DUE-DILIGENCE GEOTECHNICAL EXPLORATION FOR THE PROPOSED CITY HALL AND PARKING STRUCTURE NEWPORT BEACH, CALIFORNIA

Prepared for:

CITY OF NEWPORT BEACH

3300 Newport Boulevard
Newport Beach, California 92663

Project No. 602184-001

May 2, 2008

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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To: City of Newport Beach
3300 Newport Boulevard
Newport Beach, California 92663

Attention: Mr. David Webb

Subject: Due-Diligence Geotechnical Exploration for the Proposed City Hall and Parking Structure, Newport Beach, California

Leighton Consulting, Inc. is pleased to submit the results of our geotechnical exploration for the proposed project. Our services were provided in accordance with our proposal dated March 11, 2008 and your Notice to Proceed dated March 11, 2008. This report presents the findings from our exploration and provides recommendations to aid in the planning and design of the project.

The proposed project is deemed feasible from a geotechnical standpoint. We understand that a city hall building and a parking structure are planned. The locations of the buildings are not known at this time. The site is located on an elevated pad and we understand that the site will be graded

Our exploration indicated that the site is underlain by Quaternary age terrace deposits over bedrock. Depending on the future grading plan and location of the buildings, bedrock-terrace deposit transition may be encountered near the southern portion of the site. In order to minimize the potential of differential settlement caused by the stiffness variance between the terrace deposits and bedrock, overexcavation may be required. All buildings may be supported on conventional spread footings and slab-on-grade foundation systems. General recommendations for site grading, foundation design and other geotechnical aspects of the project are presented in this report. We recommend that a design level geotechnical exploration to be performed after the preliminary design of the buildings and grading plans are available.

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We appreciate the opportunity to be of service to you on this project. If you have any questions or if we can be of further assistance, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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Leighton

1.0 INTRODUCTION

1.1 Site Description and Proposed Improvements

The proposed city hall will be located on approximately 12 acres of vacant property north of the City's Central Library on Avocado Avenue, as shown on Figure 1. The site is located on an elevated pad between Avocado Avenue and MacArthur Boulevard with elevations ranging from 160 to 200 feet msl. The project site is bounded to the north by San Miguel Drive, to the east by MacArthur Boulevard, to the south by the Central Library and to the west by Avocado Avenue. Light vegetations are present within the southern half of the site while heavy vegetation and bushes are present within the northern area. A ravine is located between the northern and southern areas.

We understand that a city hall building and a parking structure are planned south of the ravine and a park is proposed north of the ravine. Based on our current understanding of the project plan, all structures will be on grade. It is our understanding that future site elevations are planned to be similar to Avocado Avenue and MacArthur Boulevard. As such, significant grading, with cuts up to 30 feet, may be performed during construction. Locations of the structures, grading plans and structural loadings are not available at this time.

1.2 Purpose and Scope of Investigation

The purpose of our geotechnical exploration was to provide geotechnical recommendations for planning and design of the proposed project. The exploration was performed in accordance with our proposal dated March 11, 2008 and your Notice to Proceed dated March 11, 2008. Our current exploration is limited to the areas south of the ravine.

The scope of this investigation included the following tasks:

- *Site Reconnissance* – Perform a site reconnissance to identify any geologic hazards and evaluate access for drilling equipment.
- *Background Review* – Perform a background review of readily available, relevant, geotechnical and geological literature pertinent to the site.

- *Pre-field Exploration Activities* - Contact Underground Service Alert (USA) to locate and mark existing underground utilities prior to our subsurface explorations.

- *Field Explorations* – Perform a subsurface exploration on March 27, 2008, consisted of five hollow-stem auger borings and seven test pits at the site. The hollow-stem auger borings were drilled to 30 feet to 50 feet below existing ground surface (bgs). The test pits were excavated to depths ranging from 10 feet to 16.5 feet. The hollow-stem borings and test pits were logged by a staff engineer and geologist, respectively. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a Modified-California ring sampler. Standard Penetration Tests (SPT) were conducted at selected intervals within the borings. Bulk samples of representative soil types were also collected.

  Logs of the hollow-stem auger borings and test pits are presented in Appendix A. Exploration test locations are shown on Figure 2.

- *Laboratory Tests* – Perform laboratory tests on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the subsurface materials onsite. Laboratory tests performed during this investigation include:

  - In situ moisture content and dry density;
  - Direct shear;
  - Expansion Index; and
  - Maximum dry density and moisture content.

  The results of the in situ moisture and density tests are shown on the boring logs in Appendix A. Results of all laboratory tests are presented in Appendix B.

- *Engineering Analysis* – Evaluate and analyze data obtained from our background review, field exploration, and laboratory testing program to develop recommendations for the proposed improvements.

- *Report Preparation* – Prepare a report summarizing the results of our exploration presenting our findings, conclusions and recommendations for the proposed project.

2.0 GEOLOGIC SETTING

The project site is on the northwestern flank of the northern San Joaquin Hills. The San Joaquin Hills lie within the northern part of the Peninsular Ranges geomorphic province which extends 900 miles southward from the Santa Monica Mountains to the tip of Baja California (Yerkes, et al., 1965). Regional tectonic activity has uplifted the San Joaquin Hills into an elongated arched fold (anticlinorium) trending to the northwest from San Juan Capistrano and Huntington Mesa. This anticlinal folding has occurred as this entire section of the southern California coast was uplifted by the San Joaquin Hills blind thrust fault (Grant et at., 1997, 1999, and 2002; Mueller et al., 1998). The San Joaquin Hills expose mainly Tertiary aged marine and non marine sedimentary rocks including thinly bedded shale, siltstone and sandstone of the upper Miocene-age Monterey Formation.

During Quaternary times, the eustatic fluctuations in sea level formed broad wave cut platforms upon which marine terrace sediments were deposited. Due to the continued uplift of the San Joaquin Hills some of these ancient stepped sequences of marine terrace deposits have been elevated above present day sea level. Erosion and grading activity have formed the present day landscape.

3.0 SUBSURFACE AND GROUNDWATER CONDITIONS

3.1 Subsurface Conditions

The majority of the site is underlain by terrace deposits over bedrock. Quaternary terrace deposits at the site consist of varying amounts of sand, silt and clay. In general, the terrace deposits are medium dense to very dense for granular soils and stiff to hard for cohesive soils. Tertiary age Monterey Formation bedrock was encountered beneath the terrace deposits. The bedrock at the site consists of competent sandstone and siltstone. Bedrock was encountered at depths ranging from 10 to 15 feet bgs. Based on the current understanding of the project with the finish site grade similar to the adjacent road grades, terrace deposit-bedrock transition may be encountered at the southern portion of the site.

Geologic cross-sections showing the subsurface conditions at the site are included on Figure 3. The subsurface stratigraphy is based on our observations from the borings and test pits, and our interpretation of the earth units between soil boring and test pit locations.

3.2 Groundwater Conditions

Groundwater was not encountered in any of the borings drilled to depths of 30 to 50 feet. Groundwater in the vicinity is expected to be at or near sea level, which is greater than 150 feet below the current site grade (LeRoy, 1990). However, perched water and seepage may occur within the terrace deposits, the contact between terrace deposits and underlying bedrock, and/or within sandstone units on the bedrock.

4.0 FAULTING, SEISMICITY AND POTENTIAL HAZARDS

4.1 Alquist-Priolo Earthquake Fault Zone and Nearby Faults

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (CDMG, 1977).

The closest fault to the site is the Newport Inglewood (Offshore) fault, located at approximately 2.4 miles from the site. San Joaquin Hills Blind Thrust is located less than 3.5 miles from the site. The San Andres fault is the largest fault in the region and is located approximately 52.7 miles from the site. Both active and potentially active faults found within a 62-mile (100 km) radius from the project site are listed in Appendix C.

4.2 Potential Seismic Hazards

Ground Shaking - The intensity of ground shaking resulting from an earthquake is generally characterized by using the Peak Horizontal Ground Acceleration (PHGA). To take into consideration the impact of regional faults, a probabilistic seismic hazard analysis was performed using the computer program FRISKSP (Blake, 2000) to estimate the PHGA that could occur at the site. This approach accounts for site-specific response characteristics, historical seismicity, and the geological characteristics of the regional faults under consideration. Three attenuation relationships (Abrahamson et al., 1997, Bozorgnia et al., 1999, and Sadigh et al., 1997) were used in the analysis. The results of the analyses suggest that the PHGA with a 2 percent probability of exceedance in 50 years (recurrence interval of 2,475 years) is approximately 0.70g. This level of ground motion is considered the Maximum Considered Earthquake (MCE) in accordance with the 2007 California Building Code (CBC). Results of the analyses are included in Appendix C.

Liquefaction Potential - Liquefaction is the loss of soil strength or stiffness due to a build up of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

The site is not located within a potential liquefaction hazard zone as delineated by the State of California (CDMG, 1998). The site is underlain predominantly by shallow terrace deposit over bedrock. The depth to bedrock at the site ranges from 15 feet to 20 feet, and no significant amount of loose granular soils were found within the terrace deposit. At such, we consider the potential of liquefaction at the site to be very low.

Seismically-Induced Settlement - Seismically-induced settlement occurs primarily within loose to moderately dense sandy soil due to a reduction in volume during and shortly after strong ground shaking. The majority of materials underlying the site consist of dense terrace deposits and bedrock. Accordingly, the potential for seismically-induced settlement is low.

Earthquake-Induced Lateral Spreading - Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along a gently sloping surface toward an unconfined area. Since the site has a low liquefaction potential and is relatively confined laterally, the potential for lateral spreading to occur at the site is considered low.

Seismically-Induced Landslides - The site is not located in an area mapped as potentially susceptible to seismically-induced landslides as shown on the Seismic Hazard Zones Map (CDMG, 1998). No significant slopes are located near the site. Therefore the potential of seismically induced landslides at the site is considered low.

Seismically-Induced Flooding - Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquake. Due to the absence of such structures near the site, the potential of flooding at the site is considered low.

Seiches and Tsunamis - Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on the lack of nearby enclosed water bodies near the site, the seiche risk at the site is considered negligible. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. The site has an elevation of over 160 feet above mean sea level, therefore the tsunami risk at the site is considered low.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The proposed project is feasible from a geotechnical standpoint provided that the recommendations presented in this report are properly incorporated in the planning, design and construction of the project. It is our understanding that future site elevation are planned to be similar to Avocado Avenue and MacArthur Boulevard. As such significant grading, with cuts up to 30 feet, may be performed during construction. Future buildings may be supported on bedrock, undisturbed terrace deposits or properly compacted fill.

Preliminary recommendations are provided for planning and design of the project. Additional investigation should be performed after the building footprint and grading plans are available to provide design level recommendations.

5.1 Site Grading

The bedrock-terrace deposits transition may be encountered near the southern portion of the site. If buildings are planned in this area, subgrade below the planned foundation should be overexcavated in order to provide uniform support for the buildings. The depth and lateral extent of overexcavation will be provided during design phase of the project. Groundwater is not expected to be encountered during grading. However, perched water or seepage may be encountered.

The onsite soil, free of organic material, cobbles, boulders, debris, or rock larger than 6 inches in largest dimension, is suitable for use as compacted fill. Import soil should be evaluated and tested by the geotechnical consultant before delivery to the site. In general, fill material should be low in expansion potential, non-organic, and free of debris or other deleterious materials. As the site is anticipated to be graded lower than the current elevation, we do not anticipate a significant amount of import soils will be required for the site.

5.2 Excavation Stability

All temporary excavations, including utility trenches, retaining wall excavations, and other excavations should be performed in accordance with project plans, specifications and all Occupational Safety and Health Administration (OSHA) requirements.

The terrace deposits and bedrock at the site should be readily excavated by conventional earth-moving equipment. Based on the nature of the on-site materials, excavations can be laid back in accordance with OSHA requirements before personnel are allowed to enter. Shoring may not be required if there is sufficient room for excavation lay back. It is the contractor's responsibility to ensure the stability of cuts, and the safety of all excavations.

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Shoring can be designed using the appropriate lateral earth pressures provided in Section 5.6.

The onsite soils within the proposed structural depths generally conform to OSHA soil Type B. OSHA regulations are applicable in areas with no restriction of surrounding ground deformations. Shoring should be designed for areas with deformation restrictions. The soil type should be verified or revised based on geotechnical observation and testing during construction, as soil classifications may vary over short horizontal distances. Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a minimum distance equivalent to the excavation height or 5 feet, whichever is greater, from the excavation unless the excavation is shored and these surcharges are considered in the design of the shoring system.

Additional shoring recommendations can be provided once the locations of the buildings and grading plans are available. Feasible shoring systems include cantilever shoring and soldier piles and lagging.

5.3 2007 California Building Code (CBC) Seismic Coefficients

This site is not located within a designated Alquist-Priolo Earthquake Fault Zone. However, strong ground shaking due to seismic activity is anticipated at the site. The following values are based on the 2007 CBC seismic design method. Additional seismic analyses may be necessary based on structural requirements.

California Building Code (2007) Seismic Parameters	
Site Class	C
Mapped Spectral Acceleration Parameter, S_s	1.783
Mapped Spectral Acceleration Parameter, S_1	0.653
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.3
Spectral Response Acceleration, S_{MS}	1.783
Spectral Response Acceleration, S_{M1}	0.849
Design Spectral Response Acceleration, S_{DS}	1.188
Design Spectral Response Acceleration, S_{D1}	0.566

5.4 Conventional Shallow Foundations

Upon completion of the recommended building pad preparation, the proposed structures may be supported on spread footings and a slab-on-grade system. Recommended bearing capacities will be dependant on the future foundation elevation and structural loadings of the buildings. A maximum net allowable soil bearing pressure of 3,000 psf for square pad footings and continuous strip footings can be used for preliminary design. This value may change once the final project plan is available.

The footings should have minimum widths of 2 feet and 1.5 feet for isolated square pad and continuous strip footings, respectively, with the top of the footing embedded at least 18 inches below the lowest adjacent grade. The soil bearing pressure may be increased by one-third for transient loads such as wind and seismic forces.

The post-construction total and differential static settlements will be provided once the grading plan, building layout and structural loadings are available.

Resistance to lateral loads will be provided by a combination of friction between the soil and foundation interface and passive pressure acting against the vertical portion of the footings. For calculating lateral resistance, a passive pressure of 350 psf per foot of depth to a maximum of 3,500 psf and a frictional coefficient of 0.35 may be used provided the foundations are supported within structural compacted fill as previously described. When combining frictional and passive resistance, the passive resistance should reduced by one-third. No safety factor has been incorporated in the recommended values for frictional and passive resistance.

5.5 Slab-on-Grade

At-grade floor slabs of the proposed structures may be designed and constructed as a slab-on-grade supported directly on undisturbed terrace deposits, properly compacted fill or bedrock. If bedrock-terrace deposit transition is encountered, the planned subgrade elevation should be overexcavated at least 3 feet and replaced with properly compacted fill. The structural engineer should design the slab and determine the required thickness and reinforcement based on structural load requirements.

5.6 Earth Retaining Structures

Any retaining structures planned at the site should be backfilled with granular, very low expansive soil and be constructed with a backdrain in accordance with the recommendations provided on Figure 3. The backdrain should be sloped at a minimum of one percent toward an approved non-erosive outlet. The following parameters may be used for the preliminary design of conventional retaining structures. These values may be revised based on the results of the design level exploration.

Condition	Equivalent Fluid Unit Weight (psf/ft)
Active	36 (Level Backfill)
	54 (2:1 Backfill)
At-Rest	56 (Level Backfill)
	82 (2:1 Backfill)
Passive	350 with a maximum of 3,500 psf
Coefficient of Friction	0.35

Unrestrained walls that are free to rotate or deflect may be designed using the active earth pressure. For restrained walls that are fixed against rotation, such as basement wall, the at-rest condition should be used. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time. We also recommend using the at-rest pressure for design of walls supporting settlement-sensitive structures, such as adjacent roadways and structures. The above-recommended lateral pressures were based on a soil total unit weight of 125 pounds per cubic foot (pcf). No factor of safety or load factor was applied to the lateral pressure values.

If the retaining structures are braced at the top or at specific design intervals and are constructed in a braced excavation, the earth pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25***H*** psf, where ***H*** is equal to the depth of the retaining structure being supported. Otherwise the retaining structure should be designed using the recommended at-rest pressure.

Backfill for retaining walls should be compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D1557. Relatively light construction equipment should be used to backfill the retaining walls.

Lateral pressures from other surcharge and superimposed loads (for example, from vehicle traffic and adjacent structures) should be added to the above recommended lateral earth pressures if the loads fall within a projected area of an imaginary line extended at an angle of 45 degrees from the wall foundation. Thirty percent of the surcharge load may be used for unrestrained walls and forty-five percent of the surcharge may be used for restrained walls.

Foundations for at-grade retaining walls may be designed for a maximum net allowable soils bearing pressure of 3,000 psf with a minimum embedment of 18 inches below the lowest adjacent grade.

5.7 Plan Review

The final grading and foundation plans should implement the recommendations presented in this report and should be reviewed by the project geotechnical consultant. Our recommendations may be revised, as necessary, based on future plans.

5.8 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. A design level geotechnical investigation should be performed once the building layout, grading plan and structural loadings are available. Our recommendations may be revised, as necessary, based on future plans.

5.9 Limitations

The conclusions and recommendations presented in this report have been based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either express or implied.

The conclusions and recommendations presented in this report have been based upon the subsurface conditions encountered at discrete and widely spaced locations and at specific intervals below the ground surface. Due to the inherent variance in soil conditions, variability may be encountered during construction. Where encountered during construction, such variances should be brought to our attention to determine the impact upon the recommendations presented in this report.

This report has been prepared for the use of our client for the project described in this report. The report may not be used by others without the written consent of our client and our firm.

6.0 REFERENCES

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Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

IIGER06075.0MRP

Proposed Newport Beach City Hall
Newport Beach,California

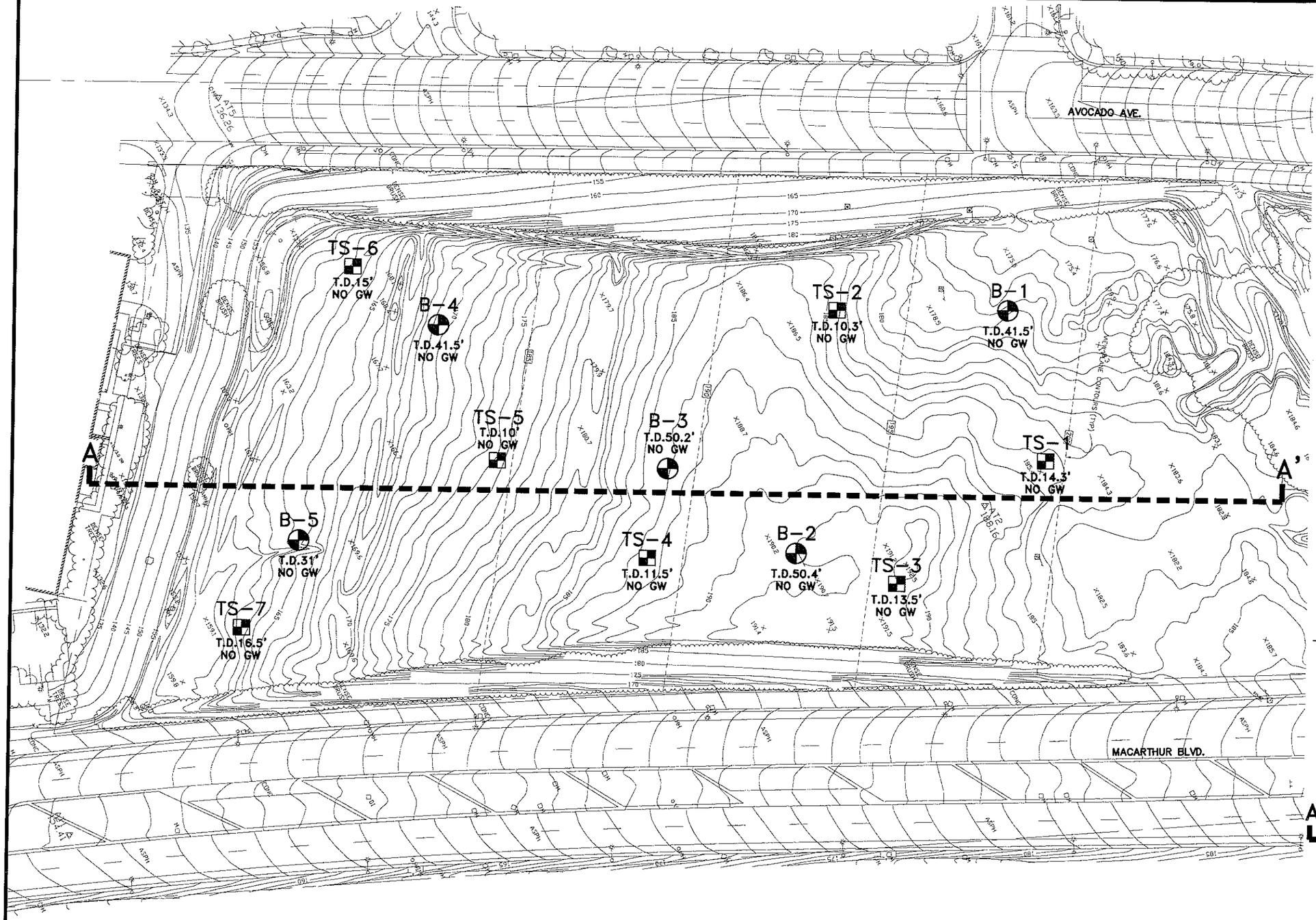
SITE LOCATION MAP

Project No.
602184-001

Date
April 2008

Figure 1

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd



LEGEND

B-5 (T.D.31.5' NO GW) — APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING WITH TOTAL AND GROUNDWATER DEPTH

TS-7 (T.D.16.5' NO GW) — APPROXIMATE LOCATION OF TEST PIT WITH TOTAL AND GROUNDWATER DEPTH

A — A' — LOCATION OF GEOLOGIC CROSS-SECTION

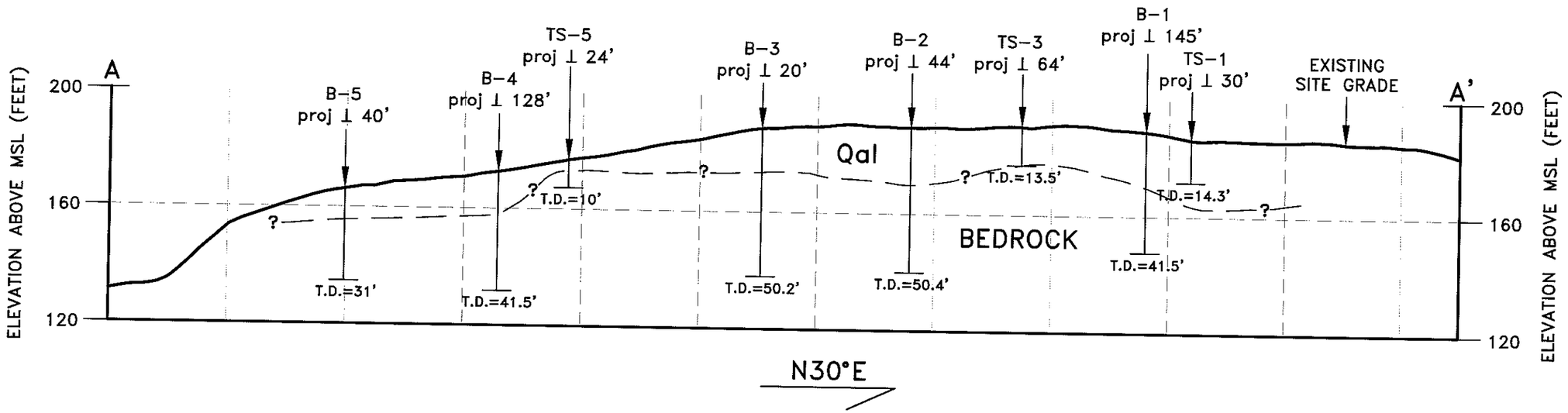
Figure 2

BORING AND TEST PIT LOCATION MAP

PROPOSED NEWPORT BEACH CITY HALL
NEWPORT BEACH, CALIFORNIA

Proj: 602184-001	Scale: 1"=80'	Date: 4/08	Leighton
Eng./Geol. VMC/ELB	Drafted By: BQT	CP By: BQT	

P:\DRAFTING\602184\001\OF_2008-04-02\FIGURE2.DWG (05-01-08 12:20:00PM) Plotted by: rbolen



LEGEND

B-5	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING SHOWING TOTAL DEPTH
TS-5	APPROXIMATE LOCATION OF TEST PIT SHOWING TOTAL DEPTH
Qal	QUATERNARY TERRACE DEPOSIT
BEDROCK	TERTIARY AGE MONTEREY FROMATION

Figure 3

GEOLOGIC CROSS-SECTION A-A'

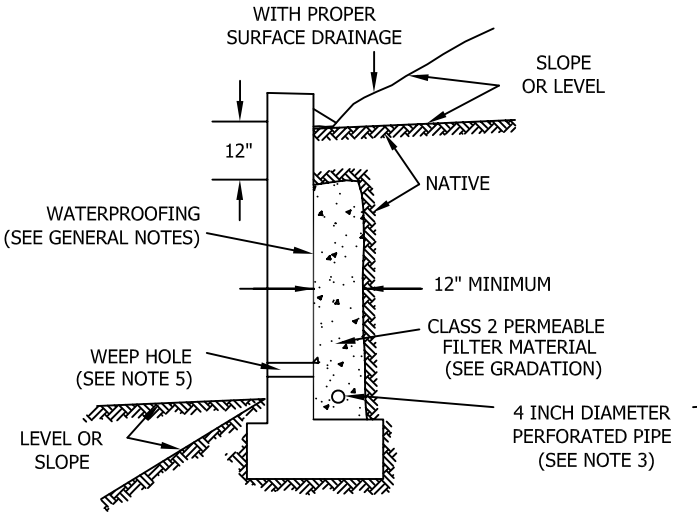
PROPOSED NEWPORT BEACH CITY HALL
NEWPORT BEACH, CALIFORNIA

Proj: 602184-001	Scale: V:1"=40' H:1"=80'	Date: 4/08	Leighton
Eng./Geol. VMC/ELB	Drafted By: RB	CP By: BQT	

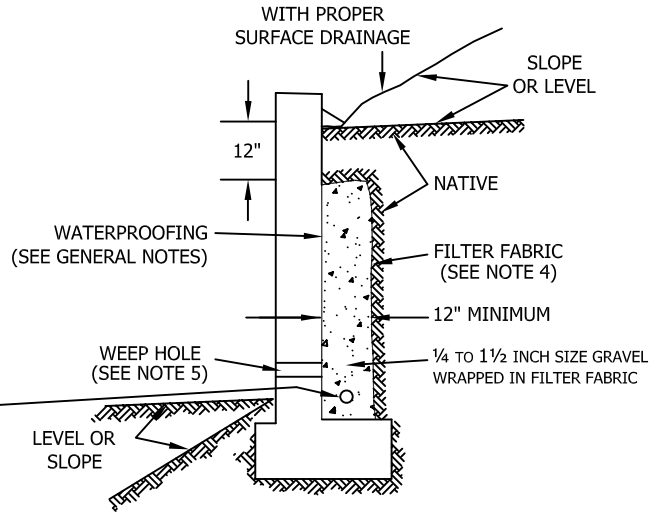
P:\DRAFTING\602184\001\OF_2008-04-02\FIGURE3.DWG (05-01-08 4:37:49PM) Plotted by: rbolen

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

P:\Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

GEOTECHNICAL BORING LOG B-1

Date 3-27-08 **Sheet** 1 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 179' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
179	0							SC	**Quaternary Terrace deposits:**	
									@ 0': Sandy CLAY/Clayey SAND, orange brown, damp, fine grained sand.	
175	5			R-1	8 15 27	118.7	11.9	CL	@ 5': Sandy CLAY, orange brown, damp, very stiff, fine grained sand.	
170	10			R-2	8 9 12	95.9	3.4	SM	@ 10': Silty SAND, light yellow brown, damp, medium dense, fine grained sand.	
165	15			S-1	4 21 37			SM	@ 15': Silty SAND, light yellow brown, damp, dense, fine grained sand, trace of well rounded gravel (pebbles).	
160				Bag-1						
	20			R-3	15 50/5"	101.0	3.4		**Bedrock: Monterey Formation:**	DS
									@ 20': SANDSTONE, light yellow brown, damp, hard, fine grained sand, very poorly cemented, friable, breaks down in hand.	
155	25			S-2	36 50/4"				@ 25': SANDSTONE, light yellow brown, damp, hard, fine grained sand, poorly cemented, friable.	
150	30									

SAMPLE TYPES:

S SPLIT SPOON — G GRAB SAMPLE
R RING SAMPLE — C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR — CS CORROSION SUITE — AL ATTERBERG LIMITS
MD MAXIMUM DENSITY — COL COLLAPSE POTENTIAL — EI EXPANSION INDEX
CN CONSOLIDATION — -200 200 WASH — RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-1

Date 3-27-08 **Sheet** 2 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 179' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
	30			R-4	50/6"	101.2	2.4		@ 30': SANDSTONE, light yellow brown, damp, very dense, fine grained sand.	
145										
	35			S-3	50/2"				@ 35': SANDSTONE, light yellow brown, damp, very dense, fine grained sand.	
140										
	40			R-5	4 25 39	102.0	10.2		@ 40': Silty SANDSTONE, mottled olive grey and yellow to orange brown, damp, hard, fine grained sand, highly fractured, fractures well healed with very fine grained sand with trace clay, moderately cemented, friable, thinly bedded, trace siltstone rock fragments.	
135									Total depth of boring: 41.5 feet No groundwater was encountered during drilling Borehole backfilled with soil cuttings and tamped	
	45									
130										
	50									
125										
	55									
120										
	60									

SAMPLE TYPES:

S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-2

Date 3-27-08 **Sheet** 1 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 191' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
190	0							CL	**Quaternary Terrace Deposits:** @ 0': Sandy CLAY, dark brown, damp, fine grained sand.	
185	5			R-1	3 5 8	100.6	14.0	SM/SC	@ 5': Silty SAND to Clayey SAND, light to dark brown, damp, loose, fine grained sand.	
180	10			R-2	10 15 22	100.3	6.2	SP	@ 10': SAND, yellow brown, damp, medium dense, fine grained sand.	DS
				Bag-1						
175	15			R-3	26 35 50	67.3	27.0	CL/SM	@ 15': Sandy CLAY, mottled yellow brown to dark reddish brown, damp, fine to medium grained sand grades to Silty SAND, light yellow brown to red brown sand, damp, very dense, fine grained sand.	
170	20			S-1	20 33 42				**Bedrock: Monterey Formation:** @ 20': SANDSTONE, orange brown to light yellow brown, damp, moderately hard, highly weathered, heavily oxidized, very poorly cemented, friable, breaks down in hand, fine grained sand.	
165	25			R-4	6 30 50/5"	102.6	2.7		@ 25': SANDSTONE, yellow brown, damp, hard, fine grained sand poorly cemented, friable.	
	30									

SAMPLE TYPES:
S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-2

Date 3-27-08 **Sheet** 2 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 191' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
									Logged By SP **Sampled By** SP	
160	30			S-2	37 50/5"				@ 30': SANDSTONE, light yellow brown, damp, hard, fine grained sand, massive, poorly cemented, friable.	
155	35			R-5	50/4"	114.2	2.7		@ 35': SANDSTONE, light yellow brown, damp, hard, fine grained sand, massive, poorly cemented, friable.	
150	40			S-3	35 50/4"				@ 40': SANDSTONE, light yellow brown, damp, hard, fine grained sand, massive, poorly cemented, friable.	
145	45			R-6	50/4"	99.0	4.5		@ 45': SANDSTONE, light yellow brown, damp, hard, fine grained sand, massive, poorly cemented, friable.	
140	50			S-4	50/5"				@ 50': SANDSTONE, light orange brown, damp, hard, fine grained sand, trace siltstone rock fragments.	
135	55								Total depth of boring: 51.5 feet No groundwater was encountered during drilling Borehole backfilled with soil cuttings and tamped	
	60									

SAMPLE TYPES:

S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-3

Date 3-27-08 **Sheet** 1 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 187' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
185	0							CL	**Quaternary Terrace Deposits:** @ 0': Sandy CLAY, dark brown, moist, fine grained sand.	
	5			R-1	7 15 31	105.4	7.1	SC	@ 5': Silty SAND to Clayey SAND, dark brown, damp, medium dense, fine grained sand.	
180	10			R-2	27 50/6"	105.4	5.5	SM	@ 10': Silty SAND, light orange brown, damp, very dense, fine grained sand, trace clay pods in matrix.	
175	15			S-1	20 33 50				**Bedrock: Monterey Formation:** @ 15': SANDSTONE, light orange brown, damp, moderately hard, fine grained sand, very poorly cemented, highly weathered, highly oxidized, friable, breaks down in hand.	
170	20			R-3	15 16 41	95.1	19.1		@ 20': SANDSTONE, light orange brown to olive grey with orange mottling, damp, hard, fine grained sand, poorly cemented, friable, highly fractured, fractures well healed with greyish white, fine grained, sandy clay, some siltstone rock fragments and lenses of greyish white sand.	
165	25			S-2	50/6"				@ 25': SANDSTONE, light yellow brown, damp, hard, fine grained sand, poorly cemented, friable.	
160	30			Bag-1						

SAMPLE TYPES:
S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-3

Date 3-27-08 **Sheet** 2 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 187' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
									Logged By SP **Sampled By** SP	
155	30			R-4	31 50/1"	102.1	9.2		@ 30': SANDSTONE, light yellow brown, damp, hard, fine grained sand, some silt, poorly cemented, friable.	DS
150	35			S-3	20 50/6"				@ 35': SANDSTONE, light yellow brown, damp, hard, fine grained sand, trace gravel.	
145	40			R-5	11 50/6"	97.4	10.2		@ 40': SANDSTONE, light yellow brown, damp, hard, fine grained sand, trace gravel, poorly cemented, friable, iron nodules and concretions.	
140	45			S-4	15 12 25				@ 45': SANDSTONE/CLAYSTONE, dark brown with yellow and grey mottling, damp, hard, fine grained sand, thinly bedded, poorly cemented, friable. @ 50': SANDSTONE/CLAYSTONE, mottled dark brown with yellow and grey, damp, hard, fine grained sand, chaotic assemblage of sandstone and claystone, disrupted bedding, poorly cemented, trace fine gravel.	
135	50			R-6	50/4"				Total depth of boring: 50.5 feet No groundwater was encountered during drilling Borehole backfilled with soil cuttings and tamped	
130	55									
	60									

SAMPLE TYPES:
S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-4

Date 3-27-08 **Sheet** 1 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 170' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
170	0			Bag-1				CL	**Quaternary Terrace Deposits:** @ 0': Sandy CLAY, dark brown, damp, fine grained sand.	
165	5			R-1	22 25 32	120.1	6.8	SC	@ 5': Clayey Gravelly SAND, dark brown, damp, hard/dense, fine grained sand, fine gravel.	
									@ 7': Encounter gravel bed.	
160	10			R-2	41 50/5"	100.7	5.8	SM	@ 10': Silty SAND, orange brown, moist, very dense, fine grained sand some siltstone rock fragments.	
155	15			S-1	21 30 35				**Bedrock: Monterey Formation:** @ 15': SANDSTONE, light orange brown, damp, moderately hard, fine grained sand, very poorly cemented, friable, breaks down in hand.	
150	20			R-3	13 50/5"	98.3	5.8		@ 20': SANDSTONE, orange brown, damp, hard, fine grained sand, fractured, fractures well healed with dark reddish brown clay, poorly cemented, friable.	
				Bag-2						
145	25			S-2	17 28 33				@ 25': SANDSTONE, hard, yellow grey, damp, fine grained sand, poorly cemented, friable.	
140	30									

SAMPLE TYPES:
S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-4

Date 3-27-08 **Sheet** 2 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 170' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
									Logged By SP **Sampled By** SP	
	30			R-4	37 50/3"	89.3	22.2		@ 30': Clayey SANDSTONE, orange brown, damp, hard, fine grained sand, poorly cemented, friable, contains olive brown siltstone rock fragments.	
135	35			S-3	7 22 22				@ 35': SANDSTONE interbedded with shale, damp, hard, fine grained sand.	
130	40			R-5	9 36 45	89.7	26.3		@ 40': SANDTONE, grey brown with yellow motting, damp, hard, fine grained sand, massive to thinly bedded clayey sandy silt, slightly fractured, fractures well healed with dark brown clay, some manganese development along fractures.	
125	45								Total depth of boring: 41.5 feet No groundwater was encountered during drilling Borehole backfilled with soil cuttings and tamped	
120	50									
115	55									
110	60									

SAMPLE TYPES:

S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-5

Date 3-27-08 **Sheet** 1 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 165' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By SP; Sampled By SP)	Type of Tests
165	0							CL	**Quaternary Terrace Deposits:** @0': Sandy CLAY, dark brown, moist, fine grained sand.	
160	5			R-1	11 20 35	123.8	10.7	CL	@ 5': Sandy CLAY to Clayey SAND, dark brown, moist, dense, fine grained sand.	
155	10			R-2	9 15 38	82.3	34.9		**Bedrock: Monterey Formation:** @ 10': SANDSTONE to thinly bedded Clayey SILTSTONE, olive orange brown, damp, hard, fine grained sand, thinly bedded siltstone within massive sandstone, sandstone moderately fractured, fractures well healed with iron oxide.	
150	15			R-3	11 21 50/3"	82.3	33.6		@ 15': SANDSTONE to thinly bedded Clayey SILTSTONE, olive orange brown, damp, hard, fine grained sand, alternating thin beds of sandstone and siltstone, highly weathered and oxidized, moderate fracturing, fractures well healed with iron oxide and manganese.	
145	20			S-1	6 7 11				@ 20': SANDSTONE, light orange brown, damp, moderately hard, fine grained sand, trace amount of clay in matrix, poorly cemented, friable, highly oxidized, massive.	
140	25			R-4	6 27 33	108.0	10.8		@ 25': SANDSTONE, orange brown, moist, dense, fine grained sand, poorly cemented, highly oxidized, friable, massive.	
135	30								@ 30': SANDSTONE, white grey, damp, hard, fine grained sand, poorly cemented, friable.	

SAMPLE TYPES:
S SPLIT SPOON G GRAB SAMPLE
R RING SAMPLE C CORE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR CS CORROSION SUITE AL ATTERBERG LIMITS
MD MAXIMUM DENSITY COL COLLAPSE POTENTIAL EI EXPANSION INDEX
CN CONSOLIDATION -200 200 WASH RV R-VALUE
SU SULFATE CONTENT

GEOTECHNICAL BORING LOG B-5

Date 3-27-08 **Sheet** 2 **of** 2

Project Proposed Newport Beach City Hall **Project No.** 602184-001

Drilling Co. Martini Drilling Corp. **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lb Auto-Hammer **Drop** 30"

Elevation Top of Hole 165' **Location** See Boring and Test Pit Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S		S-2	25 50/6"				Logged By SP Sampled By SP	
130	35								Total depth of boring: 31 feet No groundwater was encountered during drilling Borehole backfilled with soil cuttings and tamped	
125	40									
120	45									
115	50									
110	55									
105	60									

SAMPLE TYPES:

S	SPLIT SPOON	G	GRAB SAMPLE
R	RING SAMPLE	C	CORE SAMPLE
B	BULK SAMPLE		
T	TUBE SAMPLE		

TYPE OF TESTS:

DS	DIRECT SHEAR	CS	CORROSION SUITE	AL	ATTERBERG LIMITS
MD	MAXIMUM DENSITY	COL	COLLAPSE POTENTIAL	EI	EXPANSION INDEX
CN	CONSOLIDATION	-200	200 WASH	RV	R-VALUE
SU	SULFATE CONTENT				

Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	185'	Trench No.	TS-1
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Test Pit Location Map		

GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES				
			U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': CLAY with sand, dark brown, moist, stiff, fine grained sand, abundant roots within the upper 2.5 feet. @3': Silty SAND, mottled brown, grey and orange brown, moist, fine grained sand, trace amount of clay. @8': Silty SAND, brown, moist, fine grained sand. @12': SAND, mottled white and tan brown, damp, fine grained sand, noncohesive, some silt. Total depth: 14.3 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.	Quaternary Terrace Deposits	CL SM SM SP	Bag-1 @ 5'-6'			

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: Flat TREND: N45W

Log of Trench No. TS-1

Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	185'	Trench No.	TS-2
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map		

GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': Silty SAND with clay, brown, dry, fine grained sand, abundant roots.	Quaternary Terrace	SM				
	@1.3': CLAY, dark brown, moist, some fine grained sand.	Deposits	CL				
	@2.7': SAND with silt, light brown, moist, fine grained sand.						
	@7.8': Silty SAND, mottled red brown and grey, moist, fine to medium grained sand.		SP	Bag-1 @			
	@8': Encounter well rounded cobbles.		SM	3'-5'			
	@9': SANDSTONE, light yellow, moist, very dense, fine grained sand, massive, thin fractures well healed with brown clay.	Bedrock		Bag-2 @ 9'-10'			MD
	Total depth: 10.3 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.						

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: Flat TREND: N60W

Log of Trench No. TS-2

Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	191'	Trench No.	TS-3
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map		

GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': CLAY, dark brown, moist, some fine grained sand, abundant roots within the upper foot.	Quaternary Terrace Deposits	CL	Bag-1 @ 0'-2'			
	@2.8': Silty SAND, tan brown mottled with grey brown silty clay, moist, fine grained sand, some fine angular siltstone gravel.		SM	Bag-2 @ 3'-5'			
	@6': Clayey SAND, brown, moist, fine grained sand.		SC				
	@8.5': SAND, tan brown, damp, fine grained sand, non-cohesive.		SP				
	@11': Silty SAND, brown, moist, fine grained sand.		SM				
	@13': SANDSTONE, white and orange brown, fine grained sand, interbedd with thin beds of siltstone.	Bedrock					
	Total depth: 13.5 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.						

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: Flat TREND:N40W

Log of Trench No. TS-3

Project Name:	Newport Beach City Hall	Logged by:	CDL			ENGINEERING PROPERTIES				
Project Number:	602184-001	Elevation:	193'	Trench No.	TS-4					
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map							
GEOLOGIC ATTITUDES	DATE: March 27, 2008	DESCRIPTION:		GEOLOGIC UNIT		U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': CLAY, dark brown, very moist, some fine grained sand, abundant roots within the upper foot.			Quaternary Terrace Deposits		CL				
	@1.6': Silty SAND, light grey mottled with orange brown, moist, fine grained sand, oxidized.					SM	Bag-1 @ 5'-6'			
	@6.5': Encounter well rounded cobbles.					GP				
	@9': SANDSTONE, light tan brown, moist, very dense, fine grained sand, thin to thick interbeds of white sandstone.			Bedrock						
	Total depth: 11.5 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.									
GRAPHIC PRESENTATION		SCALE: 1" = 5'		SURFACE SLOPE: Flat		TREND: N50W				

Log of Trench No. TS-4

Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	172'	Trench No.	TS-5
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map		

GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': Silty SAND, tan white, dry, fine grained sand, abundant roots.	Quaternary Terrace	SM				
	@0.4': CLAY, dark brown, damp, fine grained sand, trace roots, blocky paleosol fracturing.	Deposits	CL				
	@1.6': Clayey SAND, dark brown, damp, fine grained sand.		SC				
	@2.5': Gravelly Silty SAND, brown, moist, medium grained sand, large angular and well rounded gravel.		SM				
Contact: N35E 5W	@3.7': Interbedded fine grained, white tan SANDSTONE and oxidized orange brown SILTSTONE, moist.	Bedrock					
	@7.5': Encounter small boulders.			Bag-1 @ 5'-6'			
	@8': SANDSTONE, tan white, damp, fine grained sand, friable, moderately cemented.						
	Total depth: 10.0 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.						

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: 5H:1V TREND: N50E

Log of Trench No. TS-5

Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	163'	Trench No.	TS-6
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map		

GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES				
			U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': Silty SAND, brown, dry, fine grained sand, abundant roots. @0.9': Sandy CLAY, dark brown, dry, fine grained sand. @3': Silty SAND, tan brown, dry, fine grained sand, well cemented. @8.5': End of well cemented zone. @11': Silty SAND, moist, fine to medium grained sand. Total depth: 15.0 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.	Quaternary Terrace Deposits	SM CL SM SM				

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: Flat TREND: N50W

Log of Trench No. TS-6

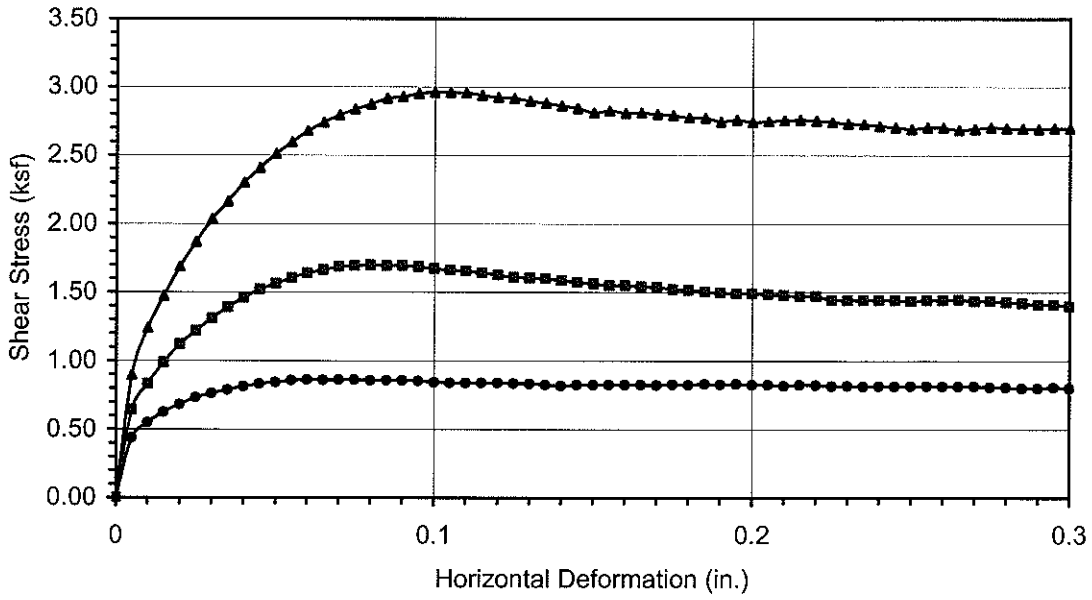
Project Name:	Newport Beach City Hall	Logged by:	CDL		
Project Number:	602184-001	Elevation:	161'	Trench No.	TS-7
Equipment:	Backhoe with 24-inch bucket	Location:	See Boring and Trench Location Map		

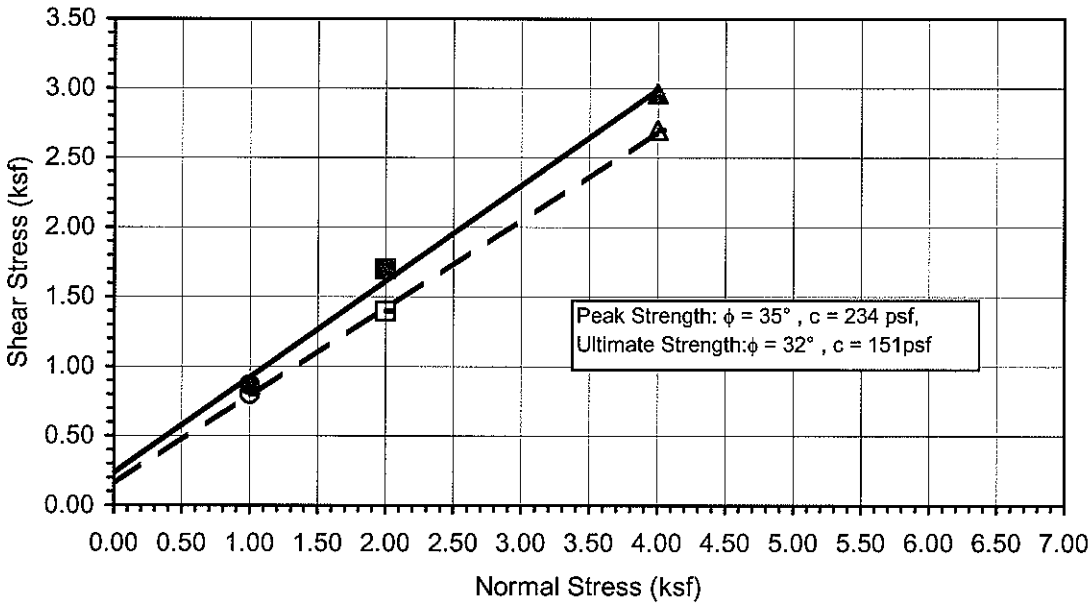
GEOLOGIC ATTITUDES	DATE: March 27, 2008 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES				
			U.S.C.S.	Sample No.	Moisture (%)	Density (pcf)	Other Tests
	@0': Silty SAND, light brown, fine grained sand, dry, abundant roots. @1.5': Silty SAND, dark brown, moist, fine grained sand, some roots. @2.3': Silty SAND, light brown, dry, fine grained sand. @3.1': Clayey SAND, dark brown, moist, fine grained sand. @5': Sandy CLAY, mottled dark brown and light brown, moist, fine grained sand, @9': Clayey SAND, brown, moist, fine grained sand. @11': Silty SAND, brown, moist, fine grained sand. @14': Encounter small boulders, well rounded. Total depth: 16.5 feet, no groundwater was encountered, the test pit was backfilled and the surface compacted by rubber tire equipment.	Quaternary Terrace Deposits	SM SM SM SC CL SC SM	Bag-1 @ 0'-5' Bag-2 @ 5'-6'			E.I.

GRAPHIC PRESENTATION SCALE: 1" = 5' SURFACE SLOPE: Flat TREND: N55W

Log of Trench No. TS-7

APPENDIX B





Boring No.	**B-1**
Sample No.	**R-3**
Depth (ft)	**20**
Sample Type: Drive	
Soil Identification: Yellowish brown silty sand'stone' (SM)	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.864	■ 1.699	▲ 2.959
Shear Stress @ End of Test (ksf)	○ 0.801	□ 1.398	△ 2.698
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.40	3.40	3.40
Dry Density (pcf)	99.5	101.2	102.3
Saturation (%)	13.3	13.8	14.2
Soil Height Before Shearing (in.)	0.9988	0.9982	0.9964
Final Moisture Content (%)	19.5	20.0	18.3

Leighton

DIRECT SHEAR TEST RESULTS

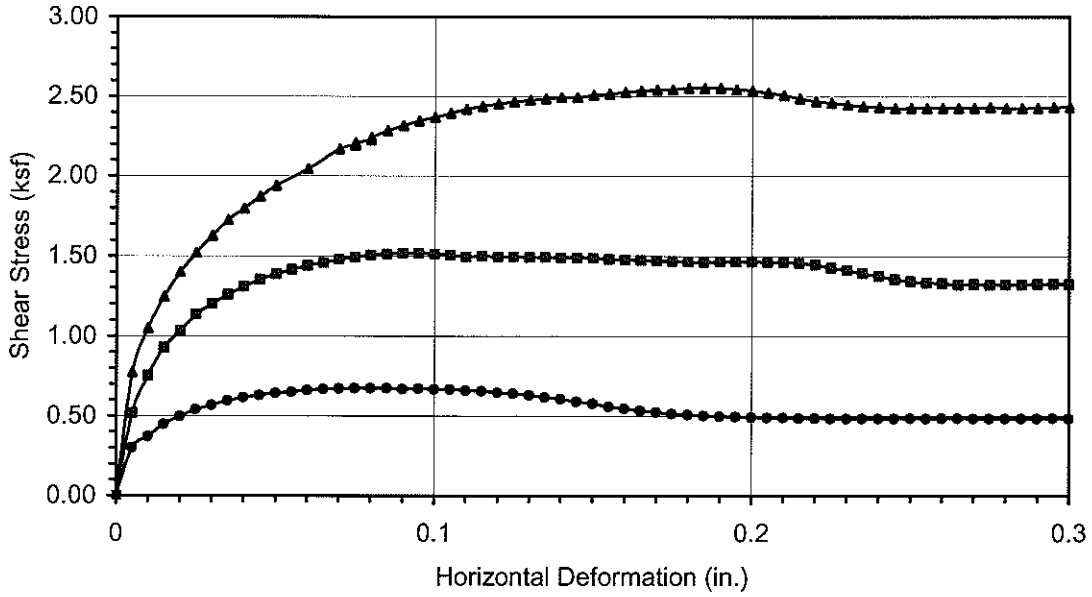
Consolidated Undrained

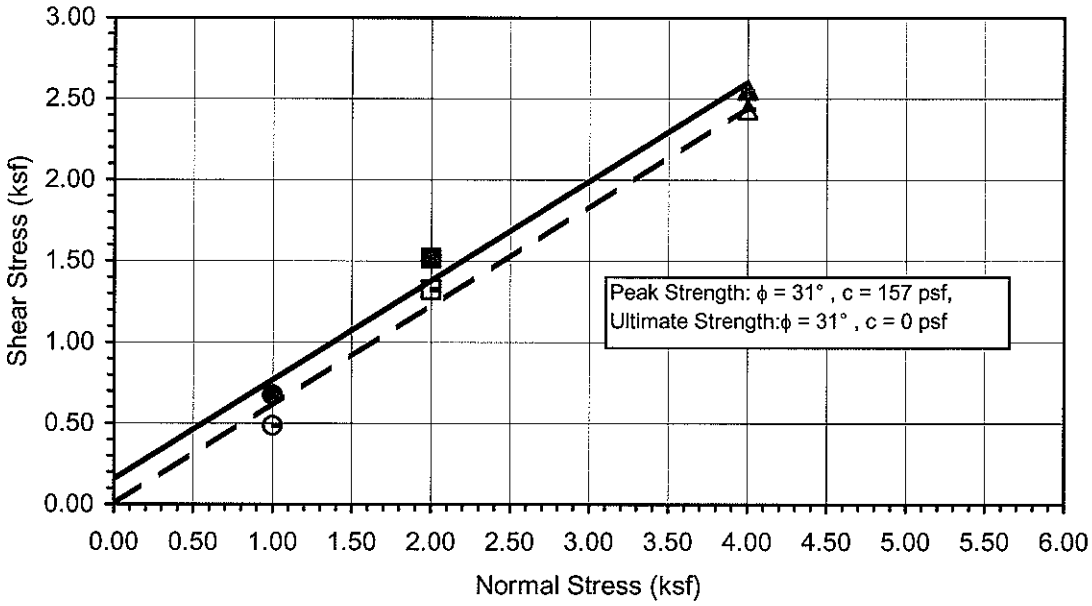
Project No.: 602184-001

Newport Beach City Hall

04-08

DS B-1 R-3





Boring No.	**B-2**
Sample No.	**R-2**
Depth (ft)	**10**
Sample Type: Drive	
Soil Identification: Yellowish brown sandy silt s(ML)	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.675	■ 1.518	▲ 2.554
Shear Stress @ End of Test (ksf)	○ 0.484	□ 1.324	△ 2.434
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.21	6.21	6.21
Dry Density (pcf)	101.4	99.8	99.6
Saturation (%)	25.4	24.4	24.2
Soil Height Before Shearing (in.)	0.9931	0.9822	0.9842
Final Moisture Content (%)	19.9	20.7	20.9

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DIRECT SHEAR TEST RESULTS

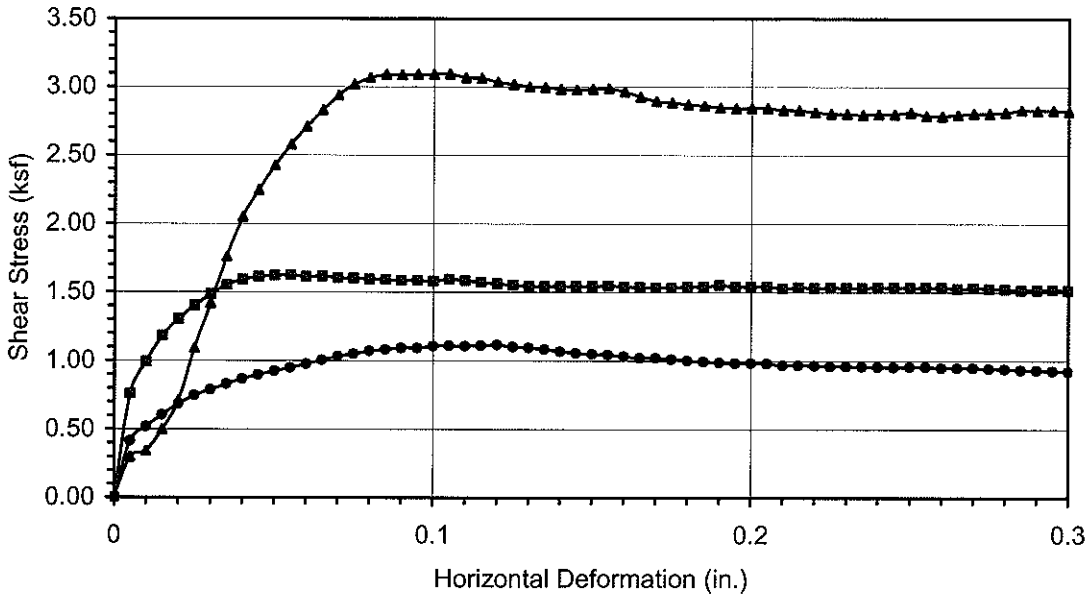
Consolidated Undrained

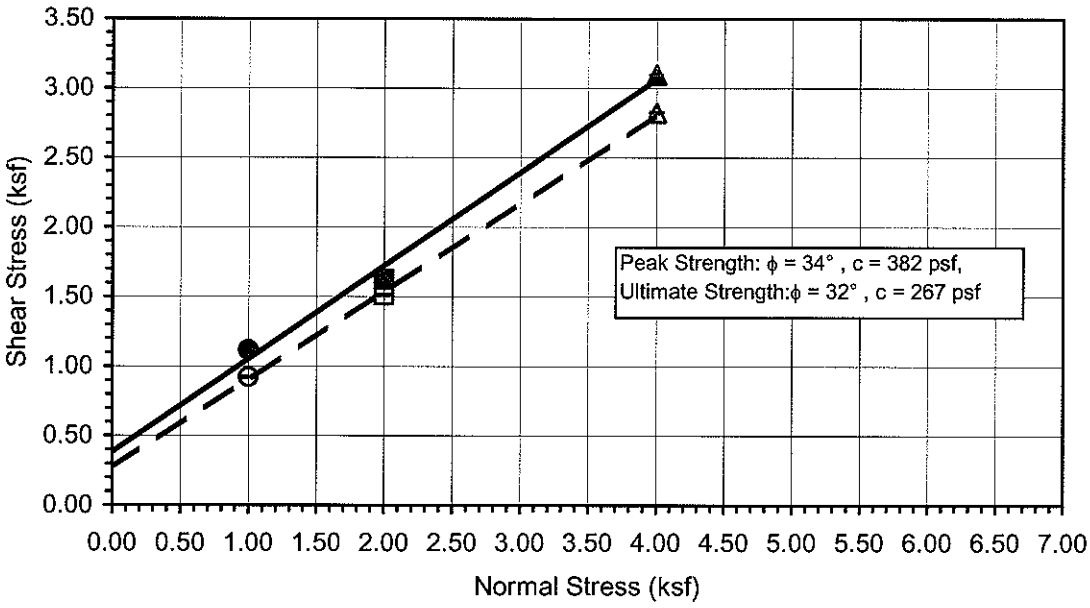
Project No.: 602184-001

Newport Beach City Hall

04-08

DS B-2 R-2





Boring No.	**B-3**
Sample No.	**R-4**
Depth (ft)	**30**
Sample Type: Drive	
Soil Identification: Yellowish brown silty clay (CL-ML)	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.116	■ 1.625	▲ 3.094
Shear Stress @ End of Test (ksf)	○ 0.921	□ 1.512	△ 2.821
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.24	9.24	9.24
Dry Density (pcf)	97.4	102.3	106.7
Saturation (%)	34.2	38.5	43.0
Soil Height Before Shearing (in.)	0.9847	0.9772	0.9734
Final Moisture Content (%)	30.8	27.4	26.9

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DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 602184-001

Newport Beach City Hall

04-08

DS B-3 R-4

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Newport Beach City Hall	Tested By:	G. Berdy	Date:	04/10/08
Project No. :	602184-001	Checked By:	LF	Date:	04/11/08
Boring No.:	TS-7	Depth (ft.)	6-7		
Sample No. :	Bag-2				
Soil Identification:	Olive brown sandy lean clay s(CL)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0280
Wt. Comp. Soil + Mold	(g)	587.10	432.30
Wt. of Mold	(g)	190.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	798.10	622.60
Dry Wt. of Soil + Cont.	(g)	725.50	551.00
Wt. of Container	(g)	0.00	190.30
Moisture Content	(%)	10.01	19.85
Wet Density	(pcf)	119.7	126.8
Dry Density	(pcf)	108.8	105.8
Void Ratio		0.549	0.593
Total Porosity		0.355	0.372
Pore Volume	(cc)	73.4	79.2
Degree of Saturation (%) [S_{meas}]		**49.2**	90.4

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
04/10/08	14:10	1.0	0	0.0725
04/10/08	14:20	1.0	10	0.0720
Add Distilled Water to the Specimen				
04/10/08	14:25	1.0	5	0.0845
04/11/08	7:30	1.0	1030	0.1005
04/11/08	8:39	1.0	1099	0.1005

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**29**

Leighton

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Newport Beach City Hall	Tested By :	G. Berdy	Date:	04/11/08
Project No.:	602184-001	Input By :	J. Ward	Date:	04/14/08
Boring No.:	TS-2	Depth (ft.)	9-10		
Sample No. :	Bag-2				
Soil Identification:	Olive yellow poorly-graded sand with silt (SP-SM)				

Preparation Method: [X] Moist [ ] Dry

[X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03330 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3736.0	3794.0	3861.0	3842.0		
Weight of Mold (g)	1885.0	1885.0	1885.0	1885.0		
Net Weight of Soil (g)	1851.0	1909.0	1976.0	1957.0		
Wet Weight of Soil + Cont. (g)	353.00	354.60	374.20	416.50		
Dry Weight of Soil + Cont. (g)	330.70	326.20	337.70	367.70		
Weight of Container (g)	53.90	54.40	54.30	54.20		
Moisture Content (%)	8.06	10.45	12.88	15.57		
Wet Density (pcf)	122.5	126.4	130.8	129.6		
Dry Density (pcf)	113.4	114.4	115.9	112.1		

Maximum Dry Density (pcf) **116.0** **Optimum Moisture Content (%)** **13.0**

PROCEDURE USED

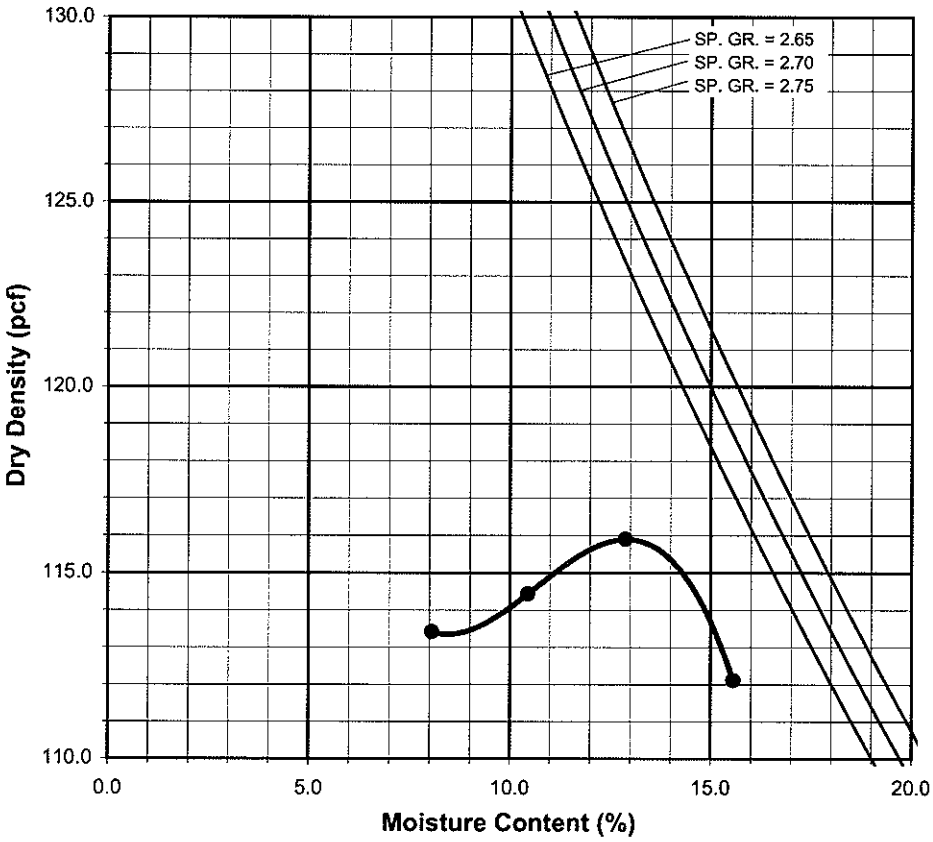
[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
[ ]
GR:SA:FI

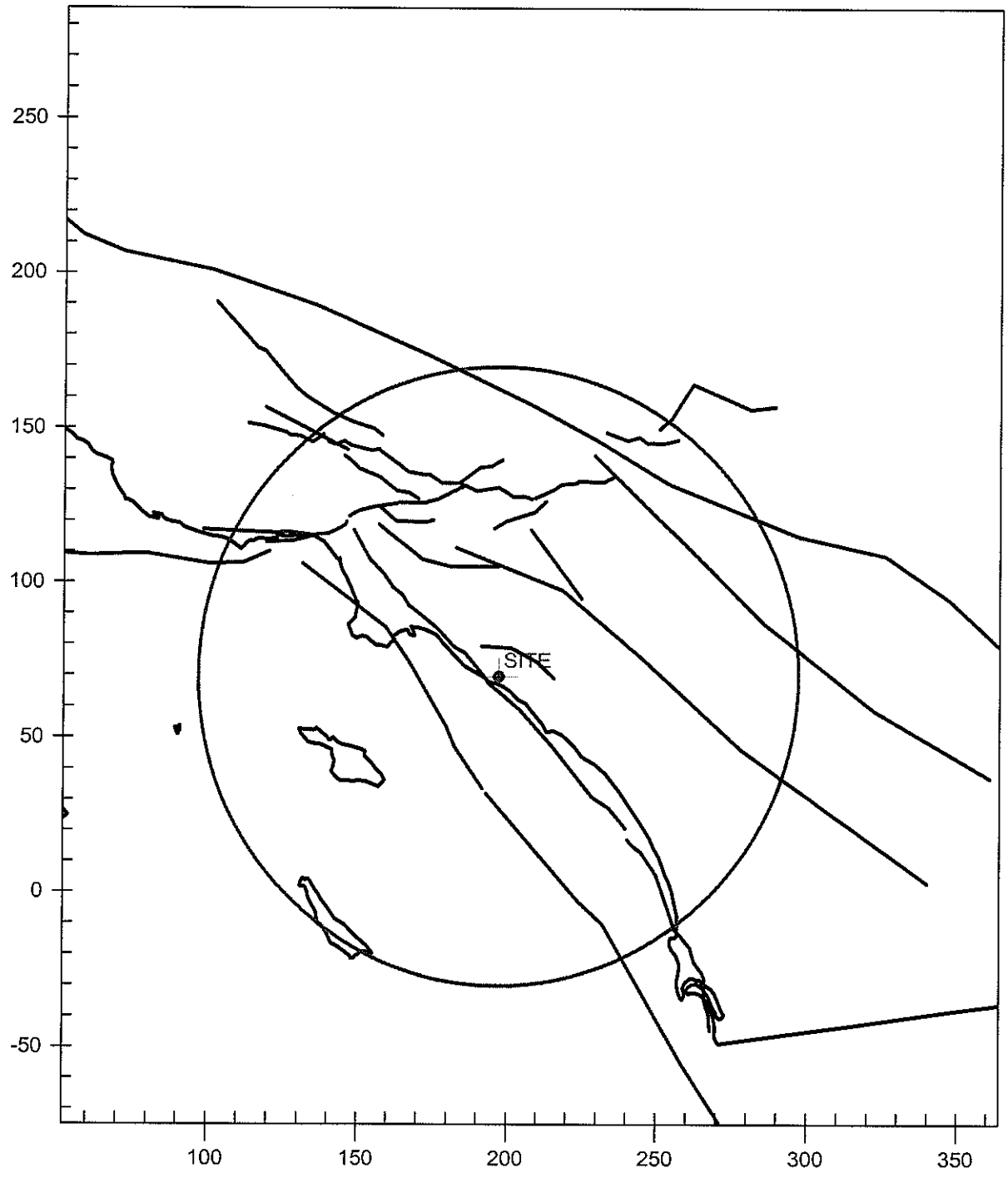
Atterberg Limits:
[ ]
LL,PL,PI



APPENDIX C

CALIFORNIA FAULT MAP

Newport Beach City Hall



CLOSEST DISTANCES BETWEEN SITE AND FAULT RUPTURES

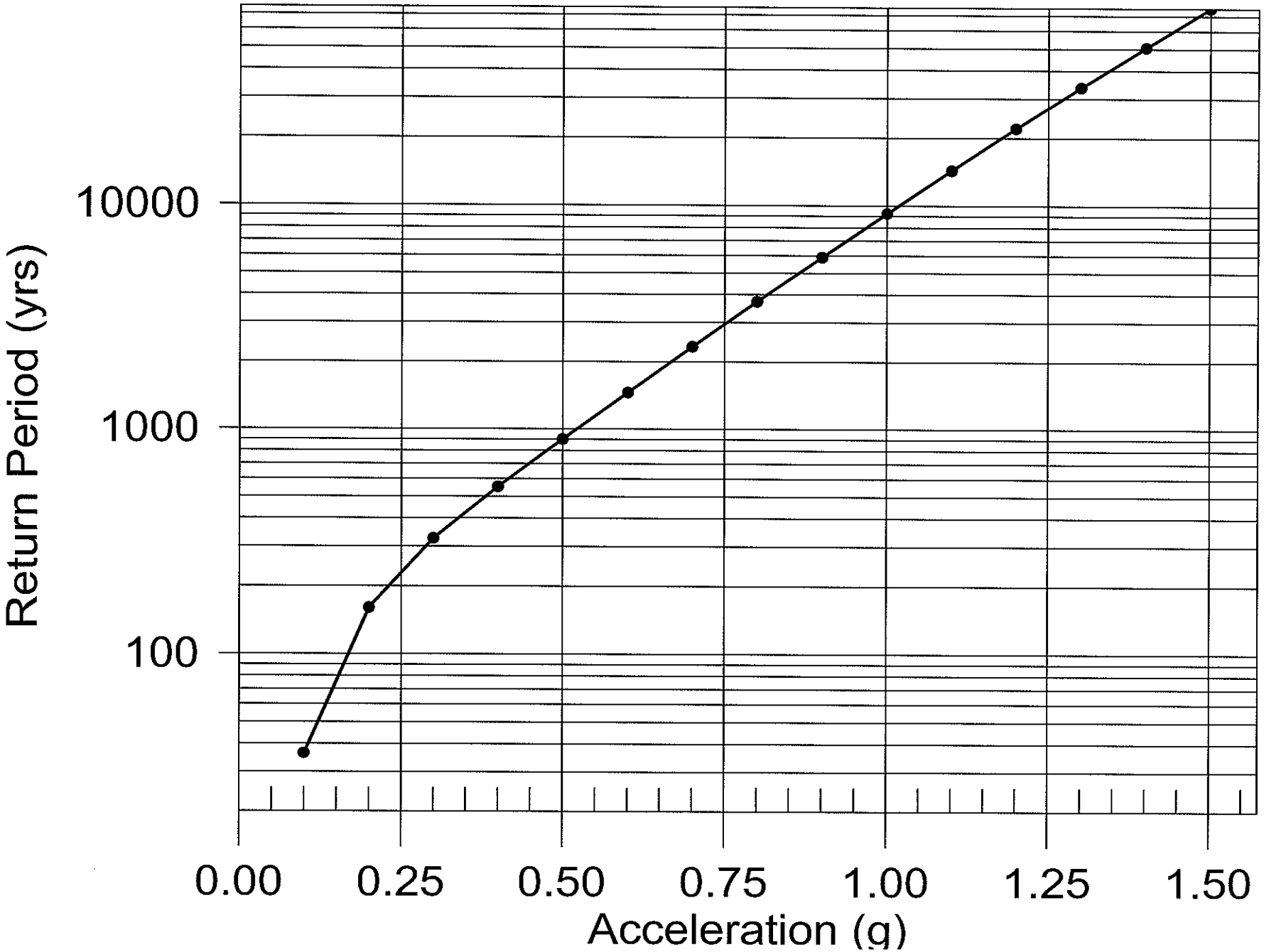
NO.	FAULT NAME	CD_1DRP	CD_2DRP	CDIST	CLODIS	CD_EPI	CD_HYPO
1	NEWPORT-INGLEWOOD (Offshore)	3.8	3.8	3.8	3.8	3.8	4.0 km
2	NEWPORT-INGLEWOOD (L.A.Basin)	4.8	4.8	4.8	4.8	5.5	5.6 km
3	SAN JOAQUIN HILLS	9.4	0.0	5.5	5.5	0.0	5.5 km
4	PALOS VERDES	23.3	23.3	23.3	23.3	23.3	23.3 km
5	WHITTIER	33.8	33.8	33.8	33.8	34.1	34.1 km
6	ELSINORE (GLEN IVY)	35.1	35.1	35.1	35.1	35.1	35.1 km
7	PUENTE HILLS BLIND THRUST	35.6	35.6	36.0	36.0	36.6	37.0 km
8	CHINO-CENTRAL AVE. (Elsinore)	37.4	30.5	34.0	34.0	31.1	34.1 km
9	CORONADO BANK	37.9	37.9	37.9	37.9	38.7	38.7 km
10	SAN JOSE	48.1	48.1	48.1	48.1	48.9	48.9 km
11	ELSINORE (TEMECULA)	48.8	48.8	48.8	48.8	49.5	49.6 km
12	UPPER ELYSIAN PARK BLIND THRUST	55.4	55.4	55.5	55.5	56.3	56.4 km
13	SIERRA MADRE	58.6	58.6	58.6	58.6	59.5	59.5 km
14	CUCAMONGA	59.2	59.2	59.2	59.2	60.5	60.5 km
15	RAYMOND	61.2	61.2	61.2	61.2	61.5	61.5 km
16	VERDUGO	63.8	63.8	63.8	63.8	65.2	65.2 km
17	CLAMSHELL-SAWPIT	64.6	64.6	64.6	64.6	65.7	65.7 km
18	HOLLYWOOD	65.9	65.9	65.9	65.9	66.7	66.7 km
19	ROSE CANYON	68.1	68.1	68.1	68.1	69.2	69.2 km
20	SANTA MONICA	71.5	71.5	71.5	71.5	71.8	71.8 km
21	SAN JACINTO-SAN BERNARDINO	74.0	74.0	74.0	74.0	74.0	74.0 km
22	SAN JACINTO-SAN JACINTO VALLEY	74.8	74.8	74.8	74.8	74.9	74.9 km
23	MALIBU COAST	77.1	77.1	77.1	77.1	78.2	78.2 km
24	SAN ANDREAS - SB-Coach. M-2b	82.8	82.8	82.8	82.8	82.8	82.8 km
25	SAN ANDREAS - Whole M-1a	82.8	82.8	82.8	82.8	82.8	83.0 km
26	SAN ANDREAS - San Bernardino M-1	82.8	82.8	82.8	82.8	82.8	82.8 km
27	SAN ANDREAS - SB-Coach. M-1b-2	82.8	82.8	82.8	82.8	82.8	82.8 km
28	SAN ANDREAS - Mojave M-1c-3	83.5	83.5	83.5	83.5	83.6	83.6 km
29	SAN ANDREAS - 1857 Rupture M-2a	83.5	83.5	83.5	83.5	83.6	83.6 km
30	SAN ANDREAS - Cho-Moj M-1b-1	83.5	83.5	83.5	83.5	83.6	83.6 km
31	ELSINORE (JULIAN)	83.7	83.7	83.7	83.7	84.7	84.7 km
32	SIERRA MADRE (San Fernando)	84.2	84.2	84.2	84.2	85.3	85.3 km
33	CLEGHORN	87.0	87.0	87.0	87.0	87.1	87.1 km
34	ANACAPA-DUME	86.9	86.9	86.9	86.9	88.2	88.2 km
35	SAN GABRIEL	87.6	87.6	87.6	87.6	88.6	88.6 km
36	NORTHRIDGE (E. Oak Ridge)	89.6	82.8	85.2	85.2	84.1	86.3 km
37	SAN JACINTO-ANZA	89.8	89.8	89.8	89.8	90.5	90.5 km
38	NORTH FRONTAL FAULT ZONE (West)	96.8	94.0	94.9	94.9	95.2	96.0 km
39	SANTA SUSANA	98.4	98.4	98.4	98.4	99.1	99.1 km

EXPLANATION

CD_1DRP = Closest distance to projection of rupture area along fault trace.
CD_2DRP = Closest distance to surface projection of the rupture area.
CDIST = Closest distance to seismogenic rupture.
CLODIS = Closest distance to subsurface rupture.
CD_EPI = Closest epicentral distance.
CD_HYPO = Closest hypocentral distance.

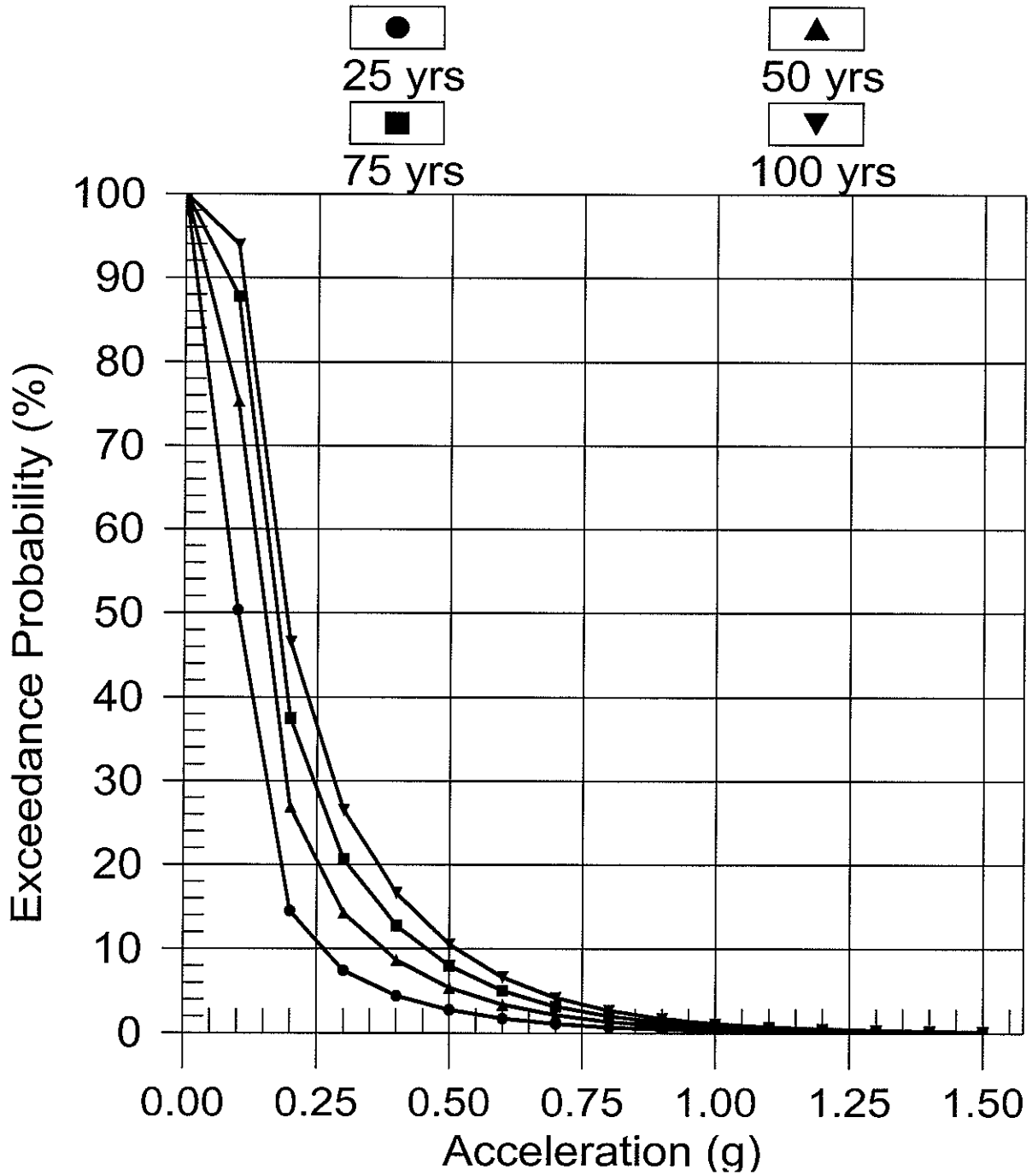
RETURN PERIOD vs. ACCELERATION

BOZ. ET AL.(1999)HOR PS COR



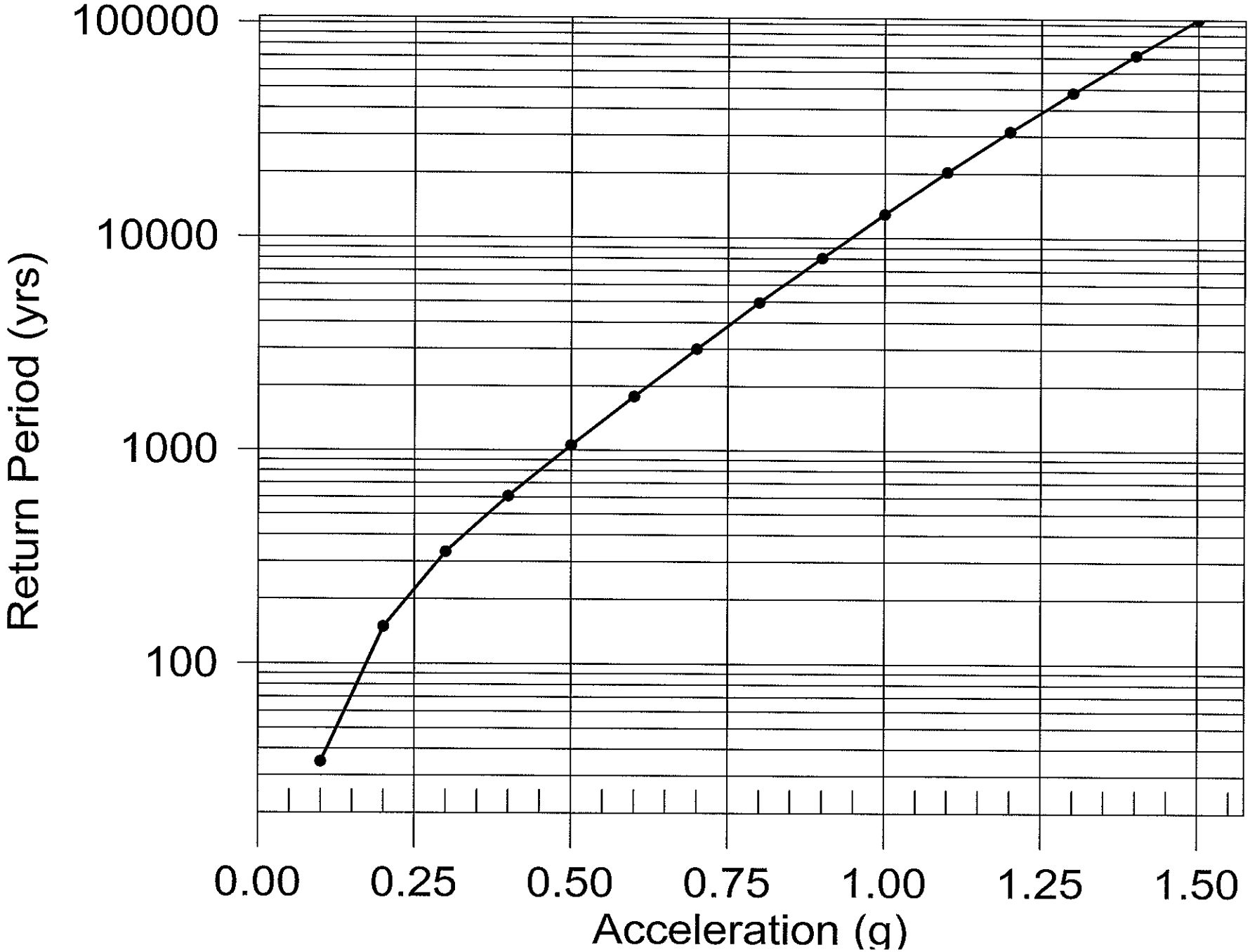
PROBABILITY OF EXCEEDANCE

BOZ. ET AL.(1999)HOR PS COR



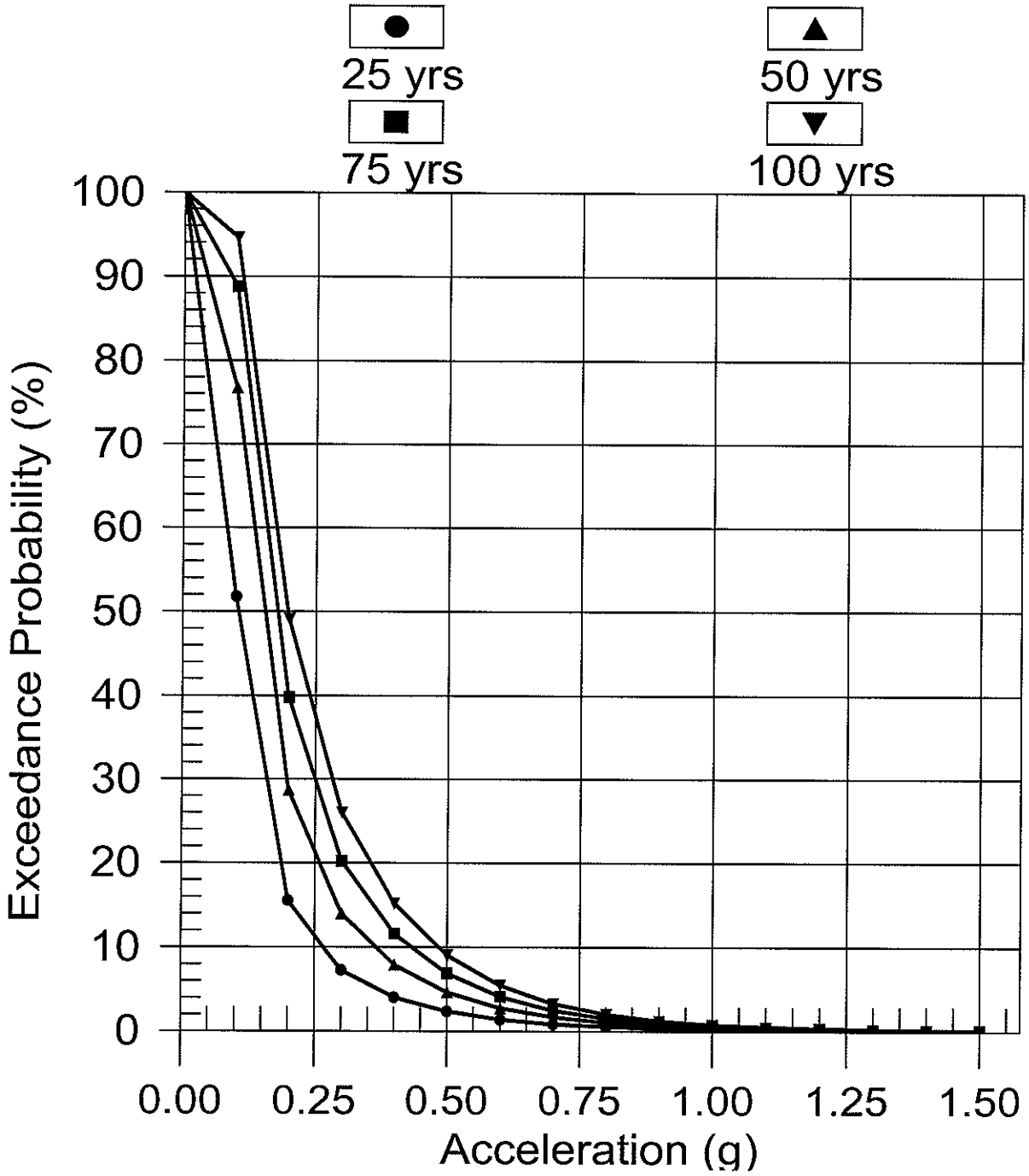
RETURN PERIOD vs. ACCELERATION

SADIGH ET AL. (1997) DEEP SOIL



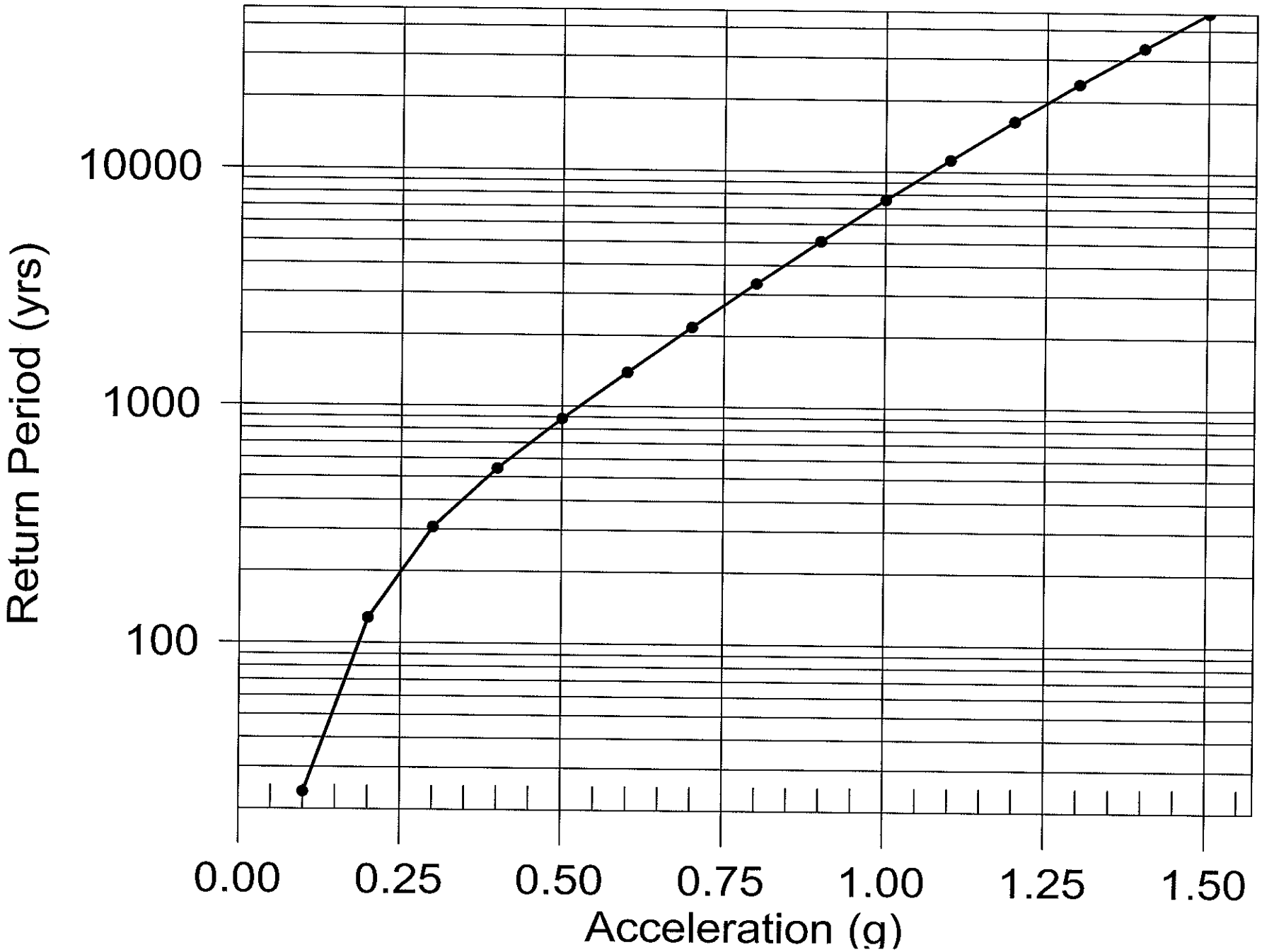
PROBABILITY OF EXCEEDANCE

SADIGH ET AL. (1997) DEEP SOIL



RETURN PERIOD vs. ACCELERATION

ABRAHAMSON & SILVA (1997) SOIL



PROBABILITY OF EXCEEDANCE
ABRAHAMSON & SILVA (1997) SOIL