Appendix E

Geotechnical Investigation

ESA

RIVERSIDE COUNTY PLANNING DEPARTMENT

Steven Weiss
Planning Director

April 2, 2015

Pages 2 (including this cover)

City of Temecula Planning Department
FAX: (951) 694-6477
Attention: Eric Jones

RE: Conditions of Approval GEO02437
City of Temecula Case No. PA14-0087
Villages at Paseo del Sol

County Geologic Report(s) GEO No.2437, submitted for this project (PA14-0087) in the City of Temecula was prepared by Converse Consultants and is entitled; "Updated Geotechnical Investigation Report for a Single Family Residential Development, Villages at Paseo del Sol, Tentative Tract No. 36483 Northwest of Temecula Pkwy and Butterfield Stage Road, City of Temecula, Riverside County, California", dated March 27, 2015.

In addition, Converse Consultants submitted the following:
"Response to Review Comments County Geologic Report No. GEO02437 City of Temecula Case No. PA14-0087 Villages at Paseo del Sol, Tentative Tract No. 36483 Northwest of Temecula Pkwy and Butterfield Stage Road, City of Temecula, Riverside County, California", dated May 20, 2015.

And "As-Built Geology and Compaction Report of Rough Grading Tracts 24182 through 24186 and 24188-1 Paseo Del Sol Master Planned Community Temecula, California", dated August 20, 1997

These documents are herein incorporated in GEO02437.

GEO02437 Concluded:

1. The project site is not located within a currently designated Riverside County or State of California Earthquake Fault Zone.
2. No major surface fault crosses through or extends towards the site.
3. The potential for surface rupture resulting from the movement of nearby major faults or currently unknown faults is not known with certainty but is considered low.
4. Site has a very high susceptibility to liquefaction.
5. Fill slopes less than thirty (30) feet in height are considered to have a low susceptibility to earthquake-induced failure.
6. The site is not considered to be at risk for lateral spreading.
7. Ground subsidence is expected to be negligible.
8. Tsunamis are not considered to be a risk.

The potential for flooding due to seiching is considered low.

GEO02437 Recommended:

1. Prior to the start of any earthwork, the site should be cleared of all vegetation and debris.
2. Existing soils are not considered suitable for the support of additional fills or structures and should be overexcavated and recompacted.

It should be noted that no engineering review of this report or formal review of provided building code information are a part of this review. Formal review of engineering design and code data will be made by the City of Temecula, as appropriate, at the time of grading and/or building permit submittal to the city.

Thank you for the opportunity to review this case for the City of Temecula. Please call me at (951) 955-6863 if you have any questions.

Sincerely,

RIVERSIDE COUNTY PLANNING DEPARTMENT
Steven Weiss, Planning Director

David L. Jones, CEG No. 2283
Chief Engineering Geologist, TLMA-Planning

cc: Converse Consultants, Fax: (909)796-7675
Applicant: Michael Rust/Cal-Paseo Del Sol, LLC, Fax: (858)622-2986

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UPDATED GEOTECHNICAL INVESTIGATION REPORT FOR A SINGLE-FAMILY RESIDENTIAL DEVELOPMENT

Villages at Paseo del Sol, Tentative Tract No. 36483
Northwest of Temecula Parkway and Butterfield Stage Road
City of Temecula, Riverside County, California
Converse Project No. 12-81-173-02

March 27, 2015

Prepared For:

Cal-Paseo Del Sol, LLC
9820 Towne Center Drive, Suite 100
San Diego, CA 92121

Prepared By:

Converse Consultants
10391 Corporate Drive
Redlands, California 92374

March 27, 2015

Mr. Michael Rust
Vice President
Cal-Paseo Del Sol, LLC
9820 Towne Center Drive, Suite 100
San Diego, CA 92121

Subject: **UPDATED GEOTECHNICAL INVESTIGATION REPORT FOR A SINGLE-FAMILY RESIDENTIAL DEVELOPMENT**
Villages at Paseo del Sol, Tentative Tract No. 36483
Northwest of Temecula Parkway and Butterfield Stage Road
City of Temecula, Riverside County, California
Converse Project No. 12-81-173-02

Dear Mr. Rust:

Converse Consultants (Converse) has prepared this updated geotechnical investigation report to present our findings, conclusions and recommendations for the Villages at Paseo del Sol project, located northwest of the intersection of Temecula Parkway and Butterfield Stage Road in the City of Temecula, Riverside County, California. This report has been updated to reflect the planned single-family residential development, current technical references, and the 2013 California Building Code seismic design requirements. This report was prepared in accordance with our proposal dated March 25, 2015 and your Professional Consulting Agreement No. 17225, effective March 27, 2015.

It is our opinion that the subject site is suitable from a geotechnical standpoint for the proposed development, provided the findings, conclusions, and recommendations presented in this report are incorporated in the preparation of the final grading plan, foundation design, and construction of the project.

It has been our pleasure to be of service on this project. If you should have any questions, please contact the undersigned at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., P.E., G. E.
Principal Engineer

Dist.: 3/Addressee (2-PDF & 1-CD)
SM/HSQ/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

Hashmi S. E. Quazi, Ph.D., P.E., G. E.
Principle Engineer

REGISTERED PROFESSIONAL ENGINEER
HASHMI S. E. QUAZI
NO. GE 2517
Exp. 6/15
GEOTECHNICAL
STATE OF CALIFORNIA

Scot Mathis, P.G., C.E.G.
Senior Geologist

REGISTERED GEOLOGIST
SCOT A. MATHIS
No. 2307
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in this report. Please refer to the pertinent section of the attached report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed 43-acre Paseo Del Sol site is located in the City of Temecula, Riverside County, California. The site is bounded to the north by De Portola Road, to the east by Butterfield Stage Road to the south by Temecula Parkway and to the west by Mantova Drive.

- The site contains three (3) superpads, with two pads on an upper level and one (1) on a lower level. The lower level is on the south of the site and the upper level is on the north. A fill slope, oriented in an east and west direction, separates the lower and upper levels. An existing drainage channel flows from the north central portion of the site, between the two (2) upper superpads, and across the lower superpad to a pond and outlet in the southwest corner of the site. The lower superpad is surrounded by an earthen berm up to approximately ten (10) feet in height.

- The project tentative tract map (RBF, 2015) indicates that the site will be reconfigured to approximately 173 single-family residential lots. The maximum cuts and fills are expected to be on the order of fifteen (15) to twenty (20) feet.

- The site was previously graded under the geotechnical observation and testing of Converse. At the time of grading the western portion of the site was identified as Tract 21482-5 and the eastern half as 21482-F. The geotechnical report of rough grading (Converse, 1997) was reviewed during the current investigation; however, the available copy of the report did not include the map indicating the lateral and vertical extent of remedial grading and fill placement. During the grading, surficial colluvium and alluvium in the northern portion of the site was excavated to expose dense older alluvium or Pauba Formation bedrock. Surficial alluvium and up to 15 feet of existing fills were excavated from the southern portion of the site to expose fine to medium grained sandy alluvium.

- Subsurface exploration consisted of drilling twelve (12) exploratory borings (BH-1 through BH-12) within the project site on May 30, 2012. Borings BH-1 and BH-11 were drilled to the planned depth of 51.5 feet bgs. Borings BH-2 through BH-10, and BH-12 were drilled to the planned depth of 16.5 feet below ground surface (bgs).

- The project site is located within the Peninsular Ranges Geomorphic Province of Southern California. The site is located on the northern margin of the west-trending Pauba Valley. The Pauba Valley is a broad alluvial valley eroded into the Pleistocene Pauba Formation by the Temecula Creek. Regional mapping (Kennedy, 2000) indicates that, except for a small area in the northwestern corner, the site is underlain by Holocene alluvial deposits. The alluvium generally consists of unconsolidated to poorly consolidated sand and gravel deposits. Previous grading has placed fill soil over the site derived from alluvium, colluvium, older alluvium, or the Pauba Formation.

- The two (2) northern, upper superpads are underlain by approximately fifteen (15) feet of fill soils on the east to approximately thiry-five (35) feet of fill on the west. The fill soils generally consist of silty sand with occasional zones of clayey sand and sand with silt. The fill is generally dense; however, variations in the fill density were encountered. The fill soils are underlain by native alluvial soils. The alluvium primarily consists of silty sand, but includes layers of sand, silt, and clay mixtures. The alluvial soils are generally medium dense to dense.

- The southern superpad is underlain by fill soil consisting of silty sand with various layers of sand, silt, and clay mixtures. The transition from fill soil to the underlying alluvial soils is not readily apparent in the borings. The presence of organics at 15.0 feet bgs is consistent with the geotechnical report documenting the previous site grading (Converse, 1997), which indicates that five (5) to fifteen (15) feet of previously existing fill soils were excavated and replaced with compacted fill. The berms surrounding the lower superpad are constructed of compacted fill.

- Groundwater was encountered during the field investigation at approximately 39.5 feet bgs (approximately 1,058.5 feet amsl) in boring BH-11. Groundwater was not encountered in the other borings. We have estimated a historical high groundwater level ranging from 1,080 feet amsl at the southwestern corner of the site to 1,100 feet amsl at the eastern site perimeter.

- The surface and subsurface soil materials for the proposed development are expected to be excavatable by conventional heavy-duty earth moving equipment.

- Results of the laboratory tests indicated that the site soils have 'slight' collapse potential, 'very low' to 'low' expansion potential and moderate shear strength. Results of corrosion tests indicate that the site soils have 'negligible' exposure to sulfate attack, and are 'moderately corrosive' to 'corrosive' to ferrous metals in contact with the soils.

- The site is not located within a currently designated Riverside County or State of California Earthquake Fault Zone. Based on a review of available geologic information no known active surface fault zone crosses or projects toward the site. CBC seismic parameters for the site are presented in the text of this report.

- Site-specific liquefaction analyses were performed using data obtained from borings BH-1 and BH-11. Groundwater depths at 40.0 feet bgs in boring BH-1 and 15.0 feet bgs in boring BH-11 have been used for the purpose of liquefaction analysis. The sediments encountered in BH-1 were not found to be susceptible to liquefaction. Boring BH-11, drilled in the lower superpad, encountered a medium dense sand layer between approximately 20.0 and 25.0 feet bgs in (1,073 to 1,078 feet amsl). This layer may be susceptible to liquefaction if subjected to ground shaking of sufficient intensity while saturated. Up to approximately 0.80 inches of liquefaction-induced settlement may occur. Surface manifestations of liquefaction, such as sand boils, are not anticipated due to the greater thickness of the overlying non-liquefiable layer in comparison to the relatively thin liquefiable layer.

- The existing superpads have irregular surface, with numerous soil piles, and are locally loose, dry, or disturbed. Such soils are not considered suitable for the support of additional fills or structures, and should be overexcavated and recompacted. The depth and limits of the overexcavation should be determined by the geotechnical consultant based on the field conditions encountered. In general, the existing grade should be overexcavated to expose a fill with a minimum density of ninety (90) percent, or an alluvium with a minimum density of eighty-five (85) percent of laboratory maximum density, or, to a depth of three (3) feet below existing grade.

- Building footings and slabs-on-grade should be uniformly supported by compacted fill. If after the planned lowering of the northern upper superpads, uncompacted alluvium soil is encountered, such alluvium should be overexcavated and recompacted to a minimum depth of three (3) feet below the bottom of the footing elevation. Such overexcavation and recompaction should extend at least five (5) feet beyond the building footprint. For sub-grade below planned asphalt concrete or Portland concrete paving, including driveways, access roads, sidewalks, street areas, curbs and gutters or other flatwork, overexcavation and recompaction in alluvium soils should be to a depth of at least two (2) feet, extending at least two (2) feet beyond the pavement or flatwork.

- All areas to receive fill should be overexcavated to the depth where the existing soils have a relative compaction of at least ninety (90) percent of the laboratory maximum dry density, or to a depth of 3.0 feet bgs. Any loose, soft, and unsuitable materials encountered during overexcavation should be removed from the site. The depth of such removal should be determined during the grading by the geotechnical consultant. After the required overexcavation as recommended in this report, or as recommended by the geotechnical consultant during remedial grading is performed, all surfaces to receive fill should be scarified to an additional depth of six inches.

- Onsite soils cleared of all debris, vegetation, rocks larger than three (3) inches, and other deleterious materials may be re-used as compacted fill. Rocks larger than one (1) inch in the largest dimension should not be placed within the upper twelve (12) inches of fill beneath footings and slabs or the upper eighteen (18) inches of fill under paved areas.

- Fill soils should be evenly spread in horizontal, 8-inch-maximum, loose lifts. The fill materials should be thoroughly mixed and moisture conditioned to within three (3) percent of optimum moisture content for granular soils and up to two (2) percent above optimum moisture content for fine-grained soils. All fill placed at the site should be compacted to at least ninety (90) percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The upper twelve (12) inches of soil below footings and slabs, and the upper twelve (12) inches of sub-grade soils underneath concrete aprons and asphalt concrete, should be compacted to at least ninety-five (95) percent of laboratory maximum dry density.

- The proposed single-family residential structures may be supported on continuous (strip) and/or isolated spread footings. Continuous and isolated spread footings should be at least eighteen (18) inches wide. The depth of embedment below lowest adjacent soil grade of interior and exterior footings should be at least eighteen (18) inches. Such footings can be designed based on an allowable net bearing capacity of 2,500 pounds per square foot (psf), plus 300 psf for each additional foot of depth and 150 psf for each foot of width. The maximum allowable bearing capacity should be limited to 3,000 psf.

- Resistance to lateral loads can be assumed to be provided by combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil and of 0.25 between soil and steel may be used with the dead load forces. Passive earth pressure of 270 psf per foot of depth may be used for the sides of footings poured against recompacted native soils. The maximum value of the passive earth pressure should be limited to 2,500 psf.

- The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. Dynamic settlement under seismic conditions is estimated to be up to 0.8 inch. The static and dynamic differential settlements can be taken as equal to one-half of the corresponding total settlement over a lateral distance of fifty (50) feet.

- Preliminary pavement design recommendations are presented in the text of this report. Due to mixing and movement during grading, the R-values of the final sub-grade soil are likely to be different than tested. We recommend performing additional R-value tests at the completion of the sub-grade preparation, which could change the proposed preliminary pavement sections.

The site is suitable from a geotechnical standpoint for the proposed development, provided that the recommendations presented in this report are incorporated into the design and construction of the project.

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1.0 INTRODUCTION

This report contains the findings of our geotechnical investigation performed for the proposed development of a 43-acre site, located northwest of the intersection of Temecula parkway and Butterfield Stage Road in the City of Temecula, Riverside County, California. The site location is shown in Figure No. 1, *Approximate Site Location Map.*

Our original investigation was based on proposed commercial and multi-family residential development (Converse, 2012). This report has been updated to reflect single-family residential development, current references, and the 2013 California Building Code seismic design requirements. No additional investigation or testing was performed.

The purpose of this investigation was to evaluate the nature and pertinent engineering properties of the subsurface materials and to provide geotechnical parameters for the design and construction of the proposed project.

This report is written for the project described herein and is intended for use solely by Cal-Paseo Del Sol, LLC and its design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

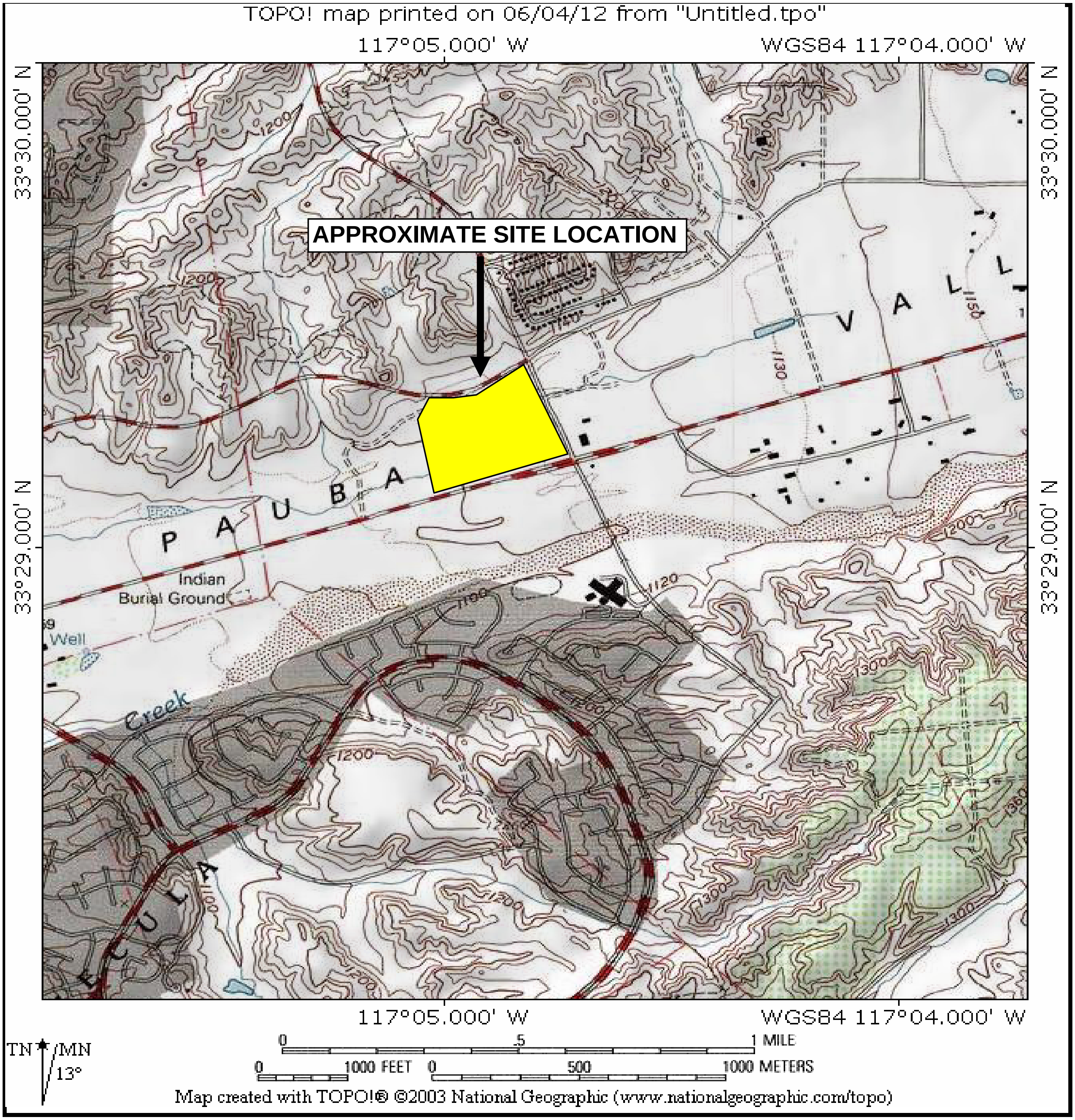
2.0 SITE DESCRIPTION

The site is located in the City of Temecula, Riverside County, California. The site is bounded to the north by De Portola Road, to the east by Butterfield Stage Road to the south by Temecula Parkway and to the west by Mantova Drive.

The site contains three superpads, with two (2) pads on an upper level on the north side of the site and one (1) on a lower level on the south side. A fill slope, oriented in an east and west direction, separates the lower and upper levels. An existing drainage channel flows from the north central portion of the site, between the two (2) upper superpads, and across the lower superpad to a pond and outlet in the southwest corner of the site. The lower superpad is surrounded by an earthen berm up to approximately ten (10) feet in height.

The topography of the northern half of the site is mostly flat, with a few rises and depressions. The slope in this area is towards the south. The southern half of the site is mostly flat, with a slight slope towards the southwest corner of the site.

Scattered concrete and asphalt debris were observed in the northern half of the site, along with several areas covered by soil piles. Animal burrows were present, particularly in the northern portion of the site. Vegetation, primarily consisting of low grass, covers



APPROXIMATE SITE LOCATION MAP

PLANNING AREA 4- APPROXIMATELY 43 ACRE SITE

City of Temecula, County of Riverside, California

For: Cal-Paseo Del Sol, LLC

Project Number:

12-81-173-01

Figure No.:

1

Converse Consultants

the entire site. General views of the project site are shown on the following photographs.

Photo 1, View to west along slope between the upper and lower levels

Photo 2, Soil piles in the northern half of the project site.

Photo 3, View to northwest from the southeast corner of the project site.

3.0 PROJECT DESCRIPTION

The project tentative tract map (RBF, 2015) indicates that the site will be reconfigured to approximately 173 single-family residential lots. The maximum cuts and fills are expected to be on the order of fifteen (15) to twenty (20) feet.

We anticipate that the residences will be one- and two-story wood-frame and stucco structures founded on continuous and/or isolated footing foundations with slab-on-grade. The vertical loads on continuous and isolated footing foundations are anticipated to be less than 2,000 pounds per linear foot and 50,000 pounds, respectively.

A west-flowing drainage channel is proposed along the southern site perimeter and a south-flowing drainage channel is proposed in the center of the site. The channels will end in a water quality basin at the southwestern corner of the site. We anticipate that the development will also include roadways, landscape areas, above-ground and underground utilities, and other improvements typically associated with residential developments.

4.0 PREVIOUS SITE GRADING

The site was previously graded under the geotechnical observation and testing of Converse. At the time of grading the western portion of the site was identified as Tract 21482-5 and the eastern half as 21482-F. The geotechnical report of rough grading (Converse, 1997) was reviewed during the current investigation; however, the available copy of the report did not include the map indicating the lateral and vertical extent of remedial grading and fill placement.

Site conditions during and following the previous grading are discussed in the following sections.

4.1 Remedial Grading

Remedial grading was conducted during the previous site grading. Surficial colluvium and alluvium in the northern portion of the site was excavated to expose dense older alluvium or Pauba Formation bedrock. Surficial alluvium and up to approximately fifteen (15) feet of existing fills were excavated from the southern portion of the site to expose fine to medium grained sandy alluvium.

4.2 Compacted Fill

Excavated site soils were placed as compacted fill. The fills were compacted to at least ninety (90) percent of the laboratory maximum dry density. Where structural fill was greater than twenty (20) feet thick, fill soils placed below twenty (20) feet from the proposed ground surface were compacted to at least ninety-five (95) percent of the laboratory maximum dry density.

4.3 Compacted Fill Thickness

The copy of the geotechnical report of rough grading (Converse, 1997) reviewed by Converse during this investigation did not include the maps showing the lateral and vertical extent of fill placement. The fill thickness was evaluated based on historical and current site topography.

Historical topographic mapping (USGS, 1986) shows that the site was relatively flat with a gentle slope to the west-southwest. Two streams flowed through the site from east to west. The site elevations ranged from approximately 1,090 feet above mean sea level (amsl) at the southwestern corner to approximately 1,110 feet amsl along the western perimeter.

As described in Section 2.0, *Site Description*, the site currently contains three superpads at two elevations. A recent topographic map (Cal-Paseo del Sol, 2012a) indicates that the southern, lower superpad is at an elevations ranging from approximately 1,090 feet amsl in the southwestern corner to approximately 1,110 feet amsl in the northeastern corner. These elevations approximate the original elevations on the historical topographic map.

The northwestern superpad has a somewhat irregular upper surface, with elevations ranging from approximately 1,140 feet amsl in the northwestern corner to approximately 1,120 feet amsl in the southeastern corner. The northeastern superpad also has an irregular surface, with numerous soil piles. Elevations range from approximately 1,130 feet near the center of the pad to 1,125 feet amsl along the eastern perimeter.

Based on our review of topographic maps and aerial photographs, up to approximately fifteen (15) to thirty-five (35) feet of fill was placed to create the two upper superpads. The lower superpad is at near the original, pre-grading elevation; however, based on the geotechnical report of rough grading, portions of the area may contain up to fifteen (15) feet of compacted fill.

4.4 Settlement Monuments

At the completion of the previous grading, settlement monuments were installed at the top and bottom of the compacted fill. Ten monuments were installed within Tract 24182, which included the current site. The location of the monuments is not known. The geotechnical report of rough grading (Converse, 1997) stated that ongoing monitoring of the monuments indicated a significant rate of ground settlement at the time of the report.

4.5 Post-Grading Site Conditions

Historical aerial photographs from 1953 to present (EDR, 2012; Google Earth, 2011) were reviewed to evaluate the site conditions following the site grading. The site was graded to approximately its present configuration between 1996 and 2002, which is consistent with the geotechnical report (Converse, 1997). Several construction office trailers were present in a fenced compound at the eastern end of the northeastern superpad from 2002 to 2007. During that time, numerous soil piles were placed on the upper superpads. The soil was either removed or spread on the site in 2007 or 2008. The site has remained essentially unchanged from 2009 to present.

5.0 SCOPE OF WORK

The scope of our previous investigation (Converse, 2012) and this update included the tasks described in the following sections.

5.1 Project Set-up

The project set-up consisted of the following:

- Conducted a site reconnaissance and stake/marked the exploration locations, such that drill rig access to all the locations is available
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities
- Arranged for a drill rig.

5.2 Subsurface Exploration

Subsurface exploration consisted of drilling twelve (12) exploratory borings (BH-1 through BH-12) within the site on May 30, 2012. Borings BH-1 and BH-11 were drilled to the planned depth of 51.5 feet bgs. Borings BH-2 through BH-10, and BH-12 were drilled to the planned depth of 16.5 feet below ground surface (bgs).

The exploratory borings were advanced using a truck mounted drill rig equipped with 8-inch diameter hollow-stem augers and a drive sampler for soil sampling. Standard penetration tests (SPT) were conducted in boring BH-1 and BH-11 starting at 20.0 feet bgs at ten (10) foot intervals. Encountered earth materials were continuously logged by a Converse geologist and classified in the field by visual examination in accordance with the Unified Soil Classification System.

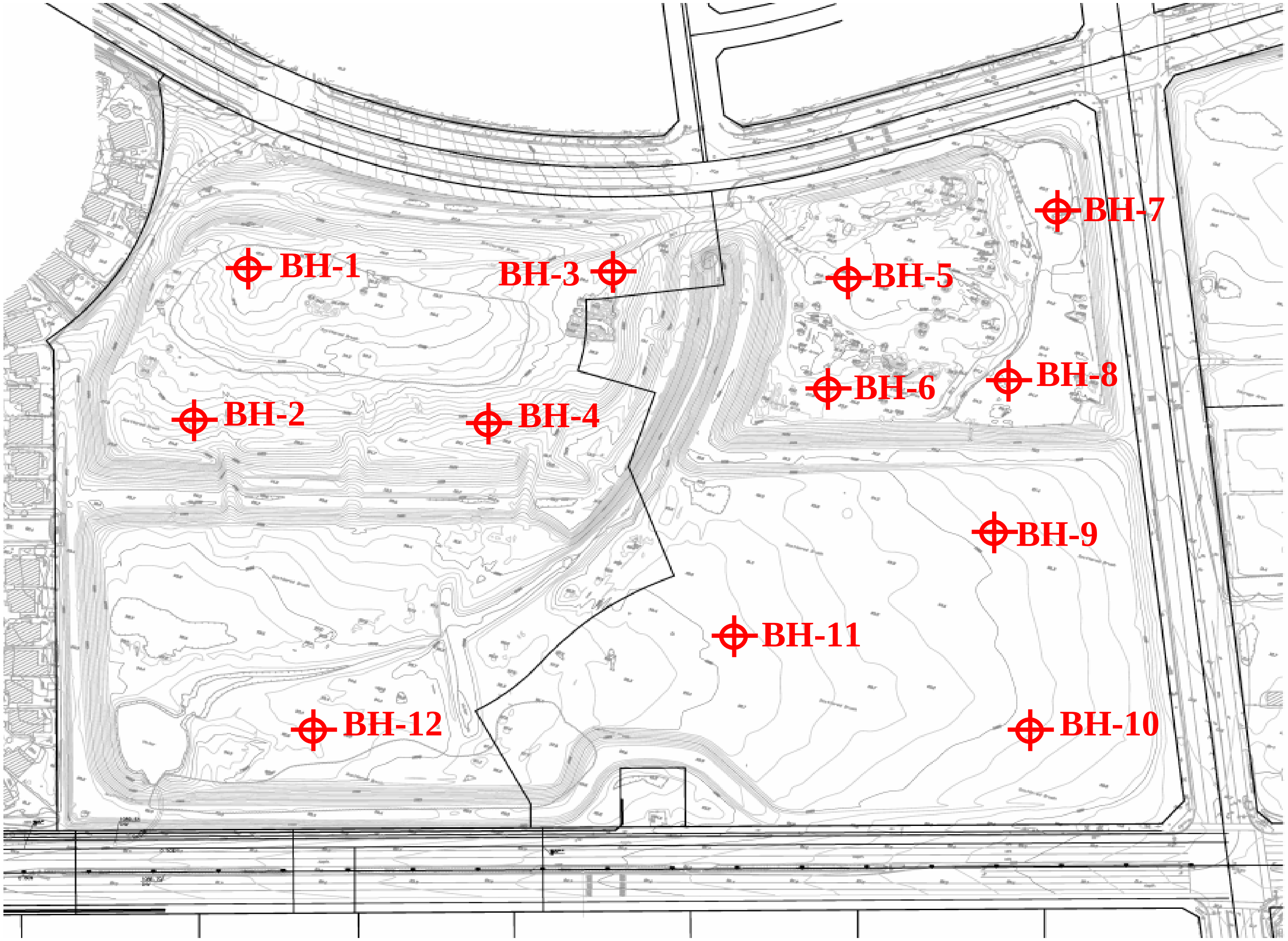
The approximate locations of the borings are presented on Figure No. 2, *Approximate Boring Location Map.* For a description of the field exploration and sampling program see Appendix A, *Field Exploration*.

5.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in the classification and to evaluate relevant engineering properties. The tests performed included:

- *In situ* Moisture Contents and Dry Densities (ASTM Standard D2216)
- Collapse Potential (ASTM Standard D5333)
- Expansion Potential (ASTM Standard D4829)
- Soil Corrosivity Tests (California Tests CT643, 422 and 417)
- R-Value (ASTM D2844)
- Percent Passing Sieve No. 200 (ASTM Standard D1140)
- Grain Size Distribution (ASTM Standard C136)
- Maximum Dry Density and Optimum Moisture Content Relationship (ASTM Standard D1557)
- Direct Shear Strength (ASTM Standard D3080)

For *in situ* moisture and density data, see the Logs of Borings in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.



APPROXIMATE BORING LOCATION MAP

Explanation	PLANNING AREA 4- APPROXIMATELY 43 ACRE SITE	Project Number 12-81-173-01
Approximate Boring Location	City of Temecula, County of Riverside, California	Scale NTS
	For: Cal-Paseo Del Sol, LLC	Date August 2012
	Converse Consultants	Figure No. 2

5.4 Analyses and Report

Data obtained from the exploratory field work and laboratory testing program, as well as the documents, reports, and maps presented in the Section 15.0, *References* were analyzed. A report (Converse, 2012) was prepared to provide findings, conclusions and recommendations developed during our investigation. The report was revised to reflect the currently planned single-family residential development, 2013 California Building Code, and other recent references.

6.0 GEOLOGIC SETTING

The regional and local geology are discussed in the following subsections.

6.1 Regional Geology

The project site is located within the Peninsular Ranges Geomorphic Province of Southern California.

The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the southwest by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Cucamonga, and San Andreas Fault Zones, all of which have been known to be active during Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

6.2 Site Geology

The site is located on the northern margin of the west-trending Pauba Valley. The Pauba Valley is a broad alluvial valley eroded into the Pleistocene Pauba Formation by the Temecula Creek.

Regional mapping (Kennedy, 2000) indicates that, except for a small area in the northwestern corner, the site is underlain by Holocene alluvial deposits. The alluvium generally consists of unconsolidated to poorly consolidated sand and gravel deposits. The surficial deposits at the site are underlain by the Pleistocene Pauba Formation, which is generally well-indurated sandstone and siltstone with occasional gravel, cobble, and boulder deposits. The mapping shows that the Pauba Formation is exposed in the northwestern corner of the site.

Previous grading, discussed in Section 4.0, *Previous Site Grading*, has placed fill soil over the site. Based on the previous geotechnical report (Converse, 1997), the fill soil was derived from alluvium, colluvium, older alluvium, or the Pauba Formation.

7.0 SITE CONDITIONS

A general description of the subsurface conditions and various materials encountered during our field exploration are presented in this section.

7.1 Subsurface Profile

The two northern, upper superpads are underlain by approximately fifteen (15) feet of fill soils on the east to approximately thirty-five (35) feet of fill on the west. The fill soils generally consist of silty sand with occasional zones of clayey sand and sand with silt. The fill is generally dense; however, variations in the fill density were encountered. The fill soils are underlain by native alluvial soils. The alluvium primarily consists of silty sand, but includes layers various sand, silt, and clay mixtures. The alluvial soils are generally medium dense to dense.

The southern superpad is underlain by fill soil consisting of silty sand with various layers of sand, silt, and clay mixtures. The transition from fill soil to the underlying alluvial soils is not readily apparent in the borings. Visible organic content was noted at approximately fifteen 15.0 feet bgs in borings BH-8 and BH-11. Concentrations of organic matter are often, but not always, associated with shallow soils. The presence of organics at fifteen 15.0 feet bgs is consistent with the geotechnical report documenting the previous site grading (Converse, 1997), which indicates that five (5) to fifteen (15) feet of previously existing fill soils were excavated and replaced with compacted fill. The berms surrounding the lower superpad are constructed of compacted fill.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawing Nos. A-2 through A-7, *Logs of Borings,* in Appendix A, *Field Exploration.*

7.2 Groundwater

Groundwater was encountered during the field investigation at approximately 39.5 feet bgs in boring BH-11. Groundwater was not encountered in the other borings. During the previous site grading, small amounts of perched groundwater were encountered during alluvial removals in canyon areas to the west of the site (Converse, 1997).

Surface water was observed in the onsite drainage channel and pond during our field exploration. Vigorous vegetation growth along the channel currently and in the historical aerial photographs indicates that the area normally contains water. Surface water should be expected to infiltrate into the subsurface, resulting in locally saturated soil and perched groundwater.

Shallow perched groundwater may be encountered during construction in the channel and pond areas. Standing groundwater is not expected to be encountered. The depth to groundwater may vary due to seasonal precipitation, groundwater pumping activity, or other factors.

Regional groundwater data (USGS, 2015) was reviewed to evaluate the historical high groundwater level at the site. A well (USGS Well No. 332908117045501) is located at the southern perimeter of the site. The well was measured only one time, in 1967, and reported a groundwater elevation of 1,078 feet amsl, or approximately fourteen (14) feet below a reported ground surface elevation of 1,092 feet amsl. Although only one historical data point was available for the well, that point correlated well with the highest groundwater levels reported in other wells in the Pauba Valley.

Due to the lack of multiple data points in the historical groundwater elevation record, it is our opinion that a conservative approach to establishing the historical high groundwater level is warranted. The highest historical groundwater elevation reported was 14.0 feet bgs. The actual historical high groundwater level is not expected to have been higher than ten (10) feet below the original, pre-grading ground surface. Therefore, we have estimated a historical high groundwater level ranging from 1,080 feet amsl at the southwestern corner of the site to 1,100 feet amsl at the eastern site perimeter.

7.3 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions presented in this report.

7.4 Excavatability

The surface and subsurface soil materials for the proposed development are expected to be excavatable by conventional heavy-duty earth moving equipment.

Although oversized materials were not encountered during drilling, we anticipate that scattered cobbles or small boulders may be encountered during excavation for the proposed developments.

The phrase "conventional heavy-duty excavation equipment" is intended to include commonly used equipment such as excavators, scrapers, and trenching machines. It does not include hydraulic hammers ("breakers"), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment models should be done by an experienced earthwork contractor.

7.5 Flooding

The site is not located within a flood zone identified by Riverside County. A flood zone associated with the Temecula Creek is located approximately 500 feet south of the site. (Riverside County, 2015) Due to the elevation of the site, flooding at Temecula Creek is not anticipated to impact the site.

8.0 LABORATORY TEST RESULTS

8.1 Physical Testing

Results of the various laboratory tests are presented in Appendix B, *Laboratory Testing Program,* except for the results of *in situ* moisture and dry density tests which are presented on the Logs of Borings in Appendix A, *Field Exploration.* The results are also discussed below:

- *In situ* Moisture and Dry Density – *In situ* dry density and moisture content of the upper ten (10) feet of soils for the project site ranged between 98 and 126 pounds per cubic feet (pcf) and between one (1) and twenty-six (26) percent, respectively.
- Collapse Potential – The collapse potential of two (2) relatively undisturbed samples were tested under a vertical stress of 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D2435/D5333 test method. The samples collected in the upper five (5) feet measured collapse of 0.4 to 1.6 percent, indicating 'Slight' collapse potential.
- Expansion Index – Two (2) representative soil samples were tested to evaluate Expansion Index (EI) in accordance with the ASTM Standard D4829. The measured EI values were eighteen (18) and thirty-three (33), which correspond to 'very low' and 'low' expansion potential.
- R-Value – Three (3) representative soil samples from the upper five (5) feet of the project site soil were tested to evaluate the R-value in accordance with the ASTM Standard D2844. The result of the R-value test will be used for preliminary pavement section recommendations. The results indicated R-values ranging from seventeen (17) to fifty-two (52).
- Percent Passing Sieve No. 200 – Two (2) representative soil samples were tested to determine the percent finer than sieve No. 200 to aid in the classification of the on-

site soils in accordance to ASTM Standard D1140. Test results indicated the soils as Silty Sand.

- Gradation Analysis – Grain size analyses were performed on five (5) representative soil samples according to ASTM Standard D422 for particles larger than seventy-five (75) micrometers. The results are presented in Drawings No. B-1a and B-1b, *Grain Size Distribution Results*, in Appendix B, *Laboratory Testing Program*.
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship of three (3) representative soil samples were tested in accordance to ASTM Standard D1557. The maximum dry density of the tested soil ranged from 122.1 to 133.5 pounds per cubic foot (pcf) and optimum moisture content ranged from 4.5 to 8.5 percent. Test results are shown on Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*.
- Direct Shear - Direct shear tests were performed on three (3) relatively undisturbed ring samples in according to ASTM Standard D3080. Results of the direct shear tests are presented in Drawings No. B-3 through B-5, *Direct Shear Test Results,* in Appendix B, *Laboratory Testing Program*. Results of direct shear test indicate that the soil tested had moderate shear strength.

For additional information on the subsurface conditions, see the Logs of Borings in Appendix A, *Field Exploration*.

8.2 Chemical Testing - Corrosivity Evaluation

Two (2) selected soil sample was tested by HDR/Schiff for corrosivity evaluation with respect to common construction materials such as concrete and steel. Tests were performed for pH, sulfate and chloride content, and saturated minimum electrical resistivity in accordance with California Test Methods 643, 422 and 417. The test results are summarized below and are presented in Appendix B, Laboratory Testing Program.

- The sulfate contents of the samples tested were 26 and 229 mg/kg.
- The pH measurements of the samples were 8.0 and 8.3.
- The chloride concentrations of the samples tested were 7 and 33 mg/kg.
- The minimum electrical resistivities when saturated were 1,500 and 2,964 ohm-cm.

9.0 FAULTING AND SEISMICITY

9.1 Faulting

The site is not located within a currently designated Riverside County or State of California Earthquake Fault Zone. Based on a review of available geologic information no known active surface fault zone crosses or projects toward the site. (CGS, 2007; Riverside County, 2015)

Several positive lines of evidence can be drawn to support the conclusion that the site does not contain active faulting. The evidence is based on the following documentation.

- Converse reviewed Riverside County fault mapping (Riverside County, 2015), which showed that the closest County and State fault zones to the site are associated with the Elsinore Fault Zone, located approximately 1.7 miles to the southwest of the site. The faults in the Elsinore Fault Zone trend northwest and do not project toward the site.
- A northeast-trending County fault zone is located approximately 1.7 miles southeast of the site and does not project toward the site.
- An approximately 0.5-mile-long, north-northwest-trending fault is mapped approximately 1.2 miles northwest of the site, projecting approximately 0.4 miles to the west of the site. The fault is not included in any County or State fault zone, indicating that it is not known to be active.
- The "Geologic Map of the Pechanga 7.5' Quadrangle, San Diego and Riverside Counties, California" (Kennedy, 2000) shows that the closest fault to the site is the Wildomar Fault, located approximately 1.7 miles southwest of the site. The Wildomar Fault and the other faults in the Elsinore Fault Zone trend northwest-southeast and do not project toward the site. Other faults are mapped over a mile to the south and east of the site. These faults generally trend northeast-southwest, and do not project toward the site.
- The map of "State of California Special Studies Zones, Pechanga Quadrangle" (CGS, 1990) indicates that the only active faults known to exist in the vicinity of the site are associated with the Elsinore Fault Zone. The closest of these faults to the site is the Wildomar Fault, located approximately 1.7 miles southwest of the site. The faults in the Elsinore Fault Zone trend northwest-southeast and do not project towards the site. There is no corresponding map of the Bachelor Mountain 7.5' quadrangle, located north of the site, indicating that the State has not established any Earthquake Fault Zones (formerly Special Studies Zones) in that area.
- Converse observed geologic conditions during grading of the site in 1996 and 1997, and stated that "Evidence of active or potentially active faulting was not observed during grading of the site. Bedrock faulting with significant offset or deformation was not observed within any of the grading areas." (Converse, 1997).

Based on the above-discussed data, the following positive lines of evidence support our conclusion that no known active surface fault crosses or projects toward the site.

- All faults zoned as active by the State of California or County of Riverside are located at least 1.7 miles from the site, and do not project toward the site.
- An approximately 0.5-mile-long fault mapped as terminating approximately 1.2 miles northwest of the site projects approximately 0.4 miles to the west of the site.
- The small fault north of the site is not zoned as active by the State or the County.
- Converse observed apparently unfaulted earth materials in the subsurface during site grading in 1996 and 1997.

The proposed site is situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site.

The following table contains a list of active and potentially active faults within 100 kilometers of the subject site. The fault parameters and distances presented in the following table are based on the output from EQFAULT (Blake, 2000), revised in accordance with CGS fault parameters (Cao et. al., 2003).

Table No. 1, Seismic Characteristics of Nearby Active Faults

Fault Name	Approximate Distance (km)	Moment Magnitude (Mw)
Elsinore-Temecula	4.4	6.8
Elsinore-Julian	13.8	7.1
Elsinore-Glen Ivy	30.2	6.8
San Jacinto-Anza	31.4	7.2
San Jacinto-San Jacinto Valley	32.0	6.9
Newport-Inglewood (Offshore)	48.1	6.9
Rose Canyon	50.9	7.2
San Jacinto – Coyote Creek	53.3	6.8
Earthquake Valley	57.5	6.5
Chino-Central Ave. (Elsinore)	58.9	6.7
San Jacinto-San Bernardino	60.7	6.7
San Andreas-Southern	60.9	7.4
San Andreas-San Bernardino	60.9	7.5
Whittier	65.7	6.8
Pinto Mountain	71.7	7.2
San Andreas-Coachella	74.6	7.2
Coronado Bank	75.8	7.6
Newport-Inglewood (L.A. Basin)	79.3	7.1
Palos Verdes	82.7	7.3
Burnt Mountain	82.9	6.5
Cucamonga	84.1	6.9
North Frontal Fault Zone (West)	84.8	7.2
North Frontal Fault Zone (East)	86.6	6.7
Eureka Peak	87.5	6.4

Fault Name	Approximate Distance (km)	Moment Magnitude (Mw)
Elysian Park Thrust	87.6	6.7
San Jacinto-Borrego	88.3	6.6
Elsinore-Coyote Mountain	88.5	6.8
Cleghorn	89.1	6.5
San Jose	89.6	6.4
Compton Thrust	90.2	6.8
Sierra Madre	93.4	7.2
Landers	97.3	7.3

9.2 Seismic Design Coefficients

Seismic parameters based on the 2013 California Building Code (CBSC, 2013), ASCE 7 (ASCE, 2010) and site coordinates 33.4870° north latitude and 117.0820° west longitude are provided in the following table.

Table No. 2, CBC Seismic Parameters

Parameter	Value
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.829g
Mapped 1-second Spectral Response Acceleration, S_1	0.734g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.829g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.102g
Design Spectral Response Acceleration for short period S_{DS}	1.219g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.734g

9.3 Secondary Effects of Seismic Activity

In general, secondary effects of seismic activity include surface fault rupture, soil liquefaction, landslides, lateral spreading, and differential settlement due to seismic shaking, tsunamis, seiches, and earthquake-induced flooding. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: The project site is not located within a currently designated Riverside County or State of California Earthquake Fault Zone (CGS, 2007; Riverside County, 2015). Based on review of available geologic information, no major surface fault crosses through or extends towards the site. The potential for surface rupture resulting from the movement of nearby major faults, or currently unknown faults, is not known with certainty but is considered low.

Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper fifty (50) feet of the ground surface, suffers a substantial reduction in its shear strength, due the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows:

- Soils must be submerged
- Soils must be primarily granular
- Soils must be loose to medium-dense
- Ground motion must be intense
- Duration of shaking must be sufficient for the soils to lose shear resistance

Regional hazard maps (Riverside County, 2015) indicate that the northern portion of the site has a moderate susceptibility to liquefaction and the southern portion of the site has a very high susceptibility to liquefaction.

We have estimated a historical high groundwater level ranging from 1,080 feet amsl at the southwestern corner of the site to 1,100 feet amsl at the eastern site perimeter. Based on historical regional data, groundwater may be anticipated in the future at depths as shallow as approximately 15.0 feet bgs in boring BH-11, or an elevation of 1,087 feet amsl. Groundwater is currently present at approximately 39.5 feet below the lower superpad ground surface in boring BH-11.

Site-specific liquefaction analyses were performed using data obtained from borings BH-1 and BH-11 (Converse, 2012). Groundwater depths at 40.0 feet bgs in boring BH-1, and 15 feet bgs in boring BH-11 were used for the purpose of liquefaction analysis. A detailed discussion of the analyses is presented in Appendix C, *Liquefaction and Settlement Analyses*.

The sediments encountered in BH-1 were not found to be susceptible to liquefaction. Boring BH-11, drilled in the lower superpad, encountered a medium dense sand layer between approximately 20.0 and 25.0 feet bgs in (1,073 to 1,078 feet amsl). This layer may be susceptible to liquefaction if subjected to ground shaking of sufficient intensity while saturated. Up to approximately 0.80 inches of liquefaction-induced settlement may occur.

Surface manifestations of liquefaction, such as sand boils, are not anticipated due to the greater thickness of the overlying non-liquefiable layer in comparison to the relatively thin liquefiable layer.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes.

The site currently includes 2H:1V (horizontal:vertical) fill slopes up to approximately twenty-five (25) feet in height. In general, properly constructed 2H:1V fill slopes less than thirty (30) feet in height are considered to have a low susceptibility to earthquake-induced failure.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved.

Due to the limited potential of the site for liquefaction, and the depth of the potentially liquefiable layer, the site is not considered to be at risk for lateral spreading.

Differential Settlement Due to Seismic Shaking: Differential settlement may occur when relatively low-density, medium- or coarse-grained sands are densified by intense seismic shaking. Based our site-specific analysis presented in Appendix C, *Liquefaction and Settlement Analyses*, seismic shaking may result in up to approximately 0.51 inches of settlement in the vicinity of BH-1 and the up to approximately 0.80 inches of settlement in the vicinity of BH-11. The potential for differential settlement may be estimated as up to half of the total settlement over a lateral distance of fifty (50) feet.

Tsunamis: Tsunamis are large waves generated in open bodies of water by fault displacement or major ground movement. Due to the inland location of the site, tsunamis are not considered to be a risk.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Due to the absence of significant bodies of water near the site, the potential for flooding due to seiching is considered low.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. Due to the distance of the site from significant bodies of water, and the elevation of the site on the margin of the Pauba Valley, it is unlikely that the site would be impacted by flooding due to earthquake-induced failure of off-site facilities.

10.0 SITE GRADING/EARTHWORK RECOMMENDATIONS

10.1 General

This section contains our general recommendations regarding earthwork and grading recommendations for the proposed development. These recommendations are based on the results of our field exploration, laboratory testing, our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may need to be modified based on observation of the actual field conditions during grading.

Prior to the start of any earthwork, the site should be cleared of all vegetation and debris. The materials resulting from clearing and grubbing operations should be removed from the site.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill. Based on these observations, removal of localized areas deeper than those documented may be required during grading. Therefore, some variations in the depth and lateral extent of excavation recommended in this report should be anticipated.

10.2 Remedial Earthwork

The existing superpads have irregular surfaces, with numerous soil piles, are locally loose, dry, or disturbed. Such soils are not considered suitable for the support of additional fills or structures, and should be overexcavated and recompacted. The depth and limits of the overexcavation should be determined by the geotechnical consultant based on the field conditions encountered.

In general, the existing grade should be overexcavated to expose a fill with a minimum density of ninety (90) percent, or an alluvium with a minimum density of eighty-five (85) percent of the laboratory maximum density, or, to a minimum depth of at least three (3) feet below existing grade.

Excavated soils generated by the recommended remedial grading are generally considered suitable for re-use as compacted fill. Prior to re-use, excavated soils should be cleared of all debris, vegetation, rocks larger than three (3) inches, and other deleterious materials.

10.3 Overexcavation

The project tentative tract map (RBF, 2015) indicates that the site will be reconfigured during the proposed development. The maximum cuts and fills are expected to be on the order of fifteen (15) to twenty (20) feet.

Building footings and slabs-on-grade should be uniformly supported by compacted fill. If after the planned lowering of the northern upper superpads, uncompacted alluvium soil is encountered, such alluvium should be overexcavated and recompacted to a minimum depth of three (3) feet below the bottom of the footing elevation. The overexcavation and recompaction should extend at least five (5) feet beyond the building footprint.

For subgrades below planned asphalt concrete or Portland concrete paving, including driveways, access roads, sidewalks, street areas, curbs and gutters or other flatwork, overexcavation and recompaction in alluvium soils should be to a depth of at least two (2) feet, extending at least two (2) feet beyond the pavement or flatwork.

Excavations should be in accordance to Section D1.5 *Excavations* in Appendix D, *Recommended Earthwork Specifications*.

10.4 Sub-grade Preparation

The sub-grade in all areas should be observed and approved by qualified geotechnical consultant.

All areas to receive fill should be overexcavated to the depth where the existing soils have a relative compaction of at least ninety (90) percent of the laboratory maximum dry density, or to a depth of 3.0 feet bgs. Any loose, soft, and unsuitable materials encountered during overexcavation should be removed from the site. The depth of such removal should be determined during the grading by the geotechnical consultant.

After the required overexcavation as recommended in this report, or as recommended by the geotechnical consultant during remedial grading is performed, all surfaces to receive fill should be scarified to an additional depth of six inches. The sub-grade soils should be moisture conditioned to within three (3) percent of optimum moisture content for granular soils and up to two (2) percent above optimum moisture content for fine-grained soils. The soils should be re-compacted to at least ninety (90) percent of the laboratory maximum dry density to produce a firm and unyielding surface.

10.5 Compacted Fill

Excavated soils are generally considered suitable for re-use as compacted fill. Prior to re-use, excavated soils should be cleared of all debris, vegetation, rocks larger than three (3) inches, and other deleterious materials. Rocks larger than one (1) inch in the largest dimension should not be placed within the upper twelve (12) inches of fill beneath footings and slabs or the upper eighteen (18) inches of fill under paved areas. Imported fill soils, if any, should meet the criteria presented in Section 11.5, *Imported Fill Materials* and should be approved by the geotechnical consultant prior to importation to the site.

Fill soils should be compacted as presented in Section D1.7 *Placement and Compaction of Structural Fill* in Appendix D, *Recommended Earthwork Specifications*.

10.6 Shrinkage and Subsidence

The existing site soils will undergo a change in volume when excavated and recompacted in accordance with the recommendations presented in this report. This volume change should be considered for earthwork calculations.

Based on our review of the *in situ* densities and an average anticipated compaction of ninety-two (92) percent, we anticipate that the loose, near-surface soils excavated during remedial and design earthwork will shrink by up to approximately ten (10) percent, while shrinkage of the previously compacted fill is anticipated to be negligible. For earthwork volume calculations, an average of five (5) percent shrinkage may be used for the upper five (5) feet of excavation and zero (0) percent for deeper excavations. The actual shrinkage will depend on, among other factors, the depth of cut and/or fill, the grading method, and the equipment utilized.

Subsidence refers to general settlement of the site due to densification of the deeper underlying soils while the shallower soils are being compacted by earthmoving equipment. Due to the previous site grading, ground subsidence is expected to be negligible.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

11.0 UTILITY TRENCH BACKFILL

The following sections present earthwork for utility trench backfill. The earthwork for utility trenches includes sub-grade preparation, pipe bedding, and trench zone backfill.

Open cuts adjacent to existing roadways and/or adjacent structures are not recommended within a 1H:1V (horizontal:vertical) plane extending beyond and down from the roadway or structure perimeter.

Spoils from the trench excavation should not be stockpiled more than six (6) feet in or within a distance H (ft) from the top of trench edge, where H is the depth of the trench in feet. Spoils, if any, should not be stockpiled behind the shoring within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

11.1 Pipeline Sub-grade Preparation

The final sub-grade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than two (2) inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

Any loose, soft and/or unsuitable materials encountered at the pipe sub-grade should be removed and replaced with an adequate bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

11.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe, to twelve (12) inches above the pipe. To provide uniform and firm support for the pipeline, free-draining granular soil should be used as pipe bedding material. For flexible pipes, excavated sandy materials may be used as bedding material. Crushed rock or gravel may be used for rigid pipes. Bedding material for the pipes should be free from oversized particles greater than one (1) inch in maximum dimension.

Pipe design generally requires sand equivalent of thirty (30) or greater for bedding materials. Specifications for bedding materials including required backfill requirements surrounding the pipe should be specified by the design engineer in accordance with the pipe manufacturer's guideline.

11.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. The detailed trench backfill recommendations are presented in Section D1.8, *Trench Backfill,* in Appendix D, *Earthwork Specifications*.

11.4 Temporary Excavations

Based on the materials encountered in the borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 3, *Slope Ratios for Temporary Excavation*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose, dry sand may have to be constructed at a flatter gradient than presented in the following table.

Table No. 3, Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0-4	1:1
4-10	1.5:1
10-20	2:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within five (5) feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of six (6) feet of stockpiled soils placed at least five (5) feet from the trench edge.

All applicable requirements of the California Construction and General Industry Safety Orders and the Occupational Safety and Health Act, current amendments, should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

11.5 Imported Fill Materials

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than twenty (20)
- Free of all deleterious materials
- Contain no particles larger than three (3) inches in the largest dimension
- Contain less than thirty (30) percent by weight retained on ¾-inch sieve
- Contain at least fifteen (15) percent fines (passing #200 sieve)
- Plasticity Index of ten (10) or less
- Corrosion potential similar or better than onsite soils

Any import fill should be tested and approved by the owner’s representative prior to delivery to the site.

12.0 DESIGN RECOMMENDATIONS

Recommendations for the design and construction of the proposed development are presented in the following sections. The recommendations provided are based on the assumption that, in preparing the site, the above earthwork recommendations will be implemented.

12.1 Shallow Foundation Design Parameters

The proposed single-family residential structures may be supported on continuous (strip) and/or isolated spread footings. Continuous and isolated spread footings should be at least eighteen (18) inches wide. The depth of embedment below lowest adjacent soil grade of interior and exterior footings should be at least eighteen (18) inches. Such footings can be designed based on an allowable net bearing capacity of 2,500 pounds per square foot (psf), plus 300 psf for each additional foot of depth and 150 psf for each foot of width. The maximum allowable bearing capacity should be limited to 3,000 psf.

The allowable net bearing capacity is defined as the maximum allowable net bearing pressure on the ground. It is obtained by dividing the net ultimate bearing capacity by a safety factor. The ultimate bearing capacity is the bearing stress at which ground fails by shear or experiences a limiting amount of settlement at the foundation. The net ultimate bearing capacity is obtained by subtracting the total overburden pressure on a horizontal plane at the foundation level from the ultimate bearing capacity.

The net allowable bearing values indicated above are for the dead load and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by thirty-three (33) percent for short duration loading, which will include loading induced by wind or seismic forces.

12.1.1 Active Earth Pressures

The active earth pressure behind any buried wall depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures. Equivalent fluid pressures are recommended in Table No. 4, *Lateral Earth Pressure*. These pressures assume a level ground surface behind the walls for a distance greater than the wall height, no surcharge, and no hydrostatic pressure.

Table No. 4, Lateral Earth Pressure

Lateral Earth Pressure, Equivalent Fluid Pressure (pcf)	
Active earth conditions (wall is free to deflect)	35
At-rest (wall is restrained)	55
Seismic pressure on unrestrained walls (at the top of reverse triangle where H is the height of the wall)	18H

If water pressure is allowed to build up behind the walls, the active pressures should be reduced by fifty (50) percent and added to a full hydrostatic pressure to compute the design pressures against the walls.

12.1.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil and of 0.25 between soil and steel may be used with the dead load forces. Passive earth pressure of 270 psf per foot of depth may be used for the sides of footings poured against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,500 psf. These lateral resistances may be increased by thirty-three (33) percent for seismic forces. Due to the low overburden stress of the soil at shallow depth, the upper one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by thirty-three (33) percent for short duration loading, which will include the effect of wind or seismic forces.

12.2 Settlement

The geotechnical report of the previous site grading (Converse, 1997) stated that ongoing monitoring of settlement monuments within or near the site indicated that the site was undergoing a significant rate of ground settlement at the time of the report. The site has remained in its current configuration for approximately fifteen (15) years, and we anticipate that the ground settlement is currently negligible.

The proposed site grading (RBF, 2015) will remove up to approximately twenty (20) feet of fill from the northern half of the site and add up to approximately fifteen (15) feet of fill to the southern half of the site. The geotechnical consultant should evaluate the final site configuration and conditions encountered during grading to determine whether post-grading settlement monitoring should be conducted.

The following estimates are provided for preliminary design purposes, and should be evaluated by the geotechnical consultant based on the final site configuration and conditions encountered during grading. The total settlement of shallow footings, designed as recommended above, from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. Dynamic settlement under seismic conditions is estimated to be up to 0.8 inches based on the analysis presented in Appendix C, *Liquefaction and Settlement Analyses*.

The static and dynamic differential settlements can be taken as equal to one-half of the corresponding total settlement over a lateral distance of forty (40) feet.

12.3 Slabs-on-Grade

The design of the slab-on-grade will depend on, among other factors, the expansive potential of the pad soils. Based on expansion index testing performed during this investigation, the expansive potential of the soil tested is very low to low. Due to mixing and movement of soil during grading, the expansion potential the final sub-grade soil may be different than this result. The final expansion potential should be verified at the completion of sub-grade preparation. We recommend performing additional expansion index tests at the completion of sub-grade preparation, to locate any areas where mitigation is required.

Thickness of slabs-on-grade should be determined by the structural engineer. The sub-grade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted.

If moisture-sensitive flooring or environments are planned, slabs-on-grade should be protected by 10-mil-thick polyethylene vapor barriers. The sub-grade surface should be free of all exposed rocks or other sharp objects prior to placement of the barrier. The barrier should be overlain by two (2) inches of sand, to minimize punctures and to aid in the concrete curing. At the discretion of the structure engineer, the sand layer may be eliminated.

In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking or curling of the slabs. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of the water/cement ratio.

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least seven (7) days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof may be used after finishing operations have been completed. The edges of concrete slabs exposed after removal of forms should be immediately protected to provide continuous curing.

12.4 Soil Corrosivity

The results of chemical testing of representative samples of site soils were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, Laboratory Testing Program and design recommendations pertaining to soil corrosivity are presented below.

The sulfate contents of the site soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-11, Table 4.2.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-11, Table 4.3.1).

We anticipate that concrete structures such as footings, slabs, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the site location and the results of chloride testing of the site soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-11, Table 4.2.1). ACI provides concrete design recommendations in ACI 318-11, Table 4.3.1.

The minimum electrical resistivities when saturated were 1,500 and 2,964 ohm-cm. These values indicate that the site soil is corrosive to ferrous metals in contact with the soil (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. UA qualified corrosion consultant should provide appropriate corrosion mitigation measures for ferrous metals in contact with the site soils.

12.5 Preliminary Pavement Design Recommendations

Three representative samples of the site soils were tested to evaluate the Resistance (R) value in accordance with the ASTM Standard D2844. The test is designed to provide a relative measure of soil strength for use in pavement design. The tests indicated R-value of seventeen (17), eighteen (18), and fifty-two (52). Due to mixing and movement during grading, the R-values of the final sub-grade soil are likely to be different than tested. We recommend performing additional R-value tests at the completion of the sub-grade preparation, which could change/increase the proposed preliminary pavement sections.

Asphalt concrete pavement sections corresponding to Traffic Indices (TIs) ranging from 5.0 to 8.0 for an R-value of seventeen (17) are presented for preliminary design in the following table.

Table No. 5, Recommended Preliminary Pavement Sections

R-value	Traffic Index (TI)	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
17	5.0	4.0	6.0
	6.0	4.0	9.0
	7.0	6.0	9.0
	8.0	6.5	12.0

Base materials should conform to Section 200-2.2, "Crushed Aggregate Base" of the current Standard Specifications for Public Works Construction (“Greenbook”) (Public Works Standards, 2015), and should be placed in accordance with Section 301-2 of the Greenbook.

Asphalt concrete materials should conform to Section 203 of the Greenbook and should be placed in accordance with Section 302.5 of the Greenbook.

Pavement sub-grade should be prepared in accordance with Section 301 of the Greenbook. The upper 12 inches of sub-grade should be compacted to a relative compaction of at least 95 percent as per ASTM Standard D1557 test method.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or sub-grade.

12.6 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. A desirable slope for surface drainage is two (2) percent in landscaped areas and one (1) percent in paved areas. Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the sub-grade soils. Gutters and downspouts should be installed on the roof, and runoff should be directed to the storm drain through non-erosive devices.

13.0 PLAN REVIEW AND CONSTRUCTION INSPECTION SERVICES

Project grading plans were not available for review at the time of this report. This report has been prepared to aid in the evaluation of the site, to prepare preliminary site grading recommendations, and to assist the engineer with the design of the proposed apartment building(s). It is recommended that this office should also be given the opportunity to review the future site plans and specifications to verify whether the recommendations presented herein are appropriate for the planned site development.

Recommendations presented herein are based upon the assumptions that earthwork monitoring will be provided by a qualified geotechnical consultant. All excavation bottoms should be observed and tested by a representative of the geotechnical consultant prior to fill placement. Structural fill and backfill should be placed and compacted during continuous observation and testing. It is recommended that footing excavations should be observed by a geotechnical consultant representative prior to placement of steel and concrete, so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials. All base course and subbase materials used for pavement structures should be tested and approved by the soils engineer.

14.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

As the project evolves, a continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that the recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

This report was prepared for Cal-Paseo Del Sol, LLC for the subject project described herein. We are not responsible for technical interpretations made by others of our exploratory information. Specific questions or interpretations concerning our findings and conclusions may require a written clarification to avoid future misunderstandings.

15.0 REFERENCES

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A

Field Exploration

APPENDIX A

FIELD EXPLORATION

The following activities were performed during our previous investigation of the site (Converse, 2012).

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling borings. During the site reconnaissance, the surface conditions were noted, and the approximate location of the test borings were established using existing boundary features. The boring locations should be considered accurate only to the degree implied by the method used to establish them. Discussion of the field investigation methods is presented below.

Twelve (12) exploratory borings (BH-1 through BH-12) were drilled within the project site on May 30, 2012. Borings BH-2 through BH-10 and BH-12 were drilled to the planned depth of at least 16.5 feet bgs. Borings BH-1 and BH-11 were drilled to the planned maximum depth of 51.5 feet bgs. The borings were advanced using a truck-mounted drill rig equipped with eight-inch diameter hollow-stem augers equipped with split-spoon drive samplers for soil sampling. Encountered earth materials were continuously logged by a Converse geologist and classified in the field by visual examination in accordance with the Unified Soil Classification System. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using a California Modified Sampler (2.4-inch inside diameter and 3.0-inch outside diameter) lined with thin sample rings. The sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling thirty (30) inches. Blow counts at each sample interval are presented on the boring logs. Samples were retained in brass rings (2.4-inch inside diameter and 1.0-inch height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of representative soils were also collected.

Standard penetration tests (SPT) were performed in Borings BH-1 and BH-11 below 20.0 feet bgs at 10-foot intervals using a split-barrel sampler (1.4-inch inside diameter and 2.0-inch outside diameter) in accordance with the ASTM Standard D1586 test method. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every six (6) inches for a total of 1.5 feet of sampler penetration are shown on the boring logs.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between driven samples are indicated in the log at the top of the next drive sample.

A key to soil symbols and terminology used in the boring logs is included as Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For Logs of Borings, see Drawings No. A-2 through A-13, *Logs of Borings.*

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, 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 DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

- STANDARD PENETRATION TEST
  Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE 2.42" I.D. sampler (CMS).
- DRIVE SAMPLE No recovery
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h
Disturb	Dist.

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er
Permeability	perm
Soil Cement	sc

Apparant Density	Very Loose	Loose	Medium	Dense	Very Dense
SPT (N)	< 4	4 - 11	11 - 30	31 - 50	> 50
CA Sampler	< 5	5 - 12	13 - 35	36 - 60	> 60
Relative Density (%)	< 20	20 - 40	40 - 60	60 - 80	> 80

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
SPT (N)	< 2	2-4	5-8	9-15	16-30	> 30
CA Sampler	< 3	3-6	7-12	13-25	26-50	> 50

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **A-1**

Log of Boring No. BH- 1

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1142 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, brown.						
		- concrete fragments			13/9/8	7	126	ma,max r
5		**CLAYEY SAND (SC):** fine to coarse-grained, yellow-brown.			6/9/15	15	123	
10					5/6/4	10	111	
15		**SILTY SAND (SM):** fine to coarse-grained, brown.			6/10/14	8	120	
20					4/7/7			
25		- trace clay			5/14/20	14	115	
30					8/11/14			

Log of Boring No. BH- 1

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1142 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal): **SILTY SAND (SM):** fine to coarse-grained, brown.			9/14/20	6	122	
40					8/11/12			
45					10/15/23	9	117	
50					6/9/8			
		End of boring at 51.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Log of Boring No. BH- 2

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1127 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, yellow-brown.						
			■		12/12/16	5	115	
5			■		9/8/9	8	115	
10			■		6/7/9	11	116	
15			■		12/10/13	10	119	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Converse Consultants

Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **A-3**

Log of Boring No. BH- 3

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1129 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, asphalt and concrete fragments, brown.						
			■		11/9/7	7	121	
5			■		3/3/6	11	114	ds,col
10		-fine to medium-grained	■		5/6/7	13	110	
15		- fine to coarse-grained	■		8/8/14	8	125	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Log of Boring No. BH- 4

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1125 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, asphalt and concrete fragments at 2' bgs, brown.						
			■		6/9/26	13	110	
5			■	▨	11/11/15	6	114	
								ma.ei
10		**SANDY SILT (ML):** fine-grained sand, brown.	■		5/7/7	14	111	
15			■		3/4/7	10	123	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Converse Consultants

Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **A-5**

Log of Boring No. BH- 5

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1130 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, yellow-brown.						
					10/16/7	6	113	ca,er r
5					10/7/6	9	119	
10		**SANDY SILT (ML):** fine-grained sand, dark brown.			7/9/15	12	120	
15					8/11/16	12	116	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Log of Boring No. BH- 6

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1127 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, brown.						
		- trace clay, trace gravel up to 1/2" in largest dimension	■		9/9/15	6	120	
5		**SAND WITH SILT (SP-SM):** fine to medium-grained, brown.	■		9/13/14	1	124	
10		**SILTY SAND (SM):** fine to coarse-grained, brown.	■		5/6/12	12	114	
15		**SANDY SILT (ML):** fine to medium-grained sand, organics, black.	■		6/10/14	19	105	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Log of Boring No. BH- 7

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1125 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to medium-grained, dark brown.						
			■		9/19/27	9	122	
5			■		9/12/19	12	111	ds
10			■		10/21/35	10	122	
15		**ALLUVIUM (Qal):** **SILTY SAND (SM):** fine to medium-grained, brown.	■		15/19/27	12	108	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

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Drawing No. **A-8**

Log of Boring No. BH- 8

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1124 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, few silt, brown.						
			■		15/17/21	5	111	
5		**CLAYEY SAND (SC):** fine-grained, dark brown.	■	▨	5/9/20	19	103	
								ma,max ei
10			■		6/8/11	15	102	
15		**ALLUVIUM (Qal):** **CLAYEY SAND (SC):** fine-grained, some organics, dark brown.	■		9/13/19	18	111	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

Log of Boring No. BH- 9

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1093 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, brown.						
					8/12/35	11	112	ca,er ma,r
5					10/18/20	10	116	
10		**ALLUVIUM:** **SANDY SILT (ML):** fine-grained sand, dark brown.			5/7/19	22	103	
15		**SAND (SP):** fine to coarse-grained, brown.			6/8/6	6	95	
		End of boring at 16.5 feet. No groundwater encountered. Borehole backfilled loose with soil cuttings on 5/30/12.						

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Project No. **12-81-173-01**

Drawing No. **A-10**

Log of Boring No. BH-10

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1096 Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Description	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
0	**FILL:** **SILTY SAND (SM):** fine to coarse-grained, brown.						
2	**SANDY SILT (ML):** fine-grained sand, light brown.	■		7/7/10	26	98	
5		■		5/10/12	11	113	col
10	**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, dark brown.	■		11/10/11	6	113	ds
15	**SANDY SILT (ML):** fine-grained sand, dark brown.	■		5/4/9	21	98	

End of boring at 16.5 feet.
No groundwater encountered.
Borehole backfilled loose with soil cuttings on 5/30/12.

Log of Boring No. BH-11

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1098 Depth to Water (ft): 39.5

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, light brown.			10/14/19	6	121	
5					10/12/17	8	118	
10		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, light brown.			12/17/11	3	102	ma,max
15		**SANDY SILT (ML):** fine-grained sand, organics, dark brown.			4/6/13	24	83	
20		**SILTY SAND (SM):** fine to coarse-grained, brown.			4/8/9			wa
25		**SANDY CLAY (CL):** fine-grained sand, dark brown.			5/6/9	22	100	
30		**SILTY SAND (SM):** fine to coarse-grained, dark brown.			4/8/12			wa

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Project No. **12-81-173-01** Drawing No. **A-12a**

Log of Boring No. BH-11

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1098 Depth to Water (ft): 39.5

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine to coarse-grained, some clay, dark brown.			3/9/31	18	114	
40		**CLAYEY SAND (SC):** fine to coarse-grained, brown.			22/32/35			
45					13/15/26	22	106	
50					20/30/36			
		End of boring at 51.5 feet. Groundwater encountered at approximately 39.5 feet. Borehole backfilled loose with soil cuttings on 5/30/12.						

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Project No. **12-81-173-01**

Drawing No. **A-12b**

Log of Boring No. BH-12

Dates Drilled: 5/30/2012 Logged by: CG Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1094 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND (SM):** fine to coarse-grained, brown.						
			■		7/12/17	13	112	
5		**ALLUVIUM:** **SANDY SILT (ML):** fine to medium-grained sand, dark brown.	■		6/10/15	17	114	
10		**SAND (SP):** fine to coarse-grained, gray.	■		8/11/11	3	105	
15			■		5/8/9	3	104	

End of boring at 16.5 feet.
No groundwater encountered.
Borehole backfilled loose with soil cuttings on 5/30/12.

Appendix

B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

The following laboratory testing was performed during our previous investigation of the site (Converse, 2012).

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Summaries of the various laboratory tests conducted are presented below.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples in accordance to ASTM Standard D2216 were used to aid in the classification of the soils and to provide quantitative measurement of the *in situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Collapse Test

To evaluate the moisture sensitivity (collapse potential) of the encountered soils, two (2) representative ring samples were loaded to approximately two (2) kips, allowed to stabilize under load, and then submerged. The test was performed in accordance to ASTM Standard D5333. The test result is presented in the following table.

Table No. B-1, Summary of Collapse Test Results

Boring No.	Depth (feet)	Soil Description	Percent Collapse (-) Swell (+)
BH-3	5-6.5	Silty Sand (SM), brown	- 1.6
BH-10	5-6.5	Sandy Silt (ML), brown	-0.4

Expansion Index Test

Two (2) representative bulk samples were tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM Standard D4829. Test results are presented in the following table.

Table No. B-2, Summary of Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-4	5-10	Silty Sand (SM)	18	Very Low
BH-8	5-10	Clayey Sand (SC), dark brown	33	Low

Soil Corrosivity Tests

Two (2) representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by HDR/Schiff in accordance to Caltrans Test Methods CT643, 422 and 417. Test results are presented in the table below and included at the end of this appendix.

Table No. B-3, Summary of Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Chloride (ppm)	Sulfate (ppm)	Minimum Electrical Resistivity (ohm-cm)
BH-5	0-5	8.0	7	26	2,964
BH-9	0-5	8.3	33	299	1,500

R-value Tests

R-value tests are performed to provide a relative measure of the soil strength for use in calculating the thickness of the pavement structural sections. Three (3) representative bulk samples were tested to determine the R-value of the near surface soil materials. The test was performed in accordance with ASTM Standard D2844. The test result is presented in the following table.

Table No. B-4, Summary of R-value Test Results

Boring No.	Depth(feet)	Soil Description	R-value
BH-1	0-5	Silty Sand (SM)	17
BH-5	0-5	Silty Sand (SM)	18
BH-9	0-5	Silty Sand (SM)	52

Percent Finer than Sieve No. 200

The percent finer than sieve No. 200 test was performed on two (2) representative soil samples to aid in the classification of the on-site soils and to estimate other engineering parameters. Testing was performed in general accordance with the ASTM Standard D1140 test method. The test results are presented in the following table.

Table No. B-5, Percent Passing Sieve #200 Results

Boring No.	Depth (feet)	Soil Classification	Percent Passing Sieve No. 200
BH-11	20.21.5	Silty Sand (SM), brown	31.7
BH-11	30-31.5	Silty Sand (SM), brown	38.7

Grain-Size Analyses

To aid in classification of the soils, mechanical grain-size analyses was performed on five (5) representative soil samples. Testing was performed in accordance with the ASTM Standard C136 test method. For test results, see Drawing No. B-1, Gra*in Size Distribution Results*.

Laboratory Maximum Density Test

Laboratory maximum dry density-optimum moisture content relationships test were performed on three (3) representative bulk samples. The test was conducted in accordance with the ASTM Standard D15570 test method. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results,* and are summarized in the following table.

Table No B-6, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-1	0-5	Silty Sand (SM), brown	7.0	133.5
BH-8	5-10	Clayey Sand (SC), dark brown	8.5	125.5
BH-11	10-15	Silty Sand (SM), brown	4.5	122.1

Direct Shear Test

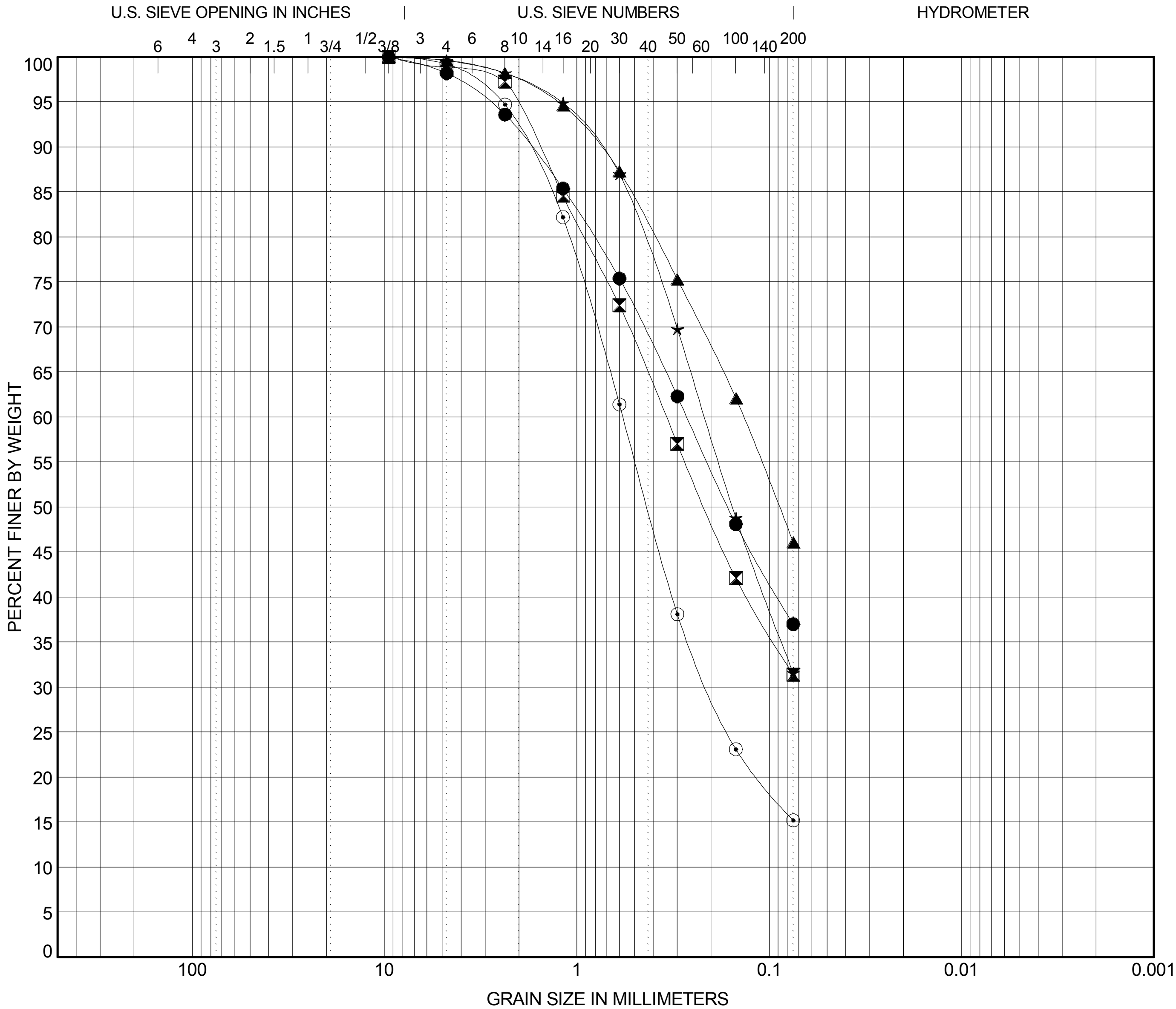
Three (3) direct shear tests were performed on representative undisturbed soil samples. For each test, three (3) ring samples were used and all samples were tested at soaked moisture conditions. The samples were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. Each sample was then sheared at a constant strain rate of 0.01 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress vs. deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-3 through B-5, *Direct Shear Test Results,* and are summarized in the table below.

Table No. B-7, Summary of Direct Shear Test Result

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-3	5.0-6.5	Silty Sand (SM)	30	100
BH-7	5.0-6.5	Silty Sand (SM)	31	150
BH-10	10.0-11.5	Silty Sand (SM)	30	100

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH- 1**	**0-5**	**SILTY SAND (SM)**					
⧗	**BH- 4**	**5-10**	**SILTY SAND (SM)**					
▲	**BH- 8**	**5-10**	**CLAYEY SAND (SC)**					
★	**BH- 9**	**0-5**	**SILTY SAND (SM)**					
⊙	**BH-11**	**10-15**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH- 1**	**0-5**	**9.5**	**0.268**			**1.8**	**61.2**	**37.0**	
⧗	**BH- 4**	**5-10**	**9.5**	**0.343**			**1.1**	**67.5**	**31.4**	
▲	**BH- 8**	**5-10**	**9.5**	**0.136**			**0.4**	**53.5**	**46.1**	
★	**BH- 9**	**0-5**	**9.5**	**0.216**			**0.4**	**68.0**	**31.6**	
⊙	**BH-11**	**10-15**	**9.5**	**0.576**	**0.206**		**0.8**	**84.0**	**15.2**	

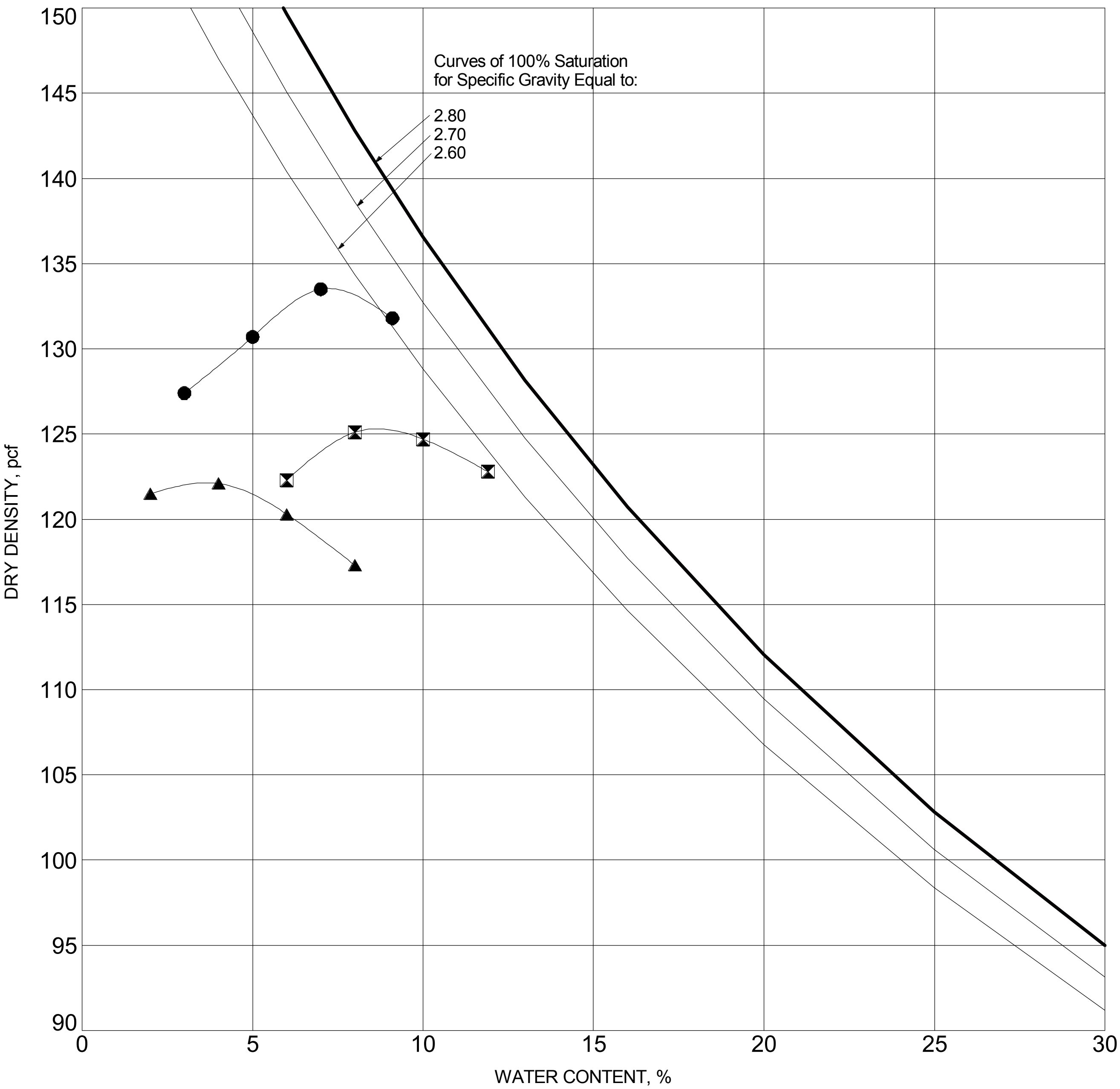
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH- 1**	**0-5**	**SILTY SAND (SM), brown**	**D1557 - A**	**7**	**133.5**
⧗	**BH- 8**	**5-10**	**CLAYEY SAND (SC), dark brown**	**D1557 - A**	**8.5**	**125.5**
▲	**BH-11**	**10-15**	**SILTY SAND (SM), brown**	**D1557 - A**	**4.5**	**122.1**

MOISTURE-DENSITY RELATIONSHIP RESULTS

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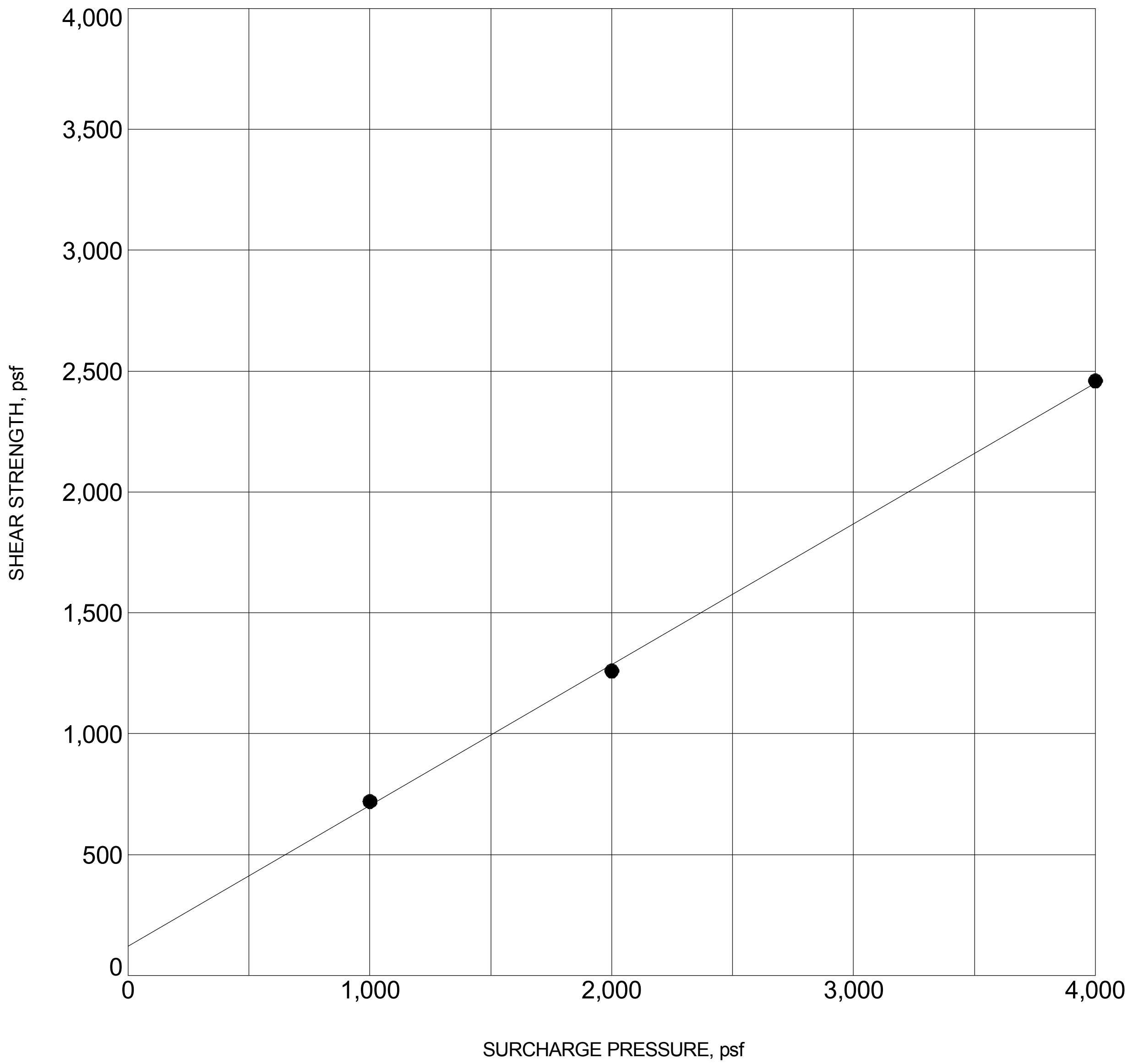
Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No.
12-81-173-01

Drawing No.
B-2



BORING NO. :	**BH- 3**	DEPTH (ft) :	**5.0-6.5**
DESCRIPTION :	**SILTY SAND (SM)**		
COHESION (psf) :	**100**	FRICTION ANGLE (degrees) :	**30**
MOISTURE CONTENT (%) :	**11.0**	DRY DENSITY (pcf) :	**114.0**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

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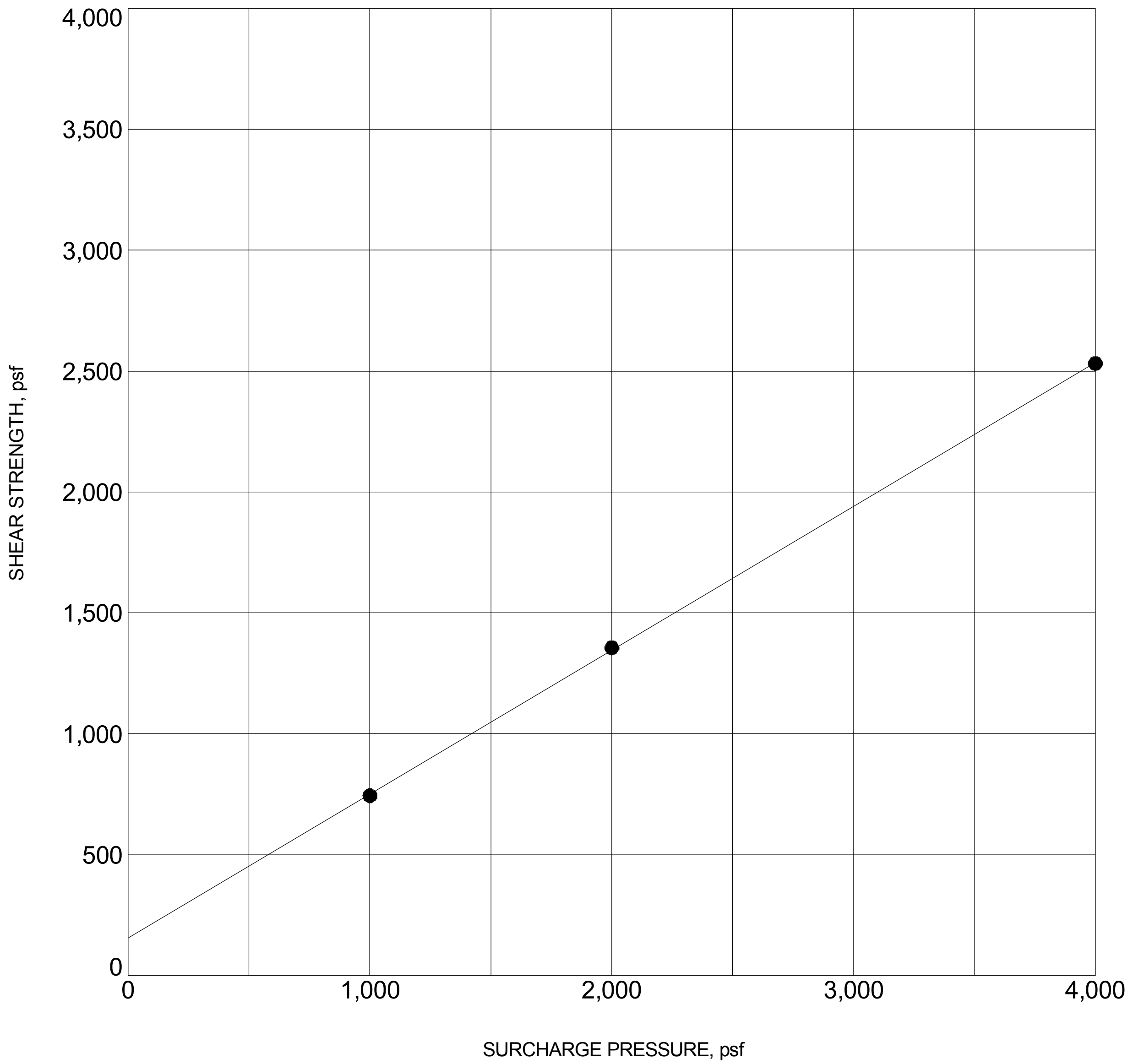
Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **B-3**



BORING NO.	:	**BH- 7**	DEPTH (ft)	:	**5.0-6.5**
DESCRIPTION	:	**SILTY SAND (SM)**			
COHESION (psf)	:	**150**	FRICTION ANGLE (degrees)	:	**31**
MOISTURE CONTENT (%)	:	**11.7**	DRY DENSITY (pcf)	:	**111.5**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

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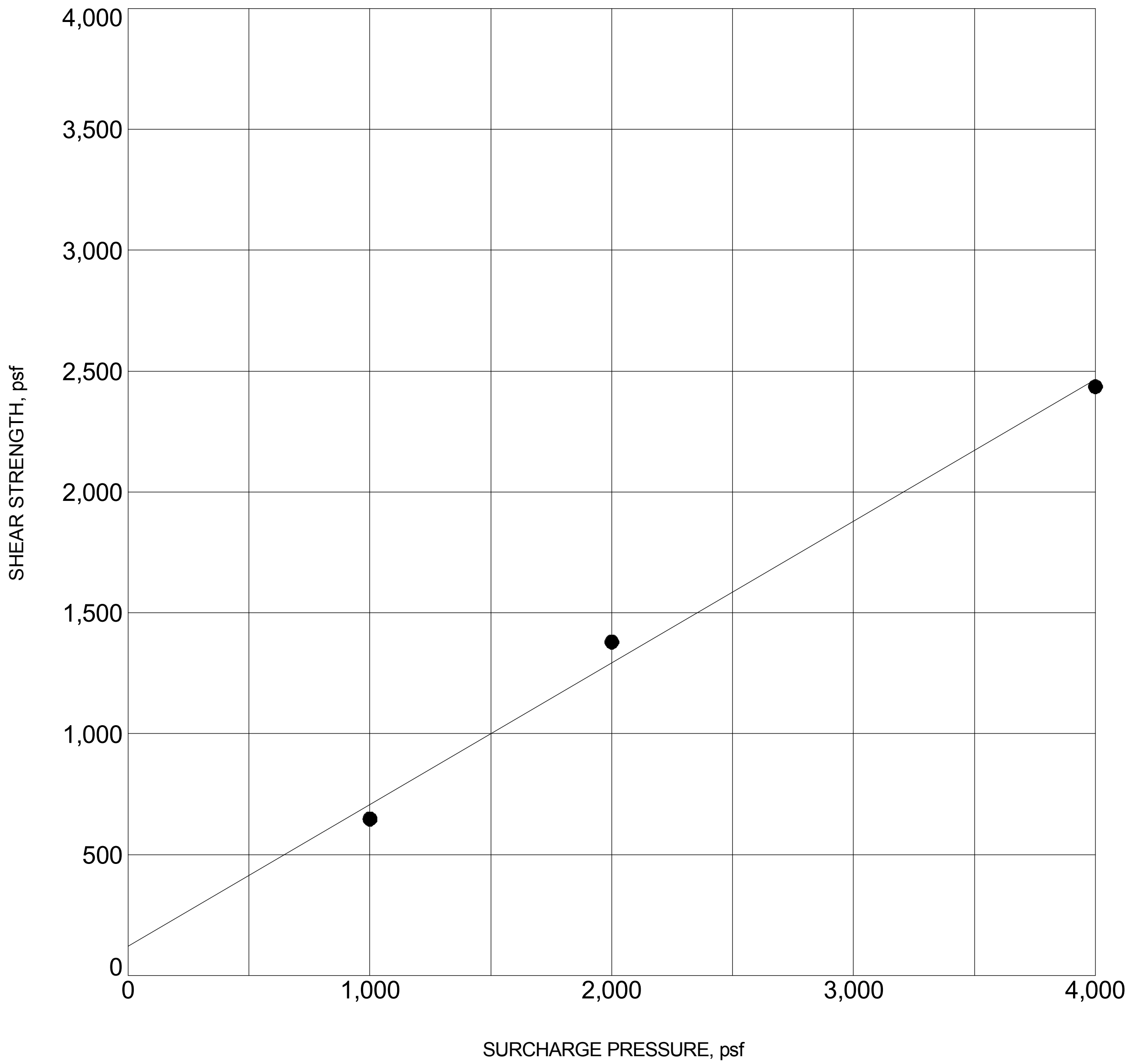
Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **B-4**



BORING NO.	:	**BH-10**	DEPTH (ft)	:	**10.0-11.5**
DESCRIPTION	:	**SILTY SAND (SM)**			
COHESION (psf)	:	**100**	FRICTION ANGLE (degrees)	:	**30**
MOISTURE CONTENT (%)	:	**10.6**	DRY DENSITY (pcf)	:	**108.8**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

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Commercial And Multi-Family Development
Planning Area 4; Approximately 43 Acre Site
City of Temecula, Riverside County, California
For: Cal-Paseo Del Sol, LLC

Project No. **12-81-173-01**

Drawing No. **B-5**

Appendix

C

Liquefaction and Settlement Analyses

APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSES

The following liquefaction and settlement analyses were performed during Converse's previous geotechnical investigation of the site.

The subsurface data obtained from exploratory borings were used to evaluate the liquefaction potential of the subject site. The boring logs are presented in Appendix A, *Field Exploration.*

Liquefaction analyses were performed using the SPT data collected from hollow stem auger borings (BH-1 and BH-11) in accordance with the method suggested in Special Publication No. 117A (CGS, 2008) and methods published by Southern California Earthquake Center (SCEC, 2008). Recorded non-standard penetration resistance from California sampler were converted to SPT blow counts by applying a conversion factor of 0.6 as suggested in Foundation Engineering Handbook (Fang, 1991). A disaggregated mean earthquake magnitude of M7.1 and peak ground acceleration (PGA) of 0.415g, where g is the acceleration due to gravity, was selected for this analysis. PGA was calculated equal to $S_{ds}/2.5$ per the California Building Code. Groundwater depths of 40 feet bgs in boring BH-1 and 15.0 feet bgs in boring BH-11 were used.

Analysis for seismically induced settlement for the proposed site was performed utilizing SPT data in the LiquefyPro computer program (CivilTech, 2011). The results of our analyses are summarized in the following table.

Boring Number	Total Depth Explored (feet)	Approximate Depth of Major Liquefaction (feet)	Dynamic Settlement (inches)
BH-1	51.5	Below 45	0.52
BH-11	51.5	20 to 25	0.80

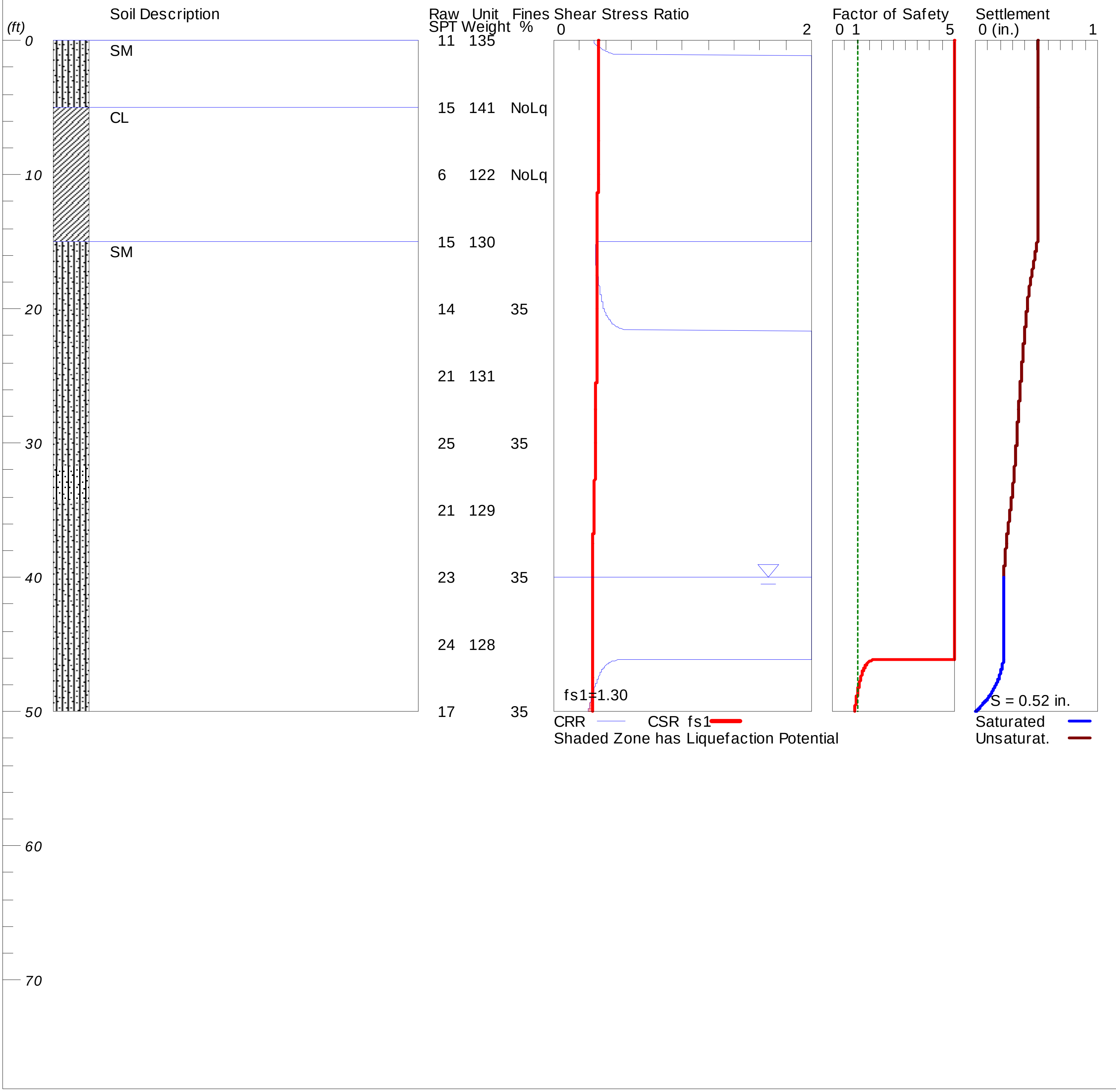
In general, the analysis indicates that liquefaction may occur within the proposed site at the depth shown on the above table. Settlement due to liquefaction is expected during a sufficiently large seismic event. The estimated dry settlement is negligible and the post-liquefaction settlement is in the range of 0.52 to 0.80 inches. Differential settlement can be estimated to be one half of the total settlement. Plate C-1 and C-2 shows the results of the liquefaction analysis.

LIQUEFACTION ANALYSIS

Liquefaction Analysis

Hole No.=BH-11   Water Depth=40 ft   Surface Elev.=1102

Magnitude=7.1
Acceleration=0.415g

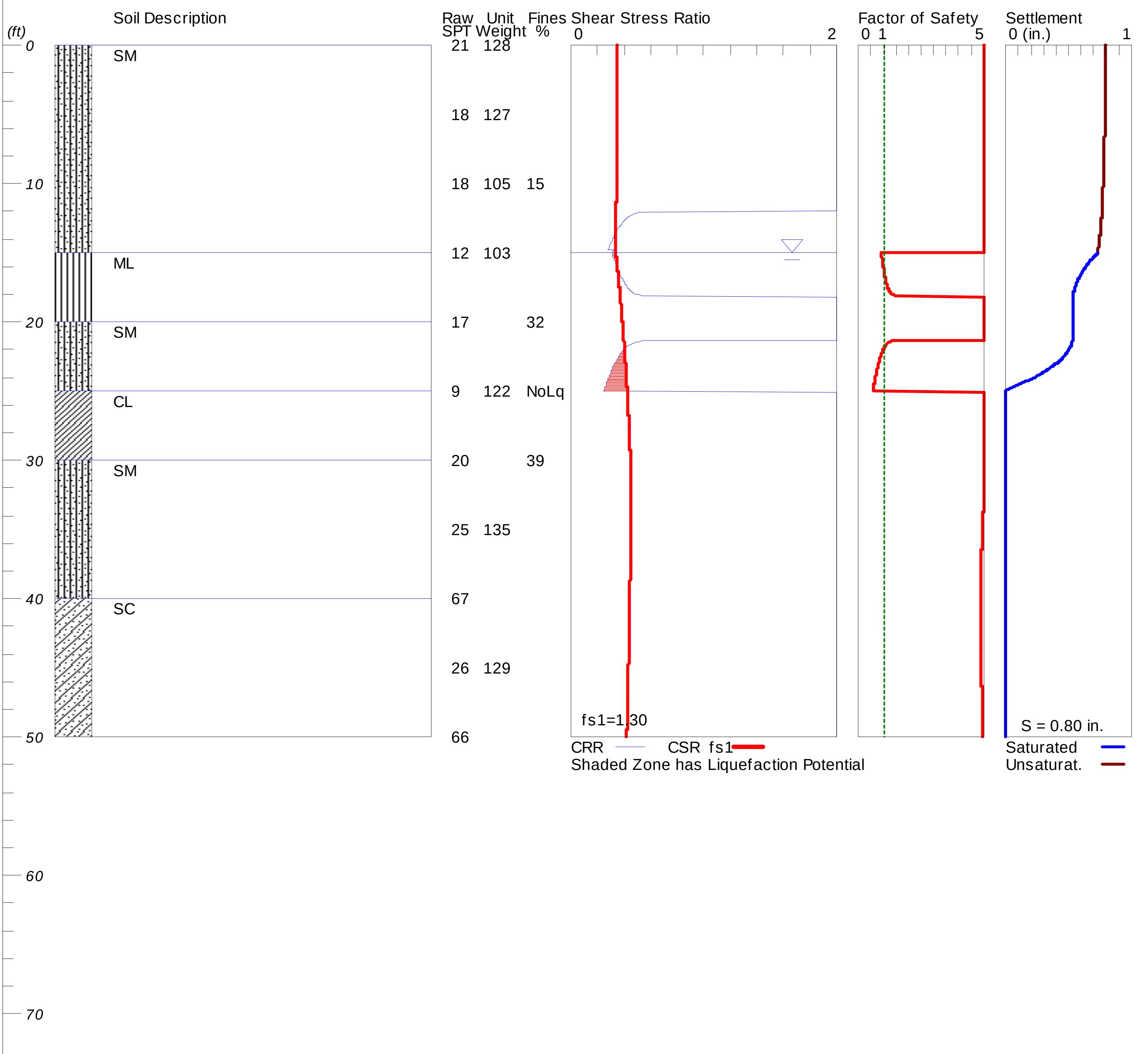


LIQUEFACTION ANALYSIS

Liquefaction Analysis

Hole No.=BH-11   Water Depth=15 ft   Surface Elev.=1102

Magnitude=7.1
Acceleration=0.415g



Appendix

D

Recommended Earthwork Specifications

APPENDIX D

RECOMMENDED EARTHWORK SPECIFICATIONS

We recommend that the following be incorporated into the project earthwork specifications.

D1.1 Scope of Work

The work includes all labor, supplies and equipment required to construct the proposed pipeline residential development in a good, workman-like manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Structural Fill
- Trench Backfill
- Observation and Testing

D1.2 Site Inspection

1. The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.

2. This *Geotechnical Investigation Report* may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report

is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the borings. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

D1.3 Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.
2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material.
3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

D1.4 Site Clearing

1. Clearing and grubbing shall consist of the removal from structure areas to be graded of all existing utilities and vegetation.
2. Organic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

D1.5 Excavations

1. Based on the results of our field exploration, surface and subsurface soil materials for the proposed development are expected to be excavatable by conventional heavy-duty earth moving equipment. Although oversized materials were not encountered during drilling, scattered cobbles and possible boulders may be encountered during grading.

2. All undocumented fill soils shall be excavated to expose dense, undisturbed, previously documented, compacted fill or native alluvium.

3. The surface of the compacted fill, where not overlain by undocumented fill, may be locally loose, dry, or disturbed. Such areas shall be excavated to expose dense, undisturbed fill.

4. Any loose sediment deposited in the bottom of the retention basin and drainage channel shall be excavated to expose dense, undisturbed alluvium. The depth and limits of the overexcavation should be determined by the geotechnical consultant based on the field conditions encountered.

5. Any surface with an existing slope steeper than 5:1 (horizontal:vertical), which is to receive additional fills, should be benched to expose dense compacted fill and to create a horizontal surface for fill placement. The lowest bench should be at least 15 feet in width and tilted at least 2 percent into the slope. Subsequent benches should not exceed 4 feet in height. Benches should be widened as needed to maintain a fill width equal to or greater than the width of the compaction equipment in use in order to facilitate compaction of all portions of the fill.

6. If loose, dry, or otherwise unsuitable materials within the fill or alluvium are exposed by the remedial grading, those materials shall be excavated. The depth and limits of the overexcavation shall be determined by the geotechnical consultant based on the field conditions encountered.

7. If required by the design grades, the soils underlying building footings shall be overexcavated to provide a compacted fill thickness of at least 3 feet and the soils underlying slabs-on-grade shall be overexcavated to provide a compacted fill thickness of at least 1 foot. Over-excavations shall extend at least five (5) feet outside building footprints or a distance equal to the actual depth of removal, whichever is greater.

8. If required by the design grades, the soils underlying asphalt concrete or Portland concrete paving, including driveways, access roads, sidewalks, street areas, curbs and gutters and other flatwork shall be overexcavated to provide a compacted fill thickness of at least 2 feet. Such over-excavation shall extend at least two (2) feet beyond the pavement edges.

D1.6 Preparation of Fill Areas

1. The sub-grade in all areas to receive fill should be observed and approved by qualified geotechnical consultant. If any loose, soft, and unsuitable materials encountered during grading should be removed from the site.

2. The sub-grade in all areas to receive fill shall be scarified to a minimum depth of twelve (12) inches, the soil moisture adjusted to within three (3) percent of optimum moisture content for granular soils and up to two (2) percent above optimum moisture content for fine-grained soils, then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.

3. For shallow foundations and slab-on-grade, the upper 24 inches of the sub-grade soils should be non-expansive (expansion index less than 20).

D1.7 Placement and Compaction of Structural Fill

1. Compacted fill placed for the support of footings and slabs-on-grade will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.

2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

   a. All fill soil particles shall not exceed three (3) inches in nominal size, and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

   b. Gravel larger than 1-inch should not be placed within the top one (1) foot of compacted fill below footings and slab-on-grade.

   c. Rocks larger than one (1) inches in the largest dimension should not be placed within the upper 18 inches of sub-grade soil under asphalt concrete paved area.

   d. Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) and moisture conditioned to within three (3) percent of optimum moisture content for granular soils and up to two (2) percent above optimum moisture content for fine-grained soils.

3. Fill soils shall be evenly spread in maximum 8-inch lifts, moisture conditioned as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

4. All fill placed at the site shall be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. Coarse-grained soils shall be moisture conditioned to within three (3) percent of

the optimum moisture content and up to two (2) percent above optimum moisture content for fine-grained soils. At least the upper 12 inches of sub-grade soils underneath the concrete apron and asphalt concrete should be compacted to at least 95 percent of the laboratory maximum dry density.

5. The upper 12 inches of soil below footing should be compacted to at least 95 percent of laboratory maximum dry density.

6. Slopes shall be constructed with fill placed in horizontal lifts, and compacted to the slope face. Finished slopes shall be compacted by appropriate means to achieve at least 90 percent compaction at the slope face.

7. Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 test method.

8. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

D1.8 Trench Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

1. Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

2. The final sub-grade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than two (2) inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

3. Pipe design generally requires sand equivalent of 30 or greater for bedding materials. Specifications for bedding materials including required backfill requirements surrounding the pipe should be specified by the design engineer in accordance with the pipe manufacturer guideline.

4. Excavated site soils free of oversize particles greater than 3 inches and deleterious

materials may be used to backfill the trench zone up to the sub-grade level.

5. Rocks larger than one (1) inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure sub-grade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in largest dimension diameter and rocks shall be well mixed with finer soil.

6. Trench backfill shall be compacted to a minimum of 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method.

7. At least the upper one (1) foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.

8. Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within three (3) percent of optimum moisture content for coarse-grained soils and up to two (2) percent optimum moisture content for fine-grained soils, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

9. The Contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

10. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

11. Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

12. It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.

13. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

D1.9 Observation and Testing

1. During the progress of earthwork for construction of the project, the Geotechnical Engineer will provide observation of the fill placement operations.
2. Field density tests will be made during earthwork operations to provide an opinion on the degree of compaction being obtained by the Contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary, until the required degree of compaction is obtained.
3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.