Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

March 2, 2009
File No. 19790

R.L. Binder Architecture and Planing
7726 West 81st Street
Playa del Rey, California 90293

Attention: Tim Young

Subject: Geotechnical Engineering Investigation
Proposed Administrative Office of the Court, Riverside Mid-County
Northeast Corner Ramsey Street and Martin Street, Banning, California

Dear Mr. Young:

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, excavations, shoring and foundation design. 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.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

GREGORIO VARELA
Staff Engineer

EDWARD F. HILL
G.E. 2126

GV/EFH:km

Distribution: (7) Addressee

E-Mail to: [ty@binderarchitects.com], Attn: Tim Young

TABLE OF CONTENTS

TABLE OF CONTENTS - continued

SECTION PAGE

SECTION

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED ADMINISTRATIVE OFFICES OF THE COURT
NORTHEAST CORNER RAMSEY STREET AND MARTIN STREET
BANNING, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included eight exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

At the time of the writing of this report, the design and alignment of the proposed structure has not been finalized. The proposed development should be reviewed by this office when it achieves more definition. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

Information concerning the proposed development was furnished by the office of R. L. Binder Architecture and Planning. The site is proposed to be developed with the administrative offices of the court for the Riverside mid-county. The structure is proposed to be two stories in height, served by a full or partial subterranean level, depending on the final design. The subterranean level is expected to occupy only a portion of the footprint of the structure. The remainder of the structure is expected to be constructed at/or near existing grade. Column loads are estimated to be between 200 and 400 kips. Wall loads are estimated to be between 3 and 6 kips per lineal foot. These loads reflect the dead plus live load, of which the dead load is approximately 75 percent. Grading will consist of excavations for the proposed subterranean level, as well as removal and recompaction of existing unsuitable soils.

SITE CONDITIONS

The property is located at the northeast corner of Ramsey Street and Martin Street in the City of Banning, California. The site is shown relative to nearby topographic features on the attached Vicinity Map. The site is bounded by Williams Street to the north, a one-story structure, a utility yard, and a vacant lot to the east, Ramsey Street to the south, and Martin Street to the west. The site is rectangular in shape and it is approximately 4.7 acres in area.

Based on information contained on a topographic survey provided by the client, the site is located on a southeasterly descending slope that varies in elevation from 2341 feet on the northwest corner to 2320 feet on the southeast corner. This translates to a natural slope gradient of less than 4 percent. The site is currently vacant. Development on the site consists of a relatively small asphalt-paved parking area located on the northwest corner of the site, and a concrete drainage ditch that crosses the northeast corner of the site. The location of these structures is shown on the attached Plot Plan.

Vegetation of the site consists of annual grasses. Drainage appears to be by sheetflow towards the City streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on January 21, 2009 by excavating two test pits, and on January 26, 2009 by drilling six borings. The test pits were excavated with the aid of hand labor to a depth of six feet. The borings varied in depth from 20 to 50 feet, and were drilled with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-8.

The location of exploratory excavations was determined by measurement from hardscape features shown in the attached Plot Plan. Elevations of the exploratory excavations were determined by interpolation of elevations contained in the ALTA/ACSM Survey of the site by PSOMAS, dated November, 2008. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in all exploratory excavations to depths ranging between 1 and 3 feet below the existing site grade. Fill materials consist of silty sands which are yellowish-brown in color, moist, medium dense, and fine to medium grained with occasional gravel.

Native soils consist of interlayered mixtures of sands and silts, which range from light-yellow to yellowish-brown to gray in color, and are moist, medium dense to very dense, and fine to medium grained with occasional gravel and cobble. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater and Caving

Groundwater was not encountered during exploration, which was conducted to a depth of approximately 50 feet below the existing ground surface. Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving was not encountered during the excavation of the percolation test pits. Caving was not encountered in the borings due to the continuously cased design of the hollowstem auger used by the drilling machine. However, caving may be experienced in large diameter excavations that encounter clean sands.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey(CGS), 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.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large 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 well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. As illustrated on the attached Earthquake Fault Zone Plate, the closest seismic fault, the San Gorgonio Pass Fault, is approximately 5,000 feet to the north of the site. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

2007 California Building Code Seismic Parameters

According to Table 1613.5.2 of the 2007 California Building Code, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile. The following table outlines the Mapped Spectral Accelerations and Site Coefficients determined based on the site coordinates, and in accordance with the 2007 CBC.

2007 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.5g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.5g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.0g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.6g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.9g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.6g

According to Section 1802.2.7, of the California Building Code, a peak ground acceleration, equivalent to Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS}) divided by 2.5, shall be utilized for liquefaction analysis. Based on the site coordinates and Site Class, Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS}) divided by 2.5 is equal to 0.4g.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Groundwater was not encountered during exploration, conducted to an approximated depth of 50 feet below the existing site grade. For purposes of liquefaction analysis, a historic high groundwater level of 10 feet below the existing site grade has been assumed.

A site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of CDMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California (Martin and Lew, 1999). The enclosed liquefaction analysis was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (Blake, 1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data. The liquefaction potential evaluation was performed by assuming a magnitude 7.1 earthquake and a peak horizontal acceleration of 0.40g. Based on the adjusted blow count data, the enclosed liquefaction analysis indicates that the soils underlying the site would not be capable of liquefaction during the design based earthquake.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying earth materials, excessive differential settlements are not expected to occur.

Tsunamis, Seiches and Flooding

The subject site is far and/or high enough from the ocean or lakes such that it would not be prone to hazards of a tsunami, seiche, or flooding from a breached upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the relatively small natural slope gradient across the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of this firm that construction of the proposed structure is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

At the time of the preparation of this report, the design and alignment of the proposed structure have not been finalized. The proposed development should be reviewed by this office when it achieves more definition. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

Up to 3 feet of fill soils were encountered during exploration of the site. Native materials underlying the existing fill soils consist of interlayered mixtures of silt, sands and gravel. Groundwater was not encountered during exploration, conducted to a depth of 50 feet below the existent grade. The site is not considered prone to liquefaction during the ground motion expected during the design based earthquake.

According to information provided by the office of R.L. Binder Architecture and Planing, the proposed subterranean level is expected to occupy only a portion of the total structure's footprint, and could be partially or completely below grade. Depending on the final design, the bottom of the subterranean level may range between 6 and 15 feet below the existing site grade. The remaining of the proposed structure is expected to be constructed at/or near existing grade.

The existing fill soils are not suitable for support of the proposed foundations, slabs, or additional fill. It is anticipated that within the footprint of the proposed basement, existing fill materials will be removed as a result of the required excavation. In areas where the structure is proposed to be constructed at/or near existing grade, all fill materials an upper soils should be removed to a minimum depth of 3 feet below the proposed foundations, or 5 feet below existing grade, whichever is deeper, and recompacted as controlled fill prior to foundation excavation. In addition, the compacted fill shall extend horizontally a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundation, whichever is greater.

The proposed structure may be supported on conventional foundations. Conventional foundations for the subterranean portion of the structure may bear in native soils expected at the level of the required excavation. Conventional foundations for the at-grade portion of the structure may be supported on the newly placed compacted fill pad.

Foundations for small outlying structures, such as property line walls, which will not be tied-in to the proposed structure may be supported on conventional foundations bearing in native earth materials.

Excavations for the proposed subterranean level and the recommended removal and recompaction are expected to range between 5 and 15 feet in depth, depending on the final design. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Temporary excavations exceeding 5 feet in height may be sloped at a uniform 1:1 gradient (45 degrees) in their entirety. Excavations which will be surcharged by adjacent traffic or structures should be shored.

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 on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

FILL SOILS

The maximum depth of fill encountered on the site was 3 feet. The existing fill may be reused as part of the recommended compacted fill blanket.

EXPANSIVE SOILS

The onsite earth materials are in the very low expansion range. The Expansion Index was found to be 1 for representative samples of the site soils. Recommended reinforcing is provided in the "Foundation Design" and Slab-On-Grade" sections of this report.

GRADING GUIDELINES

Site Preparation

All vegetation, existing fill, and soft or disturbed earth materials should be removed from the areas to receive controlled fill. The excavated areas shall be carefully observed by the geotechnical engineer prior to placing compacted fill.

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

The area where the structure is proposed to be constructed at/or near existing grade shall be excavated to a minimum depth of 3 feet below the bottom of all foundations, or 5 feet below the existing site grade, whichever is deeper. The excavation shall extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. All 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 the laboratory operated by Geotechnologies, Inc. using test method ASTM D 1557-07 or equivalent.

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 a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of relatively non-expansive soils with an expansion index of less than 20. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could effect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might effect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with ASTM D-1557-07.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 2 and 10 percent should be anticipated when excavating and recompacting the existing fill and underlying native earth materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water 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.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechncial observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by this firm during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. 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.

FOUNDATION DESIGN

Conventional

Conventional foundations for the at-grade portion of the structure may bear in the newly placed compacted fill pad. Conventional foundations for the subterranean portion of the structure may bear in the underlying native soils. Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 130 pounds per square foot. The bearing capacity increase for each additional foot of depth is 300 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities 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.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may bear in native soils. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations 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 foundations.

Foundation Reinforcement

All continuous foundations should be reinforced with a minimum of two #4 steel bars. One should be placed near the top of the foundation, and one should be placed near the bottom.

Lateral 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 0.33 may be used with the dead load forces.

Passive earth pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot.

When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ¼ inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory earth materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

At the time of writing of this report, the design of the proposed structure had not yet been finalized. It is anticipated that the depth of the subterranean level will be on the order of 6 to 15 feet below the existing site grade, depending if the final design calls for a partial or full subterranean level. Design recommendations for retaining walls up to 15 feet in height have been included in this report. 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, adjacent structures or vehicular traffic. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

For traffic surcharge, the upper 10 feet of 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.

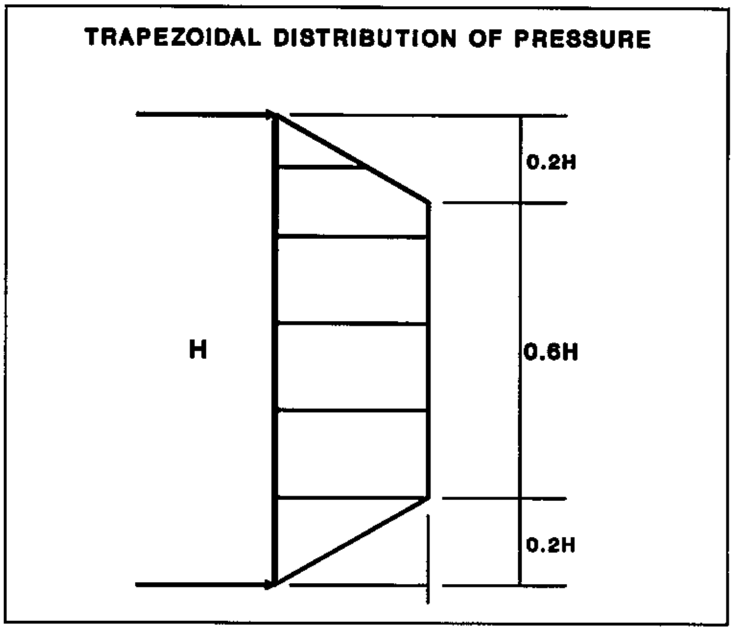
Cantilever Retaining Walls

Retaining walls supporting a backslope with a slope gradient equal or less than the natural sloping of the site may be designed utilizing a triangular distribution of pressure. Retaining walls up to 15 feet in height may anticipate an equivalent fluid pressure of 30 pounds per cubic foot.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Surcharge from any adjacent traffic, additional sloping ground or adjacent structures should be added as described above.

Restrained Retaining Walls

Restrained retaining walls may be designed to resist a trapezoidal pressure distribution of at-rest earth pressure as indicated in the diagram below.



Design restrained walls as follows:

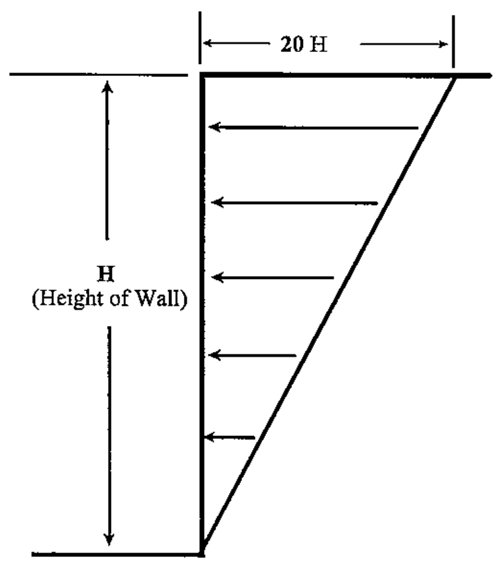
Height of Retaining Wall (feet)	Restrained Retaining Wall Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
15 feet	45H

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. The natural sloping of the site was taken into consideration when calculating this pressure. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by any adjacent traffic or structures as describe above.

Dynamic (Seismic) Lateral Forces

Retaining walls exceeding 12 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. An inverse triangular distribution should be utilized for seismic loads, with an equivalent fluid pressure of 20H pounds per cubic foot. Utilizing this inverse triangular pressure distribution, the earthquake load would be zero at the base of the wall, and would increase linearly to a maximum of 20H pounds per square foot at the top of the wall, where H is the height of the retaining wall.

DYNAMIC (SEISMIC) PRESSURE INCREMENT



Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescense or standing water inside the building. Efflorescence is a process in which a 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. Efflorescence is common to retaining walls and does not effect their strength or integrity.

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

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. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-07 or equivalent.

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.

Where retaining walls are to be constructed adjacent to property lines there is usually not enough space for emplacement of a standard pipe and gravel drainage system. As an alternative, omission of one-half of a block at the back of the wall on eight foot centers is an acceptable method of draining the walls. The resulting void should be filled with gravel. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

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 maximum density obtainable by the ASTM Designation D 1557-07 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

Excavations on the order of 5 to 15 feet in vertical depth are expected for the subterranean level and the recommended recompaction. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum height of 30 feet. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

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

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilever shoring system.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation, may be assumed to be 600 pounds per square foot per foot, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

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

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.33 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 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.

Lagging

It is possible that lagging between soldier piles could be omitted within more cohesive earth materials where the clear spacing between soldier piles does not exceed four feet. In less cohesive earth materials, such as sands and gravels, lagging would be necessary. It is recommended that a representative of this firm observe the exposed earth materials to verify their nature and establish areas where lagging could be omitted, if any.

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

Lateral Pressures

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilevered shoring up to 15 feet in height should be designed for an equivalent fluid pressure of 25 pounds per cubic foot.

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. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the shoring be designed for a maximum deflection of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and streets.

Monitoring

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

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

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 4 inches in thickness. Slabs-on-grade 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.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork 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.

Design Of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with ASTM E 1643-98 and ASTM E 1745-97 (Reapproved 2004). Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

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.

Slab Reinforcing

Concrete slabs-on-grade and outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent of the maximum density as determined by ASTM D 1557-07. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended, based on an assumed R-Value of 30:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars (TI=4)	3	4
Moderate Truck (TI=6)	4	6

Concrete paving must be a minimum of 6 inches in thickness, and shall be underlain by 6 inches of aggregate base.

Aggregate base should be compacted to a minimum of 95 percent of the ASTM D 1557-07 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

The performance of pavement is highly dependant upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

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

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. 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. Planters which are located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. This requirement is not prudent engineering practice. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

In order to establish a percolation rate for the site soils, Test Pits1 and 2 were used for percolation testing. The test pits were presoaked for 3 hours prior to the test. After the presoak, the test pits were refilled with water and the absorption of the soils was measured. Based on results of the percolation tests, a percolation rate of 16 inches per hour may be utilized for design purposes. The collected stormwater should only percolate into the underlying native soils.

The edge of the infiltration pit shall maintain a minimum distance of 20 feet away from any foundations and 50 feet away from the walls of the proposed subterranean level and property lines. The location of the proposed infiltration pit shall be reviewed and approved by this firm prior to construction.

Where percolation of stormwater into the subgrade soils is not efficient, some Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminates discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems. Please be advised that stormwater infiltration and treatment is a relatively new requirement by the various regulatory agencies and has been subject to change without notice.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechncial observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm 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.

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.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on representative samples of the onsite soils by Schiff Associates indicate that the electrical resistivities of the soils are in the mildly corrosive to moderately corrosive categories in their field moisture condition and when saturated. Soil pH values of the samples range between 7.4 and 7.6, indicating mildly alkaline condition. The soluble salt content ranged from low to moderate. Nitrate was detected in low concentrations, while the ammonium concentration was high enough to be deleterious to copper.

In summary, the soils are classified as moderately corrosive to ferrous metals and aggressive to copper. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible. Detailed results, discussion of results and recommended mitigating measures are provided within M.J. Schiff's report contained in the Appendix.

CLOSURE AND LIMITATIONS

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

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. 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.

Samples of the earth materials encountered in the exploratory excavations were collected and transported to the laboratory. 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. 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 1586-08. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

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 by ASTM D 4959-07 or ASTM D 4643-08. This information is useful in providing a gross picture of the soil consistency between exploration locations 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.

Direct Shear Testing

Shear tests are performed by ASTM D 3080-04 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, 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 Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests ASTM D 2435-04. 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 Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the ASTM D4829-08. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented in Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of ASTM D 1557-07. A soil at a selected moisture content is placed in five layers into as mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted, represent a curvilinear relationship know as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented in Plate D of this report.

REFERENCES

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluation and Mitigation of Seismic Hazards in California, CDMG Special Publication 117.

Fain, Hsai-Yang, ed., 1991, Foundation Engineering Handbook, Second Edition: New York, Van Nostrand Reinhold, pp. 630-631.

Ishihara, K. 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, Volume 1, P. 321-376, August.

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

O'Rourke, T.D., Pease, J.W. (1997), Mapping Liquefiable Layer Thickness for Seismic Hazard Assessment, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 123, no. 1, pp. 46-56.

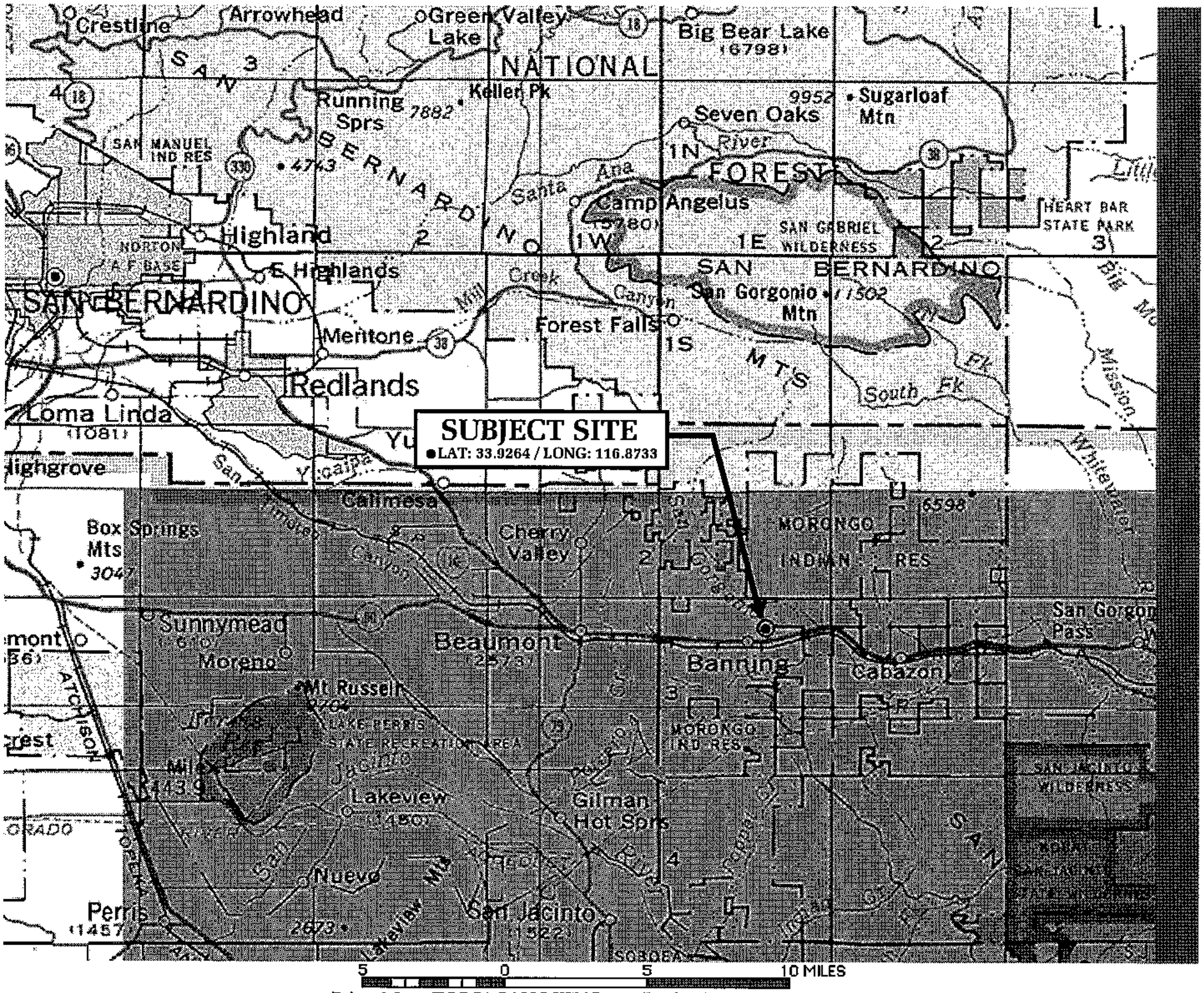
Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

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

Tokimatsu, K. and Seed, H. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.

United States Geological Survey, 2008, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.8). http://earthquake.usgs.gov/research/hazmaps/design/.

Youd, T. L. and Garris, C. T., 1995, Liquefaction-Induced Ground Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, P. 805-809.



Printed from TOPO! ©1997 Wildflower Productions (www.topo.com)

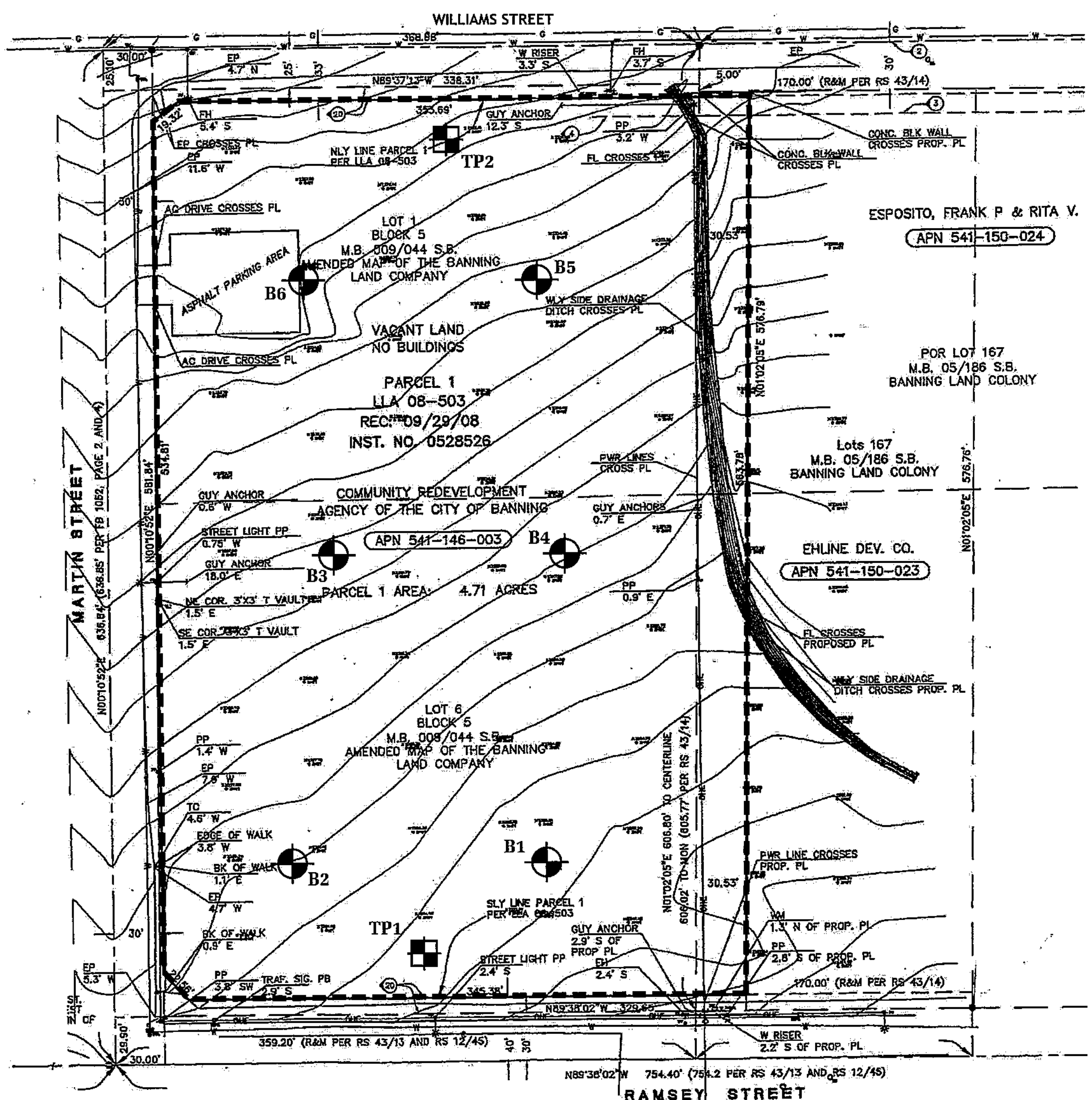
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, CABAZON, CALIFORNIA QUADRANGLE.

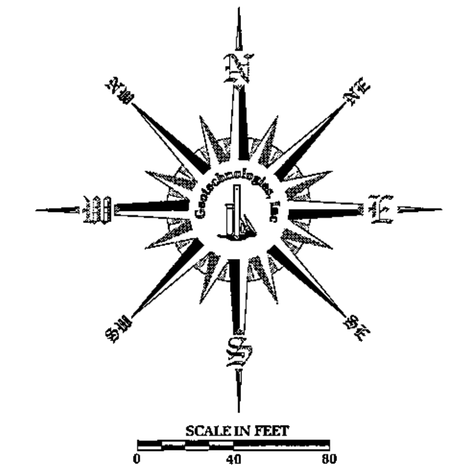
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

FILE NO. 19790





REFERENCE: ALTA/ACSM SURVEY BY PSOMAS
DATED NOVEMBER 2008

LEGEND

B6 — LOCATION & NUMBER OF BORING

TP2 — LOCATION & NUMBER OF TEST PIT

PROPERTY LINE

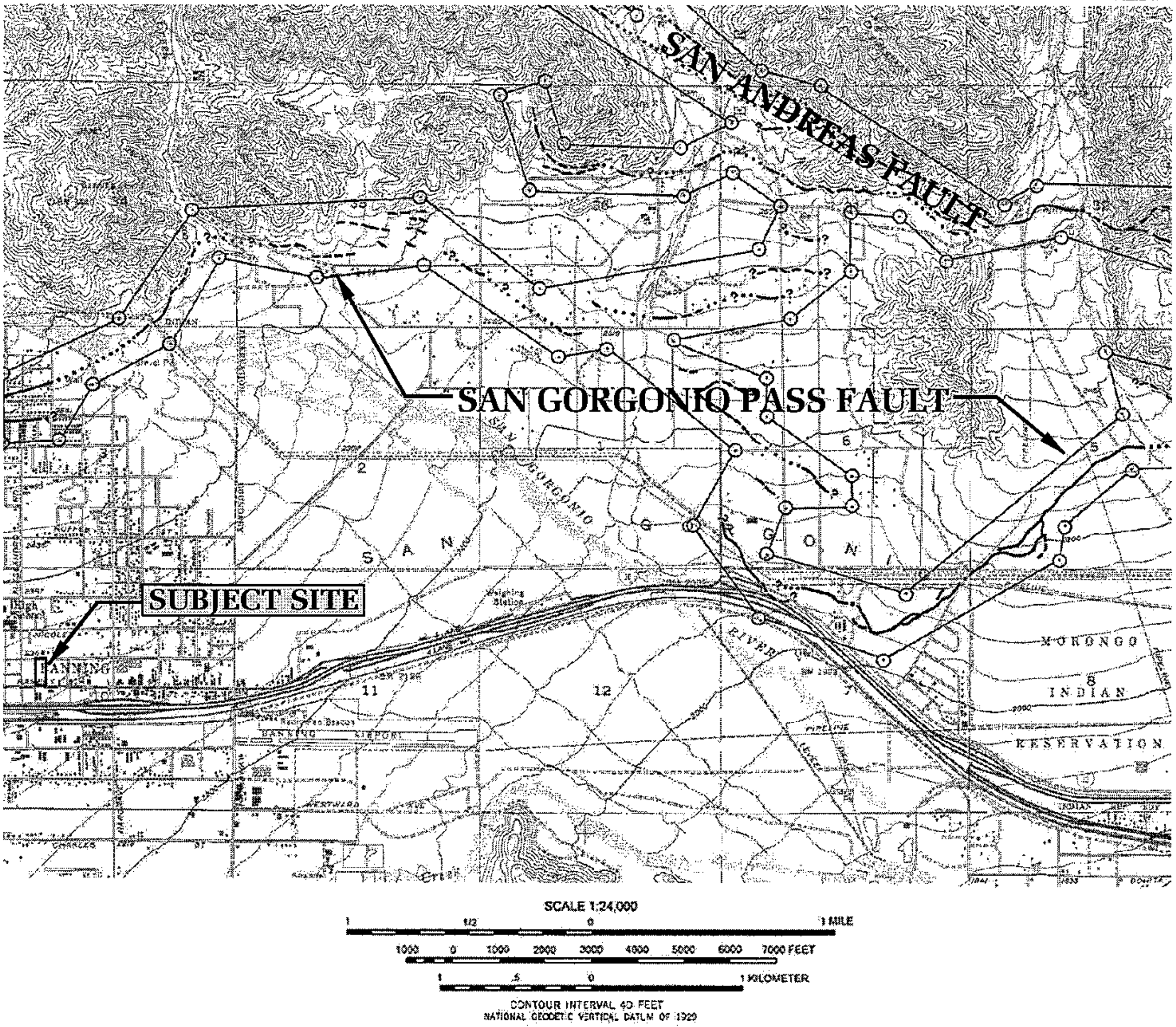
PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L.BINDER ARCHITECTURE

FILE No. 19790

DATE: February 2009



o——o Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

REFERENCE: SPECIAL STUDIES ZONES, CABAZON QUADRANGLE, CALIFORNIA, DMG, 1995.

EARTHQUAKE FAULT ZONE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

FILE NO. 19790

BORING LOG NUMBER 1

Drilling Date: 01/26/09 **Elevation: 2323.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained
1	34	5.6	111.3	1 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
				2 --		
3	30	2.3	120.3	3 --		
				4 --		
5	36	3.5	120.1	5 --		
				6 --		
7	39	3.8	116.6	7 --		
				8 --		
				9 --		
10	75/6"	5.5	110.0	10 --		
				11 --		
				12 --		
				13 --		
				14 --		
15	75/6"	2.4	132.3	15 --		
				-		dense, fine to medium grained
				16 --		
				17 --		
				18 --		
				19 --		
20	75/6"	1.5	135.2	20 --		cobble
				21 --		Total depth: 20 feet; No Water; Fill to 1 foot
				22 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				23 --		
				24 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Slide Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				25 --		SPT=Standard Penetration Test

BORING LOG NUMBER 2

Drilling Date: 01/26/09 **Elevation: 2326'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine to medium grained
				1 --		
2	28	3.6	109.5	2 --	SP	Sand, yellowish brown, moist, medium dense, fine to medium grained, slight gravel
				3 --		
4	36	6.5	124.7	4 --	SM	Silty Sand, yellowish brown, moist, medium dense, fine to medium grained, slight gravel
				5 --		
				6 --		
7	39	6.8	116.8	7 --		fine grained
				8 --		
				9 --		
10	75/6"	6.9	112.2	10 --		dense
				11 --		
				12 --		
				13 --		
				14 --		
15	32 50/6"	5.8	117.0	15 --		very dense
				16 --		
				17 --		
				18 --		
				19 --		
20	33 50/6"	2.1	122.0	20 --	SW	Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				21 --		Total depth: 20 feet No Water Fill to 2 feet
				22 --		
				23 --		
				24 --		
				25 --		

BORING LOG NUMBER 3

Drilling Date: 01/26/09 **Elevation: 2330.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained
1	35	4.5	114.2	1 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
				2 --		
3	30 50/6"	2.4	121.4	3 --		------------------
				-		very dense, slight gravel, cobble, fine to medium grained
				4 --		
5	325 50/6"	1.2	133.1	5 --		
				6 --		
7	38 50/6"	1.4	129.4	7 --		
				8 --		
				9 --		
10	32 50/6"	0.6	123.3	10 --		
				11 --		
				12 --		
				13 --		
				14 --		
15	100/6"	1.3	113.0	15 --		------------------
				-		cobble
				16 --		
				17 --		
				18 --		
				19 --		
20	80/6"	2.1	128.3	20 --		
				-		Total depth: 20 feet No Water Fill to 1 foot
				21 --		
				22 --		
				23 --		
				24 --		
				25 --		

BORING LOG NUMBER 4

Drilling Date: 01/26/09 **Elevation: 2328'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained
				-		
				1 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel, cobble
				2 --		
2.5	35 50/5"	2.7	120.4	-		
				3 --	SM	Silty Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				-		
				4 --		
				-		
5	50/6"	1.7	SPT	5 --		
				-		dense
				6 --		
				-		
				7 --		
7.5	100/3"	2.3	122.9	-		
				8 --		very dense
				-		
				9 --		
				-		
10	50/4"	1.3	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	75/6"	1.8	119.1	-		
				13 --		
				-		
				14 --		
				-		
15	50/3"	2.1	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	75/7"	2.0	135.1	-		
				18 --		light yellow to yellowish brown
				-		
				19 --		
				-		
20	50/6"	2.1	SPT	20 --		
				-	SW	Sand, yellowish brown, very dense, moist, fine to medium grained
				21 --		
				-		
				22 --		
22.5	75/7"	1.8	127.7	-		
				23 --		light gray to yellowish brown
				-		
				24 --		
				-		
25	50/5"	1.6	SPT	25 --		
				-		yellowish brown

BORING LOG NUMBER 4

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	30 50/6"	7.3	105.7			
				28	SP	Sand, yellowish brown to light gray, moist, very dense, fine grained
				29		
30	50/6"	11.2	SPT	30		
				31	SP/SM	Sand to Silty Sand, light gray to yellowish brown, moist, dense, fine grained
				32		
32.5	36 50/6"	2.5	125.1			
				33	SW	Sand with Gravel, yellowish brown to light yellow, moist, very dense, fine to medium grained
				34		
35	50/6"	2.3	SPT	35		yellowish brown
				36		
				37		
37.5	30 50/6"	7.9	108.3			
				38	SM	Silty Sand, yellowish brown, moist, very dense, fine grained
				39		
40	50/7"	6.8	SPT	40		
				41		
				42		
42.5	30 50/6"	6.0	117.2			
				43		cobble
				44		
45	50/6"	5.2	SPT	45		slight gravel
				46		
				47		
47.5	100/6"	2.8	126.4			
				48	SW	Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				49		
50	50/3"	1.7	SPT	50		
						Total depth: 50 feet; No Water; Fill to 1 foot

BORING LOG NUMBER 5

Drilling Date: 01/26/09 **Elevation: 2333.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained
1	48	4.5	115.6	1 --		
				2 --	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
3	35 50/6"	3.2	121.1	3 --		
				4 --	SM/SP	Silty Sand to Sand, yellowish brown, moist, very dense, fine grained, slight gravel, cobble
5	30 50/6"	2.3	111.4	5 --		
				6 --		fine to medium grained
7	26 50/6"	1.9	129.4	7 --		
				8 --		
				9 --		
10	100/7"	3.2	106.0	10 --		
				11 --	SM	Silty Sand, yellowish brown, moist, very dense, fine grained, cobble
				12 --		
				13 --		
				14 --		
15	36 50/6"	1.6	124.7	15 --		
				16 --	SW	Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				17 --		
				18 --		
				19 --		
20	38 50/6"	1.0	128.1	20 --		
				21 --		Total depth: 20 feet No Water Fill to 1 foot
				22 --		
				23 --		
				24 --		
				25 --		

BORING LOG NUMBER 6

Drilling Date: 01/26/09 **Elevation: 2336.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km

*reference: Survey by PSOMAS, dated 12/12/08

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
				1		
2	25 50/6"	5.1	105.6	2	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
				3		dense
4	30 50/5"	1.6	120.1	4		fine to medium grained, cobble
				5	SM/SP	Silty Sand to Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained, slight gravel
				6		
7	38 50/5"	2.0	115.7	7		
				8	SW	Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				9		
10	35 50/6"	1.8	121.3	10		
				11		
				12	SM	Silty Sand, yellowish brown, moist, very dense, fine grained
				13		
				14		
15	30 50/5"	4.3	114.3	15		
				16	SM/SW	Silty Sand to Sand with Gravel, yellowish brown, moist, very dense, fine to medium grained
				17		
				18		
				19		
20	30 50/6"	1.7	124.8	20		
				21		Total depth: 20 feet No Water Fill to 1 foot
				22		
				23		
				24		
				25		

LOG OF TEST PIT NUMBER 1

Drilling Date: 01/21/09 **Elevation: 2323.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

km *reference: Survey by PSOMAS, dated 12/12/08

Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
0 -- to 3 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
3 -- to 6 --	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
6 -- to 25 --		Total depth: 6 feet No Water Fill to 3 feet

LOG OF TEST PIT NUMBER 2

Drilling Date: 01/21/09 **Elevation: 2337.5'***

Project: File No. 19790 **R.L. Binder Architecture and Planning**

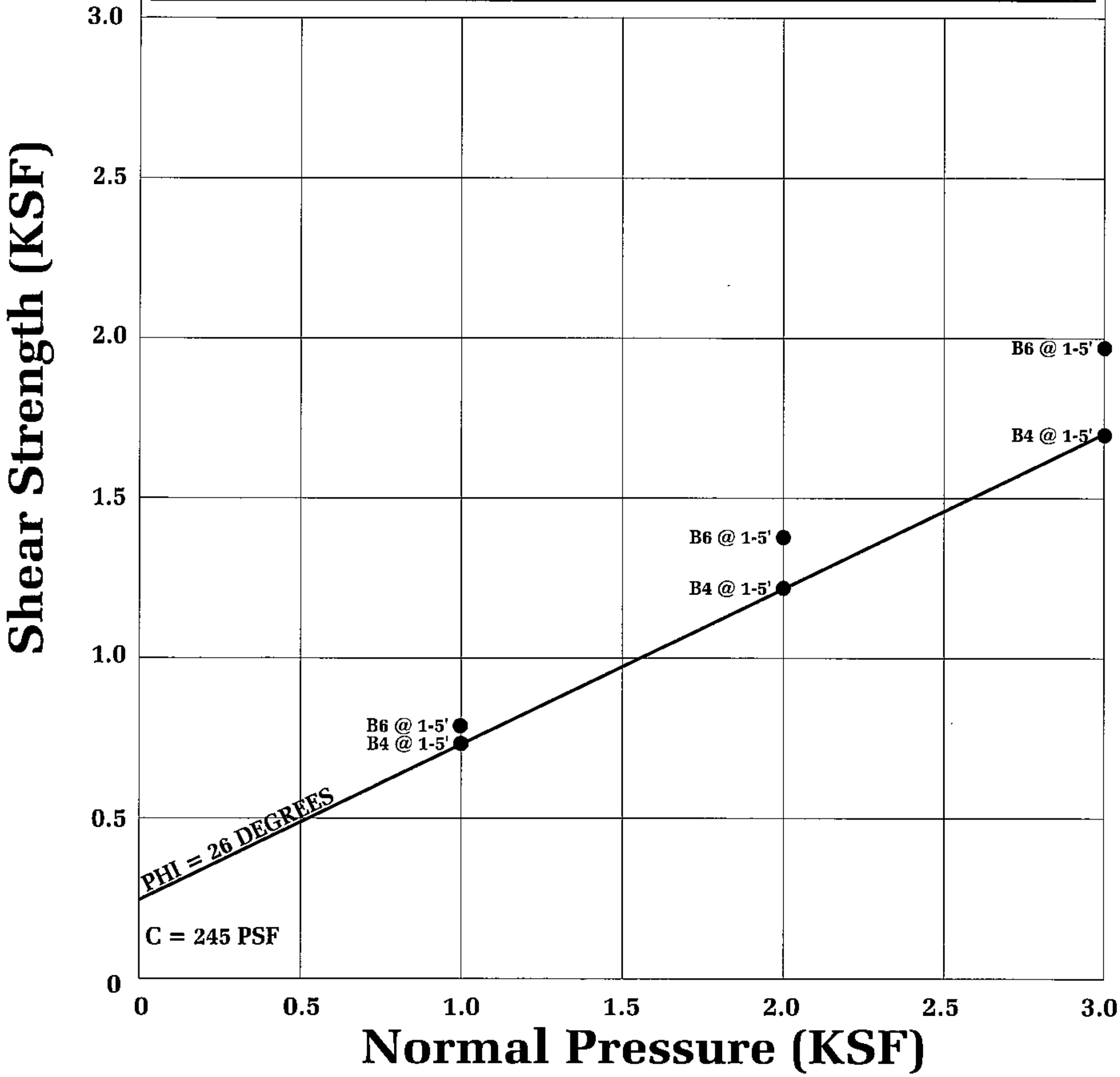
km
*reference: Survey by PSOMAS, dated 12/12/08

Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
0 --		FILL: Silty Sand, yellowish brown, slightly porous, moist, medium dense, fine grained, slight gravel
1 --	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained, slight gravel
3 --	SP	Sand, yellowish brown, moist, medium dense, fine to medium grained, slight gravel and cobble
5 --	SW	Sand with Gravel, light yellow, moist, medium dense, fine to medium grained, cobble
6 --		Total depth: 6 feet No Water Fill to 1 foot

Depth scale: 0 -- through 25 --

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B4 @ 1-5'	SM	118.8	7.8	16.0
B6 @ 1-5'	SM/SP	122.1	7.2	13.1



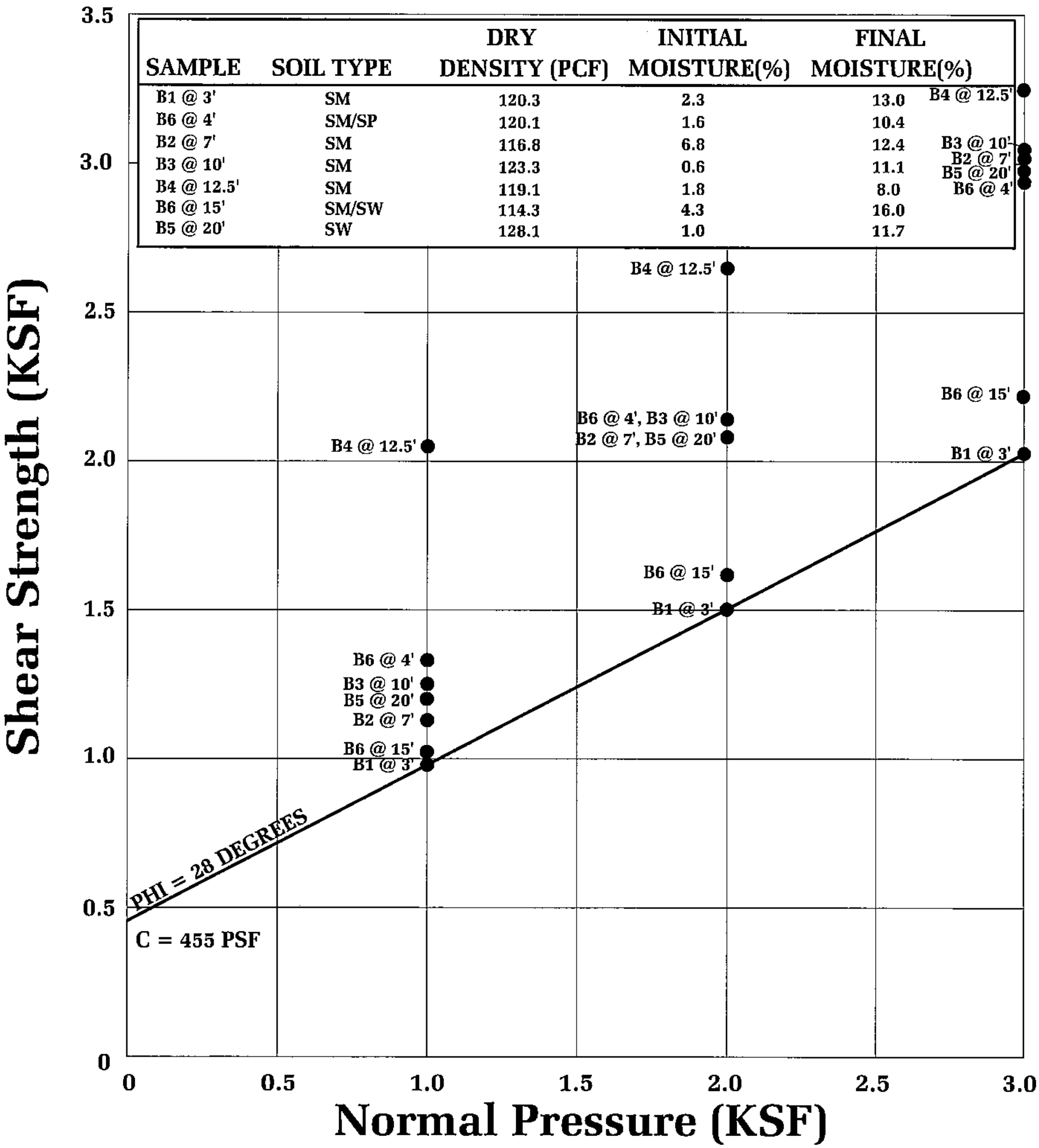
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

FILE NO. 19790 | PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 3'	SM	120.3	2.3	13.0
B6 @ 4'	SM/SP	120.1	1.6	10.4
B2 @ 7'	SM	116.8	6.8	12.4
B3 @ 10'	SM	123.3	0.6	11.1
B4 @ 12.5'	SM	119.1	1.8	8.0
B6 @ 15'	SM/SW	114.3	4.3	16.0
B5 @ 20'	SW	128.1	1.0	11.7

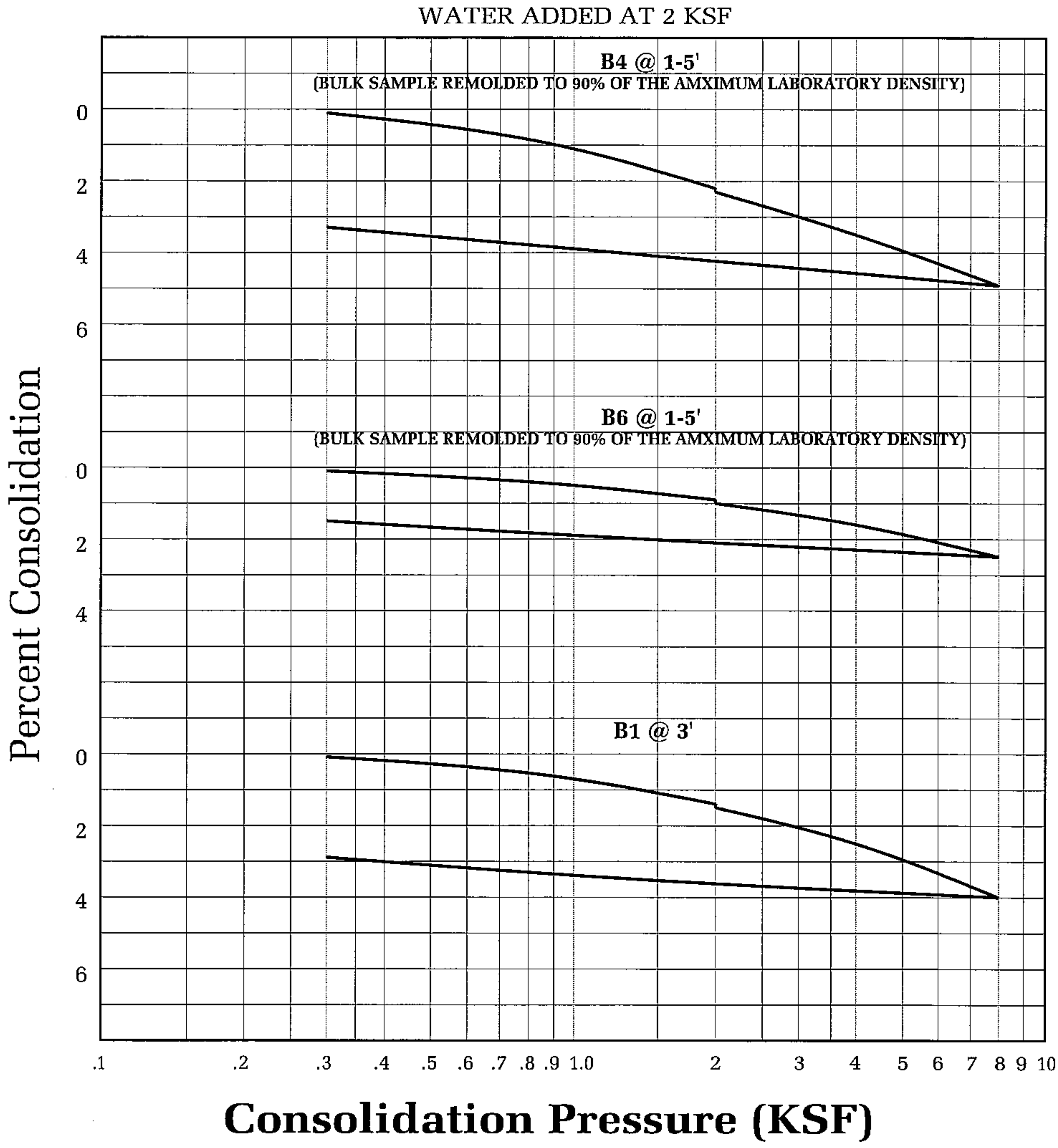
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

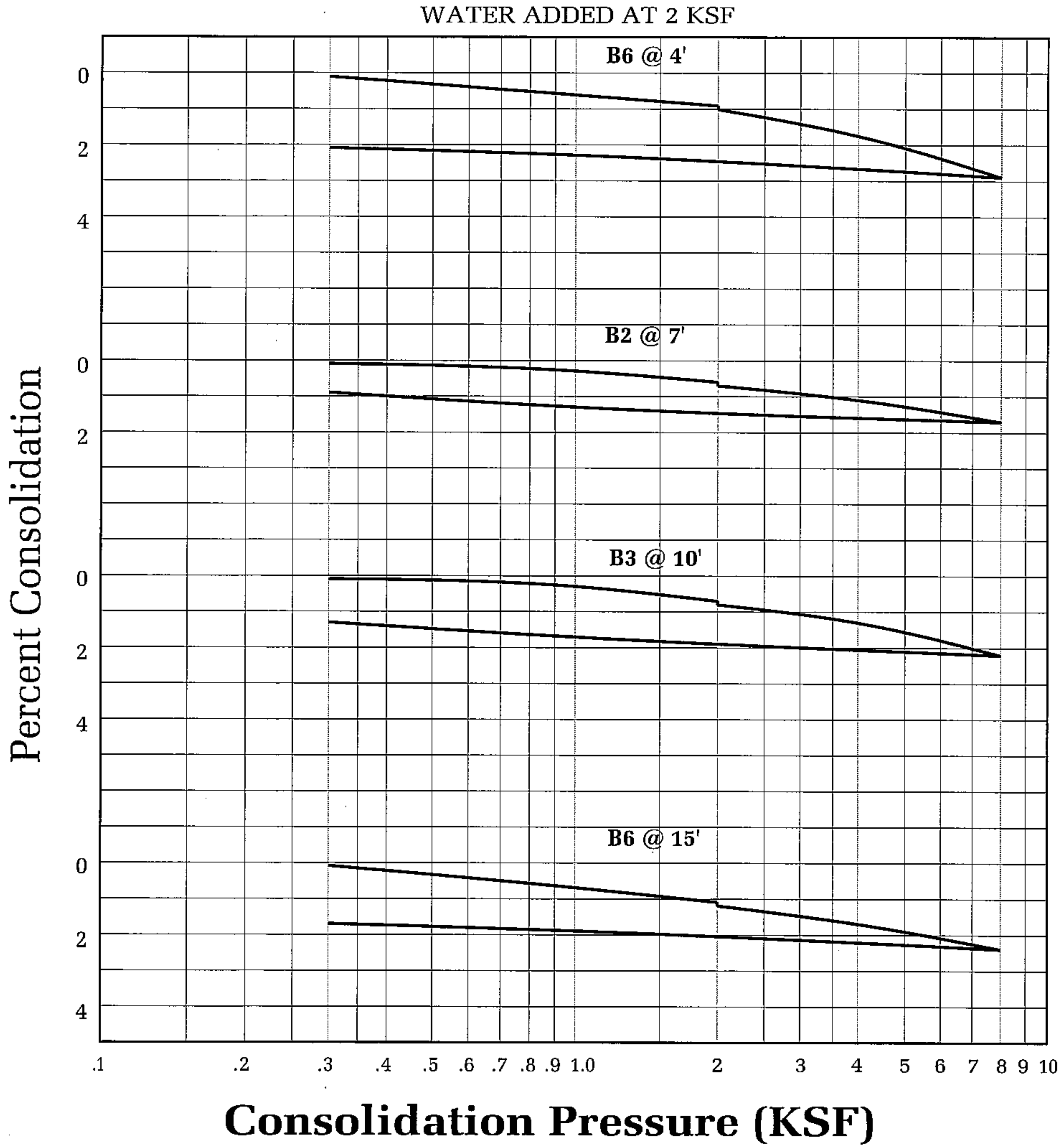
FILE NO. 19790 | PLATE: B-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE	
FILE NO. 19790	PLATE: C-1



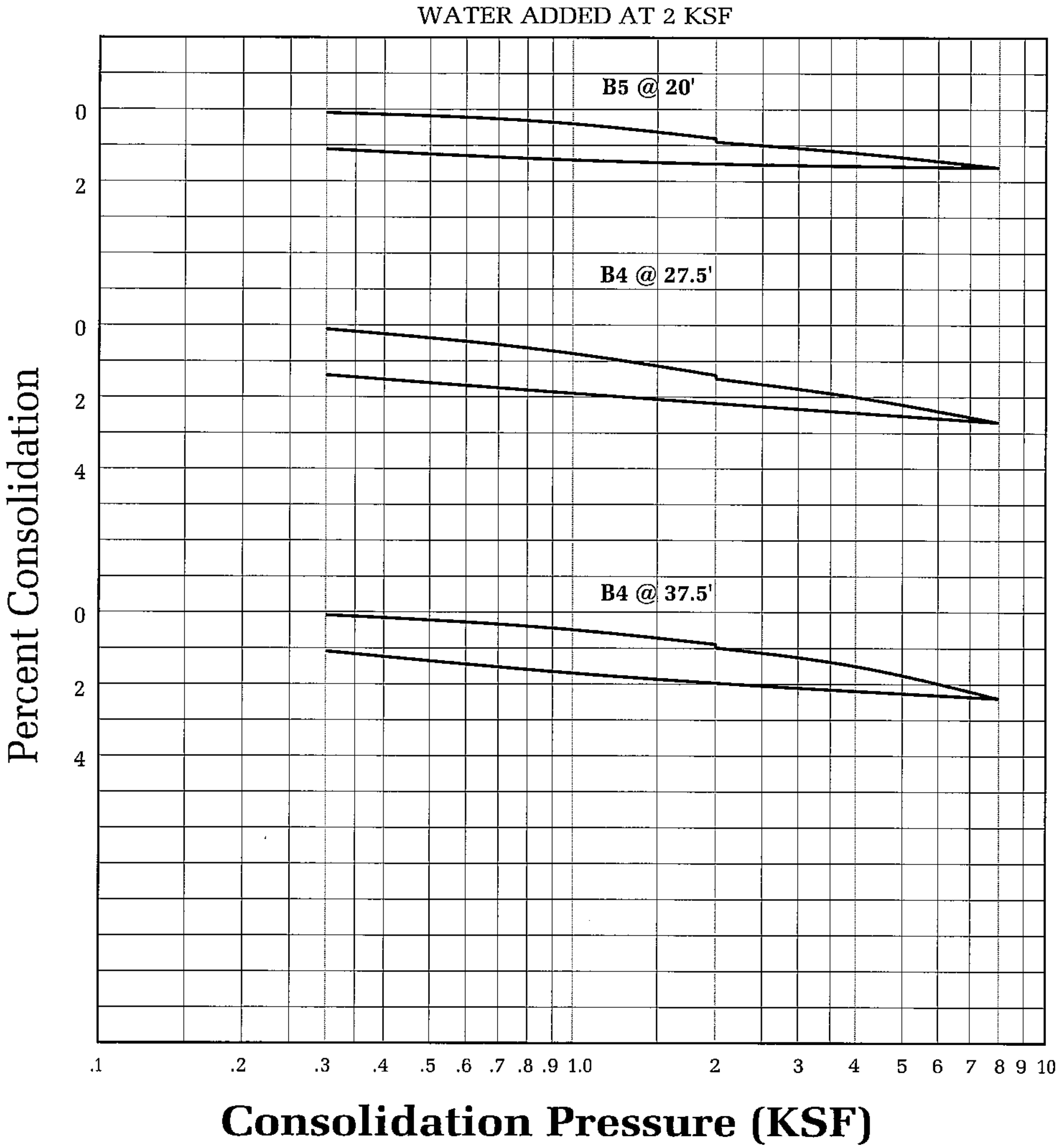
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

FILE NO. 19790

PLATE: C-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE	
FILE NO. 19790	PLATE: C-3

ASTM D-1557

SAMPLE	B4 @ 1- 5'	B6 @ 1-5'
SOIL TYPE:	SM	SM
MAXIMUM DENSITY pcf.	132.0	135.7
OPTIMUM MOISTURE %	7.8	7.2

ASTM D 4829-03

SAMPLE	B4 @ 1- 5'	B6 @ 1-5'
SOIL TYPE:	SM	SM
EXPANSION INDEX UBC STANDARD 18-2	1	1
EXPANSION CHARACTER	VERY LOW	VERY LOW

SULFATE CONTENT

SAMPLE	B4 @ 1- 5'	B6 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

R.L. BINDER ARCHITECTURE

FILE NO. 19790 | PLATE: D

Geotechnologies, Inc.

Project: R.L. Binder Architecture and Planning
File No.: 19790
Description: Liquefaction Analysis
Boring Number: 4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD | By Thomas F. Blake (1994-1996) | LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.1
Peak Horiz. Acceleration (g):	0.40
Calculated Mag.Wtg.Factor:	0.873

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	51.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Conservately Assumed for Analysis Purposes

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.00
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.4	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.227	~
2.0	120.4	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.993	0.226	~
3.0	120.4	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.989	0.224	~
4.0	120.4	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.984	0.223	~
5.0	120.4	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.979	0.222	~
6.0	120.4	0	100.0	5.0	0	0.0		1.976	177.9	~	0.975	0.221	~
7.0	120.4	0	100.0	5.0	0	0.0		1.976	177.9	~	0.970	0.220	~
8.0	122.9	0	100.0	5.0	0	0.0		1.976	177.9	~	0.966	0.219	~
9.0	122.9	0	100.0	5.0	0	0.0		1.976	177.9	~	0.961	0.218	~
10.0	122.9	0	100.0	5.0	0	0.0		1.976	177.9	~	0.957	0.217	~
11.0	122.9	0	100.0	10.0	1	0.0	175	1.356	122.1	Infin.	0.952	0.216	Non-Liq.
12.0	122.9	0	100.0	10.0	1	0.0	175	1.356	122.1	Infin.	0.947	0.215	Non-Liq.
13.0	119.1	0	100.0	10.0	1	0.0	175	1.356	122.1	Infin.	0.943	0.214	Non-Liq.
14.0	119.1	0	100.0	10.0	1	0.0	175	1.356	122.1	Infin.	0.938	0.213	Non-Liq.
15.0	119.1	0	100.0	10.0	1	0.0	175	1.356	122.1	Infin.	0.934	0.212	Non-Liq.
16.0	119.1	0	100.0	15.0	1	0.0	160	1.098	106.3	Infin.	0.929	0.211	Non-Liq.
17.0	119.1	0	100.0	15.0	1	0.0	160	1.098	106.3	Infin.	0.925	0.210	Non-Liq.
18.0	135.1	0	100.0	15.0	1	0.0	160	1.098	106.3	Infin.	0.920	0.209	Non-Liq.
19.0	135.1	0	100.0	15.0	1	0.0	160	1.098	106.3	Infin.	0.915	0.208	Non-Liq.
20.0	135.1	0	100.0	15.0	1	0.0	160	1.098	106.3	Infin.	0.911	0.207	Non-Liq.
21.0	127.7	0	100.0	20.0	1	0.0	148	0.941	101.0	Infin.	0.906	0.206	Non-Liq.
22.0	127.7	0	100.0	20.0	1	0.0	148	0.941	101.0	Infin.	0.902	0.205	Non-Liq.
23.0	127.7	0	100.0	20.0	1	0.0	148	0.941	101.0	Infin.	0.897	0.204	Non-Liq.
24.0	127.7	0	100.0	20.0	1	0.0	148	0.941	101.0	Infin.	0.893	0.203	Non-Liq.
25.0	127.7	0	100.0	20.0	1	0.0	148	0.941	101.0	Infin.	0.888	0.202	Non-Liq.
26.0	127.7	0	100.0	25.0	1	0.0	139	0.835	95.8	Infin.	0.883	0.201	Non-Liq.
27.0	127.7	0	100.0	25.0	1	0.0	139	0.835	95.8	Infin.	0.879	0.200	Non-Liq.
28.0	105.7	0	100.0	25.0	1	0.0	139	0.835	95.8	Infin.	0.874	0.199	Non-Liq.
29.0	105.7	0	100.0	25.0	1	0.0	139	0.835	95.8	Infin.	0.870	0.197	Non-Liq.
30.0	105.7	0	100.0	25.0	1	0.0	139	0.835	95.8	Infin.	0.865	0.196	Non-Liq.
31.0	105.7	0	100.0	30.0	1	0.0	131	0.765	91.8	Infin.	0.861	0.195	Non-Liq.
32.0	105.7	0	100.0	30.0	1	0.0	131	0.765	91.8	Infin.	0.856	0.194	Non-Liq.
33.0	125.1	0	100.0	35.0	1	0.0	125	0.710	85.2	Infin.	0.851	0.193	Non-Liq.
34.0	125.1	0	100.0	35.0	1	0.0	125	0.710	85.2	Infin.	0.847	0.192	Non-Liq.
35.0	125.1	0	100.0	35.0	1	0.0	125	0.710	85.2	Infin.	0.842	0.191	Non-Liq.
36.0	125.1	0	100.0	35.0	1	0.0	125	0.710	85.2	Infin.	0.838	0.190	Non-Liq.
37.0	125.1	0	100.0	35.0	1	0.0	125	0.710	85.2	Infin.	0.833	0.189	Non-Liq.
38.0	108.3	0	85.0	40.0	1	0.0	110	0.666	67.9	Infin.	0.829	0.188	Non-Liq.
39.0	108.3	0	85.0	40.0	1	0.0	110	0.666	67.9	Infin.	0.824	0.187	Non-Liq.
40.0	108.3	0	85.0	40.0	1	0.0	110	0.666	67.9	Infin.	0.819	0.186	Non-Liq.
41.0	108.3	0	85.0	40.0	1	0.0	110	0.666	67.9	Infin.	0.815	0.185	Non-Liq.
42.0	108.3	0	85.0	40.0	1	0.0	110	0.666	67.9	Infin.	0.810	0.184	Non-Liq.
43.0	117.2	0	85.0	45.0	1	0.0	106	0.629	64.2	Infin.	0.806	0.183	Non-Liq.
44.0	117.2	0	85.0	45.0	1	0.0	106	0.629	64.2	Infin.	0.801	0.182	Non-Liq.
45.0	117.2	0	85.0	45.0	1	0.0	106	0.629	64.2	Infin.	0.797	0.181	Non-Liq.
46.0	117.2	0	100.0	45.0	1	0.0	115	0.629	75.5	Infin.	0.792	0.180	Non-Liq.
47.0	117.2	0	100.0	45.0	1	0.0	115	0.629	75.5	Infin.	0.787	0.179	Non-Liq.
48.0	126.4	0	100.0	45.0	1	0.0	115	0.629	75.5	Infin.	0.783	0.178	Non-Liq.
49.0	126.4	0	100.0	45.0	1	0.0	115	0.629	75.5	Infin.	0.778	0.177	Non-Liq.
50.0	126.4	0	100.0	50.0	1	0.0	110	0.600	72.0	Infin.	0.774	0.176	Non-Liq.

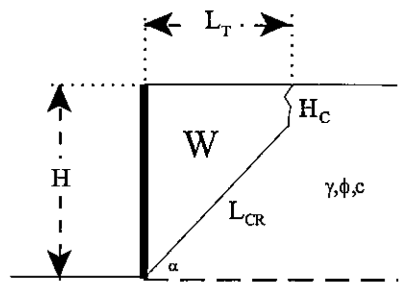
Geotechnologies, Inc.

Project: R.L. Binder Architecture and Planing
File No.: 19790
Description: Retaining Wall

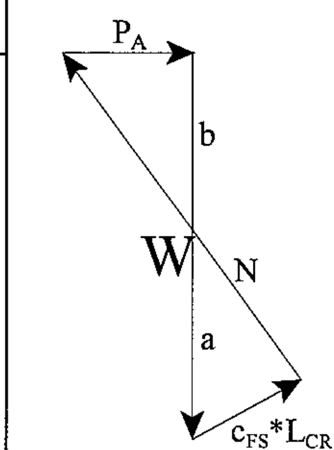
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	134.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	455.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	19.5 degrees
	(c_{FS})	303.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
35	9.8	93	12419.0	9.1	9789.3	2629.7	728.4
36	9.3	95	12780.7	9.7	9779.8	3000.8	887.9
37	8.9	97	12973.3	10.1	9657.5	3315.8	1044.3
38	8.5	97	13039.2	10.5	9461.7	3577.5	1195.8
39	8.2	97	13009.3	10.8	9219.4	3790.0	1340.8
40	8.0	96	12906.7	11.0	8949.1	3957.6	1478.3
41	7.7	95	12748.5	11.1	8663.5	4085.1	1607.7
42	7.5	94	12547.8	11.2	8371.2	4176.6	1728.5
43	7.3	92	12314.5	11.3	8078.4	4236.1	1840.3
44	7.2	90	12056.4	11.3	7789.0	4267.4	1943.1
45	7.0	88	11779.4	11.3	7505.8	4273.6	2036.8
46	6.9	86	11488.1	11.3	7230.4	4257.7	2121.1
47	6.8	83	11186.2	11.2	6963.9	4222.3	2196.3
48	6.7	81	10876.5	11.2	6706.9	4169.6	2262.2
49	6.6	79	10561.3	11.1	6459.6	4101.7	2318.9
50	6.5	76	10242.3	11.0	6221.8	4020.4	2366.5
51	6.5	74	9920.8	10.9	5993.5	3927.3	2405.0
52	6.5	72	9598.0	10.8	5774.2	3823.8	2434.4
53	6.4	69	9274.7	10.7	5563.5	3711.2	2454.7
54	6.4	67	8951.5	10.6	5361.0	3590.6	2466.1
55	6.4	64	8628.9	10.5	5166.0	3462.9	2468.4
56	6.4	62	8307.2	10.4	4978.1	3329.0	2461.7
57	6.4	60	7986.5	10.2	4796.7	3189.8	2446.0
58	6.5	57	7667.2	10.1	4621.2	3046.0	2421.3
59	6.5	55	7349.1	9.9	4450.9	2898.2	2387.6
60	6.6	52	7032.3	9.7	4285.3	2747.1	2344.7



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$
$b = W-a$
$P_A = b*\tan(\alpha-\phi_{FS})$
$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 2468.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 21.9 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: R.L. Binder Architecture and Planning
File No.: 19790

Soil Weight	γ	134	pcf
Internal Friction Angle	ϕ	28	degrees
Cohesion	c	455	psf
Height of Retaining Wall	H	15	feet

Restrained Wall Design based on At Rest Earth Pressure

P_o =	7997.7 lbs/ft	
$\sigma'_{h,\,max}$ =	44.4 H	(based on a trapezoidal distribution of pressure)
$\sigma'_{h,\,max}$ =	533.2 psf	

Design restrained wall for 45 H

Geotechnologies, Inc.

Project: R.L. Binder Architecture and Planning
File No.: 19790

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	15.0 feet
Retained Soil Unit Weight:	(γ)	134.0 pcf
Horizontal Ground Acceleration:	(k_h)	0.20 g

Seismic Increment (ΔP_{AE}):

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$
$\Delta P_{AE} =$ 2261.3 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$
$T =$ 2035.1 lbs/ft

$EFP = 2*T/H^2$
EFP = 18.1 pcf
triangular distribution of pressure, inversely applied to the proposed retaining wall.

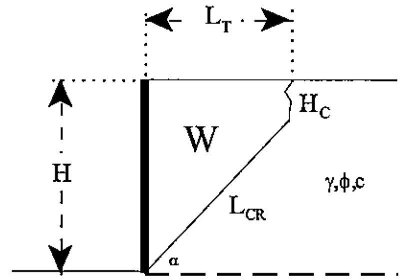
Geotechnologies, Inc.

Project: R.L. Binder Architecture and Planing
File No.: 19790
Description: Shoring up to 15 feet

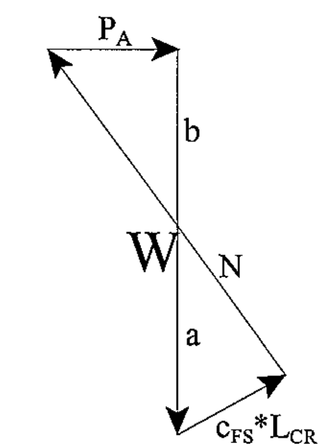
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	134.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	455.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.0 degrees
	(c_{FS})	364.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
35	14.7	6	769.5	0.5	762.5	7.0	1.5
36	13.8	24	3236.8	2.1	3099.7	137.2	31.6
37	13.0	38	5031.9	3.4	4668.1	363.8	90.4
38	12.3	47	6340.4	4.4	5711.0	629.4	168.1
39	11.7	54	7289.9	5.2	6388.9	901.0	257.6
40	11.2	59	7970.3	5.9	6810.3	1160.0	353.7
41	10.7	63	8446.2	6.5	7049.5	1396.6	452.6
42	10.4	65	8764.7	6.9	7158.7	1605.9	551.6
43	10.0	67	8961.0	7.3	7174.8	1786.2	648.6
44	9.7	68	9061.4	7.6	7124.1	1937.3	742.0
45	9.5	68	9086.1	7.8	7025.6	2060.5	830.7
46	9.2	68	9050.6	8.0	6893.4	2157.2	913.7
47	9.0	67	8966.9	8.2	6737.6	2229.3	990.5
48	8.9	66	8844.6	8.3	6565.7	2278.9	1060.6
49	8.7	65	8691.0	8.3	6383.1	2307.9	1123.5
50	8.6	64	8512.1	8.4	6193.9	2318.2	1179.0
51	8.5	62	8312.7	8.4	6001.1	2311.6	1226.9
52	8.4	60	8096.5	8.4	5806.7	2289.8	1267.0
53	8.3	59	7866.6	8.4	5612.3	2254.2	1299.2
54	8.3	57	7625.4	8.3	5419.0	2206.4	1323.5
55	8.2	55	7375.0	8.3	5227.3	2147.7	1339.8
56	8.2	53	7116.8	8.2	5037.6	2079.2	1348.0
57	8.2	51	6852.2	8.1	4850.2	2002.0	1348.2
58	8.2	49	6582.2	8.0	4665.0	1917.2	1340.3
59	8.3	47	6307.5	7.9	4481.8	1825.7	1324.4
60	8.3	45	6028.8	7.7	4300.3	1728.5	1300.4



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1348.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 12.0 pcf

Design Shoring for an Equivalent Fluid Pressure: 25 pcf

SCHIFF ASSOCIATES
FIFTY YEARS OF PROFESSIONALISM

www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

February 26, 2009 via email: gvarela@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Avenue
Glendale, CA 91201

Attention: Mr. Gregorio Varela

Re: Soil Corrosivity Study
R. L. Binder
Banning, CA
SA #09-0116SCS, GI#19790

INTRODUCTION

Laboratory tests have been completed on two soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. Schiff Associates assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed construction is a one story building with basement. The site is located at the northeast corner of Ramsey Street and Martin Street in Banning, California. The water table is reportedly greater than 50 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, Schiff Associates will be happy to work with them as a separate phase of this project.

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D513. Test results are shown in Table 1.

SOIL CORROSIVITY

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

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,000 to 10,000	Moderately Corrosive
1,000 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

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

Electrical resistivities were in the mildly corrosive to moderately corrosive categories with as-received moisture. When saturated, the resistivities were in the mildly corrosive to moderately corrosive categories. One of the two as-received resistivities was at or near their saturated values. The other resistivity dropped considerably with added moisture because the sample was dry as-received.

Soil pH values varied from 7.4 to 7.6. This range is mildly alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

Nitrate was detected in low concentration. The ammonium concentration was high enough to be deleterious to copper.

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

This soil is classified as moderately corrosive to ferrous metals, and aggressive to copper.

---

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

CORROSION CONTROL RECOMMENDATIONS

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

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of possible future cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
3. To prevent dissimilar metal corrosion cells and to facilitate the application of possible future cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
    a. Dissimilar metals.
    b. Dissimilarly coated piping (cement-mortar vs. dielectric).
    c. Above ground steel pipe.
    d. All existing piping.
4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable dielectric coating intended for underground use such as:
        i. Polyurethane per AWWA C222 *or*
        ii. Extruded polyethylene per AWWA C215 *or*
        iii. A tape coating system per AWWA C214 *or*
        iv. Hot applied coal tar enamel per AWWA C203 *or*
        v. Fusion bonded epoxy per AWWA C213.

    b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. In lieu of cathodic protection, the installation of electrical resistance (ER) probes designed for steel piping should be incorporated into the corrosion monitoring system to discern if/when cathodic protection will be warranted in the future. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to dielectric coating, ER Probes, and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevator

Implement *all* the following measures:

1. Coat hydraulic elevator cylinders as described above for steel pipe, item #4.
2. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.
3. Apply cathodic protection to hydraulic cylinders as per NACE Standard SP0169.
4. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.
5. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

OPTION 1

a. Provide a bonded dielectric coating.
b. Electrically isolate the pipeline.
c. Apply cathodic protection to steel piping as per NACE Standard SP0169.

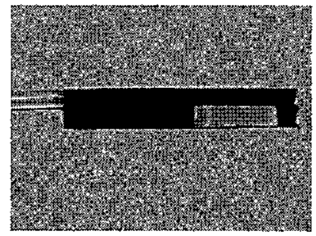
OPTION 2

a. Place the oil line in a PVC casing pipe with solvent-welded joints to prevent contact with soil and soil moisture.

Iron Pipe

Implement *all* the following measures:

1. To avoid creating corrosion problems, cast and ductile iron piping should not be placed partially in contact with both soil and concrete such as thrust blocks. To prevent contact, use a bonded dielectric coating, linear low-density polyethylene per AWWA C105, or wax tape per AWWA C217. Note, the thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.
2. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.
3. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.



4. Install electrical resistance (ER) probes designed for cast and ductile iron piping to discern if/when cathodic protection will be warranted in the future.

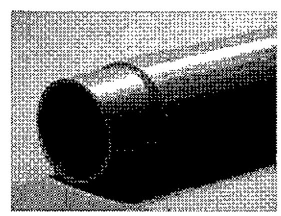
OPTION

a. As an alternative to dielectric coating, ER Probes, and possible future cathodic protection, apply a ¾-inch cement mortar coating or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

Copper Tubing

Protect buried copper tubing by *one* of the following measures:

1. Prevention of soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints.
2. Installation of a factory-coated copper pipe with a minimum 25-mil thickness such as Kamco's Aqua Shield™, Mueller's Streamline Protec™, or equal. The coating must be continuous with no cuts or defects.



3. Installation of 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE Standard SP0169.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.
2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

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

Concrete

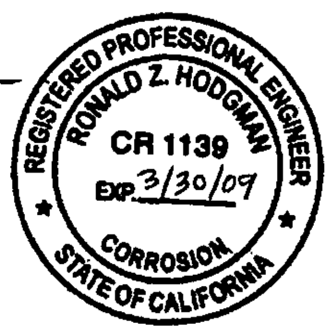
1. From a corrosion standpoint, any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.[3,4,5,6]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

CLOSURE

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

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES



Ronald Z. Hodgman

Leo Solis

Enc: Table 1

09-0116SCS RPT RZH

---

[3] 1997 Uniform Building Code (UBC) Table 19-A-4
[4] 2006 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
[5] 2006 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
[6] 2007 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
[7] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65

SCHIFF ASSOCIATES
FIFTY YEARS OF PROFESSIONALISM
www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
R.L. Binder
Your #19790, SA #09-0116SCS
16-Feb-09

Sample ID			B6 (bulk sample) 1-5' SM/SP	B4 (bulk sample) 1-5' SM
Resistivity		**Units**		
as-received		ohm-cm	30,000	8,800
saturated		ohm-cm	14,000	6,000
pH			7.6	7.4
Electrical Conductivity		mS/cm	0.16	0.22
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	107	156
magnesium	Mg^{2+}	mg/kg	17	20
sodium	Na^{1+}	mg/kg	20	23
potassium	K^{1+}	mg/kg	28	36
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	287	372
flouride	F^{1-}	mg/kg	ND	ND
chloride	Cl^{1-}	mg/kg	9.3	16
sulfate	SO_4^{2-}	mg/kg	57	77
phosphate	PO_4^{3-}	mg/kg	20	19
Other Tests				
ammonium	NH_4^{1+}	mg/kg	26	11
nitrate	NO_3^{1-}	mg/kg	2.1	1.4
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

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

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316