GEOTECHNICAL STUDY REPORT
Ammonia Station
Tapia Water Reclamation Facility
Las Virgenes Municipal Water District
Calabasas, California

Converse Project No. 11-31-349-01

January 24, 2012

PREPARED FOR

Ms. Sarah Munger
MWH Americas, Inc.
618 Michillinda Avenue, Suite 200
Arcadia, CA 91007

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Subject: **GEOTECHNICAL STUDY REPORT**
Ammonia Station
Tapia Water Reclamation Facility
Las Virgenes Municipal Water District
Calabasas, California
Converse Project Number 11-31-349-01

Dear Ms. Munger:

Enclosed is the Geotechnical Study Report prepared by Converse Consultants (Converse) for the Ammonia Station Project in the area of unincorporated Los Angeles County, Calabasas, California. The purpose of the study was to evaluate the nature and engineering properties of the subsurface soils and to provide recommendations for site earthwork, foundation design and construction for the proposed development. Our services were performed in accordance with our proposal dated December 7, 2011 (Revised.)

We appreciate the opportunity to be of continued service to MWH Americas, Inc. If you should have any questions, please do not hesitate to contact us at (626) 930-1200.

CONVERSE CONSULTANTS

William H. Chu, P.E., G.E.
Senior Vice President/Principal Engineer

Dist: 2/Addressee

SMW/SCL/GDS/WHC/mjr/amm

PROFESSIONAL CERTIFICATION

This report for the Ammonia Station Project at Tapia Water Reclamation Facility in the Calabasas area of unincorporated Los Angeles County, California, has been prepared by the staff of Converse under the professional supervision of 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.

Sean C. Lin, G.E.
Senior Engineer

Geoffrey D. Stokes, C.E.G.
Senior Engineering Geologist

William H. Chu, P.E., G.E.
Senior Vice President/Principal Engineer

EXECUTIVE SUMMARY

The following is a summary of our geotechnical study, conclusions and recommendations, as presented in the body of this report. Please refer to the appropriate sections of the 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 project consists of grading and construction of one new equipment slab foundation for the proposed Ammonia Station at Tapia Water Reclamation Facility.
- The site soils consist of fills, alluvial deposits and weathered bedrock to the maximum explored depth of 35.3 feet below existing ground surface (bgs). Fill up to a maximum observed depth of 3 feet was encountered in the boring.
- During our exploration, groundwater was encountered at the exploratory depth of 26 feet. Based upon regional groundwater data compiled by the CDMG (1998), historic high groundwater levels for the subject site are reportedly approximately 15 feet below the ground surface.
- The project site is not located within a currently designated State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zones) for surface fault rupture. No surface faults are known to project through or towards the site.
- The upper five (5) feet of mixed undocumented fill soils have an Expansion Index of 38. Expansive soil mitigation measures for foundations supported on future fill soils derived from on-site sources is needed.
- Based on our analysis, the potential seismically-induced settlement is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.8 inch. The structural engineer should consider the effect of seismically-induced settlement in the foundation design.
- Remedial grading for the pad will be needed and should include over-excavate and re-compact existing undocumented fill soils and disturbed soils due to removal of the existing structures for foundation and slab/pavement support.
- The planned ammonia station can use a structural slab (mat) foundation or conventional spread footings, supported on compacted fill.
- Site soils have "negligible" concentrations of water soluble sulfates. Accordingly, Type I or II Portland cement may be used for concrete construction. Laboratory testing indicates that site soils, in general, are considered "non-corrosive" to ferrous metals.

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1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the proposed Ammonia Station at Tapia Water Reclamation Facility in the Calabasas area of unincorporated Los Angeles County, California, as shown on Drawing No. 1, *Site Location Map*.

The purpose of the study was to evaluate the nature and engineering properties of the subsurface soils and to provide recommendations for site earthwork, foundation design and construction for the proposed development.

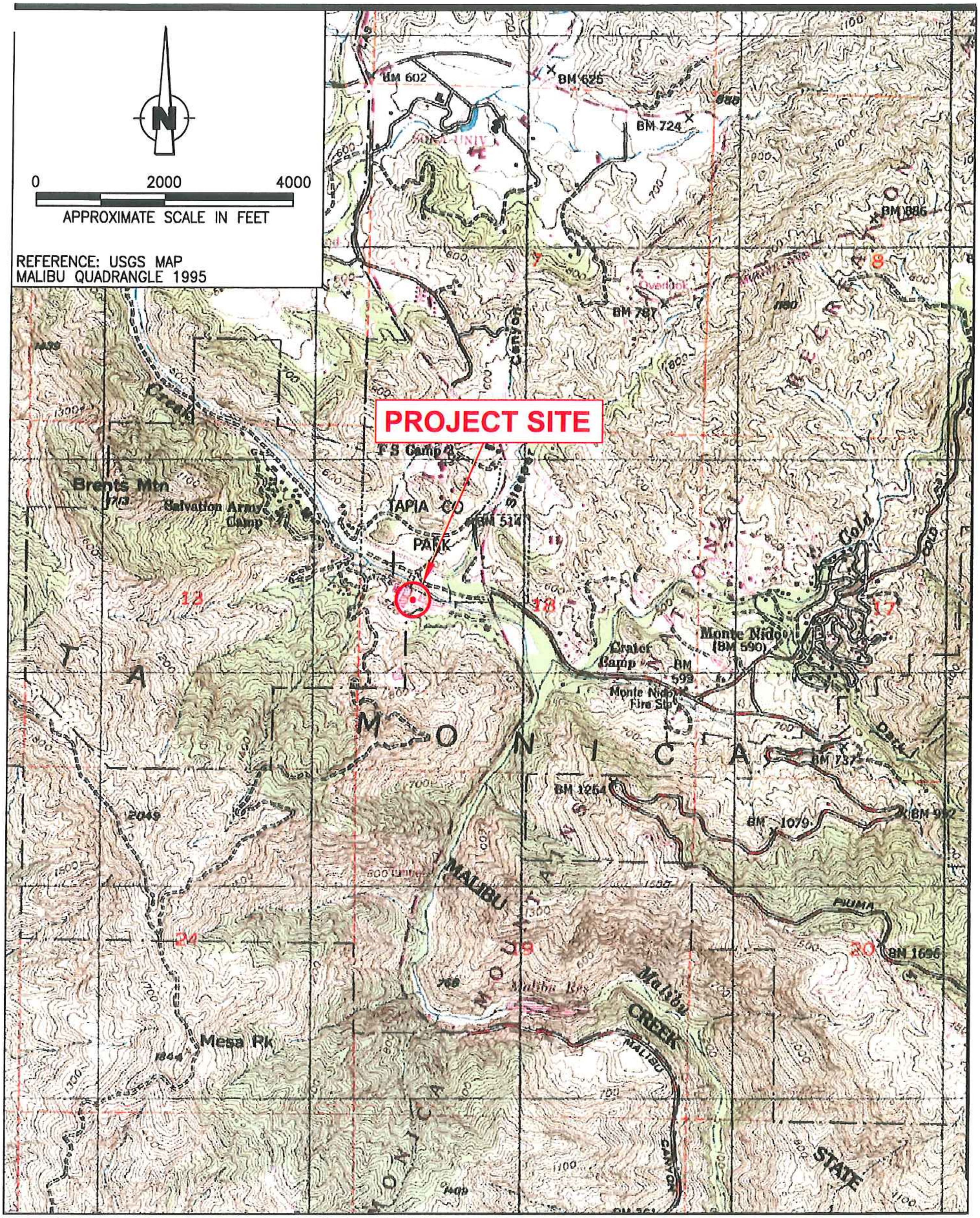
This report is written for the project described herein and is intended for use solely by MWH Americas, Inc. and its design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors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

2.0 PROJECT DESCRIPTION

The proposed project consists of grading and construction of one new equipment slab foundation for the proposed Ammonia Station at Tapia Water Reclamation Facility. No subterranean basement is proposed. The approximate location of the proposed equipment pad is depicted in Drawing No. 2, *Site Plan and Boring Location Map*.

3.0 SITE DESCRIPTION

The Tapia Water Reclamation Facility is located on the west side of Malibu Canyon Road just south of the intersection of Malibu Canyon Road and Piuma Road in the Calabasas area of unincorporated Los Angeles County, California. The site is bordered by Malibu Canyon Road to the east, an access road to the south, Malibu Creek to the north and Coral Canyon Road to the west. The project site is relatively flat and currently occupied by the existing water treatment facility. The proposed Ammonia Station is planned within the eastern portion of the site. Based on the proposed plan, the new equipment pad is located at the southwestern corner of the existing filter building located directly west of the existing balancing pond and chlorine treatment facility. Based on our review of as-built plans for the filters, the subsurface soils underneath the filter pad were improved using compaction grouting between 13 and 23 feet below ground surface to mitigate liquefaction settlement. The site coordinates are: 34.0816 degrees North Latitude, 118.7081 degrees West Longitude.

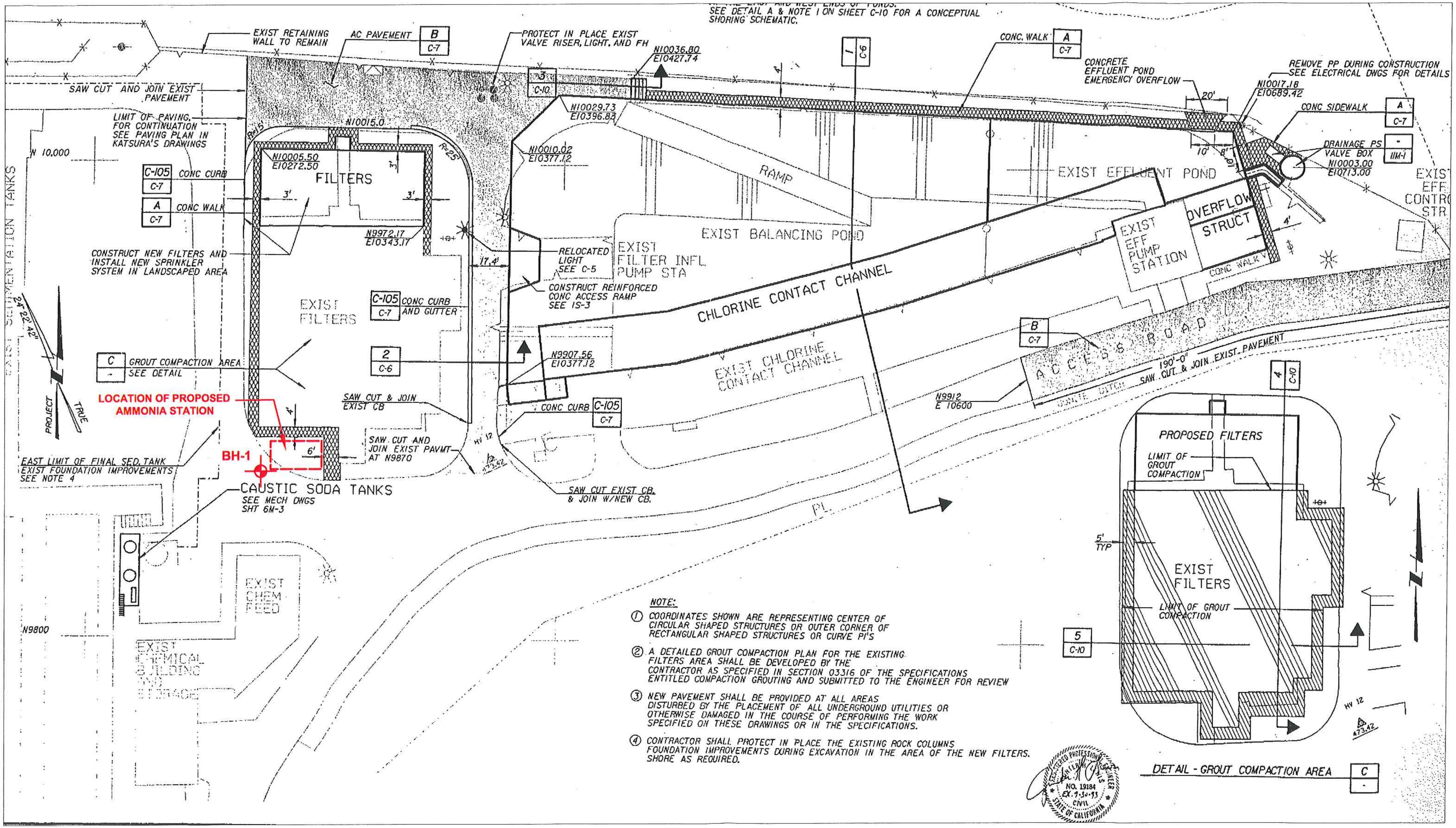


SITE LOCATION MAP

Converse Consultants

AMMONIA STATION
TAPIA WATER RECLAMATION FACILITY
CALABASAS, CALIFORNIA

Project No.	Drawing No.
11-31-349-01	1



REFERENCE: JAMES M. MONTGOMERY CONSULTING ENGINEERS, INC. 1990, SHEET C-2

LEGEND

BH-1 APPROXIMATE LOCATION OF BORING

SITE PLAN AND BORING LOCATION MAP

Converse Consultants

AMMONIA STATION
TAPIA WATER RECLAMATION FACILITY
CALABASAS, CALIFORNIA

Project No.	Drawing No.
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4.0 SCOPE OF WORK

Our scope of work consists of the tasks described in the following subsections.

4.1 Site Reconnaissance

Our field exploration included a site reconnaissance by a member of the Converse staff on December 23, 2011. The purpose of the site reconnaissance was to observe surface conditions and to select exploratory boring locations.

4.2 Subsurface Exploration

A total of one (1) boring (BH-1) was drilled on December 29, 2011 at the project site. The boring was drilled by truck mounted 8-inch diameter hollow stem auger drill rig to a depth of 35.3 feet below ground surface (bgs).

The boring was visually logged by our field engineer and sampled at regular intervals and at changes in subsurface soils. Relatively undisturbed thin-walled ring and bulk samples of representative subsurface materials were obtained from the borings for laboratory testing.

The 8-inch diameter hollow-stem auger boring was backfilled with soil cuttings following the completion of drilling. The boring backfill was densified by reverse spinning of the auger during the backfill operation and tamping of the backfill material with the auger. The boring was capped with a cold asphalt patch.

The approximate location of the exploratory boring is shown in Drawing No. 2, *Site Plan and Boring Location Map.* For a description of the field exploration and sampling program see Appendix A, *Field Exploration.*

4.3 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)
- Grain Size Distribution (ASTM Standard C136)
- Fine Content passing No. 200 Sieve (ASTM D1140)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)

- Consolidation (ASTM Standard D2435)
- Expansion Index (ASTM Standard D4829)
- Soil corrosivity tests (Caltrans 643, 422, 417, and 532)

4.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated. This report was prepared to provide our findings, conclusions, and recommendations developed during our study and evaluation.

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geologic Setting

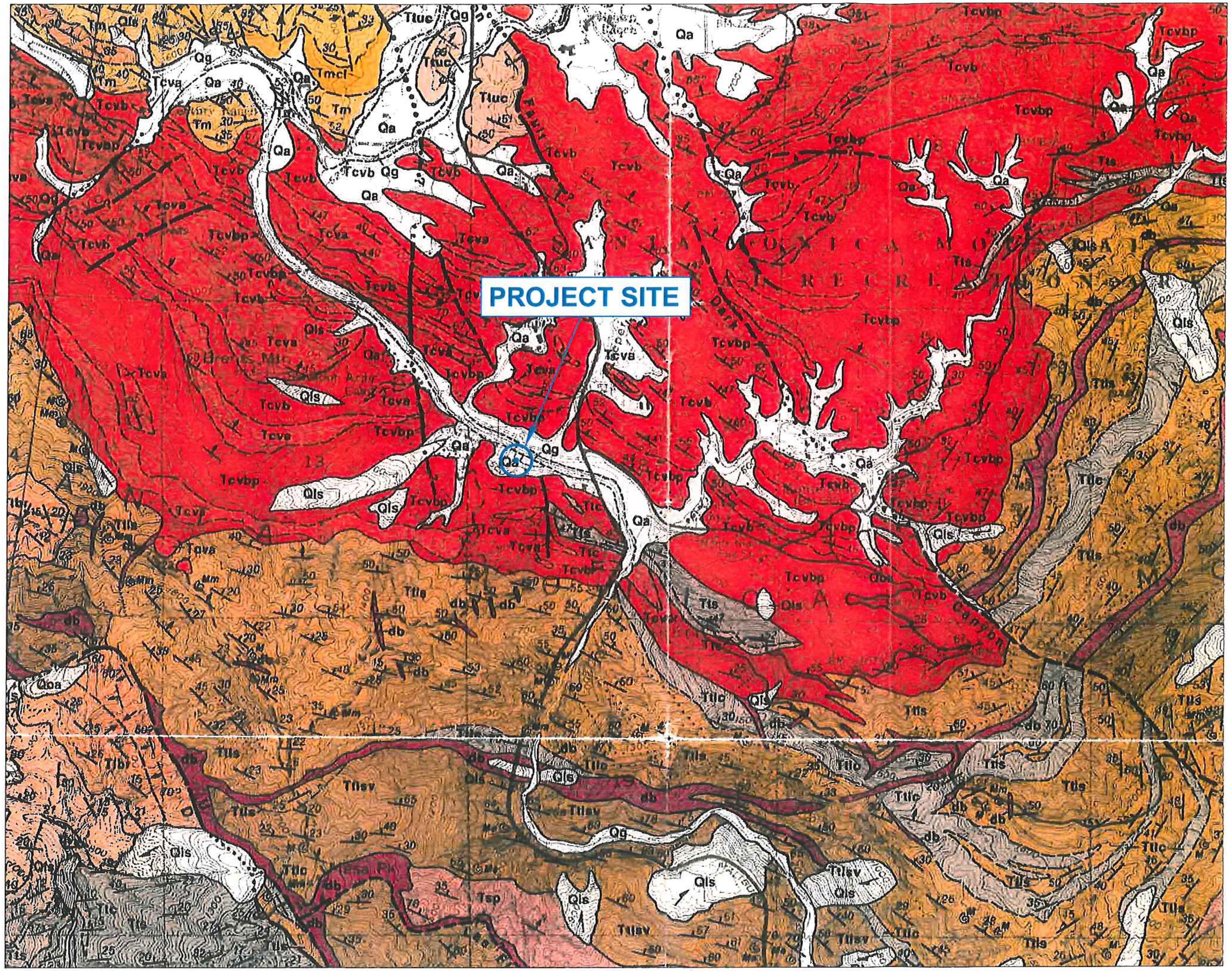
The project site is located within the central portion of the Santa Monica Mountains, which is a part of the Transverse Ranges geomorphic province of California. The mountain range trends east to west and depicts deeply incised canyons and steep slopes, indicative of rapid uplift during the Quaternary (SHZR, 2005.) Additionally, headwaters that feed into streams including Malibu Creek are located north of the crest of the Santa Monica Mountains, implying that rapid uplift occurred after streams became established (SHZR, 2005.)

Drawing No. 3, *Regional Geologic Map*, based on the Geologic Map of the Malibu Beach Quadrangle (Dibblee, 1993) has been prepared to show the location of the project site with respect to the regional geology. Review of the Geologic Map of the Malibu Beach Quadrangle (Dibblee, 1993) indicates that the site is underlain by Holocene-age (last 11,000 years) alluvial soil (map symbol Qa.)

The Santa Monica Mountains are bounded to the south by the Los Angeles Basin and the Pacific Ocean, to the north by the Simi Hills, to the north and east by the San Fernando Valley and to the west by the Oxnard Plain. Faults which have accommodated the rapid uplift of the Santa Monica Mountains include the Malibu Coast Fault, Las Flores Canyon Thrust and the Dark Canyon Fault (SHZR, 2005.). These east-west trending thrust faults and other regional faults are considered capable of seismic activity.

5.2 Subsurface Profile

The site soils consist of fills, alluvial deposits and weathered bedrock to the maximum explored depth of 35.3 feet below existing ground surface (bgs). Fill up to a maximum observed depth of 3 feet was encountered in the boring. The fill material was probably placed during construction of the existing on-site structures. Deeper artificial fill may exist



MALIBU BEACH QUADRANGLE

LEGEND

SURFICIAL SEDIMENTS

unconsolidated detrital sediments, undissected to partly dissected

Qs *beach sand*
Qg *gravel and sand of major stream channels*
Qa *alluvial gravel, sand and clay of flood plains*
af *artificial cut and fill*
Qls *landslide debris; some may be of late Pleistocene age*

OLDER SURFICIAL SEDIMENTS

unconsolidated to weakly consolidated alluvial sediments, dissected where elevated; late Pleistocene age

Qoa *older alluvium of locally derived, light gray to light brown pebble-cobble gravel, sand and silt; on coastal area deposited in part on wave-cut platform*

CONEJO VOLCANICS

(of Taliaferro 1924, Yerkes and Campbell 1979, 1980; included in middle Topanga Formation by Soper 1938, Durrell 1954; Topanga Volcanics of Truex and Hall 1969, Truex 1976)

extrusive submarine and subaerial volcanic rocks; middle Miocene age, Relizian Stage (16.6±2.4 m.y. to 13.4±0.9 m.y., Turner 1970, in Yerkes and Campbell 1979)

Tcvad *andesite-dacite breccia, exposed northwest of Century Reservoir only: light gray to tan, composed of moderately to poorly sorted, angular to subrounded cobbles and boulders of light pinkish gray to tan andesitic-dacitic rocks in detrital andesitic matrix; crudely bedded, coherent, erosion-resistant in this quadrangle; deposited subaerially(?) as reworked volcanic detritus*

Tcva *andesitic flow breccias: medium gray, pinkish gray, light brown to tan, aphanitic, very fine grained to slightly porphyritic andesitic rock; massive to vaguely stratified, composed of unsorted small to very large angular fragments of andesitic to dacitic(?) rocks in hard coherent andesitic matrix; locally includes few thin lenticular andesitic flows with platy fracture; in lower Carbon Canyon mostly shattered light gray massive andesite, possibly intrusive*

Tcvab *andesitic breccia (Ramira, Solstice and Malibu Bowl tongues of Yerkes and Campbell 1980 in ascending order in southwestern area): medium gray to pinkish brown, in part weathered light rusty brown, unstratified, composed of unsorted, very large to small angular fragments of andesite in coherent andesitic to tuffaceous (?) matrix; erosion resistant; deposited subaerially (?) as laharic (rock flow) breccia from nearby volcanic sources*

Tcvaz *(Zuma Volcanics of Yerkes and Campbell 1979, 1980) andesitic breccia south of Malibu Coast fault, similar to* Tcvab, *pinkish gray to brown; included in Conejo Volcanics*

Tcvb *basaltic flows and breccias: black, weathered dark olive brown, fine grained, composed primarily of calcic plagioclase feldspar (average An_{50}) and ferromagnesian minerals (hypersthene or augite, rare olivine and magnetite, Weigand 1982); some flows amygdaloidal with white amygdules of silica, calcite or analcime; most flows incoherent, crumbly where weathered, weakly resistant to erosion, some flows and breccias moderately coherent and more erosion resistant; some reddish; in some places unit includes thin lenses 1 or 2 meters thick of dark gray basaltic sandstone, tan arkosic sandstone, micaceous shale, carbonate rock, with oyster shell reefs; unit probably deposited in part subaerially, and in part under shallow sea*

Tcvbp *basaltic breccia, similar in color and composition to* Tcvb, *but somewhat more andesitic; unstratified to very vaguely layered; composed of small angular fragments mostly less than a foot (0.3m) and pillows as large as 2 feet (0.6m) with chilled, somewhat vitreous margins, in incoherent basaltic matrix; in large part hyaloclastic breccia (fractured from rapid chilling under sea), crumbly where weathered and weakly resistant to erosion; in few places includes thin sedimentary lenses as in* Tcvb; *deposited under sea (unit may be equivalent to unit* Tcop *of Yerkes and Campbell 1980, and to unit* Tcva *in Dibblee 1992)*

Tcvbr *basaltic breccia at Malibu Creek below juncture with Cold Creek, dark gray, composed of subangular basalt fragments in detrital basalt matrix*

Buttress unconformity of Dibblee and Ehrenspeck 1993, in southwest area of map

REFERENCE: DIBBLEE MAP
MALIBU BEACH QUADRANGLE

0 2000 4000
APPROXIMATE SCALE IN FEET

REGIONAL GEOLOGIC MAP

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at the site. The fill soils encountered consists of clayey sand. The alluvial deposits below the fill primarily consist of clayey sand, silty sand and sand extending to a depth of approximately 31-5 feet. The weathered bedrock encountered below the alluvium consists of volcanic, dark gray and brittle. For a detailed description of the materials encountered during our exploration, see Appendix A*, Field Exploration.*

5.3 Groundwater

During our exploration, groundwater was encountered at the exploratory depth of 26 feet. Based upon regional groundwater data compiled by the CDMG (1998), historic high groundwater levels for the subject site are reportedly approximate 15 feet below the ground surface.

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local conditions or during rainy seasons. Groundwater conditions below any given site vary depending on numerous factors including seasonal rainfall, local irrigation, and groundwater pumping, among other factors. The regional groundwater table is not expected to be encountered during the planned construction.

5.4 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 between or beyond the boring locations. If, during construction, subsurface conditions different from those presented in this report are encountered, this office should be notified immediately so that recommendations can be modified, if necessary.

5.5 Flood

Review of the Flood Insurance Rate Map (FIRM), Panel 1529 (1529F) of 2350, effective date September 26, 2008, from the Map Service Center (MSC) viewer, indicates that the northern portion of the site is designated as Zone “A”, “No base flood elevations determined.” The southern portion of the site is designated as Zone “D”, “Other Areas”, “Areas in which flood hazards are undetermined, but possible.”

6.0 FAULTING AND SEISMIC HAZARDS

6.1 Faulting

The subject site is situated within 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 project site is not located within a currently designated State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zones) for surface fault rupture. No surface faults are known to project through or towards the site. The closest known faults to the project site with mappable surface expressions are the Malibu Coast Fault (5 kilometers to the south). Other regional faults were included as capable fault sources for the probabilistic seismic hazard analysis for the site.

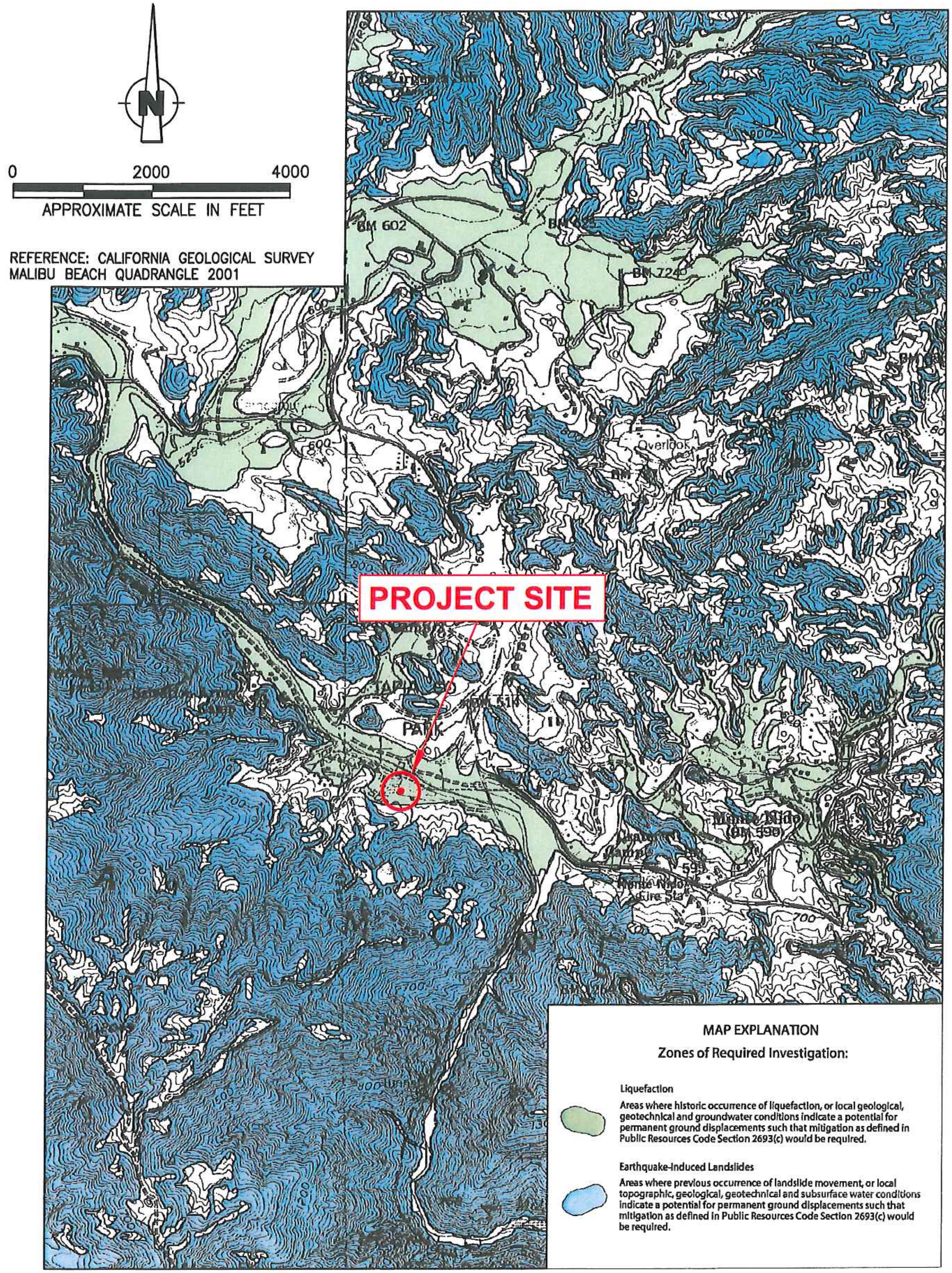
6.2 Seismic Hazards

In addition to direct effects on structures, strong ground shaking from earthquakes can also produce other side effects that include surface fault rupture, soil liquefaction, lateral spreading, seismically induced settlement, ground lurching, landsliding, earthquake-induced flooding, seiches, and tsunamis. Drawing No. 4, *Seismic Hazard Zones Map*, has been prepared to show the mapped location of potential liquefaction and earthquake-induced landslide areas near the project site. The State of California Seismic Hazard Zone Map for the Malibu Beach Quadrangle (October 17, 2001) shows the project site is located within an area of potential liquefaction. The project site is not shown with any earthquake-induced landslide areas due to the relatively flat condition of the site topography.

Results of a site-specific evaluation for each type of possible seismic hazard are explained below:

6.2.1 Surface Fault Rupture

The site is not located within a currently designated State of California Earthquake Fault Zone. Based on a review of existing geologic information, no known active fault zone crosses or projects toward the site. The potential for surface rupture resulting from the movement of the nearby major faults is considered remote.



SEISMIC HAZARD ZONE MAP

Converse Consultants	AMMONIA STATION TAPIA WATER RECLAMATION FACILITY CALABASAS, CALIFORNIA	Project No. 11-31-349-01	Drawing No. 4

6.2.2 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and consequently lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

The site is located within a mapped Seismic Hazard Zone for liquefaction. The liquefaction potential and seismic settlement analyses were performed utilizing SPT data from the upper 35 feet of soils at BH-1. The groundwater level of 15 feet below ground surface is used for analyses. The analyses were performed in accordance with the method published by Southern California Earthquake Center (March 1999) using *LiquefyPro,* Version 5.8d, 2009, by Civil Tech Software. The results of analysis indicate the site is susceptible to liquefaction between 15 and 25 feet below ground surface. The potential seismically-induced settlement, as analyzed in Boring BH-1, is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.8 inch. The structural engineer should consider the effect of seismically-induced settlement in the foundation design.

6.2.3 Lateral Spreading

Seismically induced lateral spreading involves primarily lateral movement of earth materials 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. The topography at the project site and in the immediate vicinity of the site is relatively flat, with no nearby slopes or unsupported embankments. Under these circumstances, the potential for lateral spreading at the subject site is considered negligible.

6.2.4 Seismically-Induced Slope Instability

Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The project site is relatively flat, with natural ascending slopes located more than 75 feet to the south. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be nil.

6.2.5 Tsunamis and Seiches

Tsunamis are tidal waves generated by fault displacement or major ground movement. Based on the elevation of the site (over 475 feet) and distance from the ocean (over 3.0 miles) tsunamis do not pose a hazard. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on site location away from lakes and reservoirs, seiches do not pose a hazard.

7.0 SEISMIC ANALYSIS

Seismic parameters for structural design based on the 2010 California Building Code and the site coordinates are provided on the following table:

Table No. 1, CBC Seismic Design Parameters

Seismic Parameters	
Latitude	34.0816
Longitude	118.7081
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.920g
Mapped 1-second Spectral Response Acceleration, S_1	0.771g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	1.920g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.156g
Design Spectral Response Acceleration for short period S_{ds}	1.280g
Design Spectral Response Acceleration for 1-second period, S_{d1}	0.771g

8.0 LABORATORY TESTING

Results of the various laboratory tests are discussed below.

- *In-situ* Moisture and Dry Density – Results of *in-situ* moisture and dry density tests are presented on the Logs of Borings in Appendix A, *Field Exploration.*

- Grain Size Analysis – One (1) representative sample was tested to evaluate the relative grain size distribution of sandy samples. Results are presented in Appendix B, *Laboratory Testing Program*, and indicate the samples tested are predominately clayey sand.

- Passing No. 200 – Two (2) selected samples were tested to determine the percent finer than sieve No. 200, to aid in the classification of on-site soils and for liquefaction analyses. Results are presented in Appendix A, *Log of Borings.*
- Maximum Dry Density and Optimum Moisture Content – One (1) selected sample was tested for typical moisture-density relationship. The result is presented in Appendix B, *Laboratory Testing Program.*
- Direct Shear – One (1) direct shear test was performed on a sample. Result of the direct shear test is presented in Appendix B, *Laboratory Testing Program.* The test result indicates the soil tested has moderate shear strength.
- Consolidation Test – One (1) consolidation tests were performed on a representative sample of the site soils. The result of the test is presented in Appendix B, *Laboratory Testing Program.* The upper 5 feet of soil is moderately compressible.
- Expansion Index – One (1) selected sample from the upper five (5) feet bgs of the site soil was tested to evaluate Expansion Index (EI). The test result indicates that the site soils have a low expansion potential (EI between 21 and 50).
- Soil Corrosivity – A representative sample of the site soils was tested to determine soil corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program.* Test results are also discussed in Section 11.4, *Soil Corrosivity Evaluation*
- For additional information on the subsurface conditions, see the Logs of Borings in Appendix A, *Field Exploration.*

9.0 GEOTECHNICAL EVALUATIONS AND CONCLUSIONS

Based on the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned ammonia station construction, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans, specifications, and are followed during site construction.

The following is a summary of the major geologic and geotechnical factors to be considered for the planned project:

- Undocumented fill soils were encountered in the borings with depths on the order of three (3) feet below the existing ground surface. Thicker fills or disturbed soils during removal of existing structures may exist at the site. The fill soils encountered in the borings generally consist of clayey sand.

- Remedial grading for the station pad will be needed and should include over-excavation and re-compaction of the existing undocumented fill soils.
- The planned ammonia station can use a structural slab (mat) foundation or conventional spread footings, supported on compacted fill.
- The upper five (5) feet of mixed undocumented fill soils have an Expansion Index of 38. Expansive soil mitigation measures for foundations supported on future fill soils derived from on-site sources is needed.
- Based on our analysis, the potential seismically-induced settlement is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.8 inch. The structural engineer should consider the effect of seismically-induced settlement in the foundation design.
- Site soils tested have "negligible" concentrations of water soluble sulfates, which is considered "non-corrosive" to concrete.
- Site soil tested is not considered corrosive to ferrous metal based on pH, chloride, and resistivity test results.

10.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

10.1 General Evaluation

Based on our field exploration, laboratory testing, and analyses of subsurface conditions at the site, remedial grading will be required to prepare the sites for support of the proposed structures that are constructed with conventional shallow footings. To reduce differential settlement, variations in the soil type, degree of compaction, and thickness of the compacted fill, the thickness of compacted fill placed underneath the footings should be kept uniform.

Site grading recommendations provided below are based on our experience with similar projects in the area and our evaluation of this investigation. Site preparation for the proposed structure will require removal of existing structures, improvements, concrete paving and other existing underground manmade structures and utilities.

The site soils can be excavated utilizing conventional heavy-duty earth-moving equipment. The excavated site soils, free of vegetation, shrub and debris, may be placed as compacted fill in structural areas after proper processing. Rocks larger than three inches in the largest dimension should not be placed as fill.

Soils containing organic materials should not be used as structural fill. The extent of removal should be determined by the geotechnical representative based on soil observation during grading.

10.2 Over-Excavation for Station Pad

Prior to the start of construction, all loose soil, fill and soil disturbed during demolition should be removed to firm acceptable native material or compacted fill.

In order to provide uniform support for the new structure, the minimum depth of over-excavation should be three (3) feet below the ground surface, or two (2) feet below proposed foundations, whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. Over-excavation should extend a minimum of three (3) feet beyond the limits of perimeter footings, where feasible. Excavation activities should not disturb existing utilities, buildings and remaining structures. Existing utilities should be removed and adequately capped at the project boundary line, or salvaged/rerouted as designed.

10.3 Structural Fill

Following observation of the excavation bottom, subgrade soil surfaces should be scarified to a depth of at least six inches. The scarified soil should be moisture-conditioned within three (3) percent above the optimum moisture. Scarified soil shall be compacted to a minimum 90 percent of the laboratory maximum dry density as determined by the ASTM Standard D1557 test method.

Excavated site soils, free of deleterious materials and rock particles larger than three inches in the largest dimension, should be suitable for placement as compacted fill. Any import fill should be tested and approved by Converse. The import fill should have an expansion potential less than 20.

Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned to at approximate three (3) percent above the optimum moisture content. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method. We recommend 6 inches of crushed aggregate base (Caltrans Class 2 aggregate) be placed below foundation and compacted to 95 percent of relative compaction.

10.4 Excavatability

Based on our field exploration, the earth materials at the site should be excavatable with conventional heavy-duty earth moving and trenching equipment. Some gravel and cobbles should be expected during excavation.

10.5 Expansive Soil

Based on our laboratory test result, the site soils have low expansion potential (EI =21 to 50). The soil materials with an Expansion Index higher than 20 should be mitigated. There are several mitigation measures that can be utilized to improve expansive soils at the site.

Some mitigation measures include:

- Pre-Saturation of on-site compacted subgrade soils to at approximate three (3) percent above optimum moisture content.
- Removing about two (2) feet of the underlying soils throughout the site, and replacing with imported sandy material.
- Reinforcing footing and place thicker concrete slab with moisture barrier.
- Lime treat the upper two (2) feet of the subgrade soils.

It is very important to keep the site soils moisture content around or under the edge of foundation, concrete slab, and asphalt concrete pavement at approximately the same moisture content before, during and after construction. This will reduce greatly the expansion potential of the site soils.

10.6 Pipe Backfill Recommendations

It is anticipated that the natural soils will provide a firm foundation for site utilities. Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material.

10.6.1 Pipe Subgrade Preparation

The pipe subgrade should be level, firm, uniform, free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Subgrade soil surfaces for pipeline should be scarified to a depth of at least six (6) inches and be compacted to a minimum of 90 percent relative compaction. Protruding oversize particles larger than two (2) inches in the largest dimension, if any, should be removed from the trench bottom and replaced with compacted materials. If yielding soft subgrade is encountered, we recommend over-excavate at least 18 inches, place geofabric (Mirafi HP570 or equivalent) at bottom of excavation to receive 18 inches compacted base materials (CMB or equivalent). Base material should be compacted to at least 95 percent of relative compaction.

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.

10.6.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 12 inches above the pipe. To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate or crushed rock may be used as pipe bedding material. The type and thickness of the granular bedding placed underneath and around the pipe, if any, should be selected by the pipe designer. The load on the rigid pipes and deflection of flexible pipes and, hence, the pipe design, depends on the type and the amount of bedding placed underneath and around the pipe. Care should be taken to densify the bedding material below the springline of the pipe.

Pipe design generally requires a granular material with a sand equivalent (SE) greater than 30. Bedding material for the pipes should be free from oversized particles (greater than 1-inch). Therefore, on site native materials are generally suitable to be used for pipe bedding.

Migration of fines from the surrounding native and/or fill soils must be considered in selecting the gradation of any imported bedding material. We recommend that the pipe bedding material should satisfy the following criteria:

$$D_{15} < 2.5 \text{ mm and } D_{50} < 19.0 \text{ mm}$$

Where D_{15} and D_{50} represent particle sizes of the bedding material corresponding to 15 percent and 50 percent passing by weight, respectively.

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

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

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 thickness of soil lifts/layers prior to compaction 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.

As an alternative to compacted fill for trench backfill, controlled low-strength material may be used.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by Geotechnical Consultant 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. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations. 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.

10.6.4 Flexible Pipe Joints

We recommend flexible joints should be installed to compensate possible differential settlements where the buried pipes interface with structures and for joints between the structures.

10.7 Shrinkage and Subsidence

Soil shrinkage and/or bulking as a result of remedial grading depends on several factors including the depth of over-excavation, and the grading method and equipment utilized, and average relative compaction. For preliminary estimation, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The approximate shrinkage factor for the undocumented fill soils is estimated to range from ten (10) to twenty (20) percent.
- The approximate shrinkage factor for the native alluvial soils is estimated to range from five (5) to fifteen (15) percent.
- For estimation purposes, ground subsidence may be taken as 0.15 feet as a result of remedial grading.

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 DESIGN RECOMMENDATIONS

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the earthwork and site grading recommendations provided in this report will be followed. The proposed equipments may be supported by mat foundation or continuous and isolated footings.

11.1 Slab-on-Grade (Mat) Foundations

11.1.1 Vertical Capacity

Slab-on-grade (mat) foundation founded on compacted structural fill per our earthwork and site grading recommendations, can be used. Mat foundations should be embedded at least 12 inches below the lowest adjacent grade. Greater embedment may be needed in order to resist lateral loads due to the wind and/or seismic forces. An allowable net bearing capacity of 2,000 psf may be used for mat foundations founded on compacted fill. The allowable net bearing capacity may be increased by 300 psf for each foot of depth to the maximum of 3,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

11.1.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed for foundation on supported compacted base material. An allowable passive earth pressure of 300 psf per foot of depth up to a maximum of 3,000 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

11.1.3 Modulus of Subgrade Reaction

The modulus of subgrade reaction (k) for design of mat foundations were estimated from the available soil compressibility data and published charts. For mat foundations, the following equation may be used to calculate k:

$k = k_s\,[(B+1)/2B]^2$, k value including overburden pressure
k = vertical modulus of subgrade reaction (pounds per cubic inch)
k_s= 200 pounds per cubic inch (pci)
B = foundation width (feet)

11.1.4 Settlement

Most of the slab-on-grade settlement at the project site is expected to occur immediately after the application of the load. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 0.5 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30 foot span. The potential seismically-induced settlement is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.8 inch.

11.1.5 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

11.2 Shallow Foundations

11.2.1 Vertical Capacity

As an alternative to mat foundation, continuous and square footings can be used. Continuous and isolated footings should be founded at least 15 inches below lowest adjacent final grade on compacted fill placed per our earthwork recommendations. A minimum footing width of 12 inches is recommended for continuous footings and 24 inches for isolated footings.

The allowable dead plus live load bearing value for isolated square and continuous footings founded at least 15 inches below lowest adjacent final grade is 2,000 psf. The net allowable bearing pressure can be increase by 300 psf for each additional foot of excavation depth and by 200 psf for each additional foot of width up to a maximum value of 4,500 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

11.2.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed for foundation supported on compacted base material. An allowable passive earth pressure of 300 psf per foot of depth up to a maximum of 3,000 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

11.2.3 Settlement

The static settlement of structures supported on continuous and/or spread footings founded on compacted fill will depend on the actual footing dimensions and the imposed vertical loads. Most of the footing settlement at the project site is expected to occur immediately after the application of the load. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 0.5 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30 foot span. The potential seismically-induced settlement is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.8inch.

11.2.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

11.3 Provisional Earth Pressure for Retaining Structures

The following provisional design values may be used for any utility vaults and/or walls less than 10 feet below grade. The 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 pressure. The following fluid pressures are recommended for vertical walls with no hydrostatic pressure, no surcharge, and level backfill.

Table No. 2, Earth Pressures for Retaining Structures

Equivalent Fluid Pressure	
Cantilever Wall (Active pressure)	40 (Triangular Distribution)
Restrained Wall (At-rest pressure)	60 (Triangular Distribution)

The recommended lateral pressures assume that the walls are fully back-drained to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by free draining, uniformly graded, ¾ -inch washed, crushed aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent, and should extend to about 2 feet below the finished grade. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, rigid ABS (SDR-35) or PVC A-2000, or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. If subdrain is not to be constructed, the vault wall should be designed for an equivalent fluid pressure of 90 pcf.

11.4 Soil Corrosivity Evaluation

Converse retained the Environmental Geotechnology Laboratory, Inc., located in Arcadia, California, to test one (1) bulk soil sample taken in the general area of the proposed structures. The tests included minimum resistivity, pH, soluble sulfates, and chloride content, with the results summarized on the following table:

Table No. 3, Soil Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) ppm	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-1	1-5	8.01	90	50	1,600

In accordance with the Caltrans Corrosive Guidelines (2003), the pH value, chloride content, and resistivity of the soil sample tested is in the non-corrosive range to ferrous metal.

The soluble sulfate concentration tested is in the non-corrosive range to concrete. Mitigation measures to protect concrete in contact with the soils are not anticipated. Type I or II Portland Cement may be used for the construction of the foundations and slabs.

A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed. Conventional corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

11.5 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the building with a minimum 2% slope gradient for at least 10 feet measured perpendicular to the face of the wall. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 1 percent away from the building.

12.0 CONSTRUCTION CONSIDERATIONS

12.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 12.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required.

12.2 Temporary Excavations

Based on the sandy materials encountered in the exploratory borings, sloped temporary excavations (if necessary) may be constructed according to the slope ratios presented in Table No. 4, *Slope Ratios for Temporary Excavations*. 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. 4, Slope Ratios for Temporary Excavations

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	vertical
4 – 8	1:1
8 – 15	1:1.5

*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 minimize 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, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act 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.

12.3 Shoring Design

Although not anticipated, temporary shoring may be required for the excavation due to space limitations and/or adjacent surcharge loading. Temporary shoring may consist of the use of a trench box (where feasible), conventional soldier piles and lagging. Shoring should ultimately be designed by a qualified structural engineer considering the below recommendations in their final design and others which are applicable.

Temporary cantilevered shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 30 pounds per cubic foot (pcf) for non-surcharged condition. This pressure is valid only for shoring retaining level ground. This equivalent fluid pressure is valid only for shoring supporting level ground.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. Surcharge pressures from the existing structures should be added to the above earth pressures for surcharges within a horizontal distance less than or equal to the wall height. Surcharge coefficients of 40% of any uniform vertical surcharge should be added as a horizontal earth pressure for shoring design. All shoring should be designed and installed in accordance with state and federal safety regulations.

12.4 Geotechnical Services During Construction

This report has been prepared to aid in the site preparation and site grading plans and specifications, and to assist the architect, civil and structural engineers in the design of the proposed structures. It is recommended that this office be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Recommendations presented herein are based upon the assumption that adequate earthwork monitoring will be provided by Converse. Excavation bottoms should be observed by a Converse representative prior to the placement of compacted fill. Structural fill and backfill should be placed and compacted during continuous observation and testing by this office. Footing excavations should be observed by Converse prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any recommended actions presented herein to be unsafe.

13.0 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.

Design recommendations given in this report are based on the assumption that the earthwork and site grading 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 final site grading and 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.

14.0 REFERENCES

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APPENDIX A

FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface drilling. During the site reconnaissance, surface conditions were noted, and the locations of the test borings were determined. Borings were approximately located using existing features as a guide.

Field exploration consisted of drilling four 8-inch diameter exploratory hollow stem borings (BH-1) on December 29, 2011 to depth of 35.3 feet below the existing ground surface. Soils were continuously logged and classified in the field by visual/manual examination, in accordance with the Unified Soil Classification System. Field descriptions have been modified, where appropriate, to reflect laboratory test results.

Relatively undisturbed ring and bulk samples of the subsurface soils were obtained at frequent intervals in the borings. The undisturbed samples were obtained using a California Steel Sampler (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin sample rings. The sampler was driven into the bottom of the boreholes with successive drops of a 140-pound hammer falling 30 inches by means of a mechanically driven pulley. The number of successive drops of the driving weight ("blows") required for every 6-inch of penetration of the sampler are shown on the Logs of Borings in the "blows" column.

The soil was retained in brass rings (2.4 inches in diameter and one inch in height). The central portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to the laboratory. Bulk soil samples were also collected in plastic bags and brought to the laboratory.

Standard Penetration Tests (SPTs) were also performed. In this test, a standard split-spoon sampler (1.4 inches inside diameter and 2.0 inches outside diameter) was driven into the ground with successive drops of a 140-pound hammer falling 30 inches by means of an automatic hammer. The number of successive drops of the driving weight ("blows") required for every 6-inch of penetration of the sampler are shown on the Logs of Borings in the "blows" column.

The soil retrieved from the spoon sampler was carefully sealed in waterproof plastic containers for shipment to the laboratory.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2a and A-2b, *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.

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

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

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name	Project No.	Drawing No.
TAPIA WATER RECLAMATION FACILITY	11-31-349-01	A-1

Log of Boring No. BH-1

Dates Drilled: 12/29/2011 | Logged by: DA | Checked By: SCL

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): 26

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
		4.5" ASPHALT OVER 5.5" BASE						
		FILL (Af): **CLAYEY SAND (SC):** fine to medium-grained, some silt, dark gray to graish brown.						max,ma,caer,ds,ei wa(fc=43%)
		ALLUVIUM (Qa): **CLAYEY SAND (SC):** fine-grained, some silt, dark brown.						
5					6/11/17	17	112	c
10		-increase in silt, trace gravel			7/10/12	16	109	
		SILTY SAND (SM): fine-grained, brown.						
15					3/4/4			
					20/19/23	12	103	
20		-more silt, grayish brown			3/5/5			wa(fc=40%)
		SAND WITH SILT (SP-SM): medium to coarse-grained, grayish brown.						
25					5/6/10			wa(fc=11%)
30		-more gravel			22/50(3")			
		BEDROCK: **CONEJO VOLCANICS (Tcvb):** weathered, dark gray.						
					50(3")			

Converse Consultants

Project Name: TAPIA WATER RECLAMATION FACILITY | Project No. 11-31-349-01 | Drawing No. A-2a

Log of Boring No. BH-1

Dates Drilled: 12/29/2011 | Logged by: DA | Checked By: SCL

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): 26

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
		BEDROCK: **CONEJO VOLCANICS (Tcvb):** weathered, dark gray.						
		End of boring at 35.3 due to refusal. Groundwater encountered at 26 feet. Borehole backfilled, tamped and patched with asphalt on 12-29-11.						

Converse Consultants

Project Name: TAPIA WATER RECLAMATION FACILITY

Project No. 11-31-349-01

Drawing No. A-2b

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

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. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests, performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure 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*.

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected sample. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

Percent Finer Than Sieve No. 200

The percent finer than sieve No. 200 test was performed on four (4) 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 and boring logs.

Table No. B-1, Summary of Percent Passing Sieve #200 Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Passing Sieve No. 200
BH-1*	1-5	Clayey Sand (SC)	43
BH-1	20	Silty Sand (SM)	40
BH-1	25	Sand with Silt (SP-SM)	11

* results from sieve analysis

Maximum Dry Density Test

A laboratory maximum dry density-moisture content relationship test was performed on one (1) representative bulk sample. The test was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2 *Moisture-Density Relationship Results.*

Direct Shear

Direct shear test was performed on one (1) sample remolded to 90 percent relative compaction soil samples. The test was performed at soaked moisture conditions. For this test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The 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 deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing No. B-3, *Direct Shear Test Results,* and in the following table:

Table No. B-2, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1*	1-5	Clayey Sand (SC)	30	200

*sample remolded to 90% of relative compaction

Consolidation Test

Consolidation test was performed on one (1) relatively undisturbed samples. Data obtained from this test was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the one-inch high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. The sample was tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Drawing No. B-4, *Consolidation Test Results.*

Expansion Index

One (1) selected bulk sample was tested to evaluate the expansion potential of materials encountered at the site. Test results are presented in the following table:

Table No. B-3, Expansion Index Test Results

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-1	1-5	Clayey Sand (SC)	38	Low

Soil Corrosivity

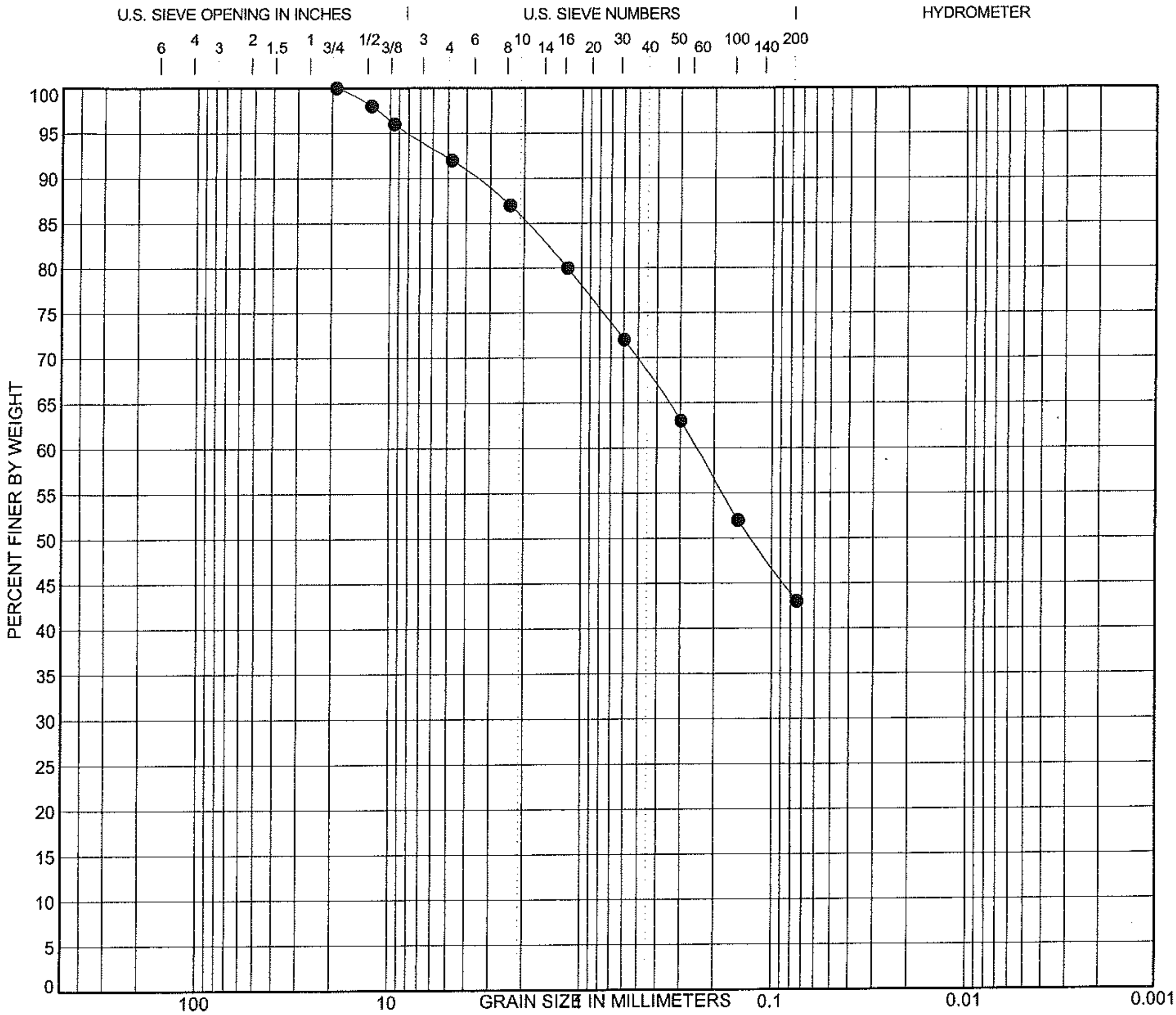
One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of this test is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL.

Table No. B-4, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) ppm	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-1	1-5	8.01	90	50	1,600

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**	**1-5**	**CLAYEY SAND (SC)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-1**	**1-5**	**19**	**0.246**			**8.0**	**49.0**	**43.0**	

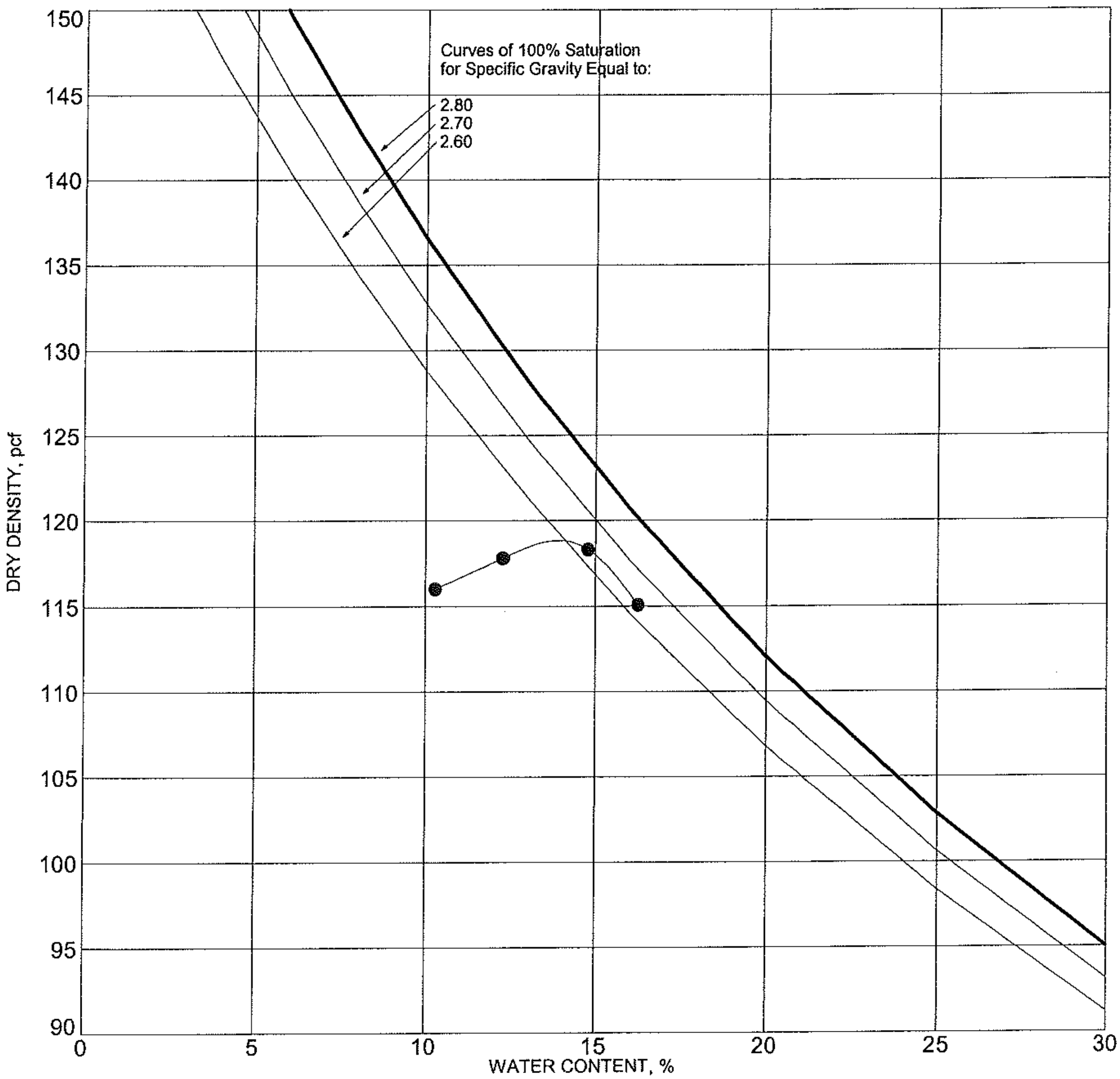
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name
TAPIA WATER RECLAMATION FACILITY

Project No.
11-31-349-01

Drawing No.
B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-1	1-5	CLAYEY SAND (SC)	D1557 Method C	13.6	118.8

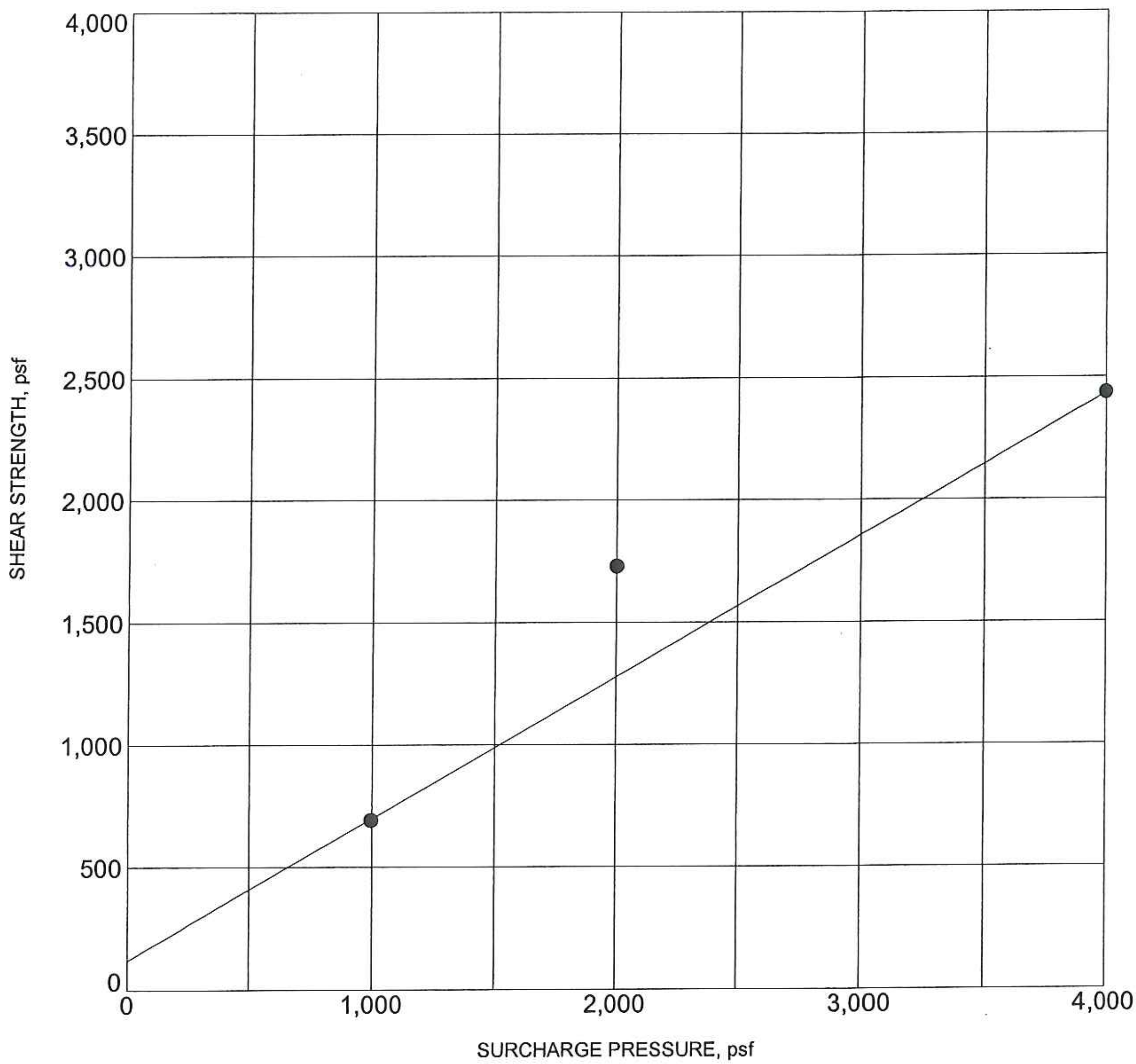
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project Name
TAPIA WATER RECLAMATION FACILITY

Project No.
11-31-349-01

Drawing No.
B-2



BORING NO.	:	BH-1	DEPTH (ft)	:	1-5
DESCRIPTION	:	CLAYEY SAND (SC)			
COHESION (psf)	:	200	FRICTION ANGLE (degrees):		30
MOISTURE CONTENT (%)	:	10.0	DRY DENSITY (pcf)	:	115.5

NOTE: Ultimate Strength. Remolded to 90% relative compaction.

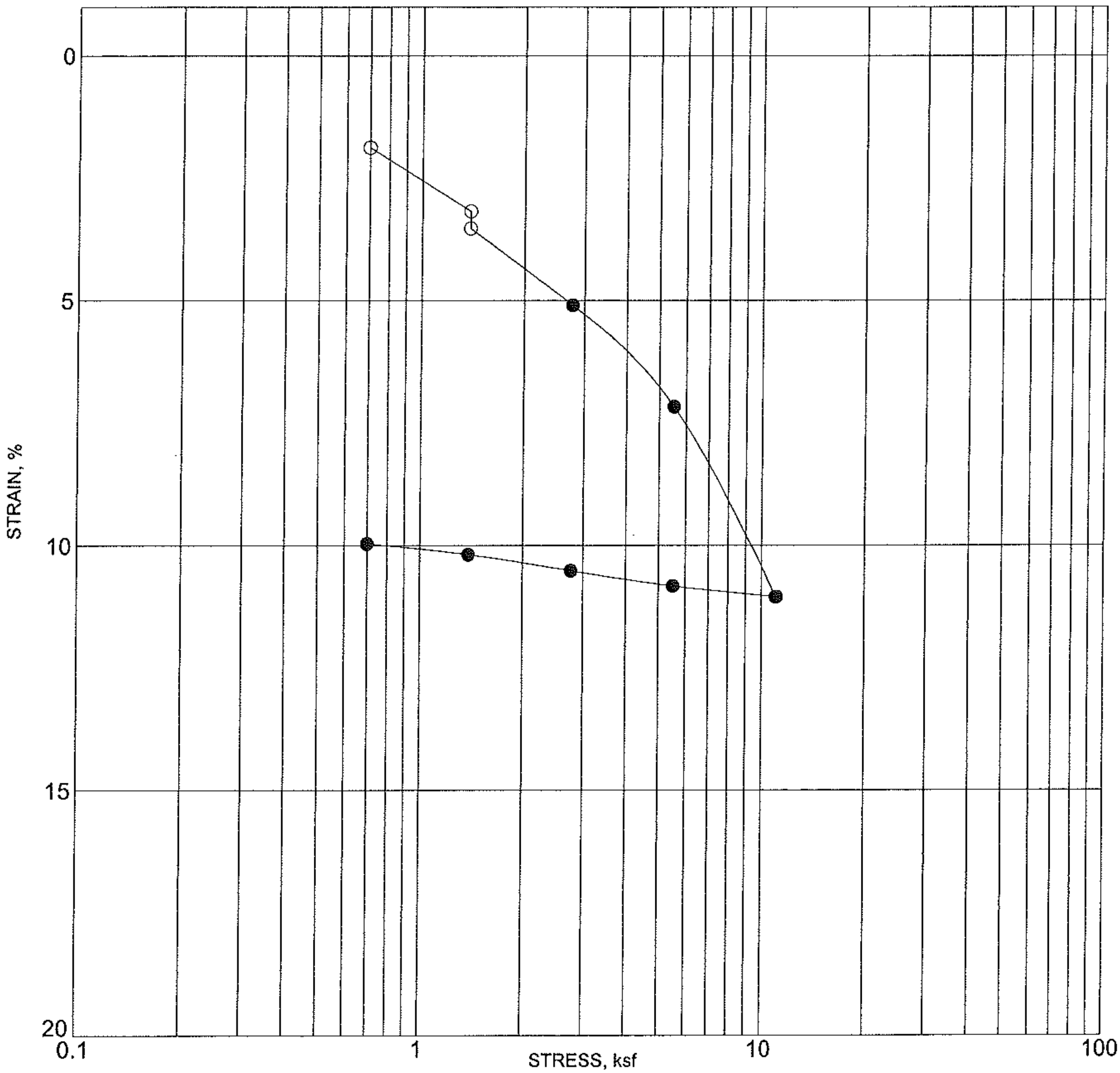
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
TAPIA WATER RECLAMATION FACILITY

Project No.
11-31-349-01

Drawing No.
B-3



BORING NO. :	BH-1	DEPTH (ft) :	5
DESCRIPTION :	CLAYEY SAND (SC)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	17.2	111.5		
FINAL	19.8	111.5		

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
TAPIA WATER RECLAMATION FACILITY

Project No. 11-31-349-01

Drawing No. B-4

APPENDIX C

EARTHWORK SPECIFICATIONS

APPENDIX C

EARTHWORK SPECIFICATIONS

C1.1 Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good, workmanlike 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 Fill
- Observation and Testing

C1.2 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 Study Report* by Converse Consultants 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 boring logs. 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.

C1.3 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; and (c) have the authority to approve or reject soils proposed for use in building areas.
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.

C1.4 Site Clearing

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

C1.5 Excavations

1. Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

C1.6 Preparation of Fill Areas

1. All organic material, organic soils, incompetent alluvium, undocumented fill soils and debris should be removed from the proposed building areas.

In order to provide uniform support for the new structure, the minimum depth of over-excavation should be three (3) feet below the ground surface, or two (2) feet below proposed foundations, whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. Over-excavation should extend a minimum of three (3) feet beyond the limits of perimeter footings, where feasible. The actual depth of removal should be determined based on observations made during grading.

2. The subgrade in all areas to receive fill shall be scarified to a minimum depth of six inches, the soil moisture adjusted within three (3) percent above optimum, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.
3. Compacted fill may be placed on native soils that have been properly scarified and recompacted as discussed above.
4. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

C1.7 Placement and Compaction of Fill

1. Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, and driveways 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. 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) at about to three percent above optimum moisture.
3. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried 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. The on-site soils shall be moisture conditioned at approximate three (3) percent above the optimum moisture content.
5. 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 compaction method.

6. 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.
7. It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

C1.8 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. Trench backfill shall be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557 test method.
3. Rocks larger than one inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in largest dimension. Rocks shall be well mixed with finer soil.
4. The pipe design engineer should select bedding material for the pipe. Bedding materials generally should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419 test method.
5. 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 between optimum and three percent above optimum, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
6. The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

7. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.
8. 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.
9. It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.
10. 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.

C1.9 Observation and Testing

1. During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.
2. Field density tests will be made during grading 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.

APPENDIX D

LIQUEFACTION/SEISMIC SETTLEMENT ANALYSIS

APPENDIX D

LIQUEFACTION/SEISMIC SETTLEMENT ANALYSIS

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

Our liquefaction analyses are based on the *Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California, Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Liquefaction Hazards in California,* dated March 1999, and *2010 California Building Code.*

The subsurface data obtained from exploratory borings were used to evaluate the liquefaction/seismic settlement potential of the area. The Logs of Borings are presented in Appendix A, *Field Exploration.* The liquefaction potential and seismic settlement analyses were performed utilizing SPT data obtained from our boring BH-1 for the upper 60 feet of soil. The analyses were performed in accordance with the method published by Southern California Earthquake Center (March 1999) using *LiquefyPro,* Version 5.8d, 2009, by Civil Tech Software. The following seismic parameters are used for liquefaction potential analyses.

Table No. D-1 Seismic Parameters Used in Liquefaction Analyses

Boring No.	Groundwater Depth* (feet)	Earthquake Magnitude* Mw	Peak Ground Acceleration** (g)
BH-1	15	7.3	0.512

* Based on Seismic Hazard Zone Report for the Malibu Beach 7.5-Minute Quadrangle
** Based on S_{DS}/2.5 per CBC 2010

The results of analysis indicate the site is susceptible to liquefaction between 15 and 25 feet below ground surface. The potential seismically-induced settlement, as analyzed in Boring BH-1, is estimated to be 1.22 inch with a potential differential dynamic settlement of 0.6 inch. The structural engineer should consider the effect of seismically-induced settlement in the foundation design.

30.00	50.00	115.00	13.00
35.00	50.00	115.00	13.00

Output Results:
Settlement of Saturated Sands=1.12 in.
Settlement of Unsaturated Sands=0.09 in.
Total Settlement of Saturated and Unsaturated Sands=1.22 in.
Differential Settlement=0.608 to 0.803 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.14	0.43	5.00	1.12	0.09	1.22
0.50	2.14	0.43	5.00	1.12	0.09	1.22
1.00	2.14	0.43	5.00	1.12	0.09	1.22
1.50	2.14	0.43	5.00	1.12	0.09	1.22
2.00	2.14	0.43	5.00	1.12	0.09	1.22
2.50	2.14	0.43	5.00	1.12	0.09	1.21
3.00	2.14	0.43	5.00	1.12	0.09	1.21
3.50	2.14	0.43	5.00	1.12	0.09	1.21
4.00	2.14	0.43	5.00	1.12	0.09	1.21
4.50	2.14	0.43	5.00	1.12	0.09	1.21
5.00	2.14	0.43	5.00	1.12	0.09	1.21
5.50	2.14	0.43	5.00	1.12	0.09	1.21
6.00	2.14	0.43	5.00	1.12	0.09	1.21
6.50	2.14	0.43	5.00	1.12	0.08	1.21
7.00	2.14	0.43	5.00	1.12	0.08	1.21
7.50	2.14	0.43	5.00	1.12	0.08	1.20
8.00	2.14	0.42	5.00	1.12	0.08	1.20
8.50	2.14	0.42	5.00	1.12	0.08	1.20
9.00	2.14	0.42	5.00	1.12	0.08	1.20
9.50	2.14	0.42	5.00	1.12	0.07	1.20
10.00	2.14	0.42	5.00	1.12	0.07	1.19
10.50	2.14	0.42	5.00	1.12	0.07	1.19
11.00	2.14	0.42	5.00	1.12	0.07	1.19
11.50	2.14	0.42	5.00	1.12	0.06	1.19
12.00	2.14	0.42	5.00	1.12	0.06	1.18
12.50	2.14	0.42	5.00	1.12	0.06	1.18
13.00	2.14	0.42	5.00	1.12	0.05	1.17
13.50	0.44	0.42	5.00	1.12	0.04	1.16
14.00	0.36	0.42	5.00	1.12	0.03	1.15
14.50	0.32	0.42	5.00	1.12	0.02	1.14
15.00	0.33	0.42	0.80*	1.12	0.00	1.12
15.50	0.33	0.42	0.79*	1.06	0.00	1.06
16.00	0.34	0.43	0.79*	0.99	0.00	0.99
16.50	0.34	0.43	0.78*	0.93	0.00	0.93
17.00	0.34	0.44	0.78*	0.86	0.00	0.86
17.50	0.34	0.44	0.77*	0.80	0.00	0.80
18.00	0.34	0.44	0.77*	0.73	0.00	0.73
18.50	0.34	0.45	0.77*	0.67	0.00	0.67
19.00	0.35	0.45	0.77*	0.60	0.00	0.60
19.50	0.35	0.46	0.77*	0.54	0.00	0.54
20.00	0.35	0.46	0.76*	0.47	0.00	0.47
20.50	0.38	0.46	0.82*	0.41	0.00	0.41
21.00	0.41	0.47	0.87*	0.37	0.00	0.37
21.50	0.42	0.47	0.89*	0.32	0.00	0.32
22.00	0.43	0.47	0.91*	0.28	0.00	0.28
22.50	0.44	0.48	0.92*	0.25	0.00	0.25
23.00	0.44	0.48	0.91*	0.21	0.00	0.21
23.50	0.43	0.48	0.88*	0.17	0.00	0.17
24.00	0.41	0.49	0.84*	0.13	0.00	0.13
24.50	0.39	0.49	0.80*	0.08	0.00	0.08

25.00	0.37	0.49	0.75*	0.02	0.00	0.02
25.50	2.14	0.50	4.32	0.00	0.00	0.00
26.00	2.14	0.50	4.30	0.00	0.00	0.00
26.50	2.14	0.50	4.28	0.00	0.00	0.00
27.00	2.15	0.50	4.28	0.00	0.00	0.00
27.50	2.15	0.51	4.25	0.00	0.00	0.00
28.00	2.15	0.51	4.22	0.00	0.00	0.00
28.50	2.15	0.51	4.20	0.00	0.00	0.00
29.00	2.14	0.51	4.17	0.00	0.00	0.00
29.50	2.14	0.52	4.15	0.00	0.00	0.00
30.00	2.14	0.52	4.12	0.00	0.00	0.00
30.50	2.13	0.52	4.11	0.00	0.00	0.00
31.00	2.13	0.52	4.10	0.00	0.00	0.00
31.50	2.13	0.52	4.09	0.00	0.00	0.00
32.00	2.13	0.52	4.09	0.00	0.00	0.00
32.50	2.12	0.52	4.08	0.00	0.00	0.00
33.00	2.12	0.52	4.07	0.00	0.00	0.00
33.50	2.12	0.52	4.06	0.00	0.00	0.00
34.00	2.12	0.52	4.06	0.00	0.00	0.00
34.50	2.11	0.52	4.05	0.00	0.00	0.00
35.00	2.11	0.52	4.05	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils