RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

GEOTECHNICAL INVESTIGATION
FOR
PROPOSED INFRASTRUCTURE IMPROVEMENTS PROJECT
SAN BERNARDINO INTERNATIONAL AIRPORT
SAN BERNARDINO, CA

for

Parsons Brinckerhoff
451 E. Vanderbilt Way, Suite 200
San Bernardino, CA 92408

February 11, 2013

RMA Group Project No. 12-406-01

RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

RMA Group Job No.: 12-406-01

February 11, 2013

Parsons Brinckerhoff
451 E. Vanderbilt Way, Suite 200
San Bernardino, CA 92408

Attention: Ronald Sklepko, Project Manager

Subject: Geotechnical Investigation for
SBIAA Infrastructure Improvements Project
San Bernardino International Airport
San Bernardino, CA

Dear Mr. Sklepko:

In accordance with your request, a geotechnical investigation has been completed for the above-referenced project. The report addresses both engineering geologic and geotechnical conditions. The results of the investigation are presented in the accompanying report, which includes a description of site conditions, results of our field exploration and laboratory testing, conclusions, and recommendations.

We appreciate this opportunity to be of continued service to you. If you have any questions regarding this report, please do not hesitate to contact us at your convenience.

Respectfully submitted,

RMA Group

PROFESSIONAL GEOLOGIST
KENNETH R. DOWELL
No. 2470
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

Kenneth Dowell, PG|CEG
Project Geologist
CEG 2470

REGISTERED PROFESSIONAL ENGINEER
SLAWOMIR W. DYMERSKI
G 2764
EXP. 9/30/2014
GEOTECHNICAL
STATE OF CALIFORNIA

Slawek Dymerski, PE|GE
Vice President
GE 2764

12130 Santa Margarita Court Rancho Cucamonga, CA 91730 | T: 909.989.1751 | F: 909.989.4287 | www.rmacompanies.com

TABLE OF CONTENTS

TABLE OF CONTENTS
(Continued)

1.00 INTRODUCTION

1.01 Purpose

A geotechnical investigation has been completed for an infrastructure improvements project at the San Bernardino International Airport in San Bernardino, California. The purpose of the investigation was to summarize geotechnical and geologic conditions at the site, to assess their potential impact on the proposed development, and to develop geotechnical and engineering geologic design parameters.

1.02 Scope of the Investigation

The general scope of this investigation included the following:

- Review of published and unpublished geologic, seismic, groundwater and geotechnical literature.
- Examination of aerial photographs.
- Contacting of underground service alert to locate onsite utility lines.
- Logging, sampling and backfilling of 12 exploratory borings drilled with a CME-75 drill rig.
- Three soil infiltration tests.
- Laboratory testing of representative soil samples.
- Geotechnical evaluation of the compiled data.
- Preparation of this report presenting our findings, conclusions and recommendations.

Our scope of work did not include a preliminary site assessment for the potential of hazardous materials onsite.

1.03 Site Location and Description

The proposed Infrastructure Improvements will be located on the north side of the existing San Bernardino International Airport in the City of San Bernardino, California. The site is generally bounded by X Street to the north, 102st Street to the east, 99th Street to the west and by Taxiway E to the south. The geographic position of the approximate center of the site is 34.1031° latitude and -117.22693° longitude. The approximate location of the site is illustrated on Figure 1.

The overall the property slopes to the southwest at approximately a 1 to 2% gradient.

At the time of our study, vegetation consisted of sparse cover of grasses and several large trees, particularly in the northern portion of the site.

Approximately the northern two-thirds of the site was vacant at the time of our investigation. Several asphalt paved roads cross the site: 99th, 101st, 102nd, X, Y and U Streets. Two vacant wooden buildings and a masonry restroom building and a water tower and a small storage building, along with several concrete building slabs are located in the northern portion of the site. The southern third of the site is covered in asphalt and concrete. A chain link fence surrounds a portion of the site at the southwest corner of 102nd and U streets.

1.04 Current and Past Land Usage

The northern two-thirds of the site are currently vacant and the southern portion of the site is currently used as a school bus yard. Prior to its current usage, the site was used as a portion of Norton Air Force Base. Based upon a review of historic aerial photographs several buildings were located in the northern two-thirds of the site. About a dozen of these buildings were demolished prior to 1990. The fenced area at the southwest corner of 100th and U Streets was used for equipment storage during the mid 1990's to the mid 2000's.

The area currently used for bus parking was previously used for car storage in 2009 and for container storage in the mid 2000's. Prior to these usages this area was used as part of the aviation operations at the airport

1.05 Planned Usage

It is our understanding that the proposed construction will consist of three general aviation aircraft hangers in the southeastern portion of the site, a San Bernardino County Sheriff Aviation Facility consisting of storage and maintenance hangers and a 2-story office building in the south western portion of the site. In addition to the proposed structures a taxiway is planned between the general aviation area and the Sheriff's aviation facility. Additional improvement will include repaving of 99th Street, Victoria Avenue (100th Street), X Street and a portion of U Street.

Our investigation was performed prior to the preparation of grading or foundation plans. To aid in preparation of this report, we utilized the following assumptions:

- Maximum foundation loads of 2 to 3 kips per linear foot for continuous footings and 60 kips for isolated spread footings.
- Cuts and fills will be less than 5 feet.

1.06 Investigation Methods

Our investigation consisted of office research, field exploration, laboratory testing, review of the compiled data, and preparation of this report. It has been performed in a manner consistent with generally accepted engineering and geologic principles and practices, and has incorporated applicable requirements of California Buildings Code. Definitions of technical terms and symbols used in this report include those of the ASTM International, the California Building Code, and commonly used geologic nomenclature. The field exploration, laboratory testing and recommended pavement structural sections in the areas subject to aircraft traffic were prepared utilizing the guidelines for the design of pavements for light aircraft contained within the Federal Aviation Administration (FAA) advisory circular 150/5320-6E.

Technical supporting data are presented in the attached appendices. Appendix A presents a description of the methods and equipment used in performing the field exploration and logs of our subsurface exploration. Appendix B presents a description of our laboratory testing and the test results. Standard grading specifications and references are presented in Appendices C and D, respectively.

2.00 FINDINGS

2.01 Geologic Setting

The site is located within the eastern portion of the Upper Santa Ana River Valley, just southwest of the foothills of the San Bernardino Mountains. The Santa Ana River Valley is a deep structural depression that is locally filled with sediment derived from the adjoining San Bernardino Mountains. According to Fife and Rodgers (1974), the alluvial deposits beneath the site are approximately 800 feet thick. The San Bernardino Mountains are basically a block that has been uplifted along bounding faults. The majority of the mountain range is underlain by Cretaceous age granitic bedrock. Other rock types within the mountains include metamorphic schists and gneisses, and marine sedimentary deposits. The bedrock units are in places overlain by Quaternary age surficial deposits composed of lake bed, stream channel and alluvial fan deposits. The Upper Santa Ana River Valley is separated from the San Bernardino Mountains be the San Andreas fault, which is located approximately 3½ miles northeast of the site at its nearest point. The map indicates alluvial soils underlying the site are late to early Holocene in age (Figure 2).

2.02 Earth Materials

The soils underlying the airfield consist primarily of artificial fill and coarse grained alluvial and aeolian sands with varying amounts of silt and gravel. The soils are classified as SM, SM-SP, SP or combinations of these types by the Unified Soils Classification System.

The artificial fill consisted of asphalt underlain by aggregate base. The asphalt ranged in thickness from 2 ½ inches to 4 inches in thickness. The aggregate base ranged from 4 to 12 inches thick. These fills were located in the southern portion of the site. In addition, thin surficial fill deposits were observed along the existing paved roads. Additional artificial fills are likely located beneath the existing buildings and areas of demolished buildings.

In the northern portion of the site, the upper 3 to 5 feet of soils are loose to medium dense and then become medium dense to dense below a depth of 2 feet beneath the site. In the southern portion of the site currently covered by asphalt, the soils are generally medium dense to dense. In general the near surface soils encountered at the site were non-corrosive to metals, had negligible soluble sulfate and chloride contents, non-expansive and generally dry.

The subsurface soils encountered in the exploratory borings drilled at the site are described in greater detail on the logs contained in Appendix A.

2.03 Existing Pavement Sections

The southern portion of the General Aviation and Sheriff's Aviation facility is paved with asphalt and currently used as a bus yard. In general within the Sheriff's Aviation Facility the pavement section consisted of 4 to 6 inches of asphalt over 10 to 12 inches of aggregate base. Along the northern 30 feet of the existing pavement the pavement section thins to 2 ½ inches of asphalt over 10 inches of base. Also in this area we encountered metal wire mesh at the bottom of the aggregate base section.

The existing pavement section within the area of the proposed General Aviation hangers and the proposed taxiway consisted of 4 inches of asphalt over 4 inches of aggregate base.

Observation of the bus yard paving indicated that it had low severity alligator cracking, low severity to medium severity block cracking and low severity joint cracking. The slurry seal on the pavement appeared to be in moderate condition.

Existing pavement sections encountered in our borings are shown in the Table below:

THICKNESS OF COMPACTED BITUMINOUS PAVING MIXTURES SPECIMENS
(Test Method: ASTM D3549)

Boring No.	Asphalt Core Thickness (inches)	Aggregate Base Thickness (inches)
B-7	4" AC	4" AB
B-8	4" AC	4" AB
B-9	6" AC	12" AB
B-10	4" AC	12" AB
B-11	2½" AC	10" AB

Pavement within the existing streets is in very poor condition.

2.04 Expansive Soils

Expansion testing performed in accordance with ASTM D4829 indicates that earth materials underlying the site have an expansion classification of very low. Results of the expansion test are presented in Appendix B. Since site grading will redistribute earth materials, potential expansive properties should be verified at the completion of rough grading.

2.05 Surface and Groundwater Conditions

No areas of ponding or standing water were present at the time of our study. Further, no springs or areas of natural seepage were found.

Groundwater was not encountered in our borings that extended to a maximum depth of 51.5 feet. According to the Matti and Carson (1986, 1991), the minimum depth to ground water in the vicinity of the site between 1973 and 1983 was on the order of 40 to 50 feet. According to Carson and Matti (1985) the depth to groundwater in 1973 to 1979 was more than 100 feet and Fife (1974) indicates depth to groundwater in 1960 at greater than 125 feet.

2.06 Faults

The site is not located within the boundaries of an Earthquake Fault Zone for fault-rupture hazard as defined by the Alquist-Priolo Earthquake Fault Zoning Act or in a County of San Bernardino or City of Highland fault rupture hazard zone. The nearest Alquist-Priolo Zone, which is located approximately 3 miles northeast of the site at its nearest point, has been established along the San Bernardino Strand of the San Andreas fault. This is also the nearest City and County designated fault rupture hazard zone. The nearest mapped fault to the site is the concealed trace of the Banning fault, located approximately 2 miles to the south (Figure 4).

The accompanying Regional Fault Map (Figure 5) illustrates the location of the site with respect to major faults in the region. The distance to notable faults within 100 kilometers of the site is presented on Table 1.

2.07 Historic Seismicity

There have been three large historic earthquakes epicentered within about 20 miles of the site. These events were epicentered in the San Bernardino area in 1858, 1907 and 1923. All of the earthquakes occurred prior to development of seismic monitoring networks and thus their locations and magnitudes are only approximate. These large historic earthquakes and others in the region are summarized in Table 1.

Seismic design parameters relative to the requirements of the 2010 California Building Code are presented in Section 3.09.

2.08 Flooding Potential

According to the City of San Bernardino General Plan (2010), the site is not located within located within a 100 year flood zone. The City General Plan Figure S-1 depicts flood zones are based on 1990 FEMA Flood Insurance Rate maps. According to the City of San Bernardino General Plan the site is located within a dam inundation area for the Seven Oaks Dam.

Control of surface runoff originating from within and outside of the site should, of course, be included in design of the project.

2.09 Landslides

Due to the low gradient of the site and immediately surrounding area, landsliding is not a hazard at this property.

3.00 CONCLUSIONS AND RECOMMENDATIONS

3.01 General Conclusion

Based on specific data and information contained in this report, our understanding of the project and our general experience in engineering geology and geotechnical engineering, it is our professional judgment that the proposed development is geologically and geotechnically feasible. This is provided that the recommendations presented below are fully implemented during design, grading and construction.

3.02 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications outlined in Appendix C, unless specifically revised or amended below. Recommendations contained in Appendix C are general specifications for typical grading projects and may not be entirely applicable to this project.

It is also recommended that all earthwork and grading be performed in accordance with Appendix J of the 2010 California Building Code and all applicable governmental agency requirements. In the event of conflicts between this report and Appendix J, this report shall govern.

3.03 Earthwork Shrinkage and Subsidence

Shrinkage is the decrease in volume of soil upon removal and recompaction expressed as a percentage of the original in-place volume. Subsidence occurs as natural ground is densified to receive fill. These factors account for changes in earth volumes that will occur during grading. Our estimates are as follows:

- Shrinkage factor = 10%-15% for soil removed and replaced as compacted fill.
- Subsidence factor = 0.1 foot.

The degree to which fill soils are compacted and variations in the insitu density of existing soils will influence earth volume changes. Consequently, some adjustments in grades near the completion of grading could be required to balance the earthwork.

3.04 Removals and Overexcavation

All vegetation, trash and debris should be cleared from the grading area and removed from the site. Prior to placement of compacted fills, all non-engineered fills and loose, porous, or compressible soils will need to be removed down to competent ground. Removal and requirements will also apply to cut areas, if the depth of cut is not sufficient to reach competent ground. Removed and/or overexcavated soils may be moisture-conditioned and recompacted as engineered fill, except for soils containing detrimental amounts of organic material. Estimated depths of removals are as follows:

- Non-engineered fill ranging from 1 to 2 feet deep was observed along the existing paved streets within the site. It is anticipated the additional fills will be encountered beneath the existing buildings and in the locations of the former buildings. Complete removal of these fills and underlying compressible native soils will need to be performed. If other non-engineered fills are encountered during grading, they will also need to be removed along with any underlying compressible native soils.

- Loose, porous and compressible native soils were encountered in the northern portion of the site to depths of about 3 to 5 feet below existing grades. The average depth of removal of these soils is expected to be 5 feet with some local areas extending deeper.

- It is expected that competent native soils will be encountered in cuts deeper than approximately 3 to 5 feet below existing grade or the base of existing non-engineered fill or ground surface in the northern section of the site. In the bus parking area currently underlain by asphalt it is expected that competent native soils will be encountered within 1 to 2 feet below the existing ground surface. Provided competent soils are exposed, these cut surfaces should be scarified to a minimum depth of 12 inches, moisture conditioned and compacted to at least 90 percent of the maximum dry density, provided that footing overexcavation requirements are met.

- Soils disturbed by demolition of existing structures will need to be over-excavated to competent native ground and then scarified to a minimum depth of 12 inches, moisture conditioned and compacted to at least 90 percent of the maximum dry density

- The asphalt and concrete currently onsite may be either processed and placed in the compacted fill, or hauled off the site. If the asphalt and concrete is use as fill material, it must be broken down to approximately 4 to 8-inch particles and mixed thoroughly with on-site soils. No large and flat pieces are to be used for fill. If asphalt is processed by grinding, it cannot be used in fills and must be removed from the site.

- Wire mesh was encountered beneath the asphalt paved portion of the Sheriff Aviation Facility. The mesh was encountered in our borings B-9, B-10 and B-11, generally at the base of the aggregate base layer. If encountered during grading, the wire mesh should be removed from soil and hauled offsite.

In addition to the above requirements, overexcavation will also need to meet the following criteria for the building pads, concrete flatwork and pavement areas:

- All footing areas, both continuous and spread, shall be undercut, moistened, and compacted as necessary to produce soils compacted to a minimum of 90% relative compaction to a depth equal to the width of the footing below the bottom of the footing or to a depth of 3 feet below the bottom of the footing, whichever is less. Footing areas shall be defined as the area extending from the edge of the footing for a distance of 5 feet.

- All floor slabs, concrete flatwork and paved areas shall be underlain by a minimum of 12 inches of soil compacted to a minimum of 90% relative compaction.

The exposed soils beneath all overexcavation should be scarified an additional 12 inches, moisture conditioned and compacted to a minimum of 90% relative compaction.

The above recommendations are based on the assumption that soils encountered during field exploration are representative of soils throughout the site. However, there can be unforeseen and unanticipated variations in soils between points of subsurface exploration. Hence, overexcavation depths must be verified, and adjusted if necessary, at the time of grading. The overexcavated materials may be moisture-conditioned and re-compacted as engineered fill.

3.05 Rippability and Rock Disposal

Our exploratory borings were advanced without difficulty to depths greater than ten feet before meeting refusal due to subsurface gravels and cobbles. Accordingly we expect that all near surface earth materials will be rippable with conventional heavy duty grading equipment and oversized materials are not expected.

3.06 Subdrains

Ground water and surface water were not encountered during the course of our investigation, the proposed grading is will not fill any large canyons and the underlying soils are fairly permeable. Consequently, installation of canyon subdrains is not expected to be necessary.

3.07 Fill and Cut Slopes

Due to the low gradient of the property, it appears that construction of cut and fill slopes will not be required. If such slopes are proposed, they should be inclined no steeper than 2 horizontal to 1 vertical.

3.08 Faulting

Since the site is not located within the boundaries of an Earthquake Fault Zone and no faults are known to pass through the property, surface fault rupture within the site is considered unlikely.

3.09 Seismic Design Parameters

Seismic design parameters have been developed in accordance with Section 1613 of the 2010 California Building Code (CBC) using the online U.S. Geological Survey Java Ground Motion Parameter Calculator (Version 5.1.0, ASCE 7 Standard) and a site location based on latitude and longitude. The calculator generates probabilistic and deterministic maximum considered earthquake spectral parameters represented by a 5-percent damped acceleration response spectrum having a 2-percent probability of exceedance in 50 years. The deterministic response accelerations are calculated as 150 percent of the largest median 5-percent damped spectral response acceleration computed on active faults within a region, where the deterministic values govern. The calculator does not, however, produce separate probabilistic and deterministic results. The parameters generated for the subject site are presented below:

2010 California Building Code (CBC) Seismic Parameters

Parameter	Value
Site Location	Latitude = 34.1031 degrees Longitude = -117.2269 degrees
Site Class	Site Class = D Soil Profile Name = Stiff soil
Mapped Spectral Accelerations (Site Class B)	S_s (0.2- second period) = 1.1739g S_1 (1-second period) = 0.779g
Site Coefficients (Site Class D)	F_a = 1.0 F_v = 1.5
Maximum Considered Earthquake Spectral Accelerations (Site Class D)	S_{MS} (0.2- second period) = 1.739g S_{M1} (1-second period) = 1.169g
Design Earthquake Spectral Accelerations (Site Class D)	S_{DS} (0.2- second period) = 1.159g S_{D1} (1-second period) = 0.779g

The above table shows that the mapped spectral response acceleration parameter a 1-second period (S_1) $\geq$ 0.75g. Therefore, for Occupancy Categories I, II and II the Seismic Design Category is E and for Occupancy Category IV the Seismic Design Category is F (CBC Table 1604.5 and Section 1613.5.6). Consequently, as required for Seismic Design Categories D through F by CBC Section 1803.5.12, lateral pressures for earthquake ground motions, liquefaction and soil strength loss have been evaluated (see Sections 3.10 and 3.16).

For preliminary design purposes, we recommend a peak ground acceleration, PGA = S_{DS}/2.5 = 1.159g/2.5 = 0.46g.

3.10 Liquefaction and Secondary Earthquake Hazards

Potential secondary seismic hazards that can affect land development projects include liquefaction, tsunamis, seiches, seismically induced settlement, seismically induced flooding and seismically induced landsliding.

Liquefaction

The site is located within liquefaction hazard zones in the City of San Bernardino General Plan and a San

Bernardino County Geologic Hazard Zone for liquefaction. It should be noted that the California Geological Survey has not yet prepared a Seismic Hazard Zone Map of potential liquefaction hazards for the quadrangle in which the site is located.

Liquefaction is a phenomenon where earthquake- induced ground vibrations increase the pore pressure in saturated, granular soils until it is equal to the confining, overburden pressure. When this occurs, the soil can completely loose its shear strength and enter a liquefied state. The possibility of liquefaction is dependent upon grain size, relative density, confining pressure, saturation of the soils, and intensity and duration of ground shaking. In order for liquefaction to occur, three criteria must be met: underlying loose, coarse-grained (sandy) soils, a groundwater depth of less than about 50 feet, and a potential for seismic shaking from nearby large-magnitude earthquake.

Groundwater was not encountered in our borings that extended to a maximum depth of 51.5 feet. The City of San Bernardino County General Plan indicates that all but the northeast corner of the site is located within a Zone of Liquefaction. According to the Matti and Carson (1986, 1991), the minimum depth to ground water in the vicinity of the site between 1973 and 1983 was on the order of 40 to 50 feet. Additionally, the Matti and Carson (1986) report is referenced on the Liquefaction Susceptibility Figure S -5 in the City of San Bernardino General Plan. According to Carson and Matti (1985) the depth to groundwater in 1973 to 1979 was more than 100 feet and Fife (1974) indicates depth to groundwater in 1960 at greater than 125 feet.

Liquefaction analysis was performed using an assumed ground water level of 40 feet based on the depth to groundwater presented in the Matti and Carson 1986 and 1991 reports. The analysis indicates that the soils are non-liquefiable.

Tsunamis and Seiches

Tsunamis are sea waves that are generated in response to large-magnitude earthquakes. When these waves reach shorelines, they sometimes produce coastal flooding. Seiches are the oscillation of large bodies of standing water, such as lakes, that can occur in response to ground shaking. Tsunamis and seiches do not pose hazards due to the inland location of the site and lack of nearby bodies of standing water.

Seismically Induced Settlement

Seismically induced settlement occurs most frequently in areas underlain by loose, granular sediments. Damage as a result of seismically induced settlement is most dramatic when differential settlement occurs in areas with large variations in the thickness of underlying sediments. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Seismic settlement was evaluated for the Design Earthquake event using an empirical method developed by Tokimatsu and Seed (1987) based on site-specific SPT blow count and grain size data obtained from our borings. We estimate ½ inch of total seismically induced ground settlement may occur at the site when subjected to a Design Earthquake event (see calculations in Appendix D). In our opinion, differential seismic settlement may be taken as one-half of the computed total seismic settlement. Calculations of seismically induced settlements are presented in Appendix D.

Seismically Induced Flooding

The Seven Oaks Dam is located approximately 7 miles east of the site. According to the City of San Bernardino

General Plan and San Bernardino County Hazard Overlay Map FH31B, the site is located within the Seven Oaks Dam inundation area. Mitigation of flooding hazards is a planning and civil engineering design issue.

Seismically Induced Landsliding

Due to the low gradient of the site, the potential for seismically induced landsliding is nil. This assumes that any slopes created during development of the site will be properly designed and constructed. It should be noted that the California Geological Survey has not yet prepared a Seismic Hazard Zone Map of potential earthquake-induced landslide hazards for the quadrangle in which the site is located.

3.11 Foundations

Isolated spread footings and/or continuous wall footings are recommended to support the proposed structures. If the recommendations in the section on grading are followed and footings are established in firm native soils or compacted fill materials, footings may be designed using the following allowable soil bearing values:

- Continuous Wall Footings:

  Footings having a minimum width of 12 inches and a minimum depth of 12 inches below the lowest adjacent grade have allowable bearing capacity of 2,000 pounds per square foot (psf). This value may be increased by 20% for each additional foot of width or depth and/or depth to a maximum value of 3,000 psf.

- Isolated Spread Footings:

  Footings having a minimum width of 24 inches and a minimum depth of 12 inches below the lowest adjacent grade have allowable bearing capacity of 3,000 psf. This value may be increased by 20% for each additional foot of width or depth to a maximum value of 4,000 psf.

- Retaining Wall Footings:

  Footings for retaining walls should be founded a minimum depth of 12 inches and have a minimum width of 12 inches. Footings may be designed using the allowable bearing capacity and lateral resistance values recommended for building footings. However, when calculating passive resistance, the upper 6 inches of the footings should be ignored in areas where the footings will not be covered with concrete flatwork. This value may also be increased by 20% for each additional foot of width or depth to a maximum value of 3,000 psf. Reinforcement should be provided for structural considerations as determined by the design engineer.

The above bearing capacities represent an allowable net increase in soil pressure over existing soil pressure and may be increased by one-third for short-term wind or seismic loads. The maximum expected settlement of footings designed with the recommended allowable bearing capacity is expected to be on the order of ½ inch with differential settlement on the order of ¼ inch.

Soils at the site are generally granular, non-plastic and have very low expansion potential in nature. Therefore, reinforcement of footings for expansive soil is not required. However, in view of the seismic setting, a nominal reinforcement consisting of one #4 bar placed within 3 inches of the top of footings and another placed within 3 inches of the bottom of footings is recommended. The structural engineer may require heavier reinforcement.

Due to the preliminary nature of the expansion tests performed for this study, we recommend additional testing be performed near the completion of rough grading to verify the test results and recommended foundation design criteria.

3.12 Foundation Setbacks from Slopes

Setbacks for footings adjacent to slopes should conform to the requirements of the California Building Code. Specifically, footings should maintain a horizontal distance or setback between any adjacent slope face and the bottom outer edge of the footing.

For slopes descending away from the foundation, the horizontal distance may be calculated by using h/3, where h is the height of the slope. The horizontal setback should not be less than 5 feet, nor need not be greater than 40 feet per the California Building Code. Where structures encroach within the zone of h/3 from the top of the slope the setback may be maintained by deepening the foundations. Flatwork and utilities within the zone of h/3 from the top of slope may be subject to lateral distortion caused by gradual downslope creep. Walls, fences and landscaping improvements constructed at the top of descending slopes should be designed with consideration of the potential for gradual downslope creep.

For ascending slopes, the horizontal setback required may be calculated by using h/2 where h is the height of the slope. The horizontal setback need not be greater than 15 feet per the California Building Code.

3.13 Slabs on Grade

Concrete floors with a minimum thickness of 4 inches are recommended for normal floor loading conditions. These floor slabs should have a minimum thickness of 4 inches and should be divided into squares or rectangles using weakened plane joints (contraction joints), each with maximum dimensions not exceeding 15 feet. Contraction joints should be made in accordance with American Concrete Institute (ACI) guidelines. If weakened plane joints are not used, then the slabs shall be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab. If heavy concentrated or moving loads are anticipated, slabs should be designed using a modulus of subgrade reaction (k) of 200 psi per inch per inch and reinforced accordingly.

Special care should be taken on floors slabs to be covered with thin-set tile or other inflexible coverings. These areas may be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab, to mitigate drying shrinkage cracks. Alternatively, inflexible flooring may be installed with unbonded fabric or liners to prevent reflection of slab cracks through the flooring.

A moisture vapor retarder/barrier is recommended beneath all slabs-on-grade that will be covered by moisture-sensitive flooring materials such as vinyl, linoleum, wood, carpet, rubber, rubber-backed carpet, tile, impermeable floor coatings, adhesives, or where moisture-sensitive equipment, products, or environments will exist. We recommend that design and construction of the vapor retarder or barrier conform to Section 1805 of the 2010 California Building Code (CBC) and pertinent sections of American Concrete Institute (ACI) guidance documents 302.1R-04, 302.2R-06 and 360R-10.

The moisture vapor retarder/barrier should consist of a minimum 10 mils thick polyethylene with a maximum perm rating of 0.3 in accordance with ASTM E 1745. Seams in the moisture vapor retarder/barrier should be overlapped no less than 6 inches or in accordance with the manufacturer's recommendations. Joints and penetrations should be sealed with the manufacturer's recommended adhesives, pressure-sensitive tape, or both. The contractor must avoid damaging or puncturing the vapor retarder/barrier and repair any punctures with additional polyethylene properly lapped and sealed.

ACI guidelines allow for the placement of moisture vapor retarder/barriers either directly beneath floor slabs or below an intermediate granular soil layer.

Placing the moisture retarder/barrier directly beneath the floor slab will provide improved curing of the slab bottom and will eliminate potential problems caused by water being trapped in a granular fill layer. Concrete slabs poured directly on a vapor retarder/barrier can experience shrinkage cracking and curling due to differential rates of curing through the thickness of the slab. Therefore, for concrete placed directly on the vapor retarded, we recommend a maximum water cement ratio of 0.45 and the use of water-reducing admixtures to increase workability and decrease bleeding.

If granular soil is placed over the vapor retarder/barrier, we recommend that the layer be at least 2 inches thick in accordance with traditional practice in southern California. Granular fill should consist of clean fine graded materials with 10 to 30% passing the No. 100 sieve and free from clay or silt. The granular layer should be uniformly compacted and trimmed to provide the full design thickness of the proposed slab. The granular fill layer should not be left exposed to rain or other sources of water such as wet-grinding, power washing, pipe leaks or other processes, and should be dry at the time of concrete placement. Granular fill layers that become saturated should be removed and replaced prior to concrete placement.

An additional layer of sand may be placed beneath the vapor retarder/barrier at the developer's discretion to minimize the potential of the retarder/barrier being punctured by underlying soils.

3.14 Miscellaneous Concrete Flatwork

Miscellaneous concrete flatwork and walkways may be designed with a minimum thickness of 4 inches. Large slabs should be reinforced with a minimum of 6x6-10/10 welded wire mesh placed at mid-height in the slab. Control joints should be constructed to create squares or rectangles with a maximum spacing of 15 feet.

Walkways may be constructed without reinforcement. Walkways should be separated from foundations with a thick expansion joint filler. Control joints should be constructed into non-reinforced walkways at a maximum of 5 feet spacing.

The subgrade soils beneath all miscellaneous concrete flatwork should be compacted to a minimum of 90 percent relative compaction for a minimum depth of 12 inches. The geotechnical engineer should monitor the compaction of the subgrade soils and perform testing to verify that proper compaction has been obtained.

3.15 Footing Excavation and Slab Preparations

All footing excavations should be observed by the geotechnical consultant to verify that they have been excavated into competent soils. The foundation excavations should be observed prior to the placement of forms, reinforcement steel, or concrete. These excavations should be evenly trimmed and level. Prior to concrete placement, any loose or soft soils should be removed. Excavated soils should not be placed on slab or footing areas unless properly compacted.

Prior to the placement of the moisture barrier and sand, the subgrade soils underlying the slab should be observed by the geotechnical consultant to verify that all under-slab utility trenches have been properly backfilled and compacted, that no loose or soft soils are present, and that the slab subgrade has been properly compacted to a minimum of 90 percent relative compaction within the upper 12 inches.

Footings may experience and overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom of the trench.

Slabs on grade and walkways should be brought to a minimum of 2% and a maximum of 6% above their optimum moisture content for a depth of 18 inches prior to the placement of concrete. The geotechnical consultant should perform insitu moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

3.16 Lateral Load Resistance

Lateral loads may be resisted by soil friction and the passive resistance of the soil. The following parameters are recommended.

- Passive Earth Pressure = 407 pcf (equivalent fluid weight).
- Coefficient of Friction (soil to footing) = 0.41
- Retaining structures should be designed to resist the following lateral active earth pressures:

Surface Slope of Retained Materials (Horizontal:Vertical)	Equivalent Fluid Weight (pcf)
Level	35
5:1	37
4:1	39
3:1	41
2:1	52

These active earth pressures are only applicable if the retained earth is allowed to strain sufficiently to achieve the active state. The required minimum horizontal strain to achieve the active state is approximately 0.0025H. Retaining structures should be designed to resist an at-rest lateral earth pressure if this horizontal strain cannot be achieved.

- At-rest Lateral Earth Pressure = 55 pcf (equivalent fluid weight)

The Mononobe-Okabe method is commonly utilized for determining seismically induced active and passive lateral earth pressures and is based on the limit equilibrium Coulomb theory for static stress conditions. This method entails three fundamental assumptions (e.g., Seed and Whitman, 1970): Wall movement is sufficient to ensure either active or passive conditions, the driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the heel of the wall and extending to the free surface of the backfill, and the driving soil wedge and the retaining structure act as rigid bodies, and therefore, experiences uniform accelerations throughout the respective bodies (U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures).

- Seismic Lateral Earth Pressure = 71 pcf (equivalent fluid weight).

The seismic lateral earth pressure given above is an inverted triangle, and the resultant of this pressure is an increment of force which should be applied to the back of the wall in the upper 1/3 of the wall height and also applied as a reduction of force to the front of the wall in the upper 1/3 of the footing depth.

3.17 Drainage and Moisture Proofing

Surface drainage should be directed away from the proposed structure into suitable drainage devices. Neither excess irrigation nor rainwater should be allowed to collect or pond against building foundations or within low-lying or level areas of the lot. Surface waters should be diverted away from the tops of slopes and prevented from draining over the top of slopes and down the slope face.

Walls and portions thereof that retain soil and enclose interior spaces and floors below grade should be waterproofed and dampproofed in accordance with CBC Section 1805A.

Retaining structures should be drained to prevent the accumulation of subsurface water behind the walls. Backdrains should be installed behind all retaining walls exceeding 3 feet in height. A typical detail for retaining wall back drains is presented in Appendix C. All backdrains should be outlet to suitable drainage devices. Retaining wall less than 3 feet in height should be provided with backdrains or weep holes. Dampproofing and/or waterproofing should also be provided on all retaining walls exceeding 3 feet in height.

3.18 Cement Type and Corrosion Potential

Soluble sulfate tests indicate that concrete at the subject site will have a negligable exposure to water-soluble sulfate in the soil. Our recommendations for concrete exposed to sulfate-containing soils are presented in the table below.

RECOMMENDATIONS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOILS

Sulfate Exposure	Water Soluble Sulfate (SO_4) in Soil (% by Weight)	Sulfate (SO_4) in Water (ppm)	Cement Type (ASTM C150)	Maximum Water-Cement Ratio (by Weight)	Minimum Compressive Strength (psi)
Negligible	0.00 - 0.10	0-150	--	--	2,500
Moderate	0.10 - 0.20	150-1,500	II	0.50	4,000
Severe	0.20 - 2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan or slag	0.45	4,500

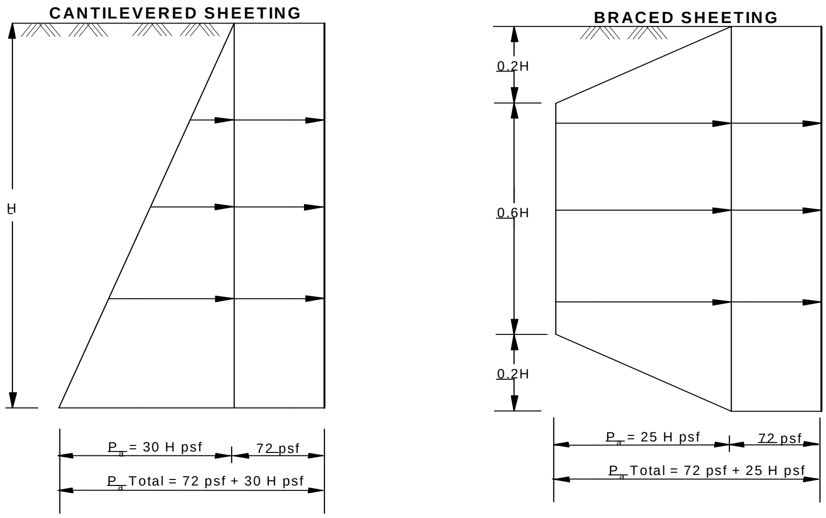
Use of alternate combinations of cementitious materials may be permitted if the combinations meet design recommendations contained in American Concrete Institute guideline ACI 318-11.

The soils were also tested for soil reactivity (pH) and electrical resistivity (ohm-cm). The test results indicate that the on-site soils have a soil reactivity ranging from 5.48 to 7.34 and an electrical resistivity ranging from 420 to 6,200 ohm-cm. A neutral or non-corrosive soil has a value ranging from 5.5 to 8.4. Generally, soils that could be considered moderately corrosive to ferrous metals have resistivity values of about 3,000 ohm-cm to 10,000 ohm-cm. Soils with resistivity values less than 3,000 ohm-cm can be considered corrosive and soils with resistivity values less than 1,000 ohm-cm can be considered extremely corrosive.

Based on our analysis, it appears that the underlying onsite soils are corrosive to ferrous metals. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. A qualified corrosion engineer should be consulted to further assess the corrosive properties of the soil.

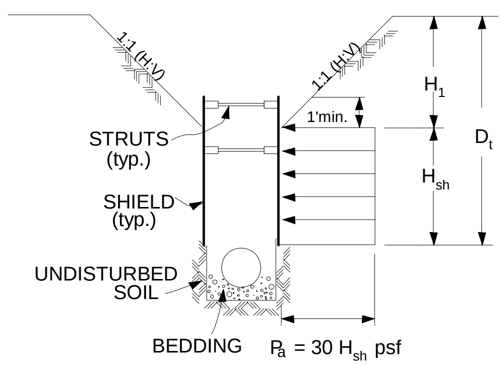
3.19 Temporary Slopes

Excavation of utility trenches will require either temporary sloped excavations or shoring. Temporary excavations in existing alluvial soils may be safely made at an inclination of 1:1 or flatter. If vertical sidewalls are required in excavations greater than 5 feet in depth, the use of cantilevered or braced shoring is recommended. Excavations less than 5 feet in depth may be constructed with vertical sidewalls without shoring or shielding. Our recommendations for lateral earth pressures to be used in the design of cantilevered and/or braced shoring are presented below. These values incorporate a uniform lateral pressure of 72 psf to provide for the normal construction loads imposed by vehicles, equipment, materials, and workmen on the surface adjacent to the trench excavation. However, if vehicles, equipment, materials, etc., are kept a minimum distance equal to the height of the excavation away from the edge of the excavation, this surcharge load need not be applied.



SHORING DESIGN: LATERAL SHORING PRESSURES

Design of the shield struts should be based on a value of 0.65 times the indicated pressure, Pa, for the approximate trench depth. The wales and sheeting can be designed for a value of 2/3 the design strut value.



HEIGHT OF SHIELD, H_{sh} = DEPTH OF TRENCH, D_t , MINUS DEPTH OF SLOPE, H_1

TYPICAL SHORING DETAIL

Placement of the shield may be made after the excavation is completed or driven down as the material is excavated from inside of the shield. If placed after the excavation, some overexcavation may be required to allow for the shield width and advancement of the shield. The shield may be placed at either the top or the bottom of the pipe zone. Due to the anticipated thinness of the shield walls, removal of the shield after construction should have negligible effects on the load factor of pipes. Shields may be successively placed with conventional trenching equipment.

Vehicles, equipment, materials, etc. should be set back away from the edge of temporary excavations a minimum distance of 15 feet from the top edge of the excavation. Surface waters should be diverted away from temporary excavations and prevented from draining over the top of the excavation and down the slope face. During periods of heavy rain, the slope face should be protected with sandbags to prevent drainage over the edge of the slope, and a visqueen liner placed on the slope face to prevent erosion of the slope face.

Periodic observations of the excavations should be made by the geotechnical consultant to verify that the soil conditions have not varied from those anticipated and to monitor the overall condition of the temporary excavations over time. If at any time during construction conditions are encountered which differ from those anticipated, the geotechnical consultant should be contacted and allowed to analyze the field conditions prior to commencing work within the excavation.

Cal/OSHA construction safety orders should be observed during all underground work.

3.20 Soil Infiltration

Four soil infiltration tests have been performed at the above-referenced site. The tests were performed using the Percolation Test Procedure as outlined in the referenced San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP).

On-site infiltration system design was not available at the time this report was prepared, although it is anticipated that on-site infiltration systems will include shallow basins and infiltration trenches no deeper than 5 feet below the ground surface. Results of the testing are summarized in the table below.

Soil Infiltration Rates

Test No.	Infiltration Rate (in/hr)
P-1	9.8
P-2	9.42
P-3	10.64
P-4	9.27

The table presents test rates. Design of the infiltration systems should include an appropriate factor of safety to account for degradation of soil conditions by fine grained materials carried by runoff, potential growth of vegetation, accumulation of trash and other appropriate considerations. The factor of safety should be determined in accordance with the methodology presented in San Bernardino County Program – Technical Guidance Document for Water Quality Management Plans (Appendix D, Section VII) using a medium concern (infiltrometer) assessment method, granular soils, relatively homogeneous soils, a groundwater depth of greater than 10 feet, and appropriate design related considerations. Per the Technical Guidance Document, the factor a safety should not be less than 2. We recommend that the slowest test rate (P-4, 9.27 in/hr) be used to determine the design rate.

The above rates apply to loose to moderately dense soils. Compaction of soils will reduce infiltration rates. Therefore soils at the bottom of the proposed infiltration basin should not be rolled or otherwise compacted, and construction traffic should be allowed in the basin. A maintenance plan should also be developed and implemented to restore infiltration properties of soils that may be impacted by sedimentation or other adverse conditions.

These factors should be considered in design and maintenance of the proposed basins. Additionally, any City of San Bernardino, San Bernardino County and State of California or other applicable Agency design criteria should be followed.

The test data sheets for the soil infiltration tests are presented in Appendix A.

3.21 Utility Trench Backfill

The onsite fill soils will not be suitable for use as pipe bedding for buried utilities. All pipes should be bedded in a sand, gravel or crushed aggregate imported material complying with the requirements of the Standard Specifications for Public Works Construction Section 306-1.2.1. Crushed rock products that do not contain appreciable fines should not be utilized as pipe bedding and/or backfill. Bedding materials should be densified to at least 90% relative compaction (ASTM D1557) by mechanical methods. The geotechnical consultant should review and approve of proposed bedding materials prior to use.

All utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Cal/OSHA construction safety orders should be observed during all underground work.

3.22 Recommended Aviation Pavement Section Design Criteria

Our recommendation for pavement design CBR is based on our analysis of the results of our field investigation and testing, laboratory testing and generally accepted engineering principals. The recommended sections were determined utilizing the FAA Guidelines for the Flexible Pavement Design contained within Advisory Circular 150/5320-6E and the FAARFIELD - Airport Pavement Design (Version 1.305, dated September 28, 2009).

The structural adequacy of runways and taxiways at airports are based on the principle that the airport pavements are designed and constructed to provide adequate support for the loads imposed by the aircrafts. The pavement must be of such quality and thickness that it will not fail under the load imposed by the aircrafts. This method utilizes the CBR and equivalency ratios for the various materials utilized in the structural section to design the asphalt overlay course, surfacing course, and base course. Our recommended design CBRs were calculated per Advisory Circular 150/5320-6E as one standard deviation below the mean of all CBR values obtained. We recommend the following CBR's for the various pavement repair conditions:

Condition	Design CBR
Remove and replace and new asphalt pavement for Apron and Taxiway	32
Grind and overlay of Apron and Taxiway existing asphalt pavement	20

Pavement design and construction methods should be based on FAA Advisory Circular 150/5320-6E on Airport Pavement Design and Evaluation, and 150/5370-10F on Standards for Specifying Construction of Airports. Subgrade compaction should be completed in accordance with Table 5-1 Subgrade Compaction Requirements for Light Load Flexible Pavements in FAA Advisory Circular 150/5320-6E.

3.23 Vehicular Pavement Sections

Sand equivalent and R-value tests were performed on anticipated subgrade soils at the site in order to provide information on their soil properties for design of pavement structural sections. Structural sections were designed using the procedures outlined in Chapter 630 of the California Highway Design Manual (Caltrans, 2012). This procedure uses the principle that the pavement structural section must be of adequate thickness to distribute the load from the design traffic index (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R-value).

Development of the design traffic indexes on the basis of a traffic study is beyond the scope of this report. Based upon your request we have utilized a traffic index of 8.0 for the proposed streets. Based upon our experience we have also used a traffic index of 6.0 for the proposed parking lots. Selection of the final pavement structural section should be based on economic considerations which are beyond the scope of this investigation. Minimum pavement sections from the City of San Bernardino Street Improvement Policy (June 1, 1987) are 2½ inches of asphalt over 4 inches of aggregate base or a full depth asphalt section with a minimum thickness of 3½ inches. The City also has a minimum Traffic Index of 5.5 for continuous local streets. Recommended structural sections based on an R-Value of 69 are as follows:

- Proposed Streets (TI=8.0, R-Value=69):
  3.5 inches of asphaltic concrete over
  4.0 inches of crushed miscellaneous base
  or
  5.5 inches of asphaltic concrete over
  12.0 inches of compacted native soil

- Parking Lots(TI=6.0, R-Value=69):
  3.0 inches of asphaltic concrete over
  4.0 inches of crushed miscellaneous base
  or
  4.0 inches of asphaltic concrete over
  12.0 inches of compacted native soil

Portland cement concrete (PCC) pavements for areas which are not subject to traffic loads may be designed with a minimum thickness of 4.0 inches of Portland cement concrete on compacted native soils. If traffic loads are anticipated, PCC pavements should be designed for a minimum thickness of 8.0 inches of Portland cement concrete on 4.0 inches of crushed miscellaneous base.

Prior to paving, the upper 12 inches of subgrade soils should be scarified, adjusted to within 2% of optimum moisture and compacted to a minimum of 95% relative compaction for full depth asphalt sections and compacted to a minimum of 90% relative compaction beneath sections including aggregate base. All aggregate base courses should be compacted to a minimum of 95% relative compaction.

3.24 Plan Review

Once a formal grading and foundation plans are prepared for the subject property, this office should review the plans from a geotechnical viewpoint, comment on changes from the plan used during preparation of this report and revise the recommendations of this report where necessary.

3.25 Geotechnical Observation and Testing During Rough Grading

The geotechnical engineer should be contacted to provide observation and testing during the following stages of grading:

- During the clearing and grubbing of the site.
- During the demolition of any existing structures, buried utilities or other existing improvements.
- During excavation and overexcavation of compressible soils.
- During all phases of grading including ground preparation and filling operations.
- When any unusual conditions are encountered during grading.

A final geotechnical report summarizing conditions encountered during grading should be submitted upon completion of the rough grading operations.

3.26 Post-Grading Geotechnical Observation and Testing

After the completion of grading the geotechnical engineer should be contacted to provide additional observation and testing during the following construction activities:

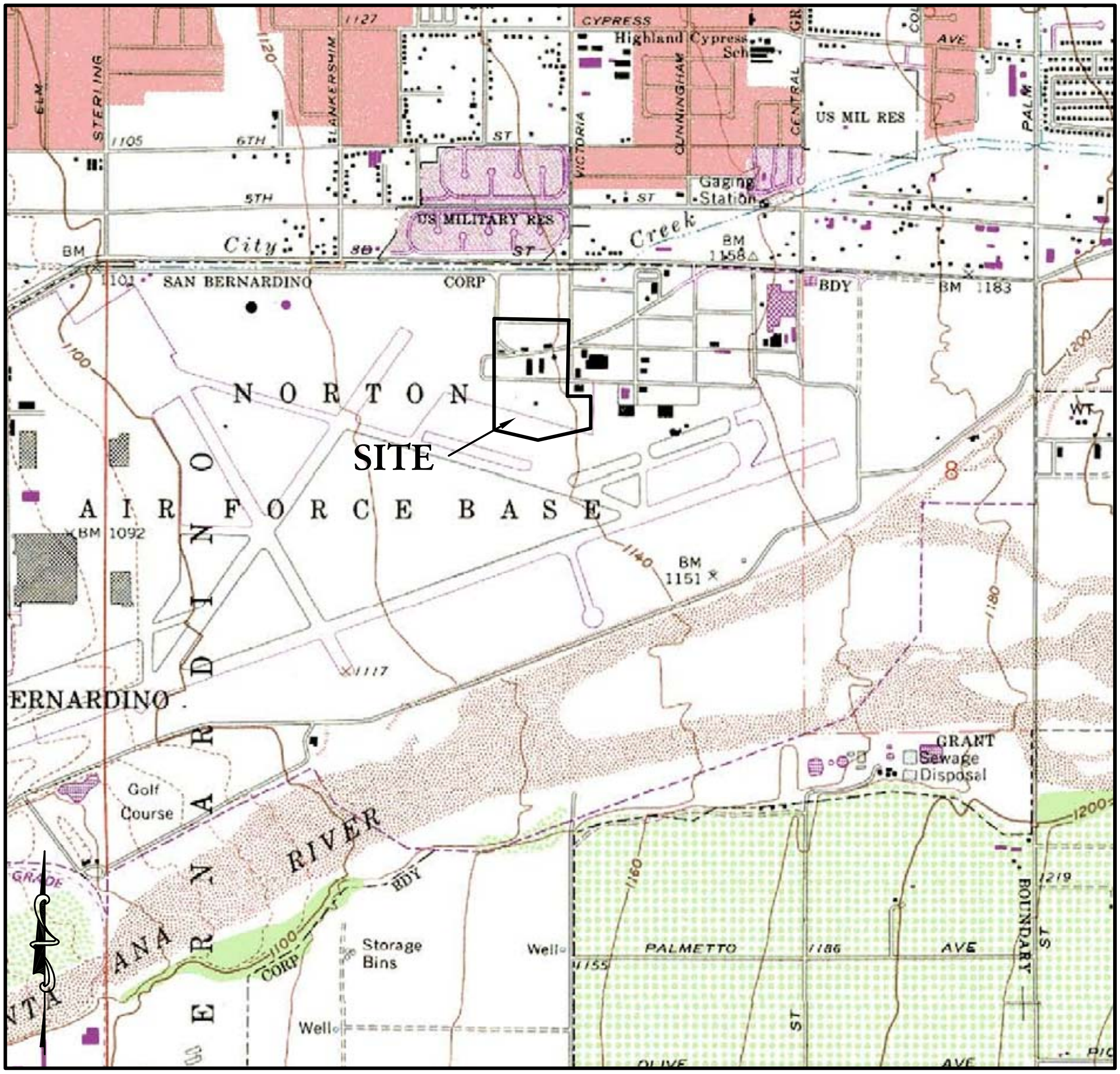
- During trenching and backfilling operations of buried improvements and utilities to verify proper backfill and compaction of the utility trenches.
- After excavation and prior to placement of reinforcing steel or concrete within footing trenches to verify that footings are properly founded in competent materials.
- During fine or precise grading involving the placement of any fills underlying driveways, sidewalks, walkways, or other miscellaneous concrete flatwork to verify proper placement, mixing and compaction of fills.
- When any unusual conditions are encountered during construction.

4.00 CLOSURE

The findings, conclusions and recommendations in this report were prepared in accordance with generally accepted engineering and geologic principles and practices. No other warranty, either expressed or implied, is made. This report has been prepared for Parsons Brinckerhoff to be used solely for design purposes. Anyone using this report for any other purpose must draw their own conclusions regarding required construction procedures and subsurface conditions.

The geotechnical and geologic consultant should be retained during the earthwork and foundation phases of construction to monitor compliance with the design concepts and recommendations and to provide additional recommendations as needed. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

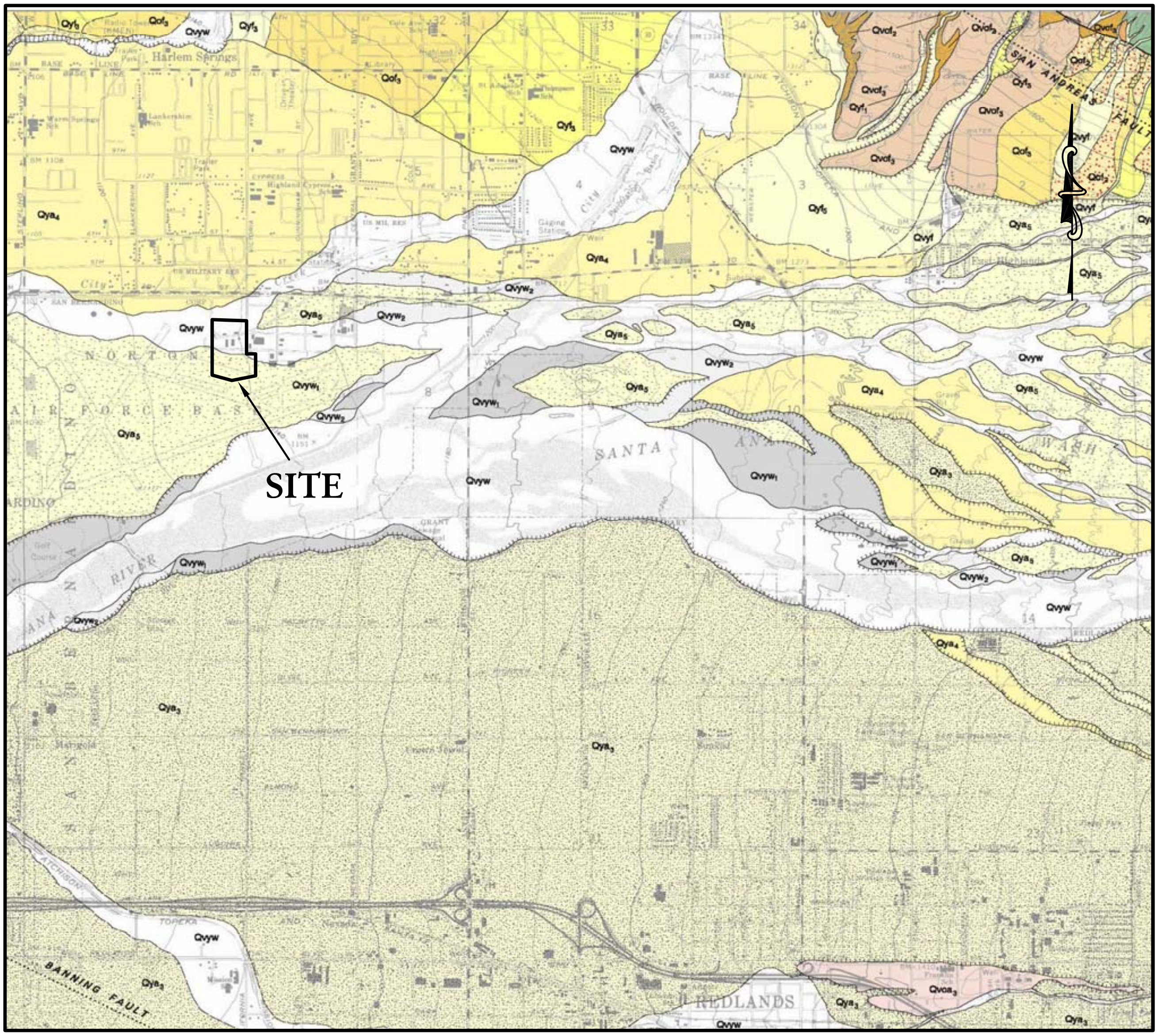
FIGURES AND TABLES



SITE LOCATION MAP

Scale: 1" = 2,000'

Base Map: U.S. Geolgocial Survey 7.5' Redlands Quadrangle



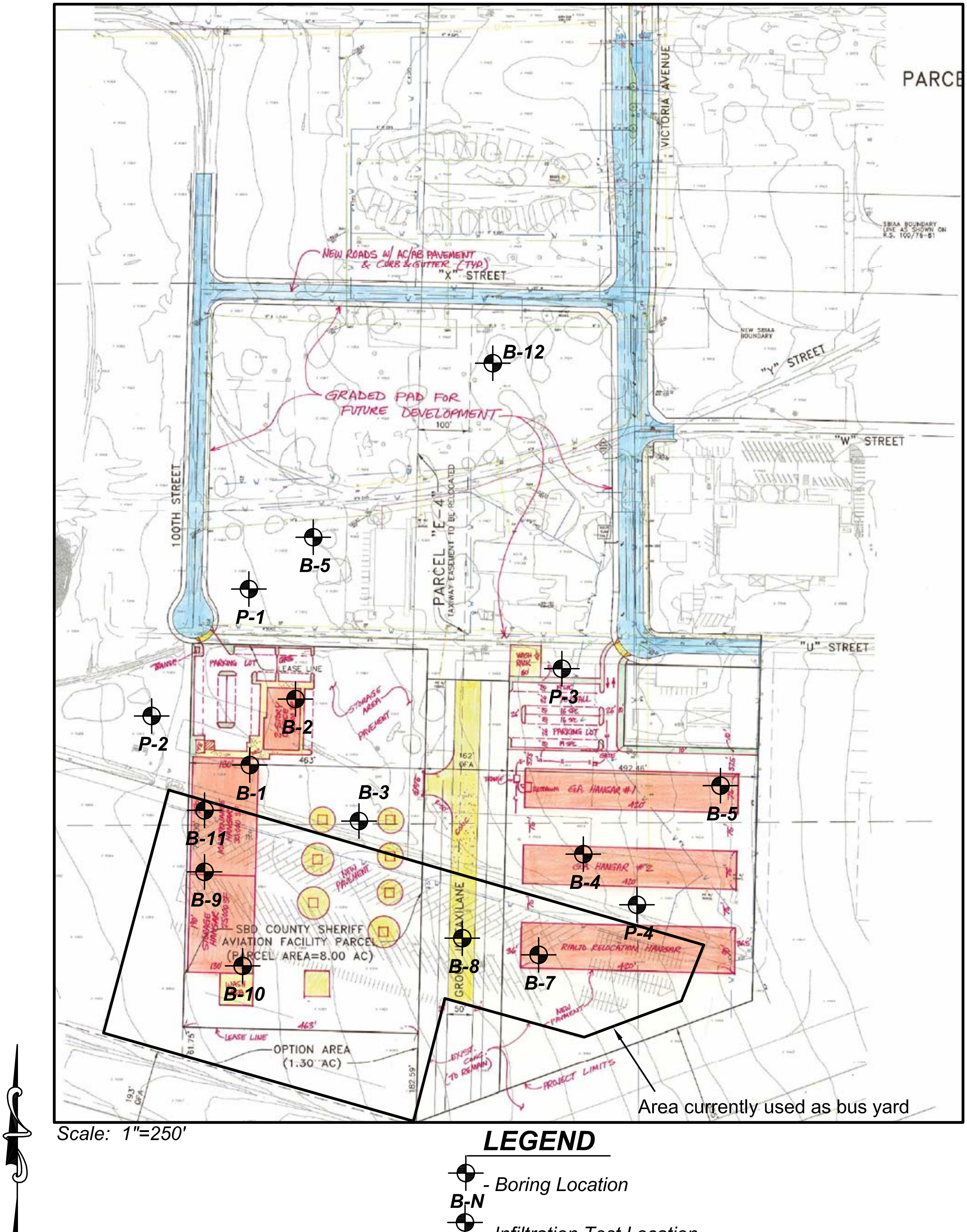
REGIONAL GEOLOGIC MAP

Scale: 1" ~ 3,000'

Partial Legend

Qvyw - Very young wash deposits (latest Holocene)
Qya5 - Young axial-valley deposits (latest Holocene)
Dotted lines indicate concealed faults

Source: Matti and others, 2003, U.S. Geological Survey OFR 03-302



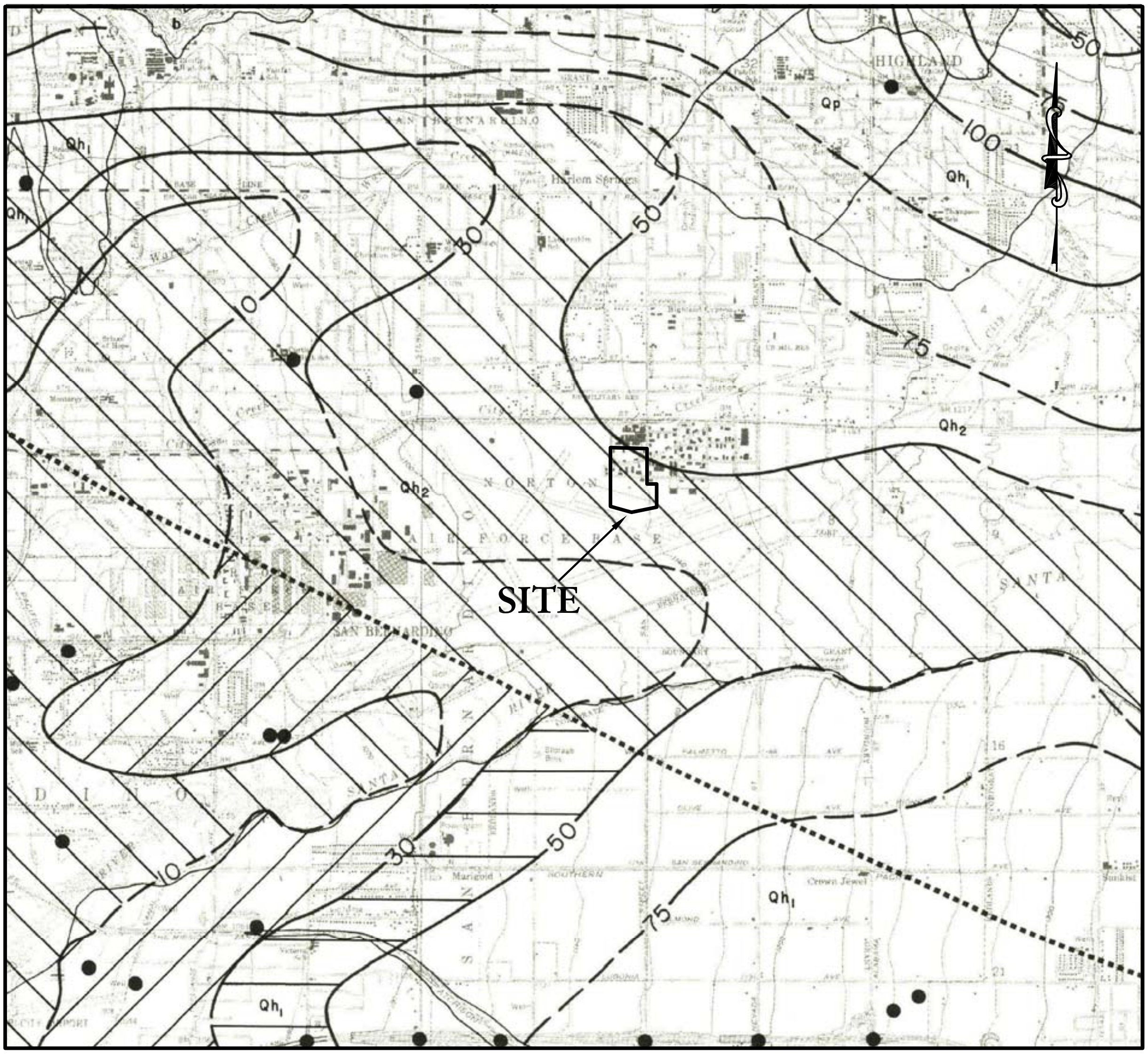
LEGEND

- Boring Location (B-N)
- Infiltration Test Location (P-N)

Base map provided by Parsons Brinkerhoff, Inc.

BORING LOCATION MAP

GEOTECHNICAL CONSULTANTS



LIQUEFACTION SUSCEPTIBILITY MAP

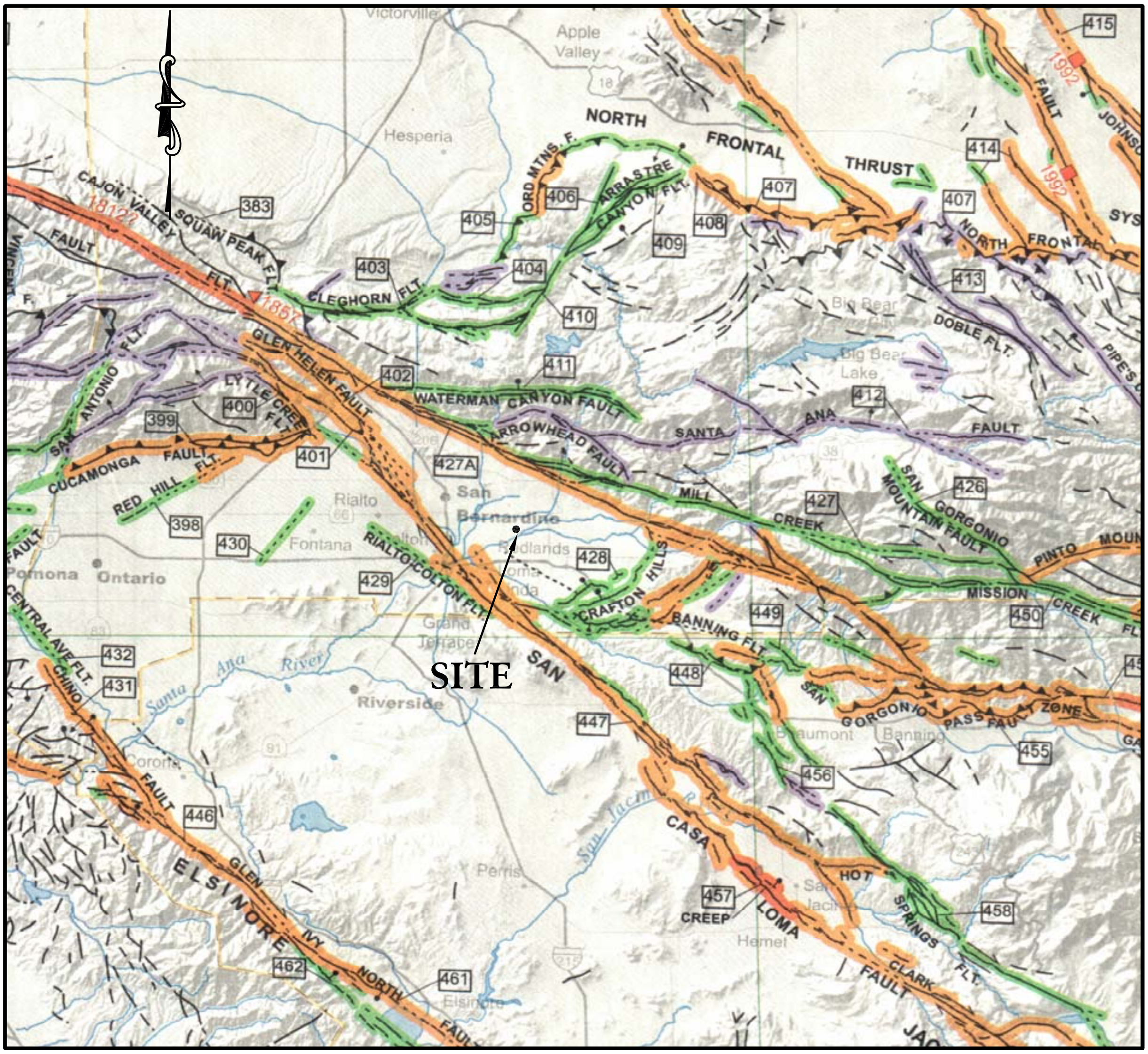
Scale: 1" ~ 4,000'

LEGEND

- Zone of High Liquefaction Susceptibility

50 - Minimum depth to groundwater during 1973-1983

Source: Matti, J.C. and Carson, S.E., 1991, Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California - A Regional Evaluation, United States Geological Survey Bulletin 1989.



REGIONAL FAULT MAP

Scale: 1" ~ 12 miles

Base Map: California Activity Map of California, California Geological Survey (Jennings and Bryant, 2010)

NOTABLE FAULTS WITHIN 100 KILOMETERS AND SEISMIC DATA

Fault Zone & geometry	Distance (km)	Distance (mi.)	Maximum Moment Magnitude	Slip Rate (mm/yr)
Calico-Hidalgo (rl-ss)	89	55	7.3	0.6
Chino-Central Ave. (rl-r-o)	43	27	6.7	1.0
Clamshell-Sawpit (r)	59	37	6.5	0.5
Cleghorn (ll-ss)	19	12	6.5	3.0
Cucamonga (r)	22	14	6.9	5.0
Elsinore - Glen Ivy (rl-ss)	46	29	6.8	5.0
Upper Elysian Park (r)	81	50	6.4	1.3
Eureka Peak (rl-ss)	77	48	6.4	0.6
Gravel Hills-Harper (rl-ss)	90	56	7.1	0.6
Helendale - S Lockhart (rl-ss)	44	27	7.3	0.6
Johnson Valley (rl-ss)	68	42	6.7	0.6
Landers (rl-ss)	74	46	7.3	0.6
Lenwood-Lockhart (rl-ss)	61	38	7.5	0.6
Newport-Inglewood (rl-ss)	85	53	6.9	1.5
North Frontal - Western (r)	24	15	7.2	1.0
North Frontal - Eastern (r)	41	25	6.7	0.5
Palos Verde (rl-ss)	31	19	7.3	3.0
Pinto Mountain (ll-ss)	47	29	7.2	2.5
Pisgah-Bullion Mtn. - Mesquite Lake	97	60	7.3	0.6
Puente Hills Blind Thrust (r)	62	39	7.1	0.7
Raymond (ll-r-o)	71	44	6.5	1.5
San Andreas (rl-ss)	5	3	7.5	24.0
San Jacinto (rl-ss)	7	4	6.7	12.0
San Joaquin Hills (r)	69	43	6.6	0.5
San Jose (ll-r-o)	43	27	6.4	0.5
Sierra Madre (r)	47	29	7.2	2.0
Verdugo (r)	85	53	6.9	0.5
Whittier (rl-ss)	47	29	6.8	2.5

Notes:

Fault geometry - (ss) strike slip, (r) reverse, (n) normal, (rl) right lateral, (ll) left lateral, (o) oblique

Fault and Seismic Data - California Geological Survey (Cao), 2003

HISTORIC STRONG EARTHQUAKES IN SOUTHERN CALIFORNIA SINCE 1812

Date	Event	Causitive Fault	Magnitude	Epicentral Distance (miles)
Dec. 12, 1812	Wrightwood	San Andreas?	7.3	41
Jan. 9, 1857	Fort Tejon	San Andreas	7.9	256
Dec. 16, 1858	San Bernardino Area	uncertain	6.0	8
Feb. 9,1890	San Jacinto	uncertain	6.3	73
May 28, 1892	San Jacinto	uncertain	6.3	75
July 30, 1894	Lytle Creek	uncertain	6.0	26
July 22, 1899	Cajon Pass	uncertain	6.4	21
Dec.25, 1899	San Jacinto	San Jacinto	6.7	25
Sept. 20, 1907	San Bernardino Area	uncertain	5.3	10
May 15, 1910	Elsinore	Elsinore	6.0	30
April 21, 1918	Hemet	San Jacinto	6.8	28
July 23, 1923	San Bernardino	San Jacinto	6.0	8
March 11, 1933	Long Beach	Newport-Inglewood	6.4	54
April 10, 1947	Manix	Manix	6.4	73
Dec. 4, 1948	Desert Hot Springs	San Andreas or Banning	6.5	51
July 21, 1952	Wheeler Ridge	White Wolf	7.3	123
Feb. 9, 1971	San Fernando	San Fernando	6.6	72
July 8, 1986	North Palm Springs	Banning or Garnet Hills	5.6	37
Oct. 1, 1987	Whittier Narrows	Puente Hills Thrust	6.0	50
Feb. 28, 1990	Upland	San Jose	5.5	28
June 28, 1991	Sierra Madre	Clamshell Sawpit	5.8	47
April 22, 1992	Joshua Tree	Eureka Peak	6.1	55
June 28, 1992	Landers	Johnson Valley & others	7.3	47
June 28, 1992	Big Bear	uncertain	6.5	24
Jan. 17, 1994	Northridge	Northridge Thrust	6.7	78
Oct. 16, 1999	Hector Mine	Lavic Lake	7.1	66

Notes:

Earthquake data: U.S. Geological Survey P.P. 1515 & online data, Southern California Earthquake Center
California Geological Survey online data
Magnitudes prior to 1932 are estimated from intensity.
Magnitudes after 1932 are moment, local or surface wave magnitudes.
Attenuation relationship - Boore et al., 1997 (mean values), values at distances > 50 miles are approxima

Site Location:

Site Longitude: 117.227
Site Latitude: 34.103

APPENDIX A

FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

A-1.00 FIELD EXPLORATION

A-1.01 Number of Borings

Our subsurface investigation consisted of 12 borings drilled with a Mobile B-61 drill rig.

A-1.02 Location of Borings

A Boring Location Map showing the approximate locations of the borings is presented as Figure 3.

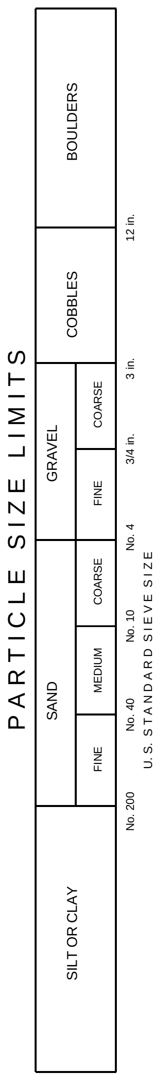
A-1.03 Boring Logging

Logs of borings were prepared by one of our staff and are attached in this appendix. The logs contain factual information and interpretation of subsurface conditions between samples. The strata indicated on these logs represent the approximate boundary between earth units and the transition may be gradual. The logs show subsurface conditions at the dates and locations indicated, and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing, and the in-place density of the various strata encountered in the exploration was determined

RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS



MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size.	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravel, gravel-sand mixtures. little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatamaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

UNIFIED SOIL CLASSIFICATION SYSTEM

I. SOIL STRENGTH/DENSITY

BASED ON STANDARD PENETRATION TESTS

Compactness of sand		Consistency of clay	
Penetration Resistance N (blows/Ft)	Compactness	Penetration Resistance N (blows/ft)	Consistency
0-4	Very Loose	<2	Very Soft
4-10	Loose	2-4	Soft
10-30	Medium Dense	4-8	Medium Stiff
30-50	Dense	8-15	Stiff
>50	Very Dense	15-30	Very Stiff
		>30	Hard

N = Number of blows of 140 lb. weight falling 30 in. to drive 2-in OD sampler 1 ft.

BASED ON RELATIVE COMPACTION

Compactness of sand		Consistency of clay	
% Compaction	Compactness	% Compaction	Consistency
<75	Loose	<80	Soft
75-83	Medium Dense	80-85	Medium Stiff
83-90	Dense	85-90	Stiff
>90	Very Dense	>90	Very Stiff

II. SOIL MOISTURE

Moisture of sands		Moisture of clays	
% Moisture	Description	% Moisture	Description
<5%	Dry	<12%	Dry
5-12%	Moist	12-20%	Moist
>12%	Very Moist	>20%	Very Moist, wet

SOIL DESCRIPTION LEGEND

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-1

Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
0–4.5	R	13	Bulk	2.9	98.6	SM		Gray-brown silty fine to coarse sand and occasional gravel, slightly mo ist, medium dense
5	R	11		1.4	112.3	SP		Dark yellow-brown fine to coarse s and
10	R	31		2.0	114.8			Increase in gravel content
12								Refusal at 12' on two attempts Total depth 12' No ground water encountered Hole backfilled with native material
15								
20								
25								

Sample Types:

R - Ring Sample
Bulk Sample
- Groundwater
T - Tube Sample
S - SPT Sample
- End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-2
Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
						SP-SM		Gray-brown fine to coarse sand with silt, medium d ense
	R	13	Bulk	1.6	106.5			
5	Nr	10				SP		Dark yellow-brown fine to coarse s and
10	R	21		7.4	98.4			Increase in gravel content
15	R	73		2.0	119.3			Gray-brown fine to coarse sand with gra vel
								Refusal at 18' on one attempt
								Increase in cobble content
20	R	80/ 10"		3.5	117.0			
								Refusal at 23' on two attempts Total depth 23' No ground water encountere d Hole backfilled with native material
25								

Sample Types:

R - Ring Sample
- Bulk Sample
- Groundwater
T - Tube Sample
S - SPT Sample
- End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-3

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Material Description

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SM		Gray-brown fine to coarse sand with silt, medium dense
	R	5		1.8	103.7			
5	R	13		3.3	103.7			
						SP		Dark yellow-brown fine to coarse sand
10	R	29		2.1	126.2			Increase in gravel content
15	R	72		1.4	117.3			
								Gray-brown fine to coarse sand with gravel
20	R	18		15.5	112.2			Gray-brown fine to coarse sand with occasional gravel, some cohesion
								Total depth 21.5' No ground water encountered Hole backfilled with native material
25								

Sample Types:

R - Ring Sample | Bulk Sample | - Groundwater

T - Tube Sample | S - SPT Sample | - End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-4
Sheet 1 of 2

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Site Geologic Map	Drive Weights:	140 lbs.
Eleveation (ft):		Drop:	30"

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SP-SM		Gray-brown fine to coarse sand with silt, medium d ense
	R	16		3.0	105.2			
5	R	20		2.7	109.0			
10	R	52		1.1	125.0			
						SP		Dark yellow-brown fine to coarse sand with occasional g ravel
15	Nr	50/6"						Cobbles encountered
20	R	60		2.8	111.4			Gray-brown fine to coarse sand with occasional gr avel
25	R	76/ 11"		2.6	115.9			

Sample Types:
R - Ring Sample
Bulk Sample
Groundwater
T - Tube Sample
S - SPT Sample
End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-4

Date Drilled:	12/6/2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Geologic Map	Drive Weights:	140 lbs.
Elevation (ft):		Drop:	30"

Material Description: This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Material Description
	S	17		21.2		SM	Brown fine to medium silty sand / sandy silt, cohesive
35	S	19		21.3			
40	S	18		21.3		CL	Brown clay, moist
45	S	52		3.1		SP	Gray-brown fine to coarse sand with silt
50	S	50/5"		3.0			Total depth 50' 5" No ground water encountered Hole backfilled with native material

Sample Types:

R - Ring Sample; Bulk Sample; - Groundwater

T - Tube Sample; S - SPT Sample; - End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-5

Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
			Bulk			SP-SM		Gray-brown fine to coarse sand with silt to silty sand, dry, medium dense
	R	13		2.0	118.5			
5	R	16		3.5	111.2			
10	R	36		2.8	123.3	SP		Dark yellow-brown fine to coarse sand
15	R	40		1.6	125.5			Increase in gravel content
20	R	67		2.5	112.0			Yellow-brown clean sand with gravel
								Total depth 21.5' No ground water encountered Hole backfilled with native material
25								

Material Description: This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample
Bulk Sample
Groundwater
T - Tube Sample
S - SPT Sample
End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-6
Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Material Description: This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Material Description
0						SM	Brown fine to medium silty sand, dry, trace of gravel, loose
2	R	23	Bulk	1.2	115.4	SP-SM	Light brown fine to medium sand with silt, dry, very slight trace of grav el, medium dense
5	Nr	15				SP	Light brown fine to coarse sand with silt, dry, poorly sorted, medium dense
10	R	50/6"		1.1	123.1		Gravel in tip
15	R	81		0.7	127.6		
16.5							Total depth 16.5' No ground water encountered Hole backfilled with native material

Depth scale: 5, 10, 15, 20, 25

Sample Types:
- R - Ring Sample
- Bulk Sample
- Groundwater
- T - Tube Sample
- S - SPT Sample
- End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-7

Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
								4" AC / 4" AB
						SP-SM		Gray-brown fine to medium sand, moist, well sorted, medium dense to d ense
	R	20	Bulk	3.5	100.1			
5	R	18		3.6	97.3			
10	R	38		2.7	110.6			Rock in sample, sand disturb ed
								Refusal at 13' on two attem pts Total depth 13' No ground water encounter ed Hole backfilled with native material
15								
20								
25								

Sample Types:

R - Ring Sample   Bulk Sample   - Groundwater

T - Tube Sample   S - SPT Sample   - End of Boring

Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-8

Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								4" AC / 4" AB
	R	14	Bulk	4.9	98.5	SP-SM		Gray-brown fine to medium sand with trace of gravel, moist, medium dense
5	R	14		4.4	104.6			
							End of Boring	Total depth 6.5' No ground water encountered Hole backfilled with native material
10								
15								
20								
25								

Material Description: This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample; Bulk Sample; Groundwater

T - Tube Sample; S - SPT Sample; End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-9
Sheet 1 of 1

Date Drilled:	12-6-2012	Drilling Equipment:	Mobile B-75
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
								6" AC / 10-12" AB
								Wire mesh encountered 10-12" into aggregate base la yer
								Total depth 1.5' No ground water encountered Hole backfilled with native material
5								
10								
15								
20								
25								

Sample Types:

R - Ring Sample; Bulk Sample; - Groundwater

T - Tube Sample; S - SPT Sample; - End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-10
Sheet 1 of 1

Date Drilled:	11-20-12	Drilling Equipment:	CME-55
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
								4" AC / 12" AB
								Wire mesh encounted at 16"
	R	19	Bulk	6.2	100.0	SM		Gray-brown fine to medium sand with minor silt, moist, occasional grav el, medium dense
5	R	12		4.6	100.4			Brown fine silty sand, moist, medium dense
						SP		Gray fine to coarse gravelly sand, slightly moist, poorly sorted, dense
10	R	29		3.0	109.8			
15	R	75		2.6	122.8			
20	R	50/5"						Total depth 20' 5" No ground water encountered Hole backfilled with native material
25								

Sample Types:

R - Ring Sample
Bulk Sample
Groundwater
T - Tube Sample
S - SPT Sample
End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-11
Sheet 1 of 1

Date Drilled:	11-20-12	Drilling Equipment:	CME-55
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
								2.5" AC / 9.5" AB
								Wire mesh encountered 9" into aggregate base layer
								Total depth 1' No ground water encountered Hole backfilled with native material
5								
10								
15								
20								
25								

Sample Types:

R - Ring Sample; Bulk Sample; Groundwater

T - Tube Sample; S - SPT Sample; End of Boring

Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-12

Sheet 1 of 1

Date Drilled:	11-20-12	Drilling Equipment:	CME-55
Logged By:	510WSC	Boring Hole Diameter:	8"
Location:	See Boring Location Map	Drive Weights:	140 lbs.
		Drop:	30"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
								This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
	R	7		1.2	99.8	SM		Brown silty fine to medium sand, dry, slight trace of gravel, loose to medium dense
5	R	22		0.6	103.3	SP		Gray fine to coarse sand with gravel to gravelly sand, dry, medium dense
10	R	73		0.8	121.4			
15	R	53		0.8	114.2			
20	R	39		1.0				Sample disturbed
25	R	77		1.1	120.5			Very dense, slight increase in gravel
								Total depth 26.5' No ground water encountered Hole backfilled with native material

Sample Types:

R - Ring Sample
Bulk Sample
Groundwater
T - Tube Sample
S - SPT Sample
End of Boring

APPENDIX B

LABORATORY TESTS

APPENDIX B

LABORATORY TESTS

B-1.00 LABORATORY TESTS

B-1.01 Maximum Density

Maximum density - optimum moisture relationships for the major soil types encountered during the field exploration were performed in the laboratory using the standard procedures of ASTM D1557.

B-1.02 Sand Equivalent

San equivalents were determined for all samples to identify which samples would be the most beneficial for California Bearing Ratio testing. Sand equivalent tests were performed using test method ASTM D2419.

B-1.03 Moisture Determination

Moisture content of the soil samples was performed in accordance to standard method for determination of water content of soil by drying oven, ASTM D2216. The mass of material remaining after oven drying is used as the mass of the solid particles.

B-1.04 Expansion Tests

Expansion index tests were performed on representative samples of the major soil types encountered by the test methods outlined in ASTM D4829.

B-1.05 Soluble Sulfates and Chlorides

Test was performed on representative sample encountered during the investigation using the ASTM D4327 procedure.

B-1.06 Soil Reactivity (pH) and Electrical Conductivity (Ec)

Representative soil sample was tested for soil reactivity (pH) and electrical conductivity (Ec) using California Test Method S3.0 and S5.0. The pH measurement determines the degree of acidity or alkalinity in the soils. The Ec is a measure of the electrical resistivity and is expressed as the reciprocal of the resistivity.

B-1.07 Particle Size Analysis

Particle size analysis was performed on representative samples of the major soils types encountered in the test holes in accordance to the standard test methods of ASTM D422. The hydrometer portion of the standard procedure was not performed and the material retained on the #200 screen was washed.

B-1.08 Direct Shear

Direct shear tests were performed on representative samples of the major soil types encountered in the test holes using the standard test method of ASTM D3080 (consolidated and drained). Tests were performed on remolded samples. Remolded samples were tested at 90 percent relative compaction.

Shear tests were performed on a direct shear machine of the strain-controlled type. To simulate possible adverse field conditions, the samples were saturated prior to shearing. Several samples were sheared at varying normal loads and the results plotted to establish the angle of the internal friction and cohesion of the tested samples.

B-1.09 California Bearing Ratio (CBR)

Test specimens were remolded to the in-place density as determined by ASTM D2937. The test specimens were soaked for four days after molding and before testing.

B-1.10 R-Value

Samples were tested for Resistance "R" Value of treated and untreated bases, subbases, and basement soils by the stabilometer in conformance with Caltrans Test Method 301.

B-1.11 Test Results

Test results for all laboratory tests performed on the subject project are presented in this appendix.

SAMPLE INFORMATION

Sample Number	Sample Description	Sample Location Boring No.	Sample Location Depth (ft)
1	Gray-brown silty sand	B-1	1-4
2	Gray-brown poorly graded sand with silt	B-2	1-4
3	Gray-brown silty sand	B-5	1-4
4	Light brown poorly graded sand with silt	B-6	2-5
5	Gray-brown poorly graded sand with silt	B-7	2-5
6	Gray-brown poorly graded sand with silt	B-8	1-4
7	Gray-brown silty sand	B-10	2-5
8	Gray-brown silty sand	B-12	1-4

MAXIMUM DENSITY - OPTIMUM MOISTURE

(Test Method: ASTM D1557)

Sample Number	Optimum Moisture (Percent)	Maximum Density (lbs/ft^3)
3	10.3	120.7
5	12.3	113.5
6	11.0	110.1
7	10.0	119.8

SAND EQUIVALENT

(Test Method: ASTM D2419)

Sample Number	Sand Equivalent
2	70
3	50
4	42
5	64
6	68
8	55

EXPANSION TEST
(Test Method: ASTM D4829)

Sample Number	Molding Moisture Content (Percent)	Final Moisture Content (Percent)	Initial Dry Density (lbs/ft³)	Expansion Index	Expansion Classification
7	7.7	15.8	106.2	0	Very Low
8	2.4	18.2	103.9	0	Very Low

SOLUBLE SULFATES AND CHLORIDES
(Test Method: ASTM D4327)

Sample Number	Soluble Sulfate (ppm)	Chloride (ppm)
1	60	3.6
3	22	3.0
7	17	1.4
8	52	8.9

SOIL REACTIVITY (pH) AND ELECTRICAL CONDUCTIVITY
(Test Method: ASTM D4972)

Sample Number	pH	Resistivity (Ω·cm)
1	6.81	1,750
3	6.75	420
7	7.34	6,500
8	5.48	2,250

PARTICLE SIZE ANALYSIS

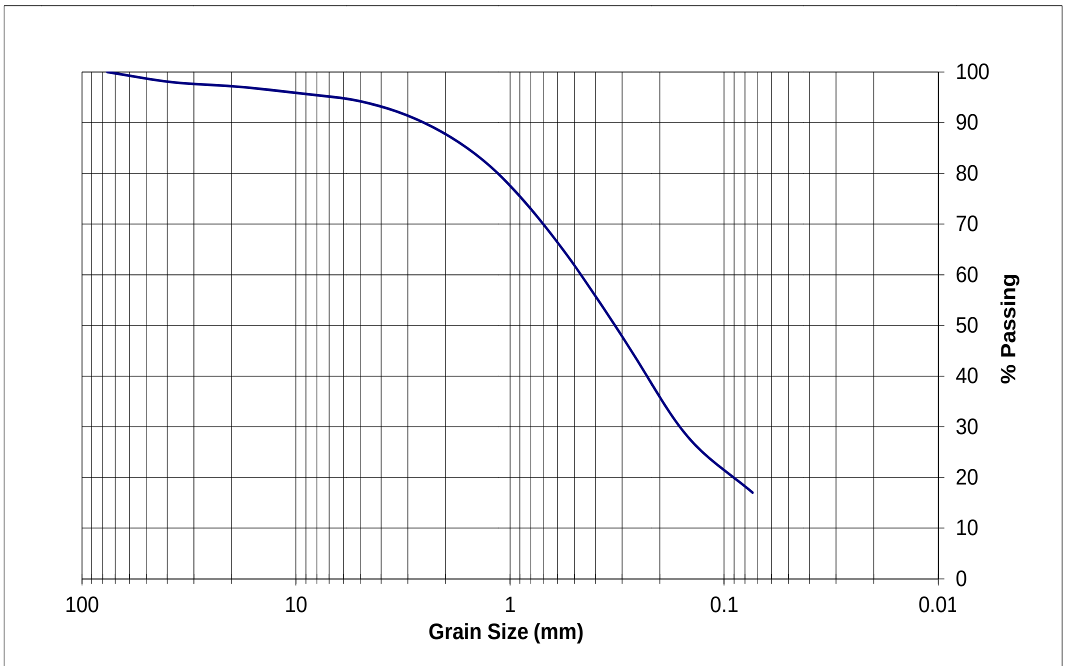
ASTM D422

Sample ID: 1

Fraction A: Dry Net Weight (g): 10138
Fraction B: Dry Net Weight (g): 509.8

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	10138	100
	1"	202	9936	98
	3/4"	291	9847	97
	1/2"	364	9774	96
	3/8"	428	9710	96
	#4	608	9530	94

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	24.6	485.2	89
	#16	71.8	438.0	81
	#30	152.1	357.7	66
	#50	251.6	258.2	48
	#100	356.1	153.7	28
	#200	417.5	92.3	17



PARTICLE SIZE ANALYSIS

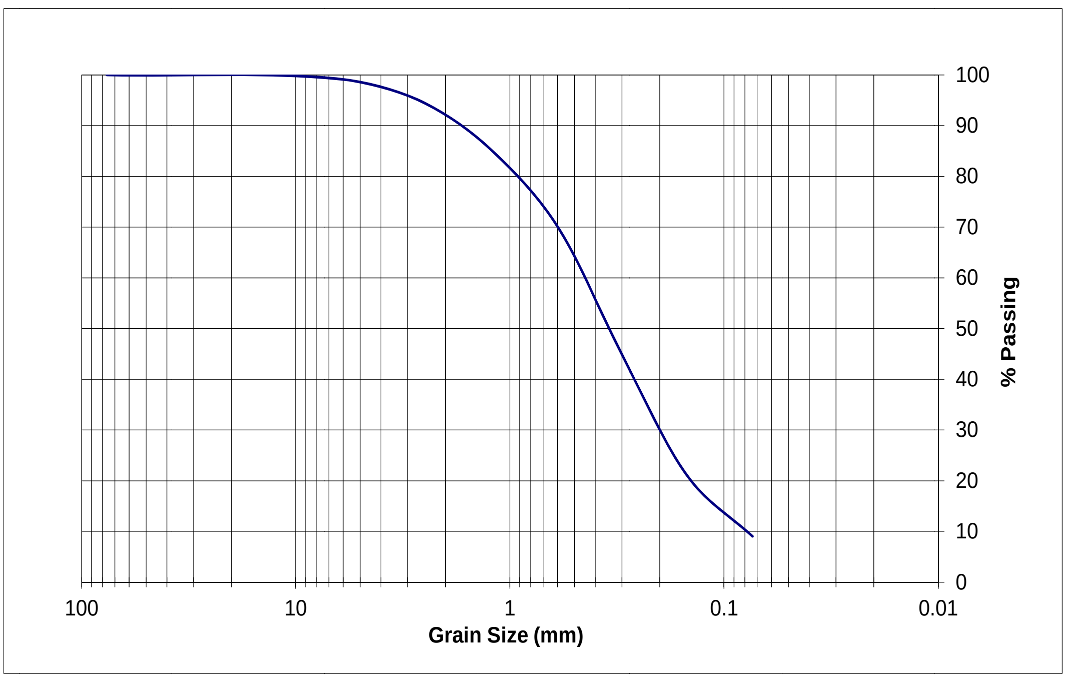
ASTM D422

Sample ID: 2

Fraction A: Dry Net Weight (g): 5181
Fraction B: Dry Net Weight (g): 493.2

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	5181	100
	1"	0	5181	100
	3/4"	0	5181	100
	1/2"	4	5177	100
	3/8"	12	5169	100
	#4	83	5098	98

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	22.1	471.1	94
	#16	68.3	424.9	85
	#30	144.2	349.0	70
	#50	269.9	223.3	45
	#100	386.9	106.3	21
	#200	447.8	45.4	9



PARTICLE SIZE ANALYSIS

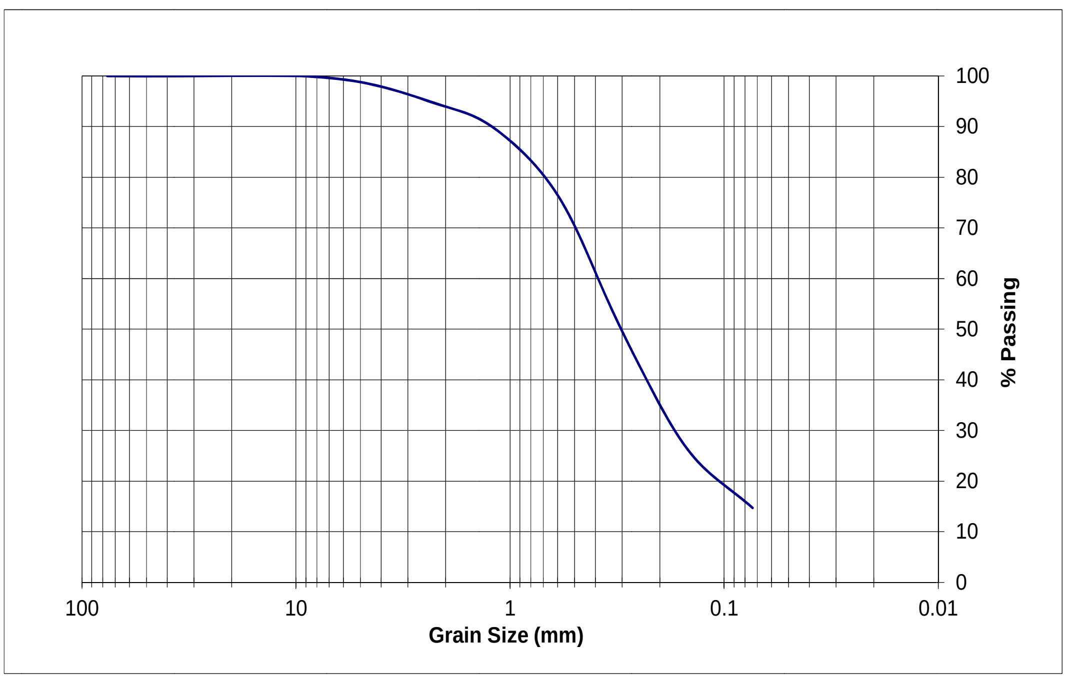
ASTM D422

Sample ID: 3

Fraction A: Dry Net Weight (g): 524.1
Fraction B: Dry Net Weight (g): 524.1

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	524.1	100
	1"	0	524.1	100
	3/4"	0	524.1	100
	1/2"	0	524.1	100
	3/8"	0	524.1	100
	#4	7.3	516.8	99

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	19.4	504.7	95
	#16	47.2	476.9	90
	#30	120.3	403.8	76
	#50	262.1	262.0	49
	#100	383.0	141.1	27
	#200	445.8	78.3	15



PARTICLE SIZE ANALYSIS

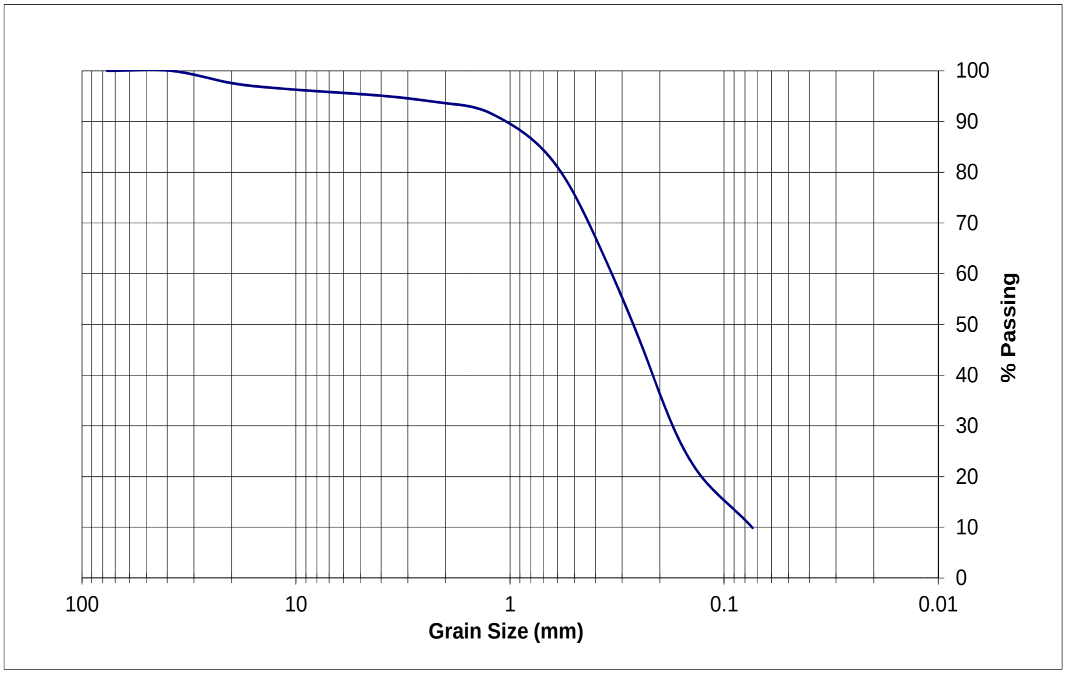
ASTM D422

Sample ID: 5

Fraction A: Dry Net Weight (g): 4934
Fraction B: Dry Net Weight (g): 482.2

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	4934	100
	1"	0	4934	100
	3/4"	127	4807	97
	1/2"	168	4766	97
	3/8"	187	4747	96
	#4	230	4704	95

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	6.7	475.5	94
	#16	20.3	461.9	91
	#30	74.8	407.4	81
	#50	204.2	278.0	55
	#100	358.6	123.6	24
	#200	432.1	50.1	10



GEOTECHNICAL CONSULTANTS

PARTICLE SIZE ANALYSIS

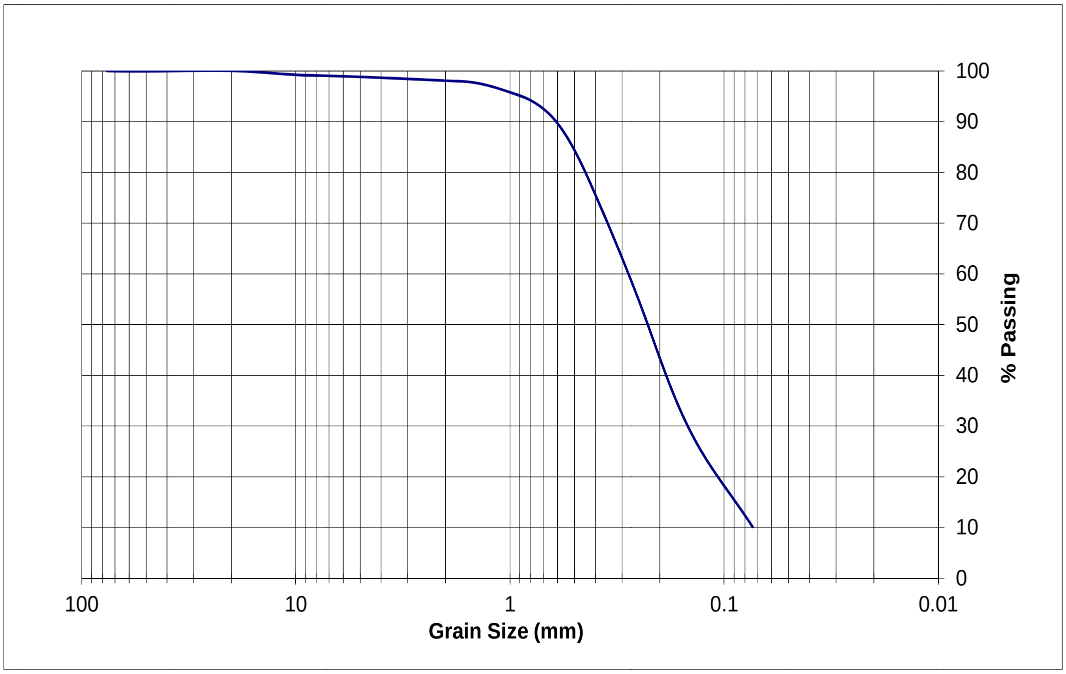
ASTM D422

Sample ID: 6

Fraction A: Dry Net Weight (g): 4830
Fraction B: Dry Net Weight (g): 498.1

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	4830	100
	1"	0	4830	100
	3/4"	0	4830	100
	1/2"	22	4808	100
	3/8"	39	4791	99
	#4	58	4772	99

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	3.0	495.1	98
	#16	10.0	488.1	97
	#30	48.1	450.0	89
	#50	181.6	316.5	63
	#100	344.5	153.6	30
	#200	447.1	51.0	10



PARTICLE SIZE ANALYSIS

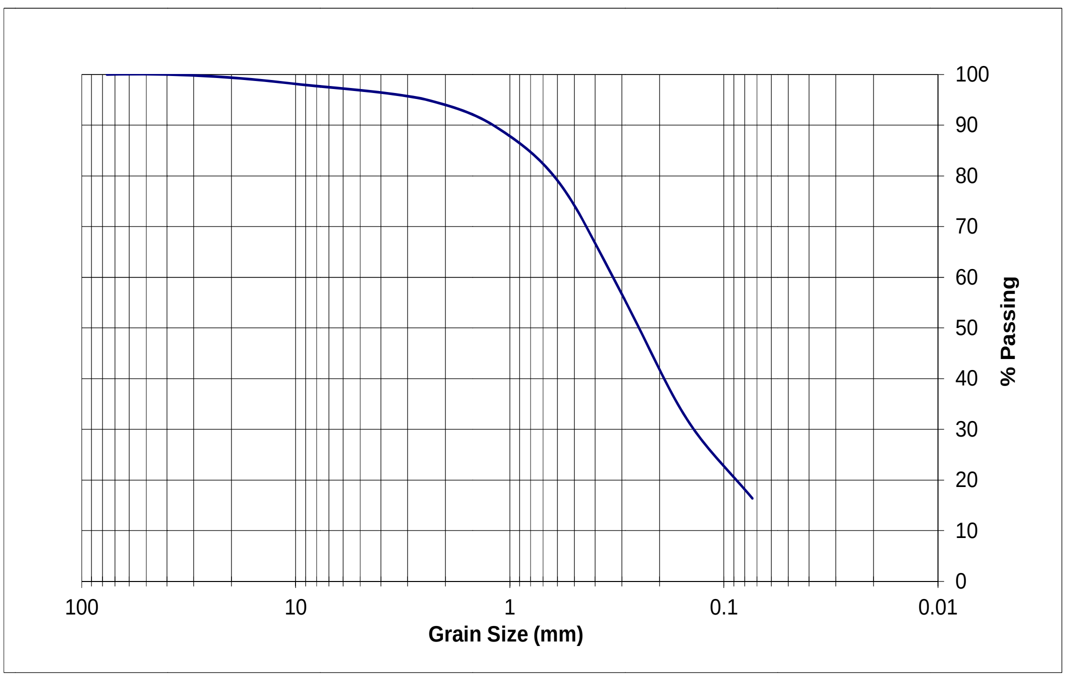
ASTM D422

Sample ID: 7

Fraction A: Dry Net Weight (g): 6730
Fraction B: Dry Net Weight (g): 502.4

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction A:	3"	0	6730	100
	1"	0	6730	100
	3/4"	45	6685	99
	1/2"	108	6622	98
	3/8"	132	6598	98
	#4	217	6513	97

	Screen Size	Net Retained Weight (g)	Net Passing Weight (g)	% Passing
Fraction B:	#8	9.8	492.6	95
	#16	35.4	467.0	90
	#30	93.8	408.6	79
	#50	209.8	292.6	56
	#100	335.2	167.2	32
	#200	417.3	85.1	16



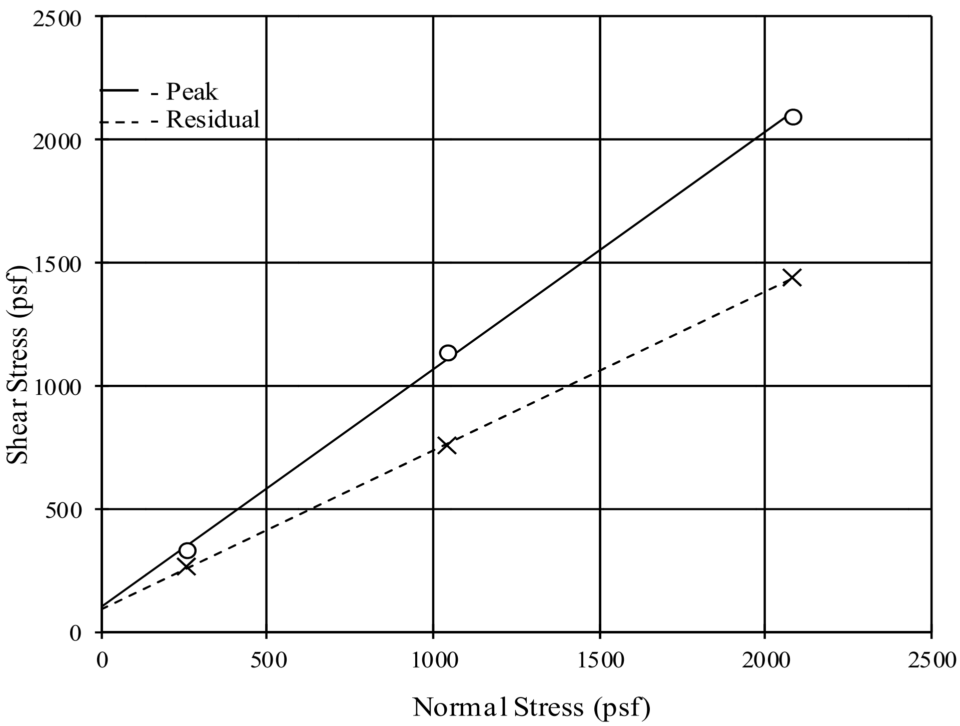
DIRECT SHEAR TEST
ASTM D3080

Sample ID: 3
Location: B-5 @ 1-4 ft

Maximum Density (pcf) =	120.7
Optimum Moisture (%) =	10.3
Remolded Density (pcf) =	114.7
Initial Moisture Content (%) =	10.0
Final Moisture Content (%) =	14.4

Normal Pressure	Peak Shear Resist	Residual Shear Resist
260	336	264
1040	1140	756
2080	2100	1440

	Peak	Residual
Cohesion (psf) =	100	90
Friction Angle (deg) =	44	33



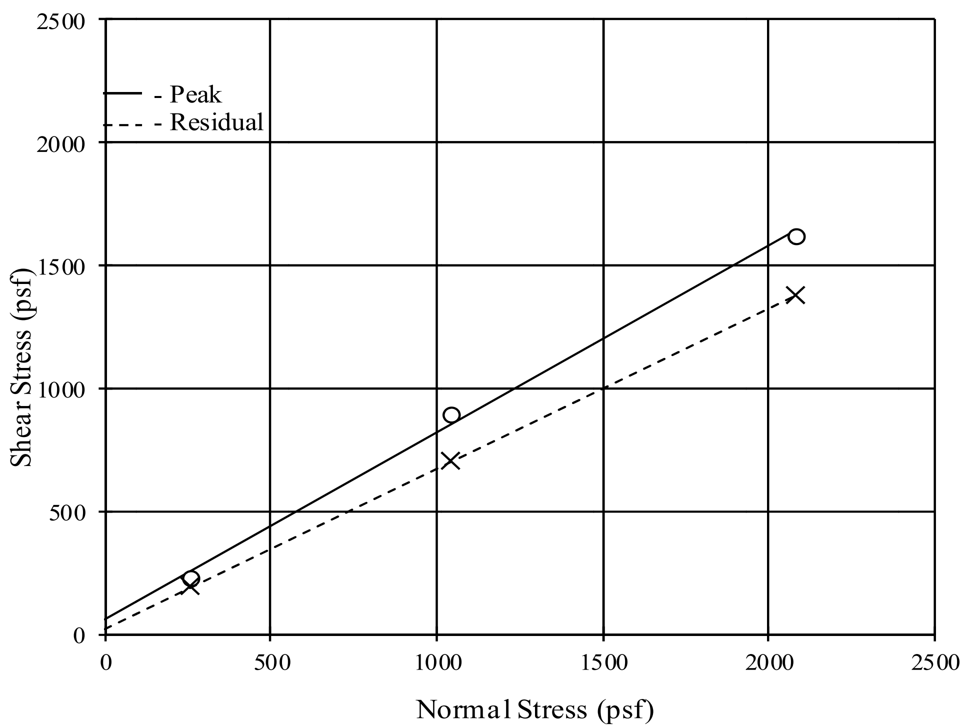
DIRECT SHEAR TEST
ASTM D3080

Sample ID: 5
Location: B-7 @ 2-5 ft

Maximum Density (pcf) =	113.5
Optimum Moisture (%) =	12.3
Remolded Density (pcf) =	107.8
Initial Moisture Content (%) =	12.3
Final Moisture Content (%) =	19.4

Normal Pressure	Peak Shear Resist	Residual Shear Resist
260	228	192
1040	900	708
2080	1620	1380

	Peak	Residual
Cohesion (psf) =	60	20
Friction Angle (deg) =	37	33



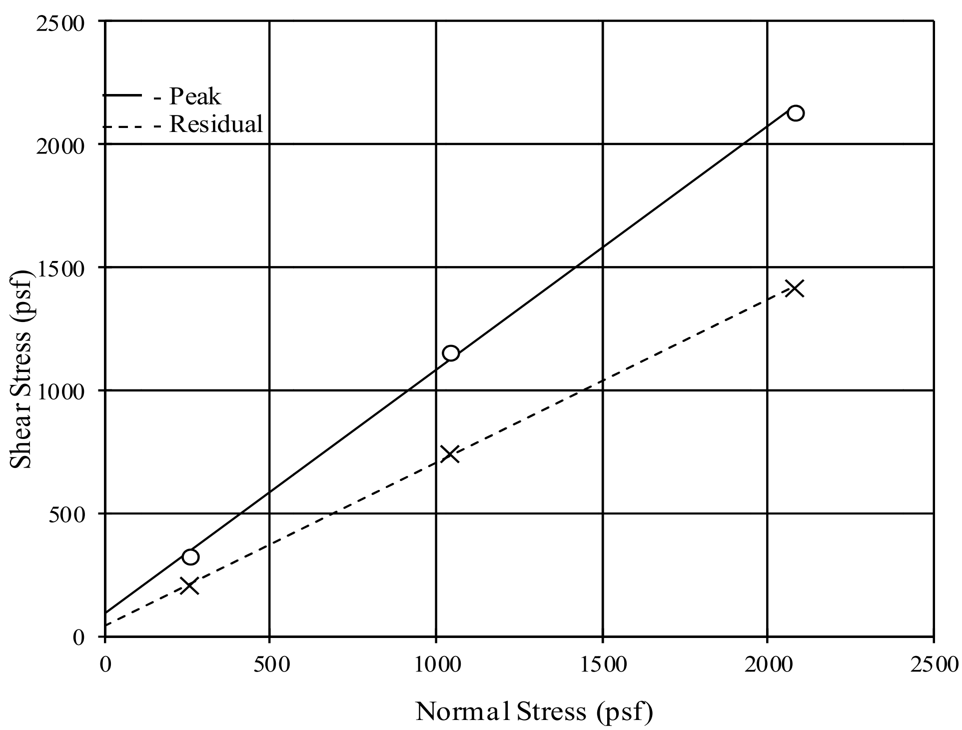
DIRECT SHEAR TEST
ASTM D3080

Sample ID: 7
Location: B-10 @ 2-5 ft

Maximum Density (pcf) = 119.8
Optimum Moisture (%) = 10.0
Remolded Density (pcf) = 113.8
Initial Moisture Content (%) = 9.7
Final Moisture Content (%) = 14.5

Normal Pressure	Peak Shear Resist	Residual Shear Resist
260	324	204
1040	1152	744
2080	2136	1416

	Peak	Residual
Cohesion (psf) =	90	40
Friction Angle (deg) =	45	34



RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

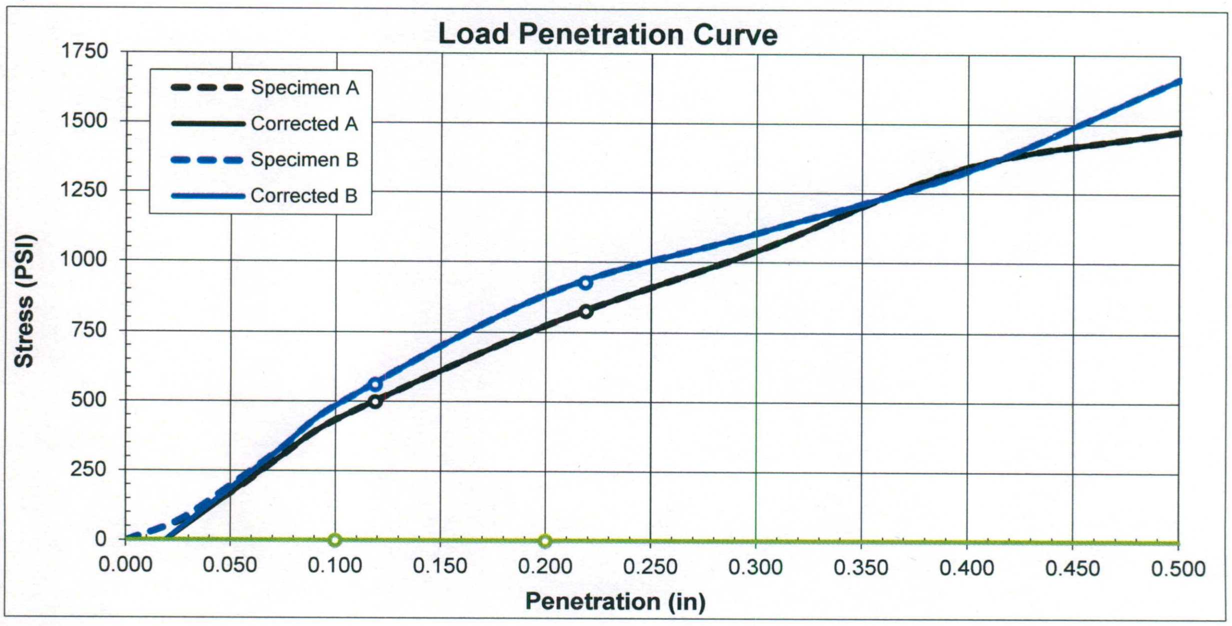
CALIFORNIA BEARING RATIO TEST
ASTM D1883

Sample ID: 3

Maximum Density (pcf): 120.7
Optimum Moisture (%): 10.3
Penetration Piston Area (in^2): 2.997

Specimen No.	No. of Blows	Remolded Dry Density (pcf)	Initial Moisture (%)	Final Moisture (%)	CBR 0.1 inch Penetration	CBR 0.2 inch Penetration	Percent Compaction
A	25	116.4	9.9	12.7	50	55	96%
B	35	118.6	10.2	11.8	56	62	97%

Corrected Penetration (inches)	Specimen A Load (lbs)	Specimen A Stress (psi)	Specimen B Load (lbs)	Specimen B Stress (psi)
0.000	0	0	0	0
0.025	200	67	210	70
0.050	510	170	590	197
0.075	920	307	1010	337
0.100	1300	434	1460	487
0.200	2320	774	2650	884
0.300	3120	1041	3310	1104
0.400	4030	1345	3990	1331
0.500	4420	1475	4990	1665



RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

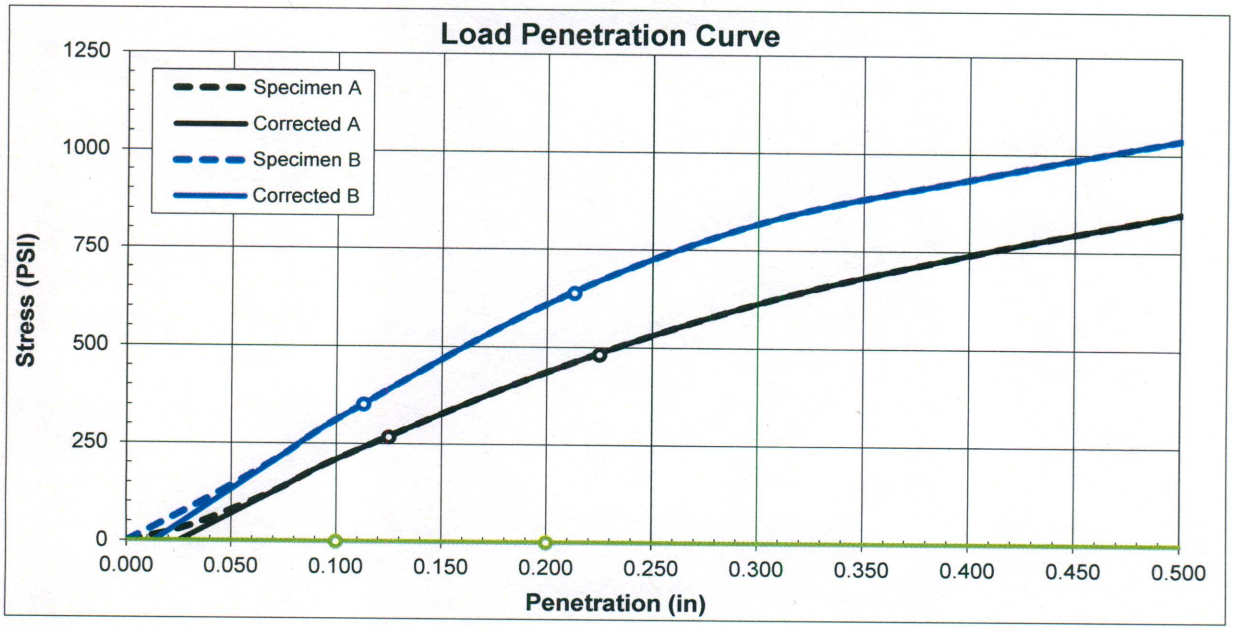
CALIFORNIA BEARING RATIO TEST
ASTM D1883

Maximum Density (pcf): 110.1
Optimum Moisture (%): 11.0
Penetration Piston Area (in^2): 2.997

Sample ID: 5

Specimen No.	No. of Blows	Remolded Dry Density (pcf)	Initial Moisture (%)	Final Moisture (%)	CBR 0.1 inch Penetration	CBR 0.2 inch Penetration	Percent Compaction
A	15	105.9	10.7	17.4	27	32	95%
B	25	109.1	9.8	16.9	35	42	98%

Corrected Penetration (inches)	Specimen A Load (lbs)	Specimen A Stress (psi)	Specimen B Load (lbs)	Specimen B Stress (psi)
0.000	0	0	0	0
0.025	90	30	210	70
0.050	240	80	430	143
0.075	420	140	670	224
0.100	630	210	940	314
0.200	1310	437	1830	611
0.300	1840	614	2450	817
0.400	2220	741	2800	934
0.500	2540	848	3110	1038



RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

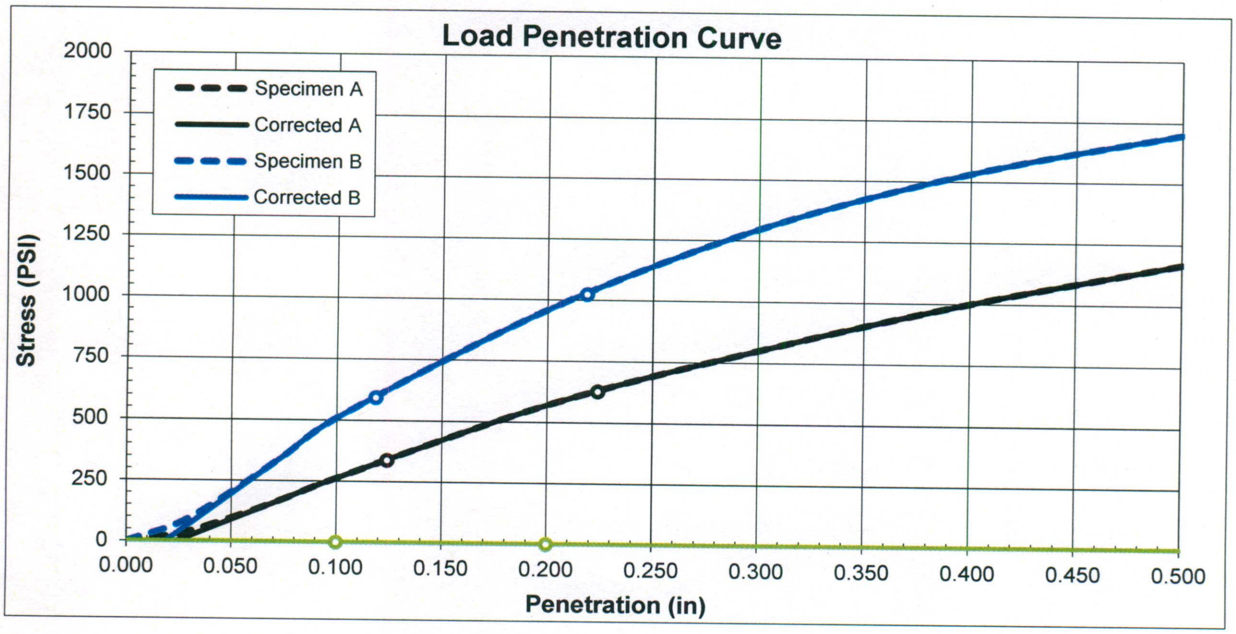
CALIFORNIA BEARING RATIO TEST
ASTM D1883

Maximum Density (pcf): 119.8
Optimum Moisture (%): 10.0
Penetration Piston Area (in^2): 2.997

Sample ID: 7

Specimen No.	No. of Blows	Remolded Dry Density (pcf)	Initial Moisture (%)	Final Moisture (%)	CBR 0.1 inch Penetration	CBR 0.2 inch Penetration	Percent Compaction
A	25	114.6	9.2	13.3	34	42	95%
B	40	116.3	9.9	12.8	59	68	96%

Corrected Penetration (inches)	Specimen A Load (lbs)	Specimen A Stress (psi)	Specimen B Load (lbs)	Specimen B Stress (psi)
0.000	0	0	0	0
0.025	90	30	220	73
0.050	290	97	610	204
0.075	530	177	1060	354
0.100	790	264	1530	511
0.200	1710	571	2880	961
0.300	2400	801	3890	1298
0.400	3000	1001	4600	1535
0.500	3500	1168	5100	1702



CTM 301 - Determination of Resistance "R" Value of Treated and Untreated Bases, Subbases and Basement Soils by Stabilometer

Sample No. 1

Specimen No	A	B	C
Moisture Content (%)	10.2	11.3	9.7
Dry Density (pcf)	114.8	113.2	114.9
Exudation Pressure (psi)	329	138	475
Stabilometer R Value	70	65	74
Expansion Pressure Dial	0	0	0

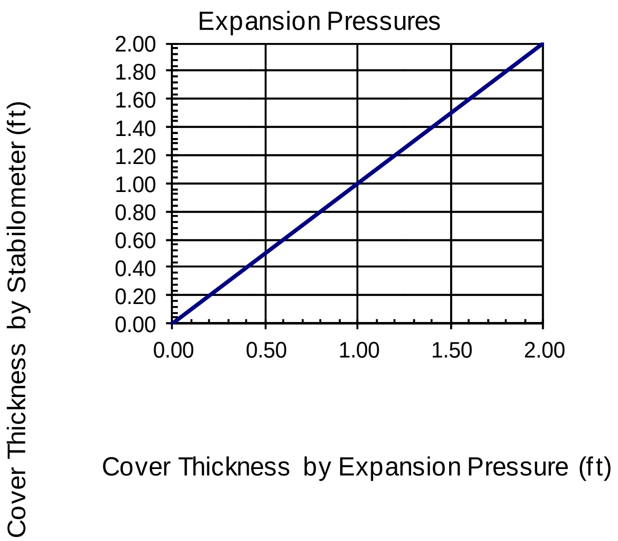
Use: Traffic Index = 5.0 Gravel Factor = 1.00

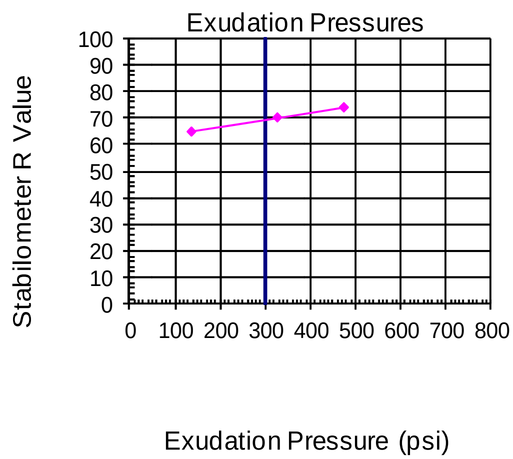
Thickness by Expansion (ft)			
Thickness by Stabilometer (ft)	0.48	0.56	0.42

Equilibrium Thick (ft) -

Equilibrium Pressure R Value **n/a**
Exudation Pressure R Value @ 300 psi **69**

Use Exudation R Value





Expansion Pressure R-Value is based on the following structural section:

Thickness of AC (ft)=	**0.25**	G_f(ac) =	2.50	W(ac) =	145
Thickness of Aggregate Base (ft)=	**0.42**	G_f(base) =	1.10	W(base) =	130
		G_f(avg) =	1.62	W(avg) =	136

CTM 301 - Determination of Resistance "R" Value of Treated and Untreated Bases, Subbases and Basement Soils by Stabilometer

Sample No. 8

Specimen No	A	B	C
Moisture Content (%)	11.8	12.1	11.0
Dry Density (pcf)	108.7	108.1	109.6
Exudation Pressure (psi)	389	205	446
Stabilometer R Value	70	68	72
Expansion Pressure Dial	0	0	0

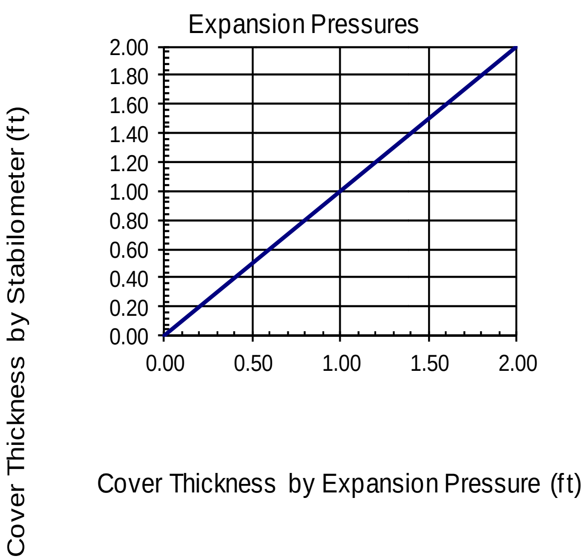
Use: Traffic Index = 5.0 Gravel Factor = 1.00

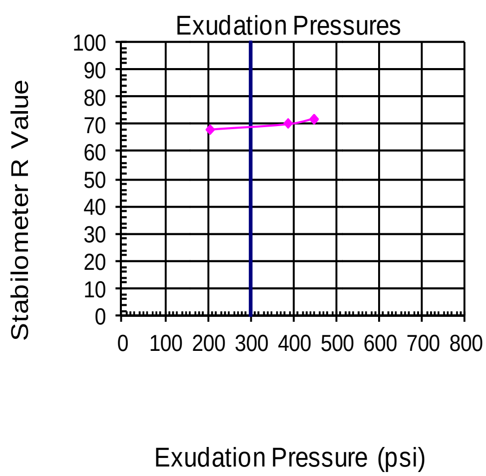
	A	B	C
Thickness by Expansion (ft)			
Thickness by Stabilometer (ft)	0.48	0.51	0.45

Equilibrium Thick (ft) -

Equilibrium Pressure R Value **n/a**
Exudation Pressure R Value @ 300 psi **69**

Use Exudation R Value





Expansion Pressure R-Value is based on the following structural section:

Thickness of AC (ft)=	**0.25**	G_f(ac) =	2.50	W(ac) =	145
Thickness of Aggregate Base (ft)=	**0.42**	G_f(base) =	1.10	W(base) =	130
		G_f(avg) =	1.62	W(avg) =	136

APPENDIX D

INFILTRATION TESTING

APPENDIX D

SOIL INFILTRATION TESTING

D-1.01 Test Method

Four soil infiltration tests have been performed at the above-referenced site. The tests were performed using the Percolation Test Procedure as outlined in the referenced San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP).

On-site infiltration system design was not available at the time this report was prepared, although it is anticipated that on-site infiltration systems will include shallow basins and infiltration trenches no deeper than 5 feet below the ground surface.

D-1.02 Infiltration Testing

The infiltration testing consisted of drilling of twelve 8 inch diameter exploratory borings and four 8 inch diameter infiltration test borings. Underground Service Alert was contacted prior to drilling of the borings. The infiltration test borings were extended to depths of 5 feet below the existing ground surface. The borings were drilled with a truck mounted drill rig equipped with hollow stem augers and were backfilled upon completion of testing. The exploratory borings were drilled to depths ranging from 1 to 51.5 feet.

Alluvial soils were encountered in the area of the test borings. The exploratory borings and test borings encountered silty sand, sand with silt and sand. Logs of the exploratory boring and infiltration test borings are included in Appendix A. Locations of the borings is shown on Figure 3.

Testing of the four infiltration test borings followed the Percolation Test Procedure included in the referenced San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP). The test borings were advanced to depths of at least 5 feet below the ground surface and the auger was rotated until the cuttings were removed from each hole. A 3 inch diameter perforated PVC pipe was then inserted into each test boring through the auger. A filter sock was installed around each pipe prior to placement in the borings to prevent siltation in the pipes during testing and to facilitate removal of the pipes at the conclusion of the testing. Water levels were measured to the nearest 0.01 of a foot using a well sounder.

The test borings were presoaked with at least 20 inches of water in the following manner:

- The presoaking time interval of the Test Borings consisted of two periods in which all of the water seeped away in less than 25 minutes.

The water levels in each test boring were refilled after each measurement, to approximately the same level. Water level measurements in the test borings were continued for a minimum of 1 hour with readings taken every 10 minutes and until a stabilized rate of drop was obtained.

D-1.03 Soil Infiltration Rates

Soil filtration rates were calculated using the procedure outlined in the San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP). The last percolation rate reading from each test boring was converted to an infiltration rate by the formula located on Page VII-29 of the San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP). See Section 3.20 for recommended design rates.

D-1.04 Groundwater Conditions

Groundwater was not encountered during our current subsurface exploration, which extended to a maximum depth of 51.5 feet below ground surface. According to the Matti and Carson (1991), the minimum depth to ground water in the vicinity of the site between 1973 and 1983 was on the order of 40 to 50 feet.

D-1.05 Design Considerations

For design purposes, we recommend use of a soil infiltration rate of 9.27 in/hr. The above infiltration rate number does not account for degradation of soil conditions by fine grained materials carried by runoff, growth of vegetation, accumulation of trash and other similar conditions that can occur during storms or between periods of basin maintenance. The factor of safety should be determined in accordance with the methodology presented in San Bernardino County Program – Technical Guidance Document for Water Quality Management Plans (Appendix D, Section VII) using a medium concern (infiltrometer) assessment method, granular soils, relatively homogeneous soils, a groundwater depth of greater than 10 feet, and appropriate design related considerations. Per the Technical Guidance Document, the factor a safety should not be less than 2. Furthermore, the infiltration system should comply with the requirement of the San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP) (dated May 19, 2011).

In addition, compaction of soil in areas proposed for water infiltration will significantly lower infiltration rates and make the tested rate inapplicable. Compaction of the infiltration site can destroy soil structure and seriously impact the infiltration facility's performance. Proper construction oversight is needed during construction to ensure that the bottoms of infiltration facilities are not overly compacted.

These factors should be considered in design and maintenance of the proposed basins. Additionally, any City of San Bernardino, San Bernardino County and State of California or other applicable Agency design criteria should be followed.

Percolation Test Data Sheet

Project:	SBIAA Infrastructure		Project No.:	12-406-01		Date:	12/6/2012
Test Hole No.:		1	Tested By:	JM			
Depth of Test Hole, D_T:		59.4	USCS Soil Classification:		SP-SM & SP		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =				
Sandy Soil Criteria*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or equal to 6"? (y/n)
1	7:32 AM	7:39 AM	7	34.56	59.4	24.84	y
2	7:40 AM	7:49 AM	9	32.4	59.4	27	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximate 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (In.)	D_f Final Depth to Water (In.)	ΔD Change in Water Level (min.)	Percolation Rate (min./in.)
1	7:51 AM	8:01 AM	10	31.4	46.8	15.4	0.33
2	8:02 AM	8:12 AM	10	29.9	43.7	13.8	0.32
3	8:13 AM	8:23 AM	10	29.9	41.8	11.9	0.28
4	8:24 AM	8:34 AM	10	29.8	42.2	12.4	0.29
5	8:39 AM	8:49 AM	10	28.9	39.7	10.8	0.27
6	8:52 AM	9:02 AM	10	29.2	39.6	10.4	0.26
7	9:04 AM	9:14 AM	10	29.3	38.9	9.6	0.25
8	9:17 AM	9:27 AM	10	28.1	38.9	10.8	0.28
9	9:30 AM	9:40 AM	10	28.8	39.1	10.3	0.26
10	9:42 AM	9:52 AM	10	29.3	38.2	8.9	0.23
11							
12							
13							
14							
15							

COMMENTS:

Infiltration Rate (in/hr) = $(\Delta D*60min/hr*r)/\Delta t\ (r+2H\ avg)$

H avg = $((D_T - D_o)-(D_T - D_o))/2$

Infiltration Rate (in/hr): 9.80

Percolation Test Data Sheet							
Project:	SBIAA Infrastructure		Project No.:	12-406-01		Date:	12/6/2012
Test Hole No.:		2	Tested By:	JM			
Depth of Test Hole, D_T:		50	USCS Soil Classification:		SP-SM & SP		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =				
Sandy Soil Criteria*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or equal to 6"? (y/n)
1	10:25 AM	10:43 AM	18	29.4	50	20.6	y
2	10:47 AM	11:13 AM	25	31.8	50	18.2	y
*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximate 30 minute intervals) with a precision of at least 0.25".							
Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (In.)	D_f Final Depth to Water (In.)	ΔD Change in Water Level (min.)	Percolation Rate (min./in.)
1	11:15 AM	11:25 AM	10	27.6	37.7	10.1	0.27
2	11:27 AM	11:37 AM	10	26.4	40	13.6	0.34
3	11:39 AM	11:49 AM	10	26.3	34.4	8.1	0.24
4	11:50 AM	12:00 PM	10	26	33.1	7.1	0.21
5	12:03 PM	12:13 PM	10	27	33.5	6.5	0.19
6	12:15 PM	12:25 PM	10	26.7	34	7.3	0.21
7							
8							
9							
10							
11							
12							
13							
14							
15							
COMMENTS:							
	Infiltration Rate (in/hr) =		$(\Delta D$*60min/hr*r$)/\Delta t$ $(r+2H$ avg$)$				
	H avg =	$((D_T - D_o)-(D_T - D_o))/2$					
					Infiltration Rate (in/hr):		9.42

Percolation Test Data Sheet

Project:	SBIAA Infrastructure		Project No.:	12-406-01		Date:	12/6/2012
Test Hole No.:		3	Tested By:	JM			
Depth of Test Hole, D_T:		55.2	USCS Soil Classification:		SP-SM & SP		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =				
Sandy Soil Criteria*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or equal to 6"? (y/n)
1	1:27 PM	1:37 PM	10	34	53.9	19.9	y
2	1:38 PM	1:48 PM	10	15	43.6	28.6	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximate 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (In.)	D_f Final Depth to Water (In.)	ΔD Change in Water Level (min.)	Percolation Rate (min./in.)
1	7:51 AM	8:01 AM	10	30.7	49.4	18.7	0.38
2	8:02 AM	8:12 AM	10	31.4	50.8	19.4	0.38
3	8:13 AM	8:23 AM	12	30.8	52.2	21.4	0.41
4	8:24 AM	8:34 AM	10	30.8	51.0	20.2	0.40
5	8:39 AM	8:49 AM	10	31.6	51.1	19.5	0.38
6	8:52 AM	9:02 AM	10	26.4	51.1	24.7	0.48
7	9:04 AM	9:14 AM	10	35.6	51.2	15.6	0.30
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

Infiltration Rate (in/hr) = $(\Delta D*60min/hr*r)/\Delta t\ (r+2H\ avg)$

H avg = $((D_T - D_o)-(D_T - D_o))/2$

Infiltration Rate (in/hr): 10.64

Percolation Test Data Sheet

Project:	SBIAA Infrastructure		Project No.:	12-406-01		Date:	12/7/2012
Test Hole No.:		4	Tested By:	JM			
Depth of Test Hole, D_T:		53.4	USCS Soil Classification:		SP-SM & SP		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =				
Sandy Soil Criteria*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or equal to 6"? (y/n)
1	7:32 AM	7:39 AM	12	28.6	53.4	24.8	y
2	7:40 AM	7:49 AM	12	22.1	38.4	16.3	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximate 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (In.)	D_f Final Depth to Water (In.)	ΔD Change in Water Level (min.)	Percolation Rate (min./in.)
1	7:56 AM	8:06 AM	10	29.6	38	8.4	0.22
2	8:08 AM	8:18 AM	10	29.3	37.2	7.9	0.21
3	8:19 AM	8:29 AM	10	30.4	39.4	9	0.23
4	8:31 AM	8:41 AM	10	30.2	39.7	9.5	0.24
5	8:42 AM	8:52 AM	10	31	40.2	9.2	0.23
6	8:55 AM	9:05 AM	10	29.8	41.2	11.4	0.28
7	9:07 AM	9:17 AM	10	28.8	35.2	6.4	0.18
8	9:19 AM	9:29 AM	10	29.2	36.6	7.4	0.20
9	9:31 AM	9:41 AM	10	30	38.4	8.4	0.22
10	9:43 AM	9:53 AM	10	31.8	40.8	9	0.22
11	9:54 AM	10:04 AM	10	31.2	40.2	9	0.22
12	10:05 AM	10:15 AM	10	30.5	37.3	6.8	0.18
13							
14							
15							

COMMENTS:

Infiltration Rate (in/hr) = $(\Delta D*60min/hr*r)/\Delta t\ (r+2H\ avg)$

H avg = $((D_T - D_o)-(D_T - D_o))/2$

Infiltration Rate (in/hr): 9.27

Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

APPENDIX D

CALCULATIONS OF LIQUEFACTION POTENTIAL
AND SEISMICALLY INDUCED SETTLEMENTS

LIQUEFACTION ANALYSIS
Standard Penetration Test

Project No.: 12-406-01
Boring No.: B-4

Fault Distance:	5 km	Hammer Type:	Auto
A_{max}:	0.464	Bore Hole Diameter:	8 inches
M_s:	7.59	SPT Liners:	No
Msf:	0.965	GW Depth (Encountered)	N/A feet
		GW Depth (Assumed)	40 feet

Soil Type	Unit Wt. γ (pcf)	Calculated Depth (ft)	Total Stress σ (TSF)	Eff Stress as tested	Eff. Stress σ (TSF)	Fines (%)	Field N (B/Ft)	CN	$(N1)_{60}$ (B/ft)	Corr. $(N1)_{60cs}$	Cyclic Resistance Ratio	Rd	Cyclic Stress Ratio	Safety Factor
SP-SM	125	2.00	0.125	0.125	0.125	17	11	2.000	27	32	0.4429	0.995	0.300	NL
SP-SM	125	4.00	0.250	0.250	0.250	17	11	2.000	27	32	0.4429	0.991	0.299	NL
SP-SM	125	6.00	0.375	0.375	0.375	17	13	1.633	26	31	0.4429	0.986	0.297	NL
SP-SM	125	8.00	0.500	0.500	0.500	17	13	1.414	23	27	0.3085	0.981	0.296	NL
SP	130	10.00	0.650	0.650	0.650	10	52	1.240	80	83	0.4429	0.977	0.295	NL
SP	130	12.00	0.780	0.780	0.780	10	52	1.132	73	76	0.4429	0.972	0.293	NL
SP	130	14.00	0.910	0.910	0.910	10	50	1.048	74	76	0.4429	0.967	0.292	NL
SP	130	16.00	1.040	1.040	1.040	10	50	0.981	69	71	0.4429	0.963	0.290	NL
SP	130	18.00	1.170	1.170	1.170	10	40	0.925	52	54	0.4429	0.958	0.289	NL
SP	130	20.00	1.300	1.300	1.300	10	40	0.877	49	51	0.4429	0.953	0.288	NL
SP	130	22.00	1.430	1.430	1.430	10	50	0.836	66	68	0.4429	0.949	0.286	NL
SP	130	24.00	1.560	1.560	1.560	10	50	0.801	63	65	0.4429	0.944	0.285	NL
SP	130	26.00	1.690	1.690	1.690	10	50	0.769	61	63	0.4429	0.939	0.283	NL
SP	130	28.00	1.820	1.820	1.820	10	50	0.741	58	60	0.4429	0.935	0.282	NL
SM	125	30.00	1.875	1.875	1.875	25	17	0.730	20	26	0.2895	0.930	0.281	NL
SM	125	32.00	2.000	2.000	2.000	25	17	0.707	19	25	0.2729	0.914	0.276	NL
SM	125	34.00	2.125	2.125	2.125	25	19	0.686	22	28	0.3311	0.897	0.271	NL
SM	125	36.00	2.250	2.250	2.250	25	19	0.667	21	28	0.3311	0.881	0.266	NL
SM	125	38.00	2.375	2.375	2.375	25	19	0.649	20	27	0.3085	0.865	0.261	NL
CL	115	40.00	2.300	2.300	2.300	80	18	0.659	20	29	0.3619	0.848	0.256	1.41
CL	115	42.00	2.415	2.415	2.353	80	19	0.643	20	29	0.3619	0.832	0.258	1.40
SM	115	44.00	2.530	2.530	2.405	25	52	0.629	54	65	0.4429	0.816	0.259	1.71
SM	125	46.00	2.875	2.875	2.688	25	52	0.590	51	61	0.4429	0.800	0.258	1.72
SM	125	48.00	3.000	3.000	2.750	25	75	0.577	72	84	0.4429	0.783	0.258	1.72
SM	125	50.00	3.125	3.125	2.813	25	75	0.566	70	83	0.4429	0.767	0.257	1.72

DYNAMIC SETTLEMENT ANALYSIS
Standard Penetration Test

Project No.: 12-406-01
Boring No.: B-4

Fault Distance:	5 km	Hammer Type:	Auto
A_{max}:	0.464	Bore Hole Diameter:	8 inches
M_s:	6.6	SPT Liners:	No
CSR Mwf	1.04	GW Depth (Encountered)	N/A feet
Dry Sand Mwf	0.79	GW Depth (Assumed)	40 feet

Soil Type	Unit Wt. γ (pcf)	Calculated Depth (ft)	Total Stress σ (TSF)	Eff Stress as tested	Eff. Stress σ (TSF)	Fines (%)	Field N (B/Ft)	CN	$(N1)_{60}$ (B/ft)	Corr. $(N1)_{60cs}$	Rd	Corr. Cyclic Stress Ratio	Volumetric Strain (%)	ΔH (in)
SP-SM	125	2.00	0.125	0.125	0.125	17	11	2.000	27	32	0.995	0.311	0.15	0.057
SP-SM	125	4.00	0.250	0.250	0.250	17	11	2.000	27	32	0.991	0.309	0.15	0.057
SP-SM	125	6.00	0.375	0.375	0.375	17	13	1.633	26	31	0.986	0.308	0.15	0.057
SP-SM	125	8.00	0.500	0.500	0.500	17	13	1.414	23	27	0.981	0.307	0.15	0.057
SP	130	10.00	0.650	0.650	0.650	10	52	1.240	80	83	0.977	0.305	0.00	0.000
SP	130	12.00	0.780	0.780	0.780	10	52	1.132	73	76	0.972	0.304	0.00	0.000
SP	130	14.00	0.910	0.910	0.910	10	50	1.048	74	76	0.967	0.302	0.00	0.000
SP	130	16.00	1.040	1.040	1.040	10	50	0.981	69	71	0.963	0.301	0.00	0.000
SP	130	18.00	1.170	1.170	1.170	10	40	0.925	52	54	0.958	0.299	0.00	0.000
SP	130	20.00	1.300	1.300	1.300	10	40	0.877	49	51	0.953	0.298	0.00	0.000
SP	130	22.00	1.430	1.430	1.430	10	50	0.836	66	68	0.949	0.296	0.00	0.000
SP	130	24.00	1.560	1.560	1.560	10	50	0.801	63	65	0.944	0.295	0.00	0.000
SP	130	26.00	1.690	1.690	1.690	10	50	0.769	61	63	0.939	0.293	0.00	0.000
SP	130	28.00	1.820	1.820	1.820	10	50	0.741	58	60	0.935	0.292	0.00	0.000
SM	125	30.00	1.875	1.875	1.875	25	17	0.730	20	26	0.930	0.291	0.15	0.057
SM	125	32.00	2.000	2.000	2.000	25	17	0.707	19	25	0.914	0.285	0.15	0.057
SM	125	34.00	2.125	2.125	2.125	25	19	0.686	22	28	0.897	0.280	0.10	0.038
SM	125	36.00	2.250	2.250	2.250	25	19	0.667	21	28	0.881	0.275	0.10	0.038
SM	125	38.00	2.375	2.375	2.375	25	19	0.649	20	27	0.865	0.270	0.10	0.038
CL	115	40.00	2.300	2.300	2.300	80	18	0.659	20	29	0.848	0.265	0.10	0.038
CL	115	42.00	2.415	2.415	2.353	80	19	0.643	20	29	0.832	0.267	0.10	0.024
SM	115	44.00	2.530	2.530	2.405	25	52	0.629	54	65	0.816	0.268	0.00	0.000
SM	125	46.00	2.875	2.875	2.688	25	52	0.590	51	61	0.800	0.267	0.00	0.000
SM	125	48.00	3.000	3.000	2.750	25	75	0.577	72	84	0.783	0.267	0.00	0.000
SM	125	50.00	3.125	3.125	2.813	25	75	0.566	70	83	0.767	0.266	0.00	0.000
												Total Dynamic Settlement (in) =		0.5

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

E-1.00 GENERAL DESCRIPTION

E-1.01 Introduction

These specifications present our general recommendations for earthwork and grading as shown on the approved grading plans for the subject project. These specifications shall cover all clearing and grubbing, removal of existing structures, preparation of land to be filled, filling of the land, spreading, compaction and control of the fill, and all subsidiary work necessary to complete the grading of the filled areas to conform with the lines, grades and slopes as shown on the approved plans.

The recommendations contained in the geotechnical report of which these general specifications are a part of shall supersede the provisions contained hereinafter in case of conflict.

E-1.02 Laboratory Standard and Field Test Methods

The laboratory standard used to establish the maximum density and optimum moisture shall be ASTM D1557.

The insitu density of earth materials (field compaction tests) shall be determined by the sand cone method (ASTM D1556), direct transmission nuclear method (ASTM D2922) or other test methods as considered appropriate by the geotechnical consultant.

Relative compaction is defined, for purposes of these specifications, as the ratio of the in-place density to the maximum density as determined in the previously mentioned laboratory standard.

E-2.00 CLEARING

E-2.01 Surface Clearing

All structures marked for removal, timber, logs, trees, brush and other rubbish shall be removed and disposed of off the site. Any trees to be removed shall be pulled in such a manner so as to remove as much of the root system as possible.

E-2.02 Subsurface Removals

A thorough search should be made for possible underground storage tanks and/or septic tanks and cesspools. If found, tanks should be removed and cesspools pumped dry.

Any concrete irrigation lines shall be crushed in place and all metal underground lines shall be removed from the site.

E-2.03 Backfill of Cavities

All cavities created or exposed during clearing and grubbing operations or by previous use of the site shall be cleared of deleterious material and backfilled with native soils or other materials approved by the soil engineer. Said backfill shall be compacted to a minimum of 90% relative compaction.

E-3.00 ORIGINAL GROUND PREPARATION

E-3.01 Stripping of Vegetation

After the site has been properly cleared, all vegetation and topsoil containing the root systems of former vegetation shall be stripped from areas to be graded. Materials removed in this stripping process may be used as fill in areas designated by the soil engineer, provided the vegetation is mixed with a sufficient amount of soil to assure that no appreciable settlement or other detriment will occur due to decaying of the organic matter. Soil materials containing more than 3% organics shall not be used as structural fill.

E-3.02 Removals of Non-Engineered Fills

Any non-engineered fills encountered during grading shall be completely removed and the underlying ground shall be prepared in accordance to the recommendations for original ground preparation contained in this section. After cleansing of any organic matter the fill material may be used for engineered fill.

E-3.03 Overexcavation of Fill Areas

The existing ground in all areas determined to be satisfactory for the support of fills shall be scarified to a minimum depth of 6 inches. Scarification shall continue until the soils are broken down and free from lumps or clods and until the scarified zone is uniform. The moisture content of the scarified zone shall be adjusted to within 2% of optimum moisture. The scarified zone shall then be uniformly compacted to 90% relative compaction.

Where fill material is to be placed on ground with slopes steeper than 5:1 (H:V) the sloping ground shall be benched. The lowermost bench shall be a minimum of 15 feet wide, shall be a minimum of 2 feet deep, and shall expose firm material as determined by the geotechnical consultant. Other benches shall be excavated to firm material as determined by the geotechnical consultant and shall have a minimum width of 4 feet.

Existing ground that is determined to be unsatisfactory for the support of fills shall be overexcavated in accordance to the recommendations contained in the geotechnical report of which these general specifications are a part.

E-4.00 FILL MATERIALS

E-4.01 General

Materials for the fill shall be free from vegetable matter and other deleterious substances, shall not contain rocks or lumps of a greater dimension than is recommended by the geotechnical consultant, and shall be approved by the geotechnical consultant. Soils of poor gradation, expansion, or strength properties shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils providing satisfactory fill material.

E-4.02 Oversize Material

Oversize material, rock or other irreducible material with a maximum dimension greater than 12 inches, shall not be placed in fills, unless the location, materials, and disposal methods are specifically approved by the geotechnical consultant. Oversize material shall be placed in such a manner that nesting of oversize material does not occur and in such a manner that the oversize material is completely surrounded by fill material compacted to a minimum of 90% relative compaction. Oversize material shall not be placed within 10 feet of finished grade without the approval of the geotechnical consultant.

E-4.03 Import

Material imported to the site shall conform to the requirements of Section 4.01 of these specifications. Potential import material shall be approved by the geotechnical consultant prior to importation to the subject site.

E-5.00 PLACING AND SPREADING OF FILL

E-5.01 Fill Lifts

The selected fill material shall be placed in nearly horizontal layers which when compacted will not exceed approximately 6 inches in thickness. Thicker lifts may be placed if testing indicates the compaction procedures are such that the required compaction is being achieved and the geotechnical consultant approves their use.
Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to insure uniformity of material in each layer.

E-5.02 Fill Moisture

When the moisture content of the fill material is below that recommended by the soils engineer, water shall then be added until he moisture content is as specified to assure thorough bonding during the compacting process.

When the moisture content of the fill material is above that recommended by the soils engineer, the fill material shall be aerated by blading or other satisfactory methods until the moisture content is as specified.

E-5.03 Fill Compaction

After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than 90% relative compaction. Compaction shall be by sheepsfoot rollers, multiple-wheel pneumatic tired rollers, or other types approved by the soil engineer.

Rolling shall be accomplished while the fill material is at the specified moisture content. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to insure that the desired density has been obtained.

E-5.04 Fill Slopes

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting of the slopes may be done progressively in increments of 3 to 4 feet in fill height. At the completion of grading, the slope face shall be compacted to a minimum of 90% relative compaction. This may require track rolling or rolling with a grid roller attached to a tractor mounted side-boom.

Slopes may be over filled and cut back in such a manner that the exposed slope faces are compacted to a minimum of 90% relative compaction.

The fill operation shall be continued in six inch (6") compacted layers, or as specified above, until the fill has been brought to the finished slopes and grades as shown on the accepted plans.

E-5.05 Compaction Testing

Field density tests shall be made by the geotechnical consultant of the compaction of each layer of fill. Density tests shall be made at locations selected by the geotechnical consultant.

Frequency of field density tests shall be not less than one test for each 2.0 feet of fill height and at least every one thousand cubic yards of fill. Where fill slopes exceed four feet in height their finished faces shall be tested at a frequency of one test for each 1000 square feet of slope face.

Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density reading shall be taken in the compacted material below the disturbed surface. When these readings indicate that the density of any layer of fill or portion thereof is below the required density, the particular layer or portion shall be reworked until the required density has been obtained.

E-6.00 SUBDRAINS

E-6.01 Subdrain Material

Subdrains shall be constructed of a minimum 4-inch diameter pipe encased in a suitable filter material. The subdrain pipe shall be Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent. Subdrain pipe shall be installed with perforations down. Filter material shall consist of 3/4" to 1 1/2" clean gravel wrapped in an envelope of filter fabric consisting of Mirafi 140N or approved equivalent.

E-6.02 Subdrain Installation

Subdrain systems, if required, shall be installed in approved ground to conform the approximate alignment and details shown on the plans or herein. The subdrain locations shall not be changed or modified without the approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in the subdrain line, grade or material upon approval by the design civil engineer and the appropriate governmental agencies.

E-7.00 EXCAVATIONS

E-7.01 General

Excavations and cut slopes shall be examined by the geotechnical consultant. If determined necessary by the geotechnical consultant, further excavation or overexcavation and refilling of overexcavated areas shall be performed, and/or remedial grading of cut slopes shall be performed.

E-7.02 Fill-Over-Cut Slopes

Where fill-over-cut slopes are to be graded the cut portion of the slope shall be made and approved by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope.

E-8.00 TRENCH BACKFILL

E-.01 General

Trench backfill within street right of ways shall be compacted to 90% relative compaction as determined by the ASTM D1557 test method. Backfill may be jetted as a means of initial compaction; however, mechanical compaction will be required to obtain the required percentage of relative compaction. If trenches are jetted, there must be a suitable delay for drainage of excess water before mechanical compaction is applied.

E-9.00 SEASONAL LIMITS

E-9.01 General

No fill material shall be placed, spread or rolled while it is frozen or thawing or during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the soils engineer indicate that the moisture content and density of the fill are as previously specified.

E-10.00 SUPERVISION

E-10.01 Prior to Grading

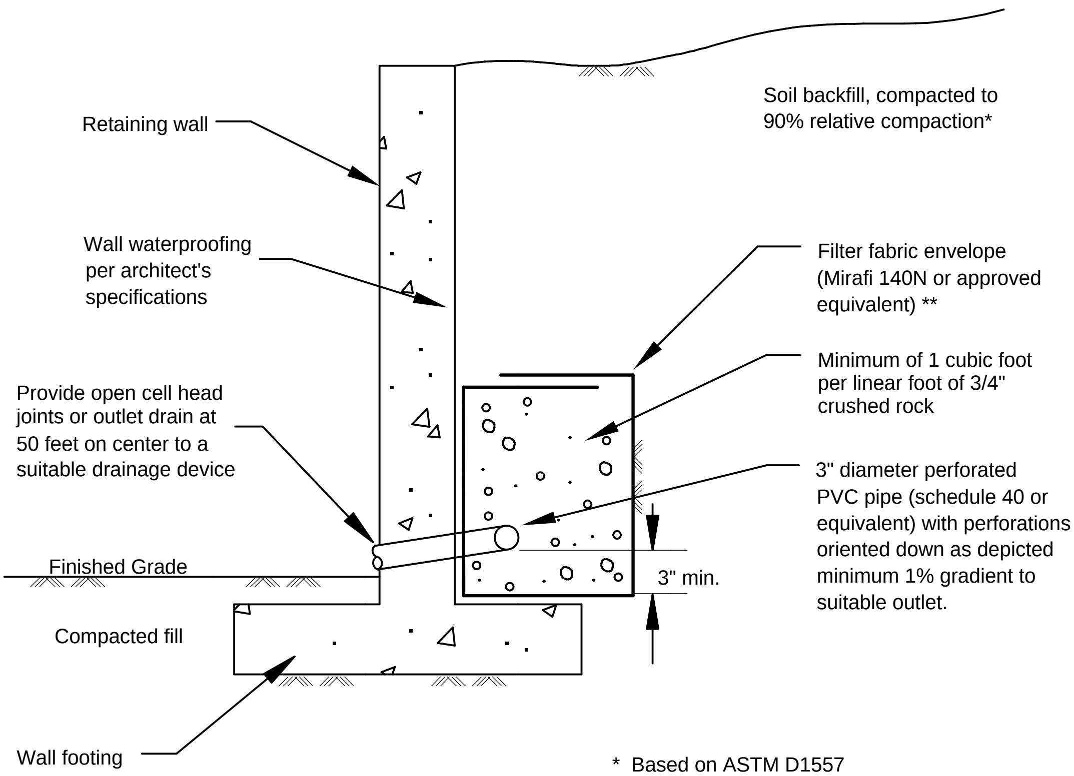
The site shall be observed by the geotechnical consultant upon completion of clearing and grubbing, prior to the preparation of any original ground for preparation of fill.

The supervisor of the grading contractor and the field representative of the geotechnical consultant shall have a meeting and discuss the geotechnical aspects of the earthwork prior to commencement of grading.

E-10.02 During Grading

Site preparation of all areas to receive fill shall be tested and approved by the geotechnical consultant prior to the placement of any fill.

The geotechnical consultant or his representative shall observe the fill and compaction operations so that he can provide an opinion regarding the conformance of the work to the recommendations contained in this report.



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2 PERMEABLE MATERIAL (CAL TRANS SPECIFICATIONS)

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

RETAINING WALL DRAINAGE DETAIL

APPENDIX F

REFERENCES

APPENDIX F

REFERENCES

1. Blake, T.F., 2000, FRISKSP Computer Program, Version 4.00.

2. Bryant, W.A. and Hart, E.W., 2007, Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology Special Publication 42, Interim Revision 2007 and online updates.

3. California Building Standards Commission, 2010 California Building Code.

4. California Department of Conservation, Division of Mines and Geology, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

5. California Department of Water Resources, 1970, Meeting Water Demands in the Bunker Hill-San Timoteo Area: Bulletin 104-5.

6. California Department of Water Resources, 2012, Online Groundwater Level Data, www.water.ca.gov/waterdatalibrary/.

7. California Division of Mines and Geology, 1977, Special Studies Zone Map of the Redlands Quadrangle, Effective January 1, 1977.

8. Cao, Y. and others, 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, June 2003.

9. City of San Bernardino, 2005, General Plan, Effective November 1, 2005.

10. Danskin, D.R., McPherson, K.R, Wollfenden, L.R., 2006, Hydrology, Description of Computer Models, and Evaluation of Selected Water-Management Alternative in the San Bernardino Area, California, USGS Open-File Report 2005-1278.

11. Dibblee, T.W., Jr., 2004a, Geologic Map of the Harrison Mountain/North ½ of Redlands Quadrangles, San Bernardino and Riverside Counties, California: Dibblee Geology Center Map #DF-126.

12. Federal Aviation Administration, Advisory Circular 150/5320-6E, Airport Pavement Design and Evaluation, date of publication September 30, 2009.

13. Federal Aviation Administration, Advisory Circular 150/5370-10F, Standards for Specifying Construction of Airports, date of publication September 30, 2011.

14. Federal Aviation Administration, FAARFIELD computer program, version 1.305, September 28, 2010.

15. Federal Emergency Management Agency, 2008, Flood Insurance Rate Map (FIRM) 06071C8707H, dated August 28, 2008.

16. Fife, D.L. and Rodgers, D.A., 1974, Generalized Map Showing Thickness of Fresh-Water Bearing Alluvium, Upper Santa Ana Valley and Maximum Credible Rock Acceleration from Earthquakes in the Vicinity of

Southwest San Bernardino County, California *in* California Division of Mines and Geology Special Report 113 (Plate 5B).

17. Google Earth, Aerial Photographs, 2011, 2009, 2007-2005, 2003, 2002, and 1995.

18. Historicaerials.com, Aerial Photographs, 2005, 1980, 1977, 1968, 1959 and 1938.

19. Historicaerials.com, Topographic Maps, 1999, 1988, 1986, 1979, 1973, 1969, 1964, 1963, 1960, 1958, 1955, 1951, 1946, 1939 and 1929.

20. Jennings, C.W., and Bryant, W.A, 2010, Fault Activity Map of California, California Geological Survey, Geologic Data Map No. 6.

21. Martin, G.R. and Lew, M., 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, Southern California Earthquake Center publication.

22. Matti, J.C., Morton, D.M., Cox, B.F. and Kendrick, K.J., 2003a, Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey OFR 03-302.

23. Matti, J.C., Morton, D.M., Cox, B.F., Carson, S.E., and Kendrick, K.J., 1992, Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey OFR 92-446.

24. Matti, J.C. and Carson, S.E., 1986 (preliminary report) 1991 (final report), Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California – A Regional Evaluation, United States Geological Survey Bulletin 1989.

25. Morton, D.M., 1978, Geologic Map of the Redlands Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey OFR 78-21.

26. Morton, D.M. and Miller, F.K., 2003, Preliminary Geologic Map of the San Bernardino 30x60 Minute Quadrangle, California: U.S. Geological Survey OFR 03-293.

27. San Bernardino County, 2010, Hazard Overlay and Geologic Hazard Overlay Map (FH31B and C).

28. San Bernardino County, 2011, Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP), dated May 19, 2011.

29. Seed, H.B. and Whitman, R.V., 1970, Design of Earth Structures for Dynamic Loads *in* American Society of Civil Engineers Specialty Conference State-of-the Art Paper, Lateral Stresses in the Ground and Design of Earth-Retaining Structures.

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

31. U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures, Publication CECW-E, Circular No. 1110-2-6058, Appendix G, http://www.usace.army.mil/publications/eng-circulars/ec1110-2-6058/

32. U.S. Geological Survey, 2011, Java Ground Motion Parameter Calculator, Version 5.0.10, ASCE 7 Standard, htpp://earthquake.usgs.gov/research/hazmaps/design/.

RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

March 29, 2013

RMA Project No.: 10-406-01

Parsons Brinckerhoff
451 E. Vanderbilt Way, Suite 200
San Bernardino, CA 92408

Attention: Ronald W. Sklepko
Senior Project Manager

Subject: Supplemental Coring Latter
Infrastructure Improvements Project
San Bernardino International Airport Authority
San Bernardino, CA

In accordance with your request we have completed supplemental data coring of the existing asphalt within the San Bernardino County Sheriff Aviation Facility Parcel at the San Bernardino International Airport. Coring was done on March 28, 2013. Locations of the cores are shown on the attached Exploration Location Map. The locations were cored and the underlying aggregate base was removed down to subgrade soils. The asphalt cores and aggregate base section was then measured. The cores were backfilled with the excavated aggregate base and patched with quick set grout.

Very truly yours,

RMA Group

PROFESSIONAL GEOLOGIST
KENNETH R. DOWELL
No. 2470
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

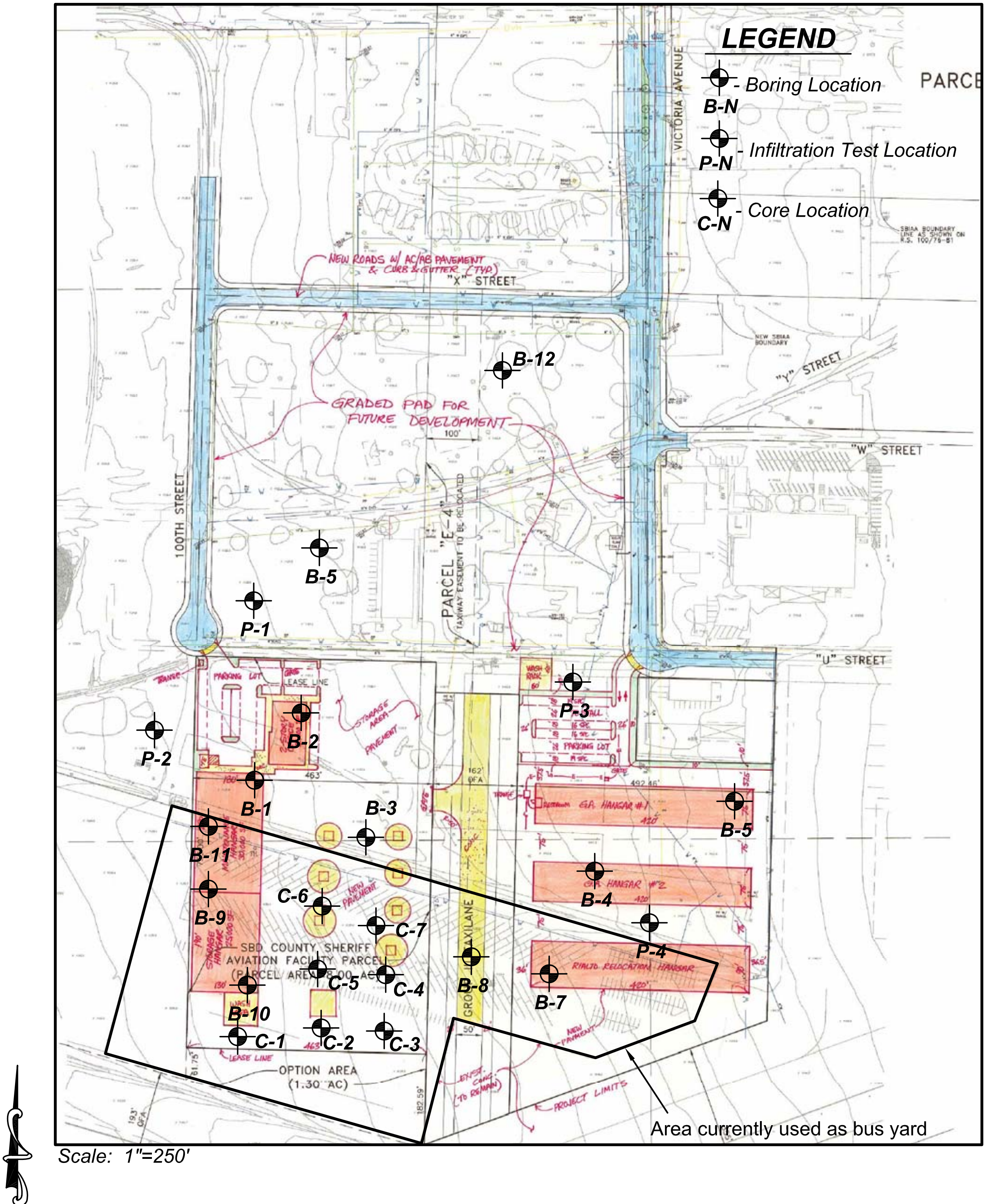
Kenneth Dowell, P.G., C.E.G.
Project Geologist

Attachments: Exploration Location Map

THICKNESS OF COMPACTED BITUMINOUS PAVING MIXTURES SPECIMENS
(Test Method: ASTM D3549)

Core No.	Asphalt Core Thickness (inches)	Aggregate Base Thickness (inches)
C-1	5" AC	6" AB
C-2	5" AC	6" AB
C-3	5" AC	5½" AB
C-4	4" AC	6" AB
C-5	5" AC	6" AB
C-6	5" AC	5¾" AB
C-7	5½" AC	7" AB

GEOTECHNICAL CONSULTANTS



Base map provided by Parsons Brinkerhoff, Inc.

EXPLORATION LOCATION MAP

San Bernardino International Airport Authority, Infrastructure Improvements Project
Parsons Brinckerhoff

RMA No.: 12-406-01
Figure 3

RMA Geotechnical Investigation

Photos of wire mesh encountered in Borings B-9, B-10 and B-11, as noted in report.