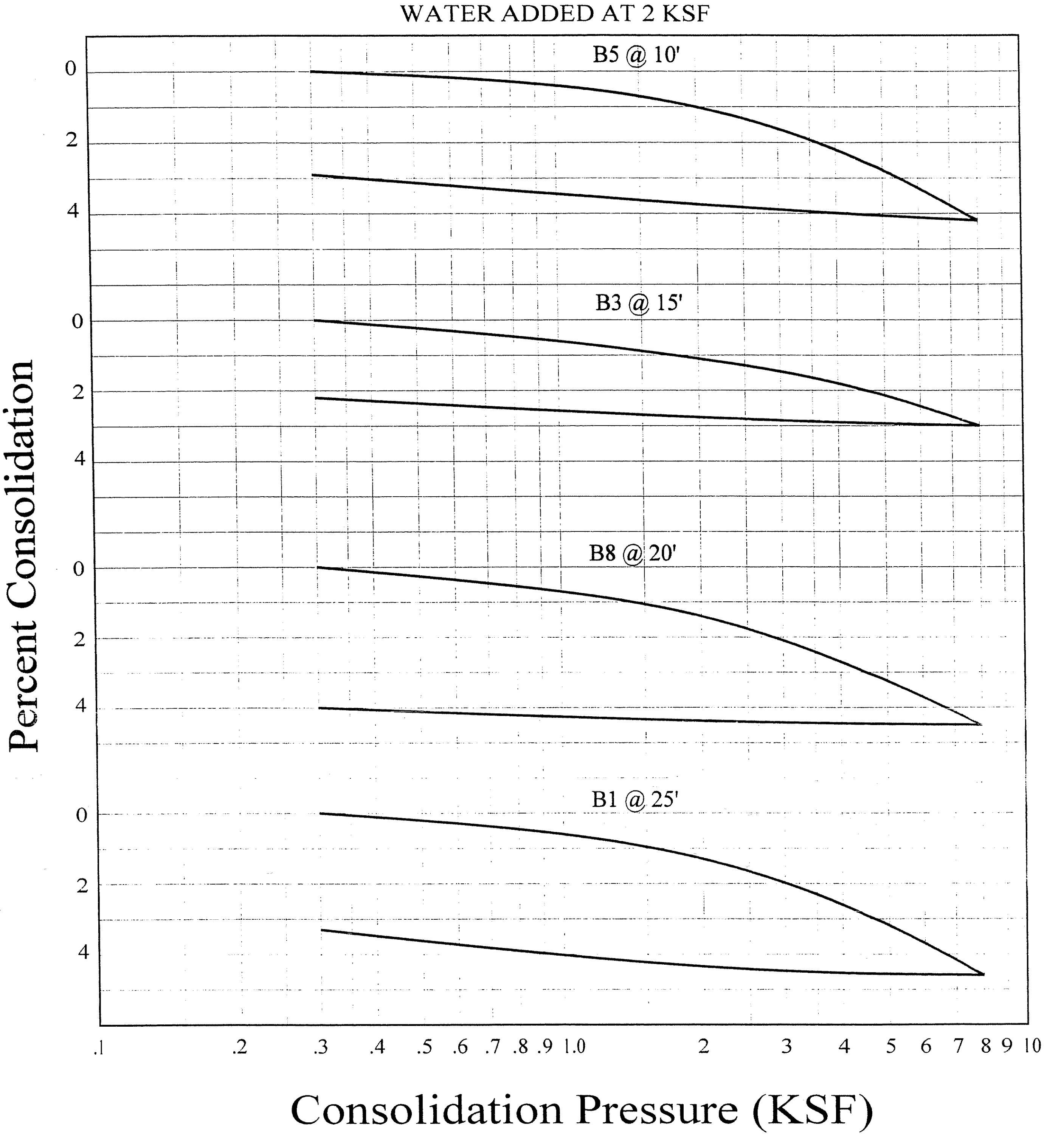
HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S PLATE: B-2



CONSOLIDATION TEST

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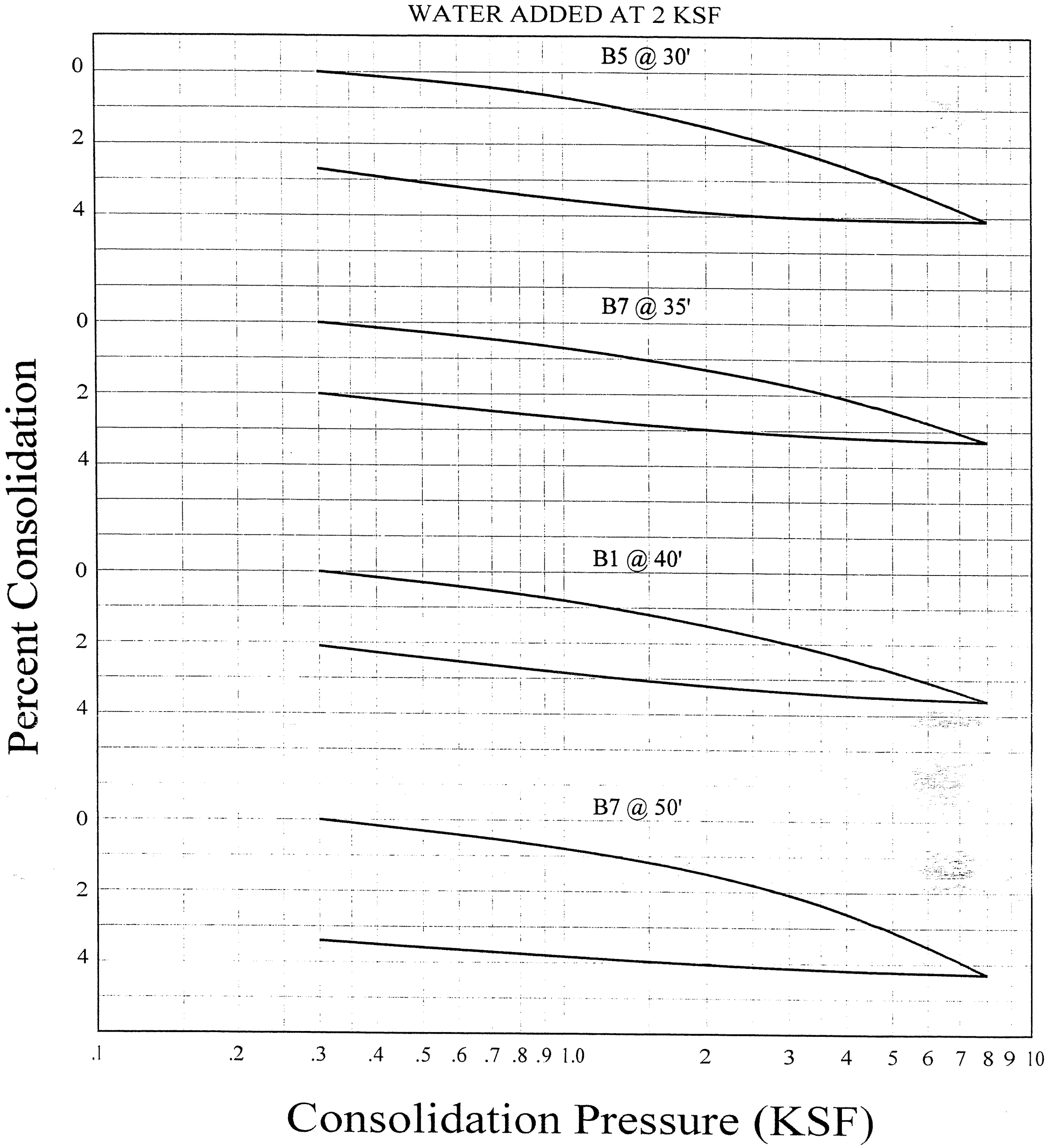
HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S PLATE: C-1



CONSOLIDATION TEST

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HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S PLATE: C-2

ASTM D-1557

SAMPLE	B7 @ 10-15'	B7 @ 15-20'
SOIL TYPE:	SW	SW
MAXIMUM DENSITY pcf.	138.0	138.0
OPTIMUM MOISTURE %	6.0	7.0

SWELL-60 POUNDS PER SQUARE FOOT

SAMPLE	B7 @ 10-15'	B7 @ 15-20'
SOIL TYPE:	SW	SW
AIR DRY%	0.2	0.2
SATURATION%	0.6	0.1
TOTAL%	0.8	0.3
EXPANSION INDEX UBC STANDARD 18-2	2	1
EXPANSION CHARACTER	VERY LOW	VERY LOW

SULFATE CONTENT

SAMPLE	B5 @ 1-5'	B7 @ 10-15'	B7 @ 15-20'
SULFATE CONTENT (percentage by weight):	< 0.10%	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

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HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S PLATE: D

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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 18568

DATE: 03-02-2004

JOB NAME: HOWARD POYOUROW **DEVELOPMENT AND CONSTRUCTION**

CALCULATION NAME: HOWARD POYOUROW **DEVELOPMENT AND CONSTRUCTION**

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 34.1012
   SITE LONGITUDE: 118.0914

SEARCH RADIUS: 60 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

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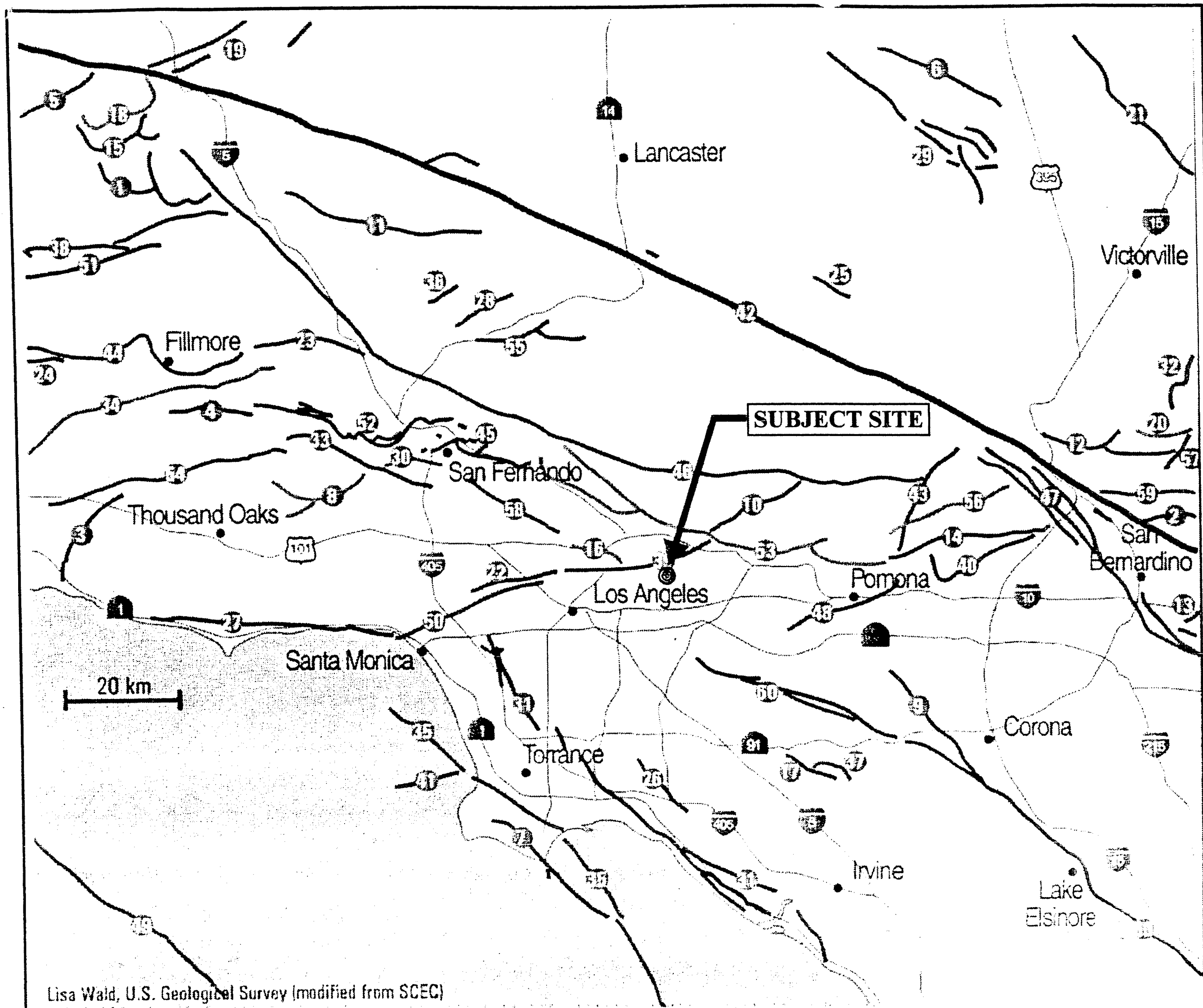
TABLE I – FAULTS IN THE VICINITY OF THE SITE

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
RAYMOND	3.4(5.5)	6.5	0.846	XI
VERDUGO	4.8(7.7)	6.7	0.749	XI
SIERRA MADRE	7.5(12.1)	7.0	0.617	X
CLAMSHELL-SAWPIT	8.3(13.3)	6.5	0.465	X
HOLLYWOOD	8.5(13.6)	6.4	0.432	X
ELYSIAN PARK THRUST	9.0(14.4)	6.7	0.475	X
WHITTIER	9.2(14.8)	6.8	0.409	X
SAN JOSE	12.6(20.3)	6.5	0.297	IX
COMPTON THRUST	15.1(24.3)	6.8	0.292	IX
NEWPORT-INGLEWOOD (L.A.Basin)	16.0(25.8)	6.9	0.266	IX
SIERRA MADRE (San Fernando)	18.3(29.5)	6.7	0.225	IX
SANTA MONICA	18.5(29.8)	6.6	0.209	VIII
SAN GABRIEL	18.6(29.9)	7.0	0.248	IX
CHINO-CENTRAL AVE. (Elsinore)	19.5(31.4)	6.7	0.210	VIII
CUCAMONGA	20.2(32.5)	7.0	0.242	IX
NORTHRIDGE (E. Oak Ridge)	22.1(35.5)	6.9	0.207	VIII
PALOS VERDES	24.7(39.8)	7.1	0.200	VIII
MALIBU COAST	25.7(41.3)	6.7	0.154	VIII
SAN ANDREAS - 1857 Rupture	27.5(44.3)	7.8	0.286	IX
SAN ANDREAS - Mojave	27.5(44.3)	7.1	0.180	VIII
SANTA SUSANA	28.6(46.0)	6.6	0.127	VIII
ELSINORE-GLEN IVY	31.2(50.2)	6.8	0.127	VIII
HOLSER	33.8(54.4)	6.5	0.094	VII
SAN JACINTO-SAN BERNARDINO	34.9(56.1)	6.7	0.102	VII
SAN ANDREAS - San Bernardino	35.3(56.8)	7.3	0.160	VIII
SAN ANDREAS - Southern	35.3(56.8)	7.4	0.172	VIII
ANACAPA-DUME	35.8(57.6)	7.3	0.154	VIII
NEWPORT-INGLEWOOD (Offshore)	36.7(59.1)	6.9	0.114	VII
CLEGHORN	38.8(62.4)	6.5	0.075	VII
OAK RIDGE (Onshore)	40.9(65.9)	6.9	0.099	VII
SIMI-SANTA ROSA	42.9(69.1)	6.7	0.079	VII
SAN CAYETANO	45.8(73.7)	6.8	0.078	VII
SAN ANDREAS - Carrizo	47.8(76.9)	7.2	0.106	VII
SAN JACINTO-SAN JACINTO VALLEY	49.3(79.3)	6.9	0.078	VII
NORTH FRONTAL FAULT ZONE (West)	49.6(79.9)	7.0	0.082	VII
ELSINORE-TEMECULA	53.1(85.4)	6.8	0.065	VI
SANTA YNEZ (East)	56.9(91.5)	7.0	0.071	VI
CORONADO BANK	58.3(93.9)	7.4	0.097	VII

-END OF SEARCH- 38 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE RAYMOND FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.4 MILES (5.5 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8465 g



1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek fault
16 Eagle Rock fault
17 El Modeno fault
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

SOUTHERN CALIFORNIA FAULT MAP

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HOWARD POYOUROW
DEVELOPMENT AND CONSTRUCTION

FILE NO. 18568-S

FIGURE I

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*     E Q S E A R C H   *
*                       *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 18568

DATE: 03-02-2004

JOB NAME: HOWARD POYOUROW **DEVELOPMENT AND CONSTRUCTION**

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 34.1012
SITE LONGITUDE: 118.0914

SEARCH DATES:
START DATE: 1800
END DATE: 2004

SEARCH RADIUS:
60.0 mi
96.6 km

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

TABLE II – HISTORICAL EARTHQUAKE EPICENTER

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	1.006	XI	0.5(0.8)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.455	X	2.0(3.2)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.693	XI	2.9(4.6)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.156	VIII	8.7(14.0)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.141	VIII	9.7(15.7)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.120	VII	11.4(18.4)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.120	VII	11.4(18.4)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.120	VII	11.4(18.4)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.149	VIII	12.2(19.7)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.151	VIII	12.9(20.7)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.121	VII	13.8(22.2)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.058	VI	20.0(32.3)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.071	VI	22.1(35.6)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.059	VI	22.5(36.3)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.059	VI	23.7(38.1)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.064	VI	23.8(38.3)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.045	VI	24.2(39.0)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.049	VI	24.2(39.0)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.057	VI	24.2(39.0)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.049	VI	24.2(39.0)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.045	VI	24.2(39.0)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.045	VI	24.4(39.3)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.045	VI	24.4(39.3)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.050	VI	25.1(40.5)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.147	VIII	26.6(42.8)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.040	V	27.7(44.6)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.040	V	27.7(44.6)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.071	VI	27.7(44.6)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.048	VI	27.7(44.6)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.071	VI	27.7(44.6)
DMG	34.4110	113.4010	02/09/1971	14 041.8	8.4	6.40	0.115	VII	27.7(44.6)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.053	VI	29.0(46.6)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.034	V	29.5(47.4)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.039	V	30.4(48.9)
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.034	V	31.2(50.2)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.071	VI	31.2(50.3)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.148	VIII	31.3(50.4)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.068	VI	32.1(51.7)
DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.035	V	32.6(52.5)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.029	V	33.1(53.3)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.039	V	33.6(54.0)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.031	V	33.7(54.2)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.079	VII	34.2(55.0)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.132	VIII	34.5(55.6)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.027	V	35.0(56.3)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.026	V	35.4(57.0)
GSP	34.3780	118.6180	01/19/1994	211144.9	11.0	5.10	0.028	V	35.6(57.3)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.085	VII	36.5(58.7)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.029	V	36.9(59.3)
GSP	34.3690	118.6720	04/26/1997	103730.7	16.0	5.10	0.026	V	37.9(61.1)

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GSP	34.3260	118.6980	01/17/1994	233330.7	9.0	5.60	0.039	V	37.9(61.1)
GSP	34.3940	118.6690	06/26/1995	084028.9	13.0	5.00	0.023	IV	38.7(62.2)
GSP	34.3770	118.6980	01/18/1994	004308.9	11.0	5.20	0.027	V	39.5(63.6)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.033	V	40.1(64.5)
GSB	34.3790	118.7110	01/19/1994	210928.6	14.0	5.50	0.033	V	40.2(64.7)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.030	V	43.3(69.7)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.024	IV	45.2(72.8)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.017	IV	48.3(77.8)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.038	V	48.3(77.8)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.017	IV	48.3(77.8)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.046	VI	48.6(78.3)
MGI	34.0000	119.0000	12/14/1912	0 0 0.0	0.0	5.70	0.027	V	52.4(84.4)
DMG	34.0000	119.0000	09/24/1827	4 0 0.0	0.0	7.00	0.075	VII	52.4(84.4)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.034	V	52.9(85.1)
DMG	34.0650	119.0350	02/21/1973	144557.3	8.0	5.90	0.030	V	54.0(86.9)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.030	V	57.1(91.8)

****
-END OF SEARCH- 66 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

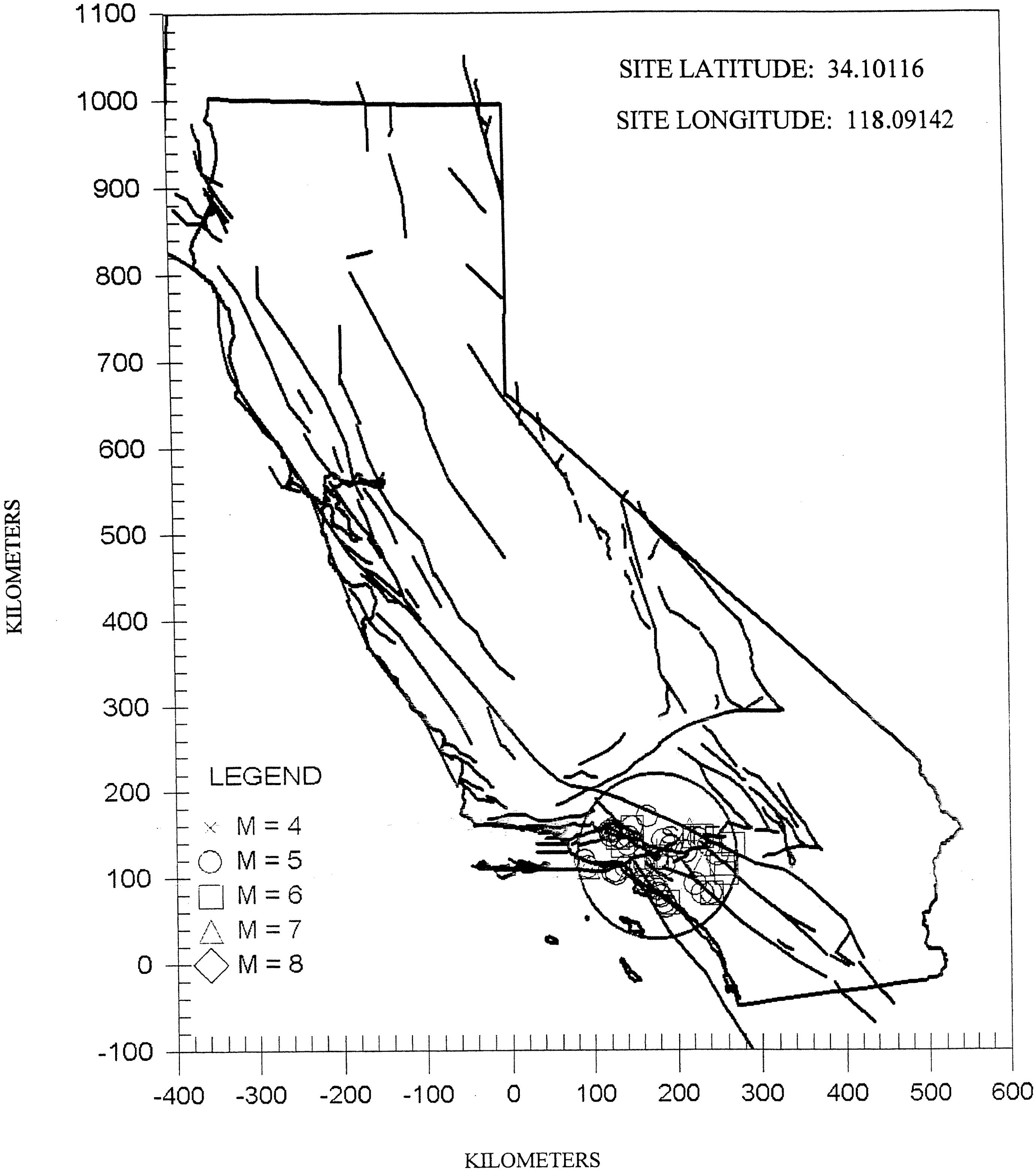
TIME PERIOD OF SEARCH: 1800 TO 2004

LENGTH OF SEARCH TIME: 205 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 0.5 MILES (0.8 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 1.006 g



EARTHQUAKE EPICENTERS MAP

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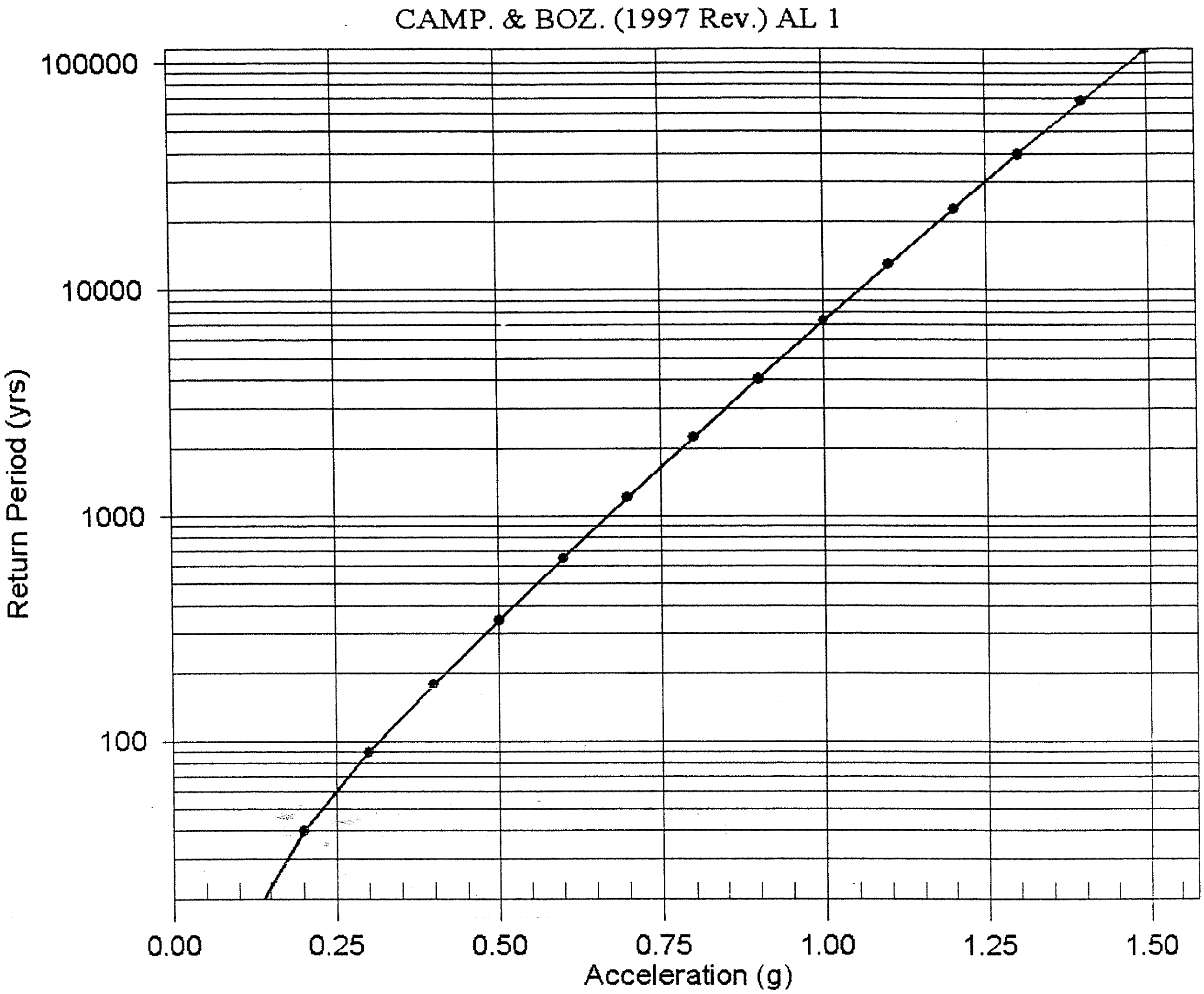
FIGURE II



RETURN PERIOD vs. ACCELERATION

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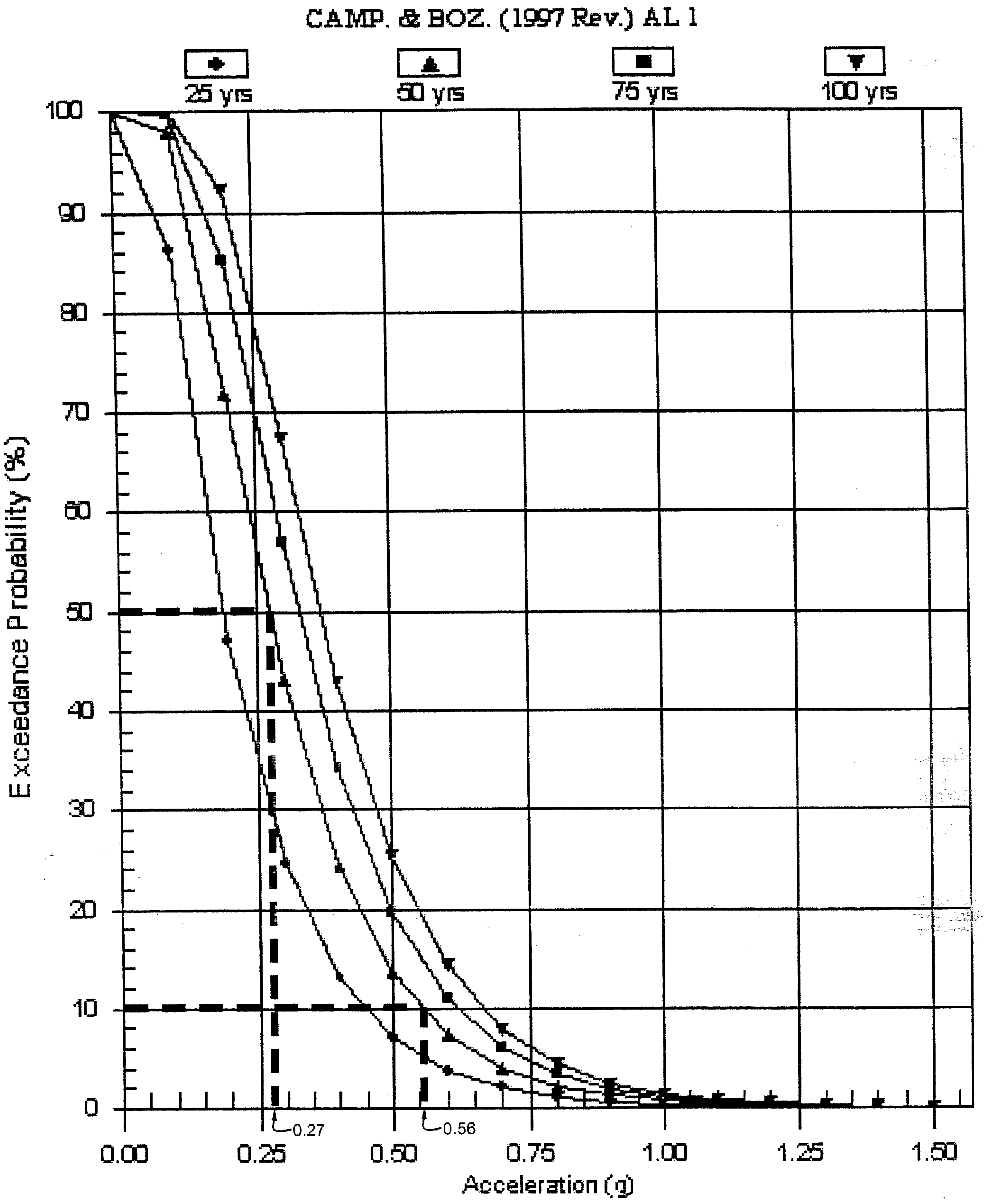
FIGURE III



PROBABILITY OF EXCEEDANCE

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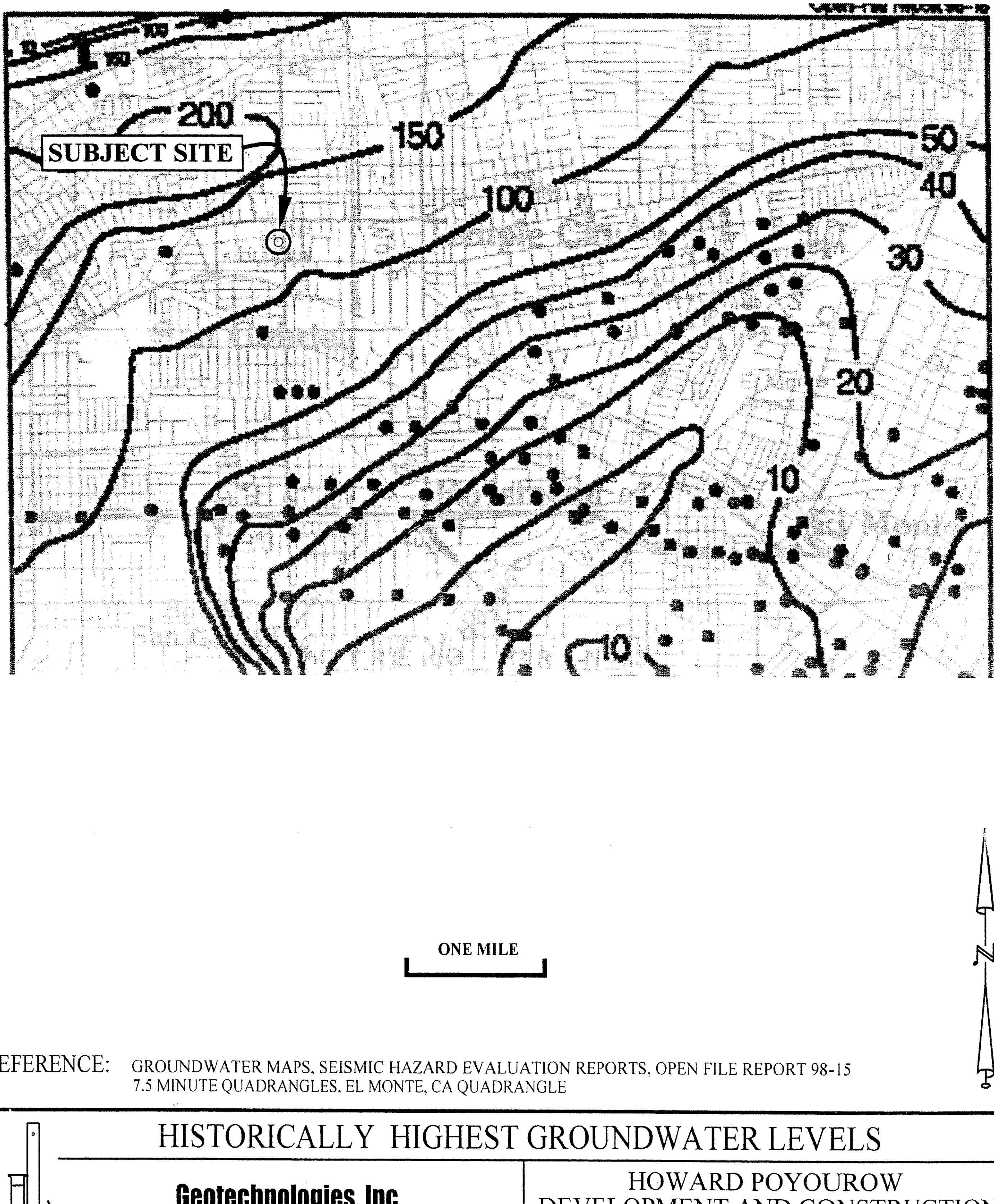
FILE NO. 18568-S	FIGURE IV



REFERENCE: GROUNDWATER MAPS, SEISMIC HAZARD EVALUATION REPORTS, OPEN FILE REPORT 98-15
7.5 MINUTE QUADRANGLES, EL MONTE, CA QUADRANGLE

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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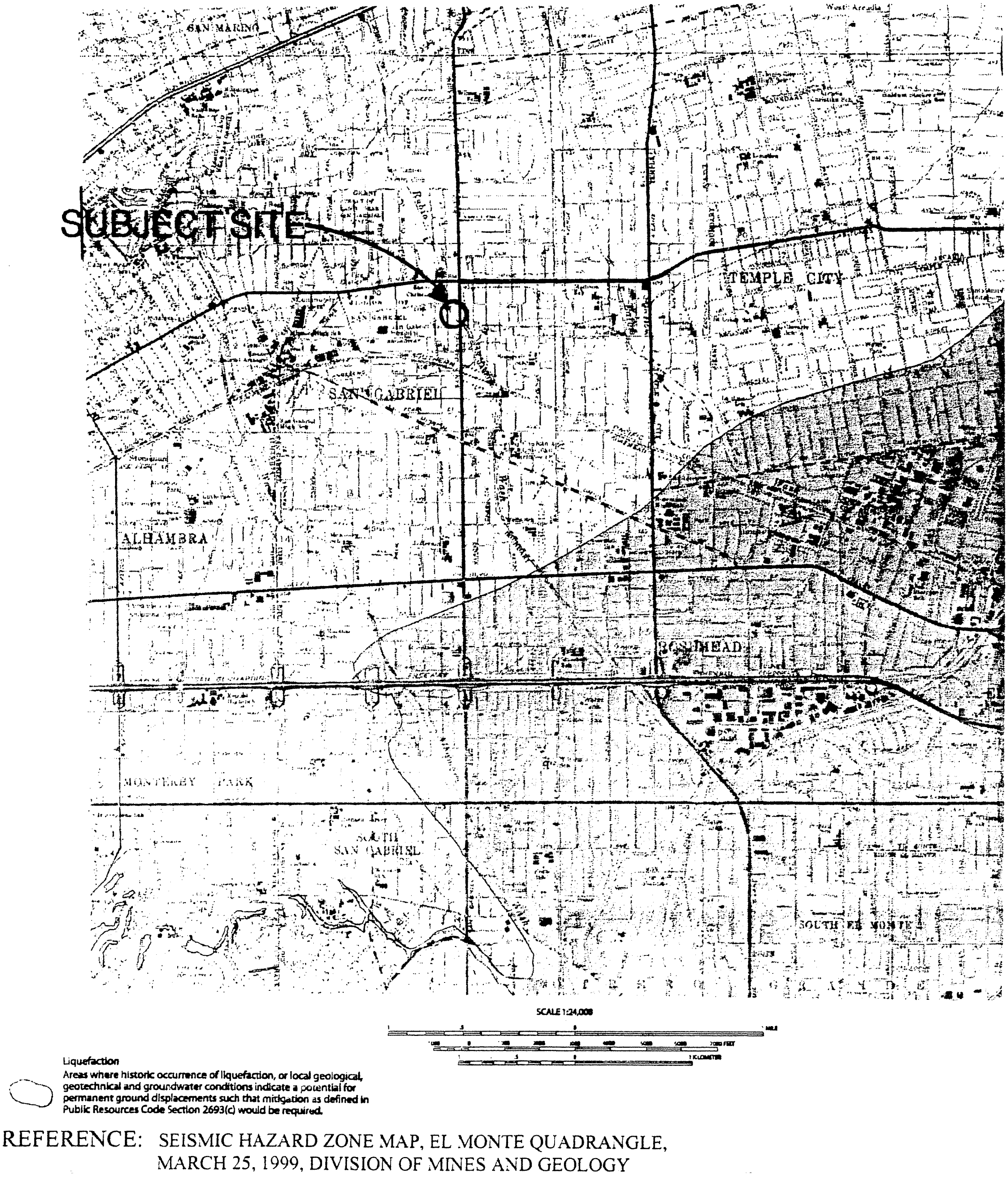

REFERENCE: SEISMIC HAZARD ZONE MAP, EL MONTE QUADRANGLE, MARCH 25, 1999, DIVISION OF MINES AND GEOLOGY

SEISMIC HAZARD ZONE MAP

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