Appendix E:
Geology and Soils

THIS PAGE INTENTIONALLY LEFT BLANK

Geotechnical Investigation

2071 Tice Valley Boulevard
Walnut Creek, California

Report No. 195053 has been prepared for:

Pulte Group

6210 Stoneridge Mall Road, 5th Floor, Pleasanton, California 94588

August 13, 2012

Jim P. Tomkins, E.I.T Staff Engineer	Scott M. Leck, P.E., G.E. Principal Geotechnical Engineer	Erin L. Steiner, P.E.,G.E. Senior Project Engineer Quality Assurance Reviewer

405 Clyde Avenue, Mountain View, California 94043-2209 Main: 650.967.2365 Fax: 650.967.2785
website: www.trcsolutions.com

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
2071 TICE VALLEY BOULEVARD
WALNUT CREEK, CALIFORNIA

1.0 INTRODUCTION

In this report, we present the results of our geotechnical investigation for the 2071 Tice Valley Boulevard development to be located in Walnut Creek, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed development.

For our use, we received a drawing titled "Site Exhibit Tice Valley Boulevard", dated June 14, 2012 by Civil Engineering Associates.

1.1 Project Description

As presently planned, the project consists of construction of approximately 81 three-story townhome residences. Site grading is anticipated to consist of cuts and fills of about 2 feet or less. Additional improvements will include streets, underground utilities and landscape areas.

1.2 Scope of Services

Our scope of services was presented in detail in our agreement with you dated July 10, 2012. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling four borings and retrieving soil samples for observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

Environmental services were not included as part of this study.

2.0 SITE CONDITIONS

2.1 Exploration Program

Subsurface exploration was performed on August 1, 2012 using conventional, truck-mounted drilling to investigate, sample, and log subsurface soils. Four exploratory borings were drilled to depths of approximately 30 feet. Our borings were permitted and backfilled in accordance with the Contra Costa Valley Water District guidelines. The approximate locations of the borings are shown on the Site Plan, Figure 2. Logs of our borings and details regarding our field investigation are included in Appendix A. Our laboratory tests are discussed in Appendix B.

2.2 Surface

We also performed a brief surface reconnaissance during our site exploration. The site consists of a 6.4-acre parcel located at 2071 Tice Valley Boulevard. The site is located just south of Tice Valley Boulevard, and at the time of our field exploration, this site was occupied by a school and landscape nursery. Access roads, a gymnasium and other landscape areas were also observed during our investigation. Based on our observation, the site appears relatively level with minor grade variation for drainage purposes.

2.3 Subsurface

An existing pavement section consisting of about 3 to 4 inches of asphalt concrete over 2 to 6 inches of aggregate base was present at the location of Borings EB-2 and 3. Exposed at the ground surface or present below the pavement sections, our explorations generally encountered medium stiff to hard lean to fat clays. A 4-5 foot thick layer of gravel was encountered in Boring EB-3, which may be fill. Below the upper clays and gravel, the explorations encountered interbedded medium dense to dense clayey sands and medium stiff to very stiff clays and sandy clays.

A Plasticity Index (PI) test was performed on a representative near surface clayey soil sample in Boring EB-2 at about 2 feet. The test result exhibited a PI of 33 indicating the near surface clayey soils at the site have high plasticity and expansion potential. Moisture contents of the upper 5 feet of soils varied from 17 to 32 percent. Two samples tested indicated percent passing the #200 sieve of 26 percent.

2.4 Ground Water

Free ground water was encountered during our subsurface exploration in all of the borings at depths of approximately 11 to 20 feet. The ground water depth was measured at the time of drilling and may not reflect a stabilized level. All borings were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made. The site has not in an area that has been mapped by the California Geological Survey (CGS) for depth of historic high ground water levels. Based on our ground water data at this and other sites in the area, we have assumed a design ground water depth of 10 feet for our liquefaction analyses.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). Fault rupture through the site, therefore, is not anticipated.

The closest active or potentially active faults to the site are the Calaveras Fault located approximately 1 mile to the southeast, the Concord-Green Valley Fault located approximately 2½ miles to the east and the Hayward Fault, located approximately 8½ miles to the west.

3.2 Maximum Estimated Ground Shaking

The current California Building Code (CBC) method indicates a peak ground surface acceleration of 0.40g, which is equal to the Design Spectral Response Acceleration Parameter (S_{DS}) divided by 2.5 as discussed in Section 1803.5.12 of the 2010 CBC.

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2007) estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2007 and 2037. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region.

The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco located more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is not in a area which has been mapped by the State of California as having potential for seismically induced liquefaction hazards. During cyclic ground shaking such as earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix and result in liquefaction. Liquefied soil may lose shear strength and lead to large shear deformations and/or flow failure (Youd et al, 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil. The layers of sand identified in the borings were found to either be too dense or contain too many soil fines to liquefy. Therefore, the liquefaction potential at the site is judged to be low.

3.5 Seismically-Induced Dry Sand Densification

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Our explorations did not encounter any loose cohesionless soils above the design ground water depth; therefore, we judge the probability of significant differential settlement of non-saturated sand layers at the site to be low.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel or excavation. In soils, this movement is generally due to failure along a weak plane and may often be associated with liquefaction. There are no creeks or open bodies of water within an appropriate distance from the site for lateral spreading to occur, and the potential for liquefaction is low. For these reasons, the probability of lateral spreading occurring at the site during a seismic event is low.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted four samples collected during our subsurface investigation to an analytical laboratory for chloride, soluble sulfate,pH and electrical resistivity testing. The results of these tests are summarized in Table 2 below.

Table 2. Results of Corrosivity Testing

Sample No.	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
EB-1, 2A	3½	8	57	7.2	869	Corrosive	Negligible
EB-2, 3A	5½	5	43	7.3	1,153	Corrosive	Negligible
EB-3, 2A	3½	97	4,320	7.4	837	Corrosive	Severe
EB-4, 1B	2	< 2	4	5.8	1,582	Corrosive	Negligible

Note: mg/kg = milligrams per kilogram = parts per million (ppm)
*Resistivity measured at 100% saturation

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 3 below.

Table 3. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 500	Very Severe Corrosion
501 to 2,000	Corrosive
2,001 to 8,000	Moderately Corrosive
8,001 to 32,000	Mildly Corrosive
>32,000	Progressively Less Corrosive

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 4.

Table 4. Relationship Between Sulfate Concentration and Sulfate Exposure (Table 4.2.1 of ACI)

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

1= seawater

Acidity is an important factor of soil corrosivity; the lower the pH (the more acidic the environment), the higher the soil corrosivity with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 2, soil resistivity results range from 869 to 1,582 ohm-centimeters. Based on these test results and resistivity correlations presented in Table 3, the corrosion potential to buried metallic improvements may be characterized as corrosive. We recommend that a corrosion engineer be consulted regarding corrosion protection for any underground metallic pipelines proposed for the project.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered severe for the native subsurface materials sampled at the location of Boring EB-3. We recommend that additional laboratory testing be performed once mass grading has been completed to further evaluate the corrosion potential to concrete. Depending on the laboratory test results, it may be necessary to design concrete that will be in contact with on-site soils in accordance with Section 4.2 of the ACI.

5.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

5.1 Conclusions

From a geotechnical engineering viewpoint, the proposed development may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

The primary geotechnical concerns at the site are as follows:

- The presence of highly expansive soils
- The presence of undocumented fill
- Compaction difficulties due to high moisture content of existing on-site soil

- The presence of existing buildings and pavements which will require demolition prior to site development
- Shallow ground water

We have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the development.

5.1.1 Expansive Soils

To reduce the potential for damage to the planned structures due to the presence of highly expansive near-surface soils, we recommend foundations be designed as post-tensioned mats and have sufficient reinforcement. Detailed recommendations are presented in the following sections of this report. Site flatwork will require a non-expansive fill (NEF) section below them.

5.1.2 Undocumented Fill

As previously discussed, gravelly soils on the order of 4- to 5-feet-thick are present at a depth of 3 feet in Boring EB-3. Based on the blow counts, this material appears to be relatively well compacted. In addition, this soil has high sulfate content not identified at other soils at the site. To support structures on a shallow foundation system, the existing area around the location of Boring EB-3 should be pot-holed during site grading to determine the lateral extend of this material and field density tests should be performed. If the material is found to be at least compacted to at least 90 percent relative density, it can remain in place and be used for structural support. Foundations that come into contact with this material should be designed to resist the adverse affects of the high sulfate content of the soils. Detailed recommendations for mitigating undocumented fills are provided in Section 6.2.

5.1.3 Compaction Difficulties Due to High Moisture Content

Based on the boring logs the moisture content of the upper 3 feet of soil is likely 5 to 10 percent above the optimum moisture content. It is our experience that the on-site soils are very moisture-sensitive and are very difficult to compact when significantly above the optimum moisture content. General guidelines addressing this potential concern are discussed in Sections 6.4 and 6.8 of this report.

5.1.4 Demolition Debris

A significant amount of debris above and below grade is anticipated as a result of the site demolition required prior to site grading. The debris should be collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. It has been our experience that some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of over-excavation and backfill to rebuild over-excavated areas, along with other similar work items. See Section 6.6 for recommendations regarding re-use of recycled materials on site.

5.1.5 Shallow Ground Water

We anticipate that ground water exists at a depth of approximately 10 feet below the existing ground surface. Our experience with similar sites in the vicinity indicates that shallow ground water could significantly impact underground construction. These impacts typically consist of potentially wet and unstable subgrade soils, difficulty achieving compaction, and difficult underground utility installation. Dewatering of utility trenches and subgrade stabilization may be required in some isolated areas of the site, depending on the time of year that construction begins and the depth of the excavation.

5.2 Plans, Specifications and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. Since subsurface materials encountered in the relatively small diameter and widely spaced explorations may vary significantly from other subsurface materials on the site, we also recommend that a representative of our firm observe and test the geotechnical aspects of the project during construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation, and if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. We recommend that trees and shrubs designated to be removed should include the entire rootball and all roots larger that ½-inch in diameter. Depressions resulting from removal of trees and shrubs should be cleaned of loose soils and roots, and properly backfilled in accordance with the "Compaction" section of this report. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures be carried out under our observation and that backfill be tested during placement.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 4 inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscape areas, if desired.

6.2 Removal of Undocumented Fill

As previously discussed, gravelly soils on the order of 4- to 5-feet-thick are present at a depth of 3 feet in Boring EB-3. Based on the blow counts, this material appears to be relatively well compacted. In addition, this soil has high sulfate content not identified at other soils at the site. To support structures on a shallow foundation system, the existing area around the location of

Boring EB-3 should be pot-holed during site grading to determine the lateral extend of this material and field density tests should be performed. If the material is found to be at least compacted to at least 90 percent relative density, it can remain in place and be used for structural support.

All fills not found to be properly compacted should be removed down to the native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed building areas should be removed in their entirety. As an alternative, it may be feasible to abandon (in-place) underground utilities within the proposed building areas provided the utility does not conflict with new improvements, is completely grouted, and previous fills associated with the utility do not pose a risk to the structures. Existing underground utilities outside the proposed building areas may be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

Fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) and increasingly higher with increasing diameter.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, foundations, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

Because of the high in-situ moisture content of the native soils relative to laboratory optimum water content, as a minimum, the grading contractor should plan to aerate the soils prior to reuse as engineered fill. Additionally, the contractor should minimize wheels loads on the subgrade during earthwork to reduce the potential for pumping. There is a high potential of subgrade yielding/pumping and the contractors should plan accordingly. Vibratory smooth-drum roller equipment should be used with caution as this equipment will contribute to pumping of already wet subgrade soils. Additional recommendations for treatment of wet soils are presented in Section 6.8.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 20 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of the proposed import fill should be submitted to us at least ten days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.6 Reuse of On-site Recycled Materials

If desired to reuse existing asphalt and/or concrete as engineered fill, we recommend that it be ground up to meet the gradation requirements for its intended use. If laboratory testing indicates that the recycled material meets Caltrans Class 2 aggregate base specifications, it may be used as aggregate base beneath private pavements and sidewalks. We should further evaluate the proposed use of recycled materials prior to the work being performed. The City of Walnut Creek should be consulted regarding the use of any recycled materials within the public right-of-way. Fill containing recycled asphalt should not be used within the top two feet of residential lots or unpaved common areas.

6.7 Compaction

All fill as well as scarified surface soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content at least 1 percent over laboratory optimum, except for the native expansive clays. The native expansive clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in un-compacted thickness. Each successive lift should be firm and non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base [and sub-base] should be compacted to at least 95 percent relative compaction except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

6.8 Wet Weather Conditions/Wet Soils

It should be understood that earthwork such as fill placement and trench backfill may be very difficult during wet weather, especially for fill materials with a significant amount of clay. In addition, we identified moisture contents of 5 to 10 percent over optimum moisture content in the upper 5 feet of the site soils. As the water content in the soil increases significantly above the optimum moisture content, the soils will become soft, yielding, and difficult to compact. Therefore, we recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

There are several alternatives to facilitate fill placement and trench backfill if earthwork is performed during the wet winter season, and the moisture content of the fill materials increases significantly above optimum moisture.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction.
- Over-excavation the fill and replace with suitable on-site or import materials with an appropriate moisture content.
- Install a geo-synthetic (geotextile or geogrid) to reduce surface yielding and reinforce soft fill.
- Chemically treat with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

6.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. Contractors should plan on drying the native clay soils prior to reuse as engineered fill. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. If native expansive soil is used for trench backfill, it should be compacted to between 87 to 92 percent at a moisture content at least 3 percent over optimum. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with expansive subgrade material.

6.10 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

On a preliminary basis, and based on the soils encountered in the upper 5 feet during our site exploration, site soils can be classified as Type B based on soil classification proposed by OSHA. A representative of TRC should be retained to verify soil conditions in the field at the time of excavation to finalize our classification of soil type.

6.11 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscape areas and ½ percent in pavement areas, should be provided to direct surface water away from foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.12 Landscaping Considerations

As the near-surface soils are highly expansive, we recommend greatly restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering,
- Using low flow-rate spray sprinkler heads, or drip irrigation systems,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that the landscape architect incorporate these items into the landscaping plans, and that we review the plans before construction.

6.13 Construction Observation

A representative from our company should observe and test the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials and the placement and compaction of fill. To facilitate your construction schedule, we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

7.0 FOUNDATIONS

Provided the site is prepared in accordance with the recommendations of this report, the proposed residential structures may be supported on post-tensioned mat slabs as discussed below. Retaining and sound walls may be supported on shallow footing foundations which are also discussed below.

7.1 2010 CBC Site Class and Site Seismic Coefficients

Chapter 16 of the 2010 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations the site is underlain by deep alluvial soils, which corresponds to a soil profile type D. Based on this information and local seismic sources, the site may be characterized for design using the information in Table 5.

Table 5. 2010 CBC Site Class and Site Seismic Coefficients

Latitude: 37.876 N Longitude:122.069 W	CBC Table/ Figure	Factor/ Coefficient	Value
Soil Profile Type	Table 1613.5.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.5(3)	S_s	1.5
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.5(4)	S_1	0.6
Site Coefficient	Table 1613.5.3(1)	F_a	1.0
Site Coefficient	Table 1613.5.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameter	Equation 16A-37	S_{MS}	1.5
Adjusted MCE Spectral Response Parameter	Equation 16A-38	S_{M1}	0.9
Design Spectral Response Acceleration Parameter	Equation 16A-39	S_{DS}	1.0
Design Spectral Response Acceleration Parameter	Equation 16A-40	S_{D1}	0.6

7.2 Post-Tensioned Mats

We recommend the proposed residential structures be supported on post-tensioned mats bearing on prepared natural soil or compacted fill. The building pads should be prepared in accordance with the recommendations presented in the "Subgrade Preparation" and "Compaction" sections of this report. Before mat construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for mat support. The use of a post-tensioned mat foundation system, as opposed to a conventionally reinforced mat, should reduce the amount of objectionable slab cracks, but will not prevent the development of bending stresses in the mat due to differential movement of the supporting subgrade. A post-tensioned mat distributes the differential movement of the supported structure over a longer span, through bending of the mat.

Post-tension mats should be designed with the criteria presented in Table 6 using an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. The structural engineer should determine the mat thickness and reinforcing in accordance with the anticipated use and loading. The design should take into consideration soil-structure interaction and site specific soil characteristics based on the results of our investigation.

Table 6. Post-Tension Mat Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation (ft.)	7.7	4.0
Differential Soil Movement (in.)	0.5	1.25

The above design criteria are based on the procedures developed by the Post-Tensioning Institute (PTI) presented in the 2005, 3rd edition, 2nd printing of "Design of Post-Tensioned Slabs-On-Ground."

According to the PTI manual, the following assumptions were made when using this design procedure:

- The site will be sloped such that the water flows away from the foundation for a distance of 10 ft from the perimeter,
- No vegetation over 6 feet tall shall be planted within 20 feet of the foundation perimeter unless specifically accounted for the design,
- Downspouts shall be tied directly into storm drains or other means to direct excessive moisture away from the foundation,
- The site shall be maintained during the design life of the foundation, which includes systematic watering on all sides of the foundations and during dry periods, observing to ensure that the soil does not pull away from the foundation,
- The foundation should not be constructed over a cut/fill transition in expansive or compressible soils without proper considerations for swell and/or settlement potential, and
- Stoops, porches, patios, etc, shall be cast independently of the slab foundation unless it is the foundation for a supporting member, such as a column.

In addition, since the post-tensioned mat is a flexible foundation system, consideration should be given to compatibility with roof trusses, load concentrations, brittle exterior siding, drainage and utility connections. The design procedures provided in the PTI manual are only valid if the above assumptions are taken into account. Proper site preparation and maintenance are vital to the success of the foundation design.

Underground utilities paralleling the mat, located 4 feet or shallower, should be located at least 4 feet outside of the edge of the mat slab. Utility plans should be reviewed by us prior to trenching for conformance to this requirement.

If desired to minimize floor moisture in habitable areas, we recommend that a moisture barrier system be constructed beneath the mats. The recommendations presented in the "Moisture Protection Considerations" should be considered.

7.4 Conventionally-Reinforced Mat Foundations

Alternately, the proposed residences may be supported on conventionally reinforced mat foundations. Mat foundations may be designed in accordance with the edge and center lift criteria presented in the Post-Tensioned Mat section above. All mats should be designed with a thickened edge at least 12 inches wide and 12 inches thick. The thickened edge should be considered from top to bottom of the mat.

All mats may be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

All mats should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the subgrade of the mat foundation prior to placement of reinforcing steel.

If desired to minimize floor moisture in habitable areas, we recommend that a moisture barrier system be constructed beneath the mats. The recommendations presented in the "Moisture Protection Considerations" should be considered.

7.3 Mat Foundation Settlement

Total settlement due to static loading should be less than about ½- inch. As discussed previously, sand layers were found to be too dense to liquefy.

7.4 Lateral Loads

Lateral loads may be resisted by friction between footings or mat foundations and the supporting subgrade. An ultimate frictional resistance of 0.45 may be used for design of foundations. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an ultimate passive pressure based on an equivalent fluid pressure of 450 pounds per cubic foot (pcf) be used in design. Passive resistance should be neglected within the upper 12 inches of the soil for foundations. An appropriate factor of safety should be applied to the ultimate values shown above, as determined by the structural engineer.

7.5 Building Pad Moisture Conditioning

Due to the high expansion potential of the near surface soils, we recommend that finished pads be moisture conditioned to at least 3 percent over optimum in the upper 12 inches of the building pad prior to placing the moisture barrier system. The moisture content of the finished pads should be checked within 24 hours prior to the construction of the moisture barrier.

7.6 Moisture Protection Considerations

Since the long-term performance of concrete mat foundations depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete mat of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with mat foundation construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class C requirements should be placed directly below the mat. The vapor barrier should extend to the edge of the mat. If the mat is 8 inches thick or less, at least 4 inches of free-draining gravel, such as ½- or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E1643 requirements.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist cured for a minimum of seven days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute and Portland Cement Association, and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. The application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 RETAINING WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

8.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½- to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

8.3 Foundations

8.3.1 Footings

The retaining walls and sound walls may be supported on conventional continuous spread footings bearing on natural, undisturbed soil or compacted fill. All footings should have a minimum width of 12 inches and should extend at least 18 inches below lowest adjacent finished grade. Because of the high expansion potential of the near-surface soils, this relatively deeper footing is recommended to place bearing surfaces below the zone of significant moisture fluctuation in order to reduce the effects of heave or shrinkage.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe the all footing excavations before reinforcing steel is placed.

We estimate that total foundation movement under static loads will be less than ½-inch, with post-construction differential movement of less than ¼-inch over a horizontal distance of 25 feet. As discussed in the "Liquefaction" section, we estimate that liquefaction is unlikely as the sand layers were found to be too dense to liquefy.

Lateral loads may be resisted by friction between footings and the supporting subgrade. An ultimate frictional resistance of 0.45 may be used for design of foundations. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an ultimate passive pressure based on an equivalent fluid pressure of 450 pounds per cubic foot (pcf) be used in design. Passive resistance should be neglected within the upper 12 inches of the soil for foundations. An appropriate factor of safety should be applied to the ultimate values shown above, as determined by the structural engineer.

8.3.2 Drilled Piers

Alternately, retaining walls and sound walls can be supported on drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of at least 12 inches and extend at least 8 feet below the existing ground surface Piers may be designed for an allowable skin friction of 400 pounds per square foot (psf) for combined dead plus live loads with a one-third increase allowed for either transient wind or seismic loading. Piers should have a minimum center-to-center spacing of at least three pier diameters. Grade beams should be designed to span between piers in accordance with structural requirements.

Resistance to uplift loads will be developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 300 psf be used.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the excavation of all piers be performed under our direct observation to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

Due to the presence of sands and gravels, casing of each shaft may be necessary. Any ground water encountered should be removed from excavations prior to concrete placement. If it can't ve removed, then concrete will need to be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole.

Total settlement for the recommended pier foundations should not exceed ½- inch and post construction differential settlement across the walls founded on piers should be less than ¼- inch in 50 feet due to static loads.

Lateral loads exerted on structures supported on piers and grade beams may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 450 pounds per cubic foot (pcf) acting against two times the projected diameters of the individual pier shafts below rough pad grade, with a maximum of 2,000 psf at depth. The upper 12 inches of soil should be neglected when calculating the lateral capacity of the piers.

8.4 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

9.0 PAVEMENTS

9.1 Asphalt Concrete

Using estimated traffic indices for various pavement-loading requirements, and a design R-Value of 5, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 7.

Table 7. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 5

Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are moderately to highly expansive, some increased maintenance and reduction in pavement life can be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

9.2 Portland Cement Concrete Pavements

Recommendations for Portland Cement Concrete (PCC) pavements are presented below in the table below. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 8. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
4	5
57	5½
480	6

Our design is based on an R-value of 5 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

9.3 Pavement Cut-off

Because the native soils at the site are moderately to highly expansive, surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it may be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cut-off barrier may be eliminated. If desired to install pavement cut-off barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

9.5 Exterior Concrete Flatwork and Sidewalks

Exterior slabs-on-grade, such as pedestrian walkways, patios, and sidewalks, will experience seasonal movement due to the expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that moderate slab movement or localized cracking and/or distress could still occur.

1. The minimum recommendation for concrete flatwork constructed on expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by

scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

2. Concrete flatwork should be at least 4 inches thick and underlain by at least 6 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction.

3. Use a maximum control joint spacing of no more than 8 feet in each direction and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

10.0 LIMITATIONS

This report has been prepared for the sole use of Pulte Group specifically for design of the 2071 Tice Valley Boulevard project in Walnut Creek, California. The opinions, conclusions and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between explorations do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

11.1 Literature

American Concrete Institute, 2008, *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*, An ACI Standard, first printing, January.

California Building Code, 2010, *Structural Engineering Design Provisions*, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada*, International Conference of Building Officials, February.

California Division of Mines and Geology, 1997, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117, March.

Campbell, K.W. and Bozorgnia, Y., 1994, *Near-Source Attenuation of Peak Horizontal Acceleration from Worldwide Accelerograms Recorded from 1957 to 1993*, Proceedings, Fifth U.S. National Conference on Earthquake Engineering, Vol. III, Earthquake Engineering Research Institute, pp. 283-292.

Ishihara, K. and Yoshimine, M., 1990, *Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes*, Soils and Foundations, 32 (1): 173-188.

Portland Cement Association, 1984, *Thickness Design for Concrete Highway and Street Pavements*: report.

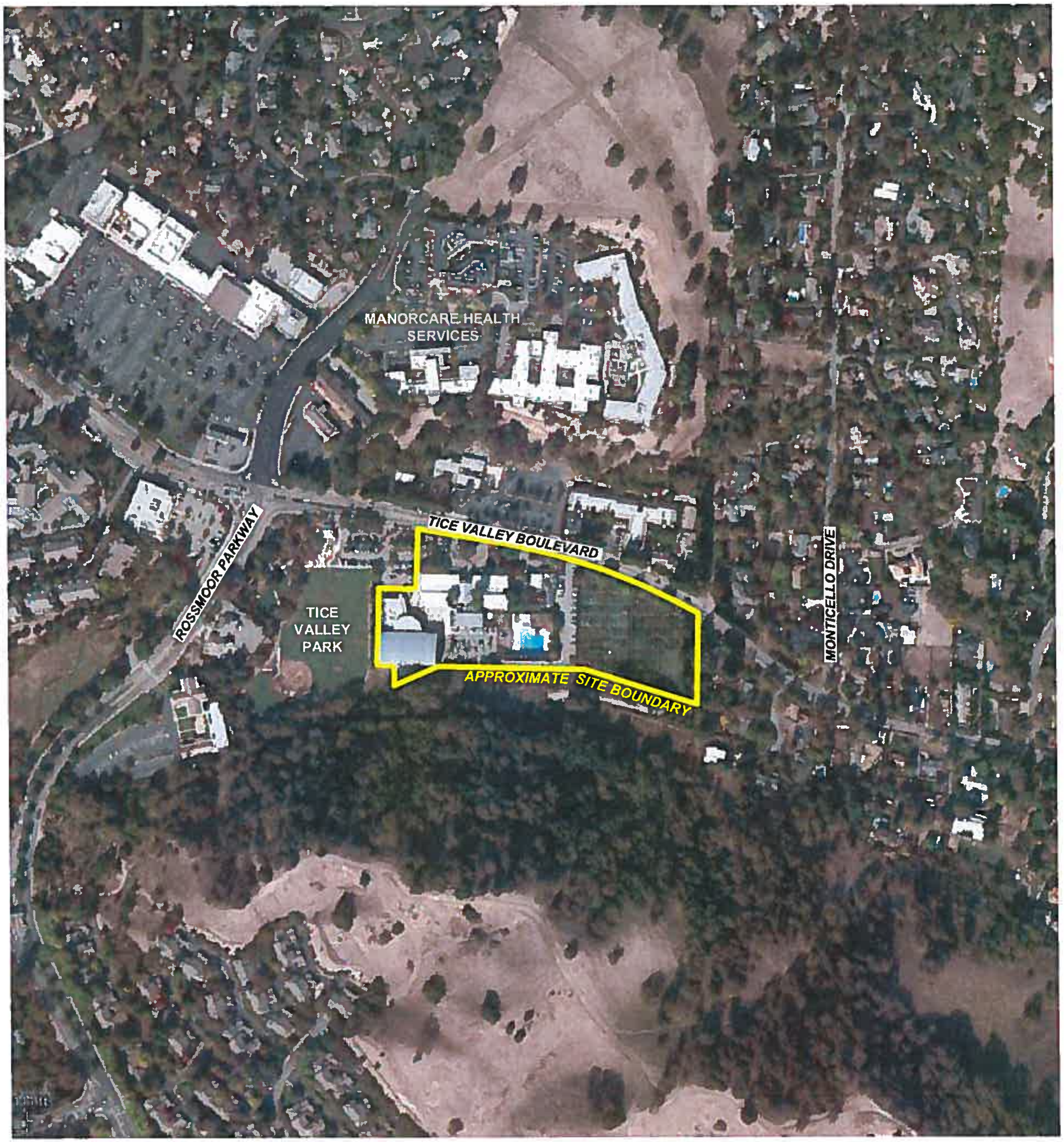
State of California Department of Transportation, 2006, *Highway Design Manual*, September.

Working Group (2007) on California Earthquake Probabilities, 2008, *The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S Geological Survey*, Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution No. 1138, 2008.

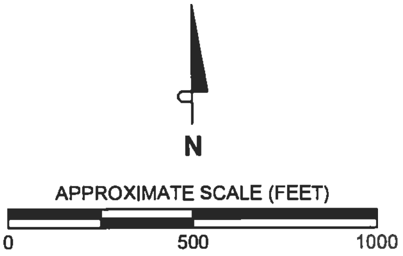
Youd, T.L. and Garris, C.T., 1995, *Liquefaction-Induced Ground-Surface Disruption:* Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805-809.

Youd et al, 2001, *Liquefaction Resistance of Soils*: Summary Report from the 1996 NCEER and 1998 NCEER/NSF *Workshops on Evaluation of Liquefaction Resistance of Soils*, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October.

* * * * * * * * * *



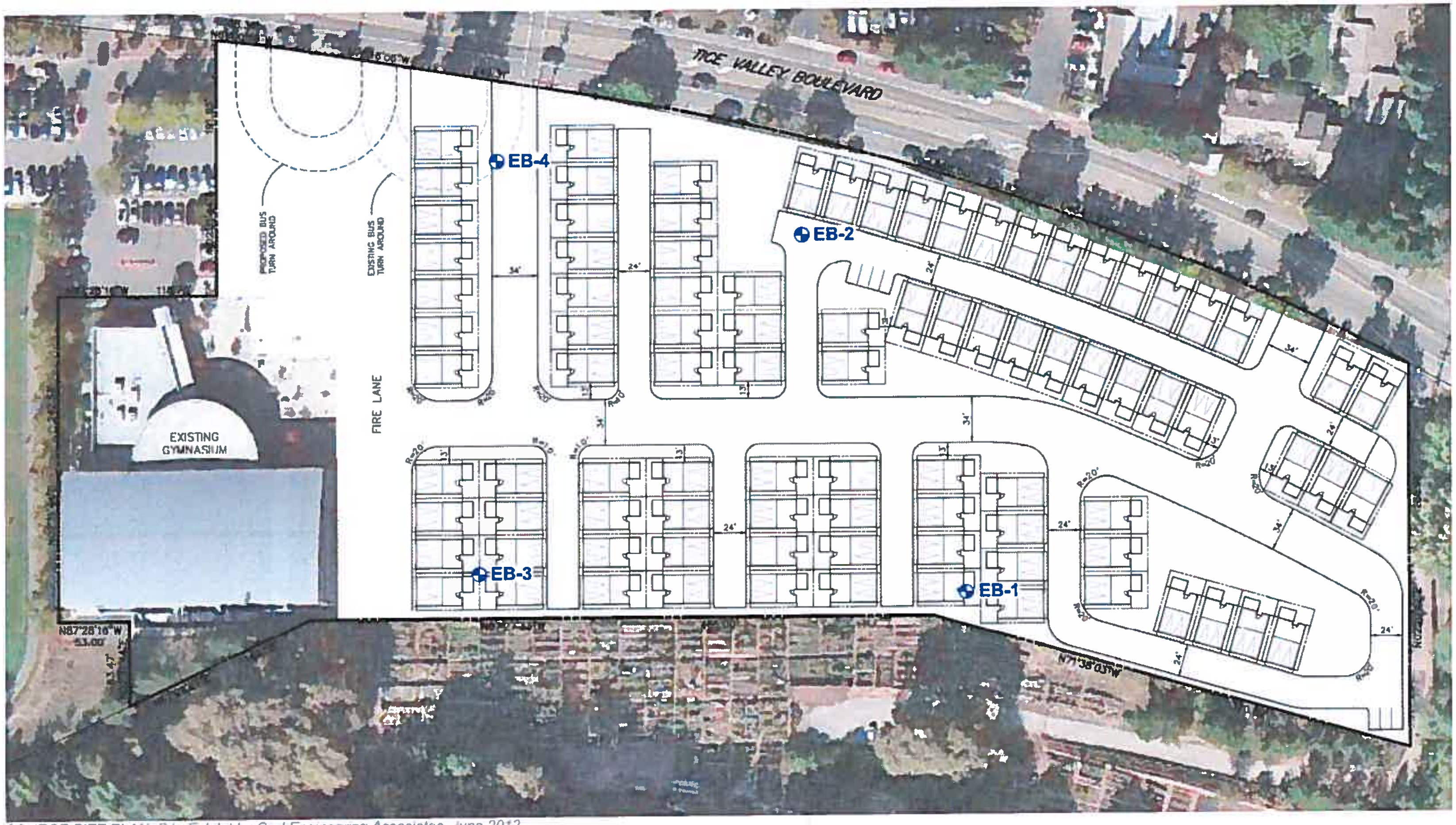
SOURCE AERIAL PHOTO: Google Earth, October 2011



VICINITY MAP

2071 Tice Valley Boulevard
Walnut Creek, California

TRC | 195053 | **FIGURE 1**



SOURCE SITE PLAN: Site Exhibit by Civil Engineering Associates, June 2012

LEGEND

● Approximate location of exploratory boring

N

SCALE (FEET)

0 40 80

SITE PLAN

2071 Tice Valley Boulevard
Walnut Creek, California

TRC 195053 **FIGURE 2**

APPENDIX A
FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on August 1, 2012 to maximum depth of 30 feet. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil interpretations, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing site boundaries. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * * *

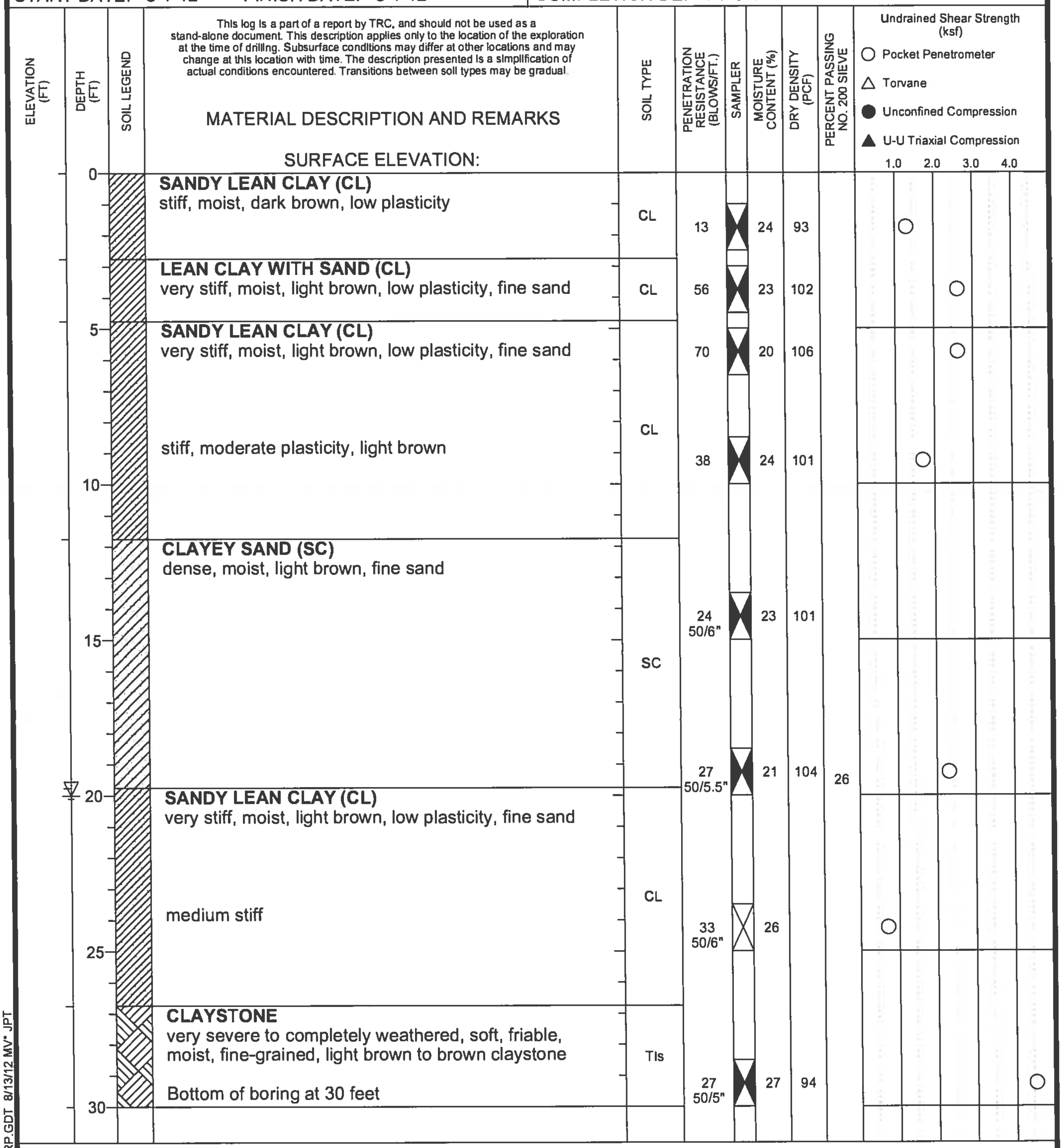
EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: JPT
START DATE: 8-1-12 FINISH DATE: 8-1-12

PROJECT NO: 195053
PROJECT: 2071 Tice Valley Boulevard
LOCATION: Walnut Creek, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.



MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
0	**SANDY LEAN CLAY (CL)** stiff, moist, dark brown, low plasticity	CL	13	24	93	
	LEAN CLAY WITH SAND (CL) very stiff, moist, light brown, low plasticity, fine sand	CL	56	23	102	
5	**SANDY LEAN CLAY (CL)** very stiff, moist, light brown, low plasticity, fine sand	CL	70	20	106	
	stiff, moderate plasticity, light brown		38	24	101	
10						
	CLAYEY SAND (SC) dense, moist, light brown, fine sand	SC	24 50/6"	23	101	
15						
			27 50/5.5"	21	104	26
20	**SANDY LEAN CLAY (CL)** very stiff, moist, light brown, low plasticity, fine sand	CL				
	medium stiff		33 50/6"	26		
25						
	CLAYSTONE very severe to completely weathered, soft, friable, moist, fine-grained, light brown to brown claystone Bottom of boring at 30 feet	Tls	27 50/5"	27	94	
30						

GROUND WATER OBSERVATIONS:
∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 20.0 FEET

LA CORP.GDT 8/13/12 MV' JPT

TRC

EXPLORATORY BORING: EB-2

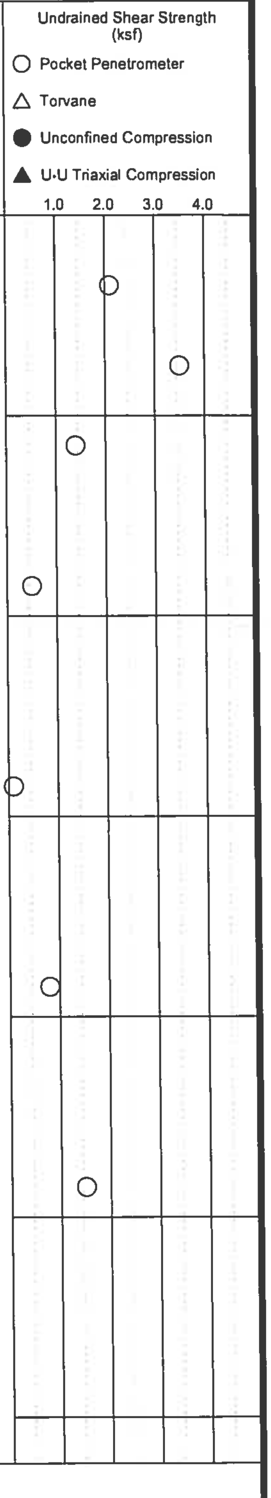
Sheet 1 of 1

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: JPT
START DATE: 8-1-12 FINISH DATE: 8-1-12

PROJECT NO: 195053
PROJECT: 2071 Tice Valley Boulevard
LOCATION: Walnut Creek, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
0	**3.0" of AC over 6.0" of AB**	AC				
	FAT CLAY (CH) very stiff, moist, dark "brownish-gray", high plasticity Plasticity Index = 33, Liquid Limit = 55	CH	34	25	98	
	SANDY LEAN CLAY (CL) very stiff, moist, dark brown, low plasticity, fine sand	CL	28	20	105	
5	**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand	SC	24	32	94	
	LEAN CLAY (CL) stiff, moist, brown, moderate plasticity	CL				
	SANDY LEAN CLAY (CL) medium stiff, moist, brown, low plasticity, fine sand	CL	13	27	95	
10						
	very soft		12	30	90	
15						
	medium stiff		23	27	93	
20						
			35	25	103	
25						
	CLAYEY SAND (SC) dense, moist to wet, gray, fine to coarse sand	SC	52	23		
	Bottom of boring at 30 feet					
30						



GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 11.0 FEET

LA CORP.GDT 8/13/12 MV" JPT

TRC

EXPLORATORY BORING: EB-3

Sheet 1 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 195053
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: 2071 Tice Valley Boulevard
LOGGED BY: JPT	LOCATION: Walnut Creek, CA
START DATE: 8-1-12 FINISH DATE: 8-1-12	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	0	**4.0" of AC over 2.0" of AB**	AC						
		LEAN CLAY WITH SAND AND GRAVEL (CL) hard, moist, brown, low plasticity, fine to coarse sand, fine to coarse gravel (sub-angular)	CL	45	⧗	15	112		○
		CLAYEY GRAVEL (GC) [FILL?] dense, moist, brown, fine to coarse sand, fine to coarse gravel (sub-angular)	GC	68	⧗	7	99		
	5	**POORLY GRADED GRAVEL (GP) [FILL?]** medium dense, moist, brown, fine to coarse sand, fine to coarse gravel (sub-angular)	GP	31	⊠	7			
		SANDY LEAN CLAY (CL) stiff, moist, dark brown, low plasticity, fine sand	CL	16	⧗	23	102		○
	10								
		stiff		35	⧗	24	99		○
	▽ 15								
		CLAYEY SAND (SC) medium dense, moist to wet, brown, fine to coarse sand	SC	28	⧖	20		32	
	20								
		SANDY LEAN CLAY (CL) very stiff, moist to wet, brown, low plasticity, fine sand	CL	34 50/6"	⧗	20	108		○
	25	**CLAYEY SAND (SC)** dense, moist to wet, brown to gray, fine to coarse sand	SC						
		SANDY LEAN CLAY (CL) very stiff, moist, brown, low plasticity, fine sand	CL						
		CLAYEY SAND (SC) dense, moist, brown, fine sand	SC	67	⧖	26			○
	30	*Continued Next Page*							

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 15.0 FEET

LA CORP.GDT 8/15/12 MV' JPT

TRC

EXPLORATORY BORING: EB-3 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: JPT
START DATE: 8-1-12 FINISH DATE: 8-1-12

PROJECT NO: 195053
PROJECT: 2071 Tice Valley Boulevard
LOCATION: Walnut Creek, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	1.0 2.0 3.0 4.0
	30		**SANDY LEAN CLAY (CL)** medium stiff, moist, brown, low plasticity, fine sand Bottom of boring at 30 feet	CL						
	35									
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 15.0 FEET

LA CORP.GDT 8/15/12 MV' JPT

TRC

EXPLORATORY BORING: EB-4

Sheet 1 of 2

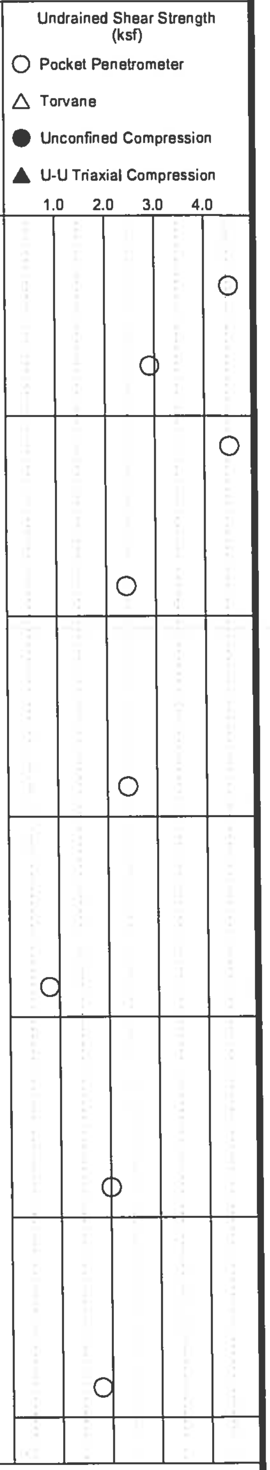
DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: JPT
START DATE: 8-1-12 FINISH DATE: 8-1-12

PROJECT NO: 195053
PROJECT: 2071 Tice Valley Boulevard
LOCATION: Walnut Creek, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual

SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0		**FAT CLAY (CH)** hard, moist, dark "brownish-gray", high plasticity	CH	30		17	96	
			LEAN CLAY (CL) very stiff, moist, dark brown, low to moderate plasticity	CL	28		21	102	
	5				35		19	106	
			SANDY LEAN CLAY (CL) hard, moist, brown, low plasticity, fine sand	CL					
	10		**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity	CL	24		31	92	
	15		dark brown, moderate plasticity		35		25	100	
			SANDY LEAN CLAY (CL) very stiff, moist, light brown, low plasticity, fine sand	CL					
	20		**LEAN CLAY (CL)** medium stiff, moist-wet, light brown, low plasticity	CL	36		23	101	
					26		31	91	
	25		**SANDY LEAN CLAY (CL)** very stiff, moist to wet, brown, low to moderate plasticity, fine sand	CL					
			CLAYEY SAND (SC) medium dense, moist to wet, gray, fine to coarse sand	SC					
	30		**SANDY LEAN CLAY (CL)** stiff, moist to wet, brownish-gray, low plasticity, fine to	CL	19		29		



Continued Next Page

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 13.5 FEET

LA CORP.GDT 8/13/12 MV' JPT

EXPLORATORY BORING: EB-4 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: JPT
START DATE: 8-1-12 FINISH DATE: 8-1-12

PROJECT NO: 195053
PROJECT: 2071 Tice Valley Boulevard
LOCATION: Walnut Creek, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression 1.0 2.0 3.0 4.0
	30		coarse sand Bottom of boring at 30 feet							
	35									
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:

∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 13.5 FEET

APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

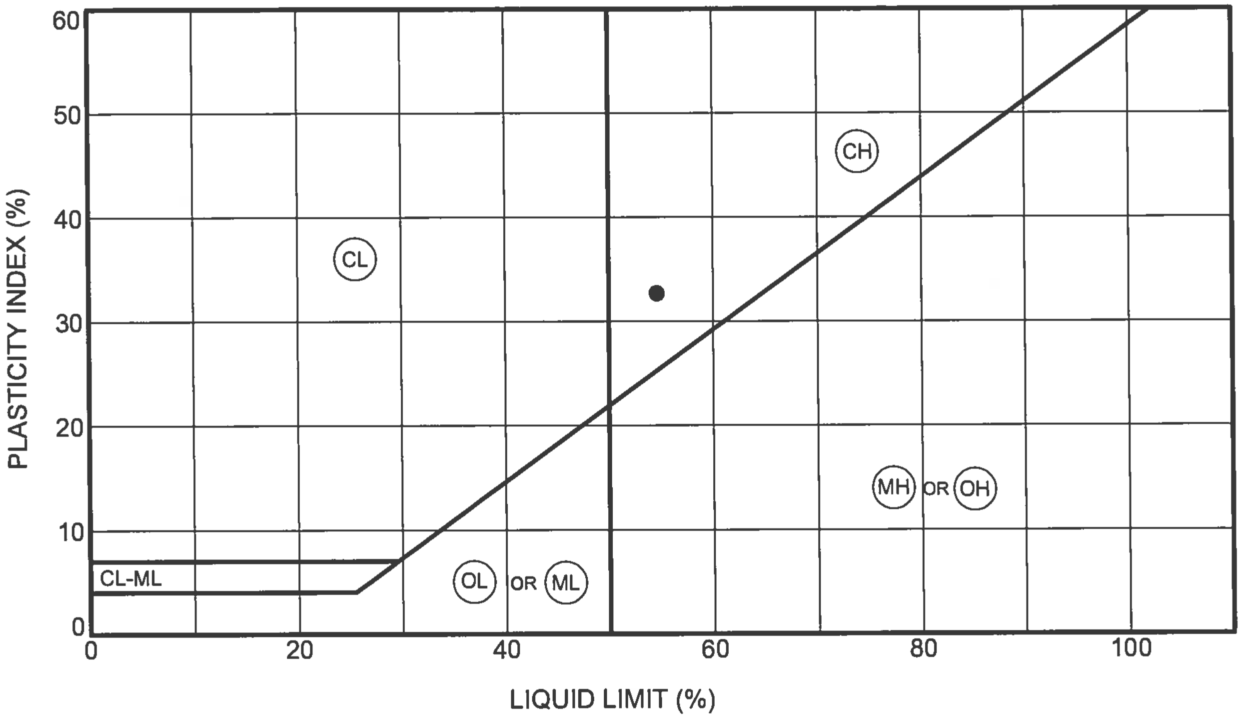
Moisture Content: The natural water content was measured (ASTM D2216) on 32 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on 26 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index (PI) test (ASTM D4318) was performed on a sample of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The PI was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on the Plasticity Chart of this appendix and on the log of the boring at the appropriate sample depth.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 2 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

* * * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-2**	**2.0**		**55**	**22**	**33**		

LA CORP.GDT 8/9/12 MV* JPT

TRC

PLASTICITY CHART AND DATA

Project: Tice Valley Boulevard

Location: Walnut Creek, CA

Project No.: 195053

FIGURE B-1

THIS PAGE INTENTIONALLY LEFT BLANK