TERRADYNE
Engineers, Geologists & Environmental Scientists

Report of Geotechnical Investigation and Percolation-Infiltration Study
Proposed 7-Eleven Development
900 W Mission Avenue
Escondido, San Diego County, CA 92025
Terradyne Project No.: L201027

Mr. Chris Post
ATC Design Group
1277 Pacific Oaks Place, Suite 102
Escondido, CA 92029

Terradyne Project No: L201027 **June 17, 2020**

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Re: **Report of Geotechnical Investigation and Percolation-Infiltration Study**
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Terradyne Project No.: L201027

Dear Mr. Post,

In accordance with your request, Terradyne, Inc., Inc. has performed this Report of Geotechnical Investigation and Percolation-Infiltration Study at the subject site. The purpose of our investigation was to evaluate the geotechnical conditions at the site in the areas of proposed construction and to provide geotechnical parameters for design and construction.

Based on our investigation, it is our opinion that the proposed construction is feasible from the geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development.

We appreciate and wish to thank you for the opportunity to serve you on this project. Please do not hesitate to contact us if we can be of additional assistance during the Construction Materials Testing and Quality Control phases of construction.

Respectfully Submitted,
Terradyne Engineering, Inc.

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EXECUTIVE SUMMARY

The soil conditions at the site of the proposed 7-Eleven Development located 900 W Mission Avenue, Escondido, San Diego County, CA 92025, were explored by drilling seven (7) geotechnical borings to the maximum depth of 22.0 ft below existing grade. Four (4) of the geotechnical borings were converted percolation/infiltration test borings. Laboratory tests were performed on selected samples to evaluate the engineering characteristics of various soil strata encountered in our borings.

This report presents a description of subsurface conditions encountered at the site, recommended foundation systems, and design and construction criteria influenced by the subsurface conditions. It is based on data obtained from field investigations, laboratory test results and our previous experience with similar sites.

- Based on the review of the available references, the site is not located in an Earthquake Fault Zone, and has not been evaluated for seismic landslide hazards nor a liquefaction susceptibility although based on site geotechnical and geological conditions the site appears to have a low probability of liquefaction and landsliding.

- Our review of the available references indicate that the mapped active fault nearest to the site is the Oceanside Section of the Newport-Inglewood-Rose Canyon Fault Zone, located at approximately 15.3 miles to the southwest of the subject site at the closest point, and described as capable of a Magnitude M_W 6.0 – 7.2 earthquake (SCEDC, 2013). Other mapped active faults near the subject site are the San Diego Trough Fault Zone, located at approximately 41 miles to the southwest of the site. The Julian Section of the Elsinore Fault Zone, located at approximately 20.3 miles to the east-northeast of the site at the closest point. This fault is described as capable of a magnitude 6.8 to 8.0 earthquake. As noted above the subject property is not within a State of California Fault Zone (CGS, 2018).

- Foundation support for the new convenience store building could be derived by utilizing a rigid shallow conventional continuous or spread foundation system embedded within the newly placed fill compacted to 92%. For the design of the structure, modulus of subgrade reaction (k_1) of 100 psi/in is recommended. An allowable bearing capacity of 2000 psf may be used for foundation bearing on properly compacted fill soil. The upper three (3) feet of subgrade within the building should be over excavated and recompacted to 92%. The excavation should also be extended five (5) feet outside the building footprint.

- From a geotechnical standpoint, we are of the opinion that the proposed construction/site grading is not expected to have an adverse impact on adjacent properties and vice versa.

- Groundwater was not encountered in our borings during field exploration on May 22, 2020.

Detailed descriptions of subsurface conditions, engineering analysis, and design recommendations are included in this report.

1.0 INTRODUCTION

Terradyne Engineering Inc., (TEI) conducted an onsite field exploration on May 20, 2020 that included drilling, logging and sampling of seven (7) hollow stem auger geotechnical borings to a maximum depth of 22.0 feet below existing elevations (referenced as B-1 through B-7 Appendix B). Four (4) of the geotechnical borings were converted to percolation-infiltration test borings (referenced as P-1 through P-4), for the proposed building development located at the northeast portion of the property.

This report describes: the evaluation performed; the results and opinions of the findings; and Terradyne Engineering Inc., (TEI) geotechnical recommendations for design and construction of the proposed structures.

2.0 PROPOSED CONSTRUCTION

Based on the available plans and information, the proposed construction will consist of a 4,088 square feet building structure, a four (4) island pump station and canopy, underground fuel tanks and related improvements (trash enclosure, asphalted drive and parking areas and landscaping. The site will also include a stormwater management system. This proposed building will be located generally in the northeastern portion of the property. Access is planned from the south, and east of the property. Figure 4, depicts the proposed improvements for the site.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our geotechnical investigation was based upon the planning information provided to us by the client, and consisted of field, laboratory and engineering evaluation of the site's subsurface soil and groundwater conditions and provide geotechnical engineering recommendations for the design and construction of the proposed building and associated improvements. Our scope of services includes the following:

1) Review of readily available documents pertinent to the subject site (References).

2) The excavation and sampling of seven (7) exploratory engineering borings to a maximum depth of 22.0-ft below existing ground elevations. The borings were excavated in the vicinity of the proposed building structure, canopy, pump islands underground storage tanks and parking areas. The soils encountered in the excavations were logged by our field Geologist and relatively undisturbed and bulk samples were collected at selected intervals in the various soil types to the maximum depth of the exploration.

3) The conversion of four (4) engineering borings into percolation-infiltration wells.

4) Percolation/infiltration testing.

5) Laboratory analysis of the collected samples.

6) Observation of the groundwater conditions during drilling operations.

7) Geotechnical analysis of the data and information obtained according to the project requirements; and

6) Preparation of this report presenting our findings, conclusions and recommendations, pertinent to the proposed building and paving sections for drive and parking areas.

The Scope of Services does not include an environmental assessment of the presence or absence of wetlands and/or hazardous or toxic materials in the soil, surface water, groundwater, or air, in the proximity of this site. Any statements in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions are strictly for the information of the client.

4.0 SITE DESCRIPTION

Based on review of the property details provided and aerial photographs of the site, the parcel under investigation consists of a developed property located to the northwest of the intersection of W. Mission Avenue and Rock Springs Road in the City of Escondido San Diego County California (APN Number 228-220-13 and 228-220-43). Site Topography grades gently to the south-southwest with site elevations ranging from approximately +647 to +652 feet above mean sea level. At time of our site investigation, the property was occupied by commercial type building along the southern portion of the property, a body shop with a one-story garage type building and a portable type building structure. The site was also occupied by several vehicles throughout the site. It is our understanding that the proposed structures are to be constructed at elevations similar to those currently existing at the subject site.

Review of the USGS Valley Center California 7.5-minute topographic quadrangle (Figure 1, Appendix A) and the Google Earth Pro®, database indicates the subject property is located on an alluvial valley. The subject property is approximately situated at 33.126786° north latitude and 17.098006° west longitude (Google Earth Pro®, 2020).

5.0 GEOTECHNICAL INVESTIGATION

5.1 Field Exploration

The field exploration by Terradyne Engineering Inc., was completed on May 20, 2020. Seven (7) hollow-stem auger borings were advanced to a maximum depth of 22.0 feet below ground surface. The locations of these exploratory borings (referenced as Boring B-1 through Boring B-7) are shown on the Boring Locations Map (Figure 3 and 4, Appendix A).

5.1.1 Engineering Borings

The exploratory boring excavations were advanced using a truck mounted drill rig with an 8-inch diameter hollow-stem auger. Drive samples recovered from all borings, were obtained using a Modified California Drive Sampler (2.5-inches inside diameter and 3-inches outside diameter) with thin brass liners, and a Standard Penetrometer (2-inches outside diameter and 1-3/8-inches inside diameter). The samplers were driven 12 to 18 inches into the soil by a 140-pound hammer free-falling for a distance of 30-inches.

Representative bulk and relatively undisturbed samples were taken of earth materials encountered in this field investigation. Recovered samples were placed in transport containers and returned to our laboratory for further classification and testing. The soils classifications listed in the excavation logs are a result of visual classification of soil with field moisture content. The classifications were assigned in accordance with ASTM D-2488: "Description of Soils (Visual-Manual Method)" and all applicable field soil-identification procedures described therein. These may or may not correspond precisely to those indicated by subsequent laboratory methods. Classifications, made in the field from auger cuttings and drive samples, were verified in the laboratory after further examination and testing of samples.

Conditions between boring locations may vary considerably and it should be expected that site conditions may or may not be precisely represented by any one of the borings. Soil deposition processes and topographic forming processes are such that soil and rock types and conditions may change in small vertical intervals and short horizontal distances. Stratification lines, as indicated on the Boring Logs, represent approximate changes in soil and rock composition, moisture and color, as approximated by field personnel logging the drilling operation and by the engineer in the laboratory from sample recovery data and by observation of the samples. Actual depths to changes in the field may differ from those indicated on the logs, or transitions may occur in a gradual manner and may not be sharply defined by a readily obvious line of demarcation.

All seven borings were backfilled with native soil on May 20, 2020. Earth materials encountered in this investigation consisted of fill and alluvial sediments, silty sands (see Figure C, Appendix A).

5.1.2 Percolation Borings and Testing

Terradyne Engineering Inc., (TEI) directed the drilling and conversion of four (4) borings into percolation-infiltration test borings, each located in the general area of potential storm water infiltration BMPs. The location of these percolation-infiltration test borings (referenced herein as ‘P-1’ to ‘P-4’) is shown on Figures 3 and 4 (Appendix A).

The percolation-infiltration borings were drilled with a truck mounted 8-inch hollow stem auger to the level of the base of proposed storm water infiltration BMPs. Field measurements were taken to confirm that the borings were excavated to approximately 8-inches in diameter. Logs of the percolation test borings are provided in Appendix B.

The boreholes were logged by a TEI geologist, who observed and recorded exposed soil cuttings and the boring conditions. Samples were obtained for identification and classification utilizing the Standard Penetration Test (‘SPT’, after ASTM D1586).

Once the test borings were drilled to the design depth, the borings were converted to percolation wells placing an approximately 2-inch layer of ¾-inch gravel on the bottom, then extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The ¾-inch gravel was used to fill the annular space around the perforated pipe to at least 12-inches below existing finish grade to minimize the potential of soil caving.

The percolation test holes were pre-soaked before testing and immediately prior to testing. The pre-soak process consisted of filling the hole twice with water before testing as recommended in *Appendix D of the Escondido Storm Water Design Manual (BMP Design Manual, 2016).*

Consecutive measurements indicated that more than 6 inches of water percolated in 25 minutes in all the infiltration test borings. Water levels were recorded every 10 minutes, for a minimum of 1 hour (minimum of 6 readings), or until the water percolation stabilized as recommended in *Appendix D of the Escondido Storm Water Design Manual (BMP Design Manual, 2016)*. After each reading, the water level was raised to close to the previous water level to maintain a near constant head before subsequent readings. Water level (depth) measurements were obtained from the top of the pipe. Table 1 (following page) abstracts the scope of the percolation testing.

Table 1. Summary of the Percolation Borings and Testing

Boring	Approx. Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elev. (feet, msl)	Subsurface Unit Tested[1,2]
P-1	+649.0	5.0	+644.0	Qoa
P-2	+648.0	5.0	+643.0	Qoa
P-3	+648.0	5.0	+643.0	Qoa
P-4	+648.0	5.0	+643.0	Qoa

Notes to Table 3-1:

1. 'Qoa' indicates older alluvium deposits.
2. All borings penetrated into older alluvium deposits (Qoa).

Upon completion of all work, the pvc pipe was removed from each percolation boring and backfilled with soil cuttings and match the existing surfacing.

5.2 Site Geology

Regional Geologic maps of the area (DMG, 1999, Geologic Map of the Valley Center 7.5' Quadrangle San Diego County, California), indicate that the subject site is located in an area underlain by Quaternary (Pleistocene) older flood plain deposits (Qoa), consisting of moderately well consolidated poorly sorted flood plain deposits. Figure 2 (Appendix A), reproduces geologic mapping of the site vicinity showing the site layout and the mapped location of the older alluvial deposits (Qoa).

5.3 Faulting and Seismicity

Our review of the available references indicate that the mapped active fault nearest to the site is the Oceanside Section of the Newport-Inglewood-Rose Canyon Fault Zone, located at approximately 15.3 miles to the southwest of the subject site at the closest point, and described as capable of a Magnitude M_W6.0 – 7.2 earthquake (SCEDC, 2013). Other mapped active faults near the subject site are the San Diego Trough Fault Zone, located at approximately 41 miles to the southwest of the site. The Julian Section of the Elsinore Fault Zone, located at approximately 20.3 miles to the east-northeast of the site at the closest point. This fault is described as capable of a magnitude 6.8 to 8.0 earthquake as noted above the subject property is not within a State of California Fault Zone (CGS, 2018). As noted above the subject property is not within a State of California Fault Zone (CGS, 2018).

5.4 General Subsurface Conditions

A field log was prepared for each of the borings. The logs include information concerning the boring method, samples attempted and recovered, and the presence of various materials (such as silt, clay, sand or gravel) and groundwater observations. It also includes an interpretation of the subsurface conditions between samples. Therefore, the log includes both factual and interpretive information. The final log and key to classification terms and symbols are included in Appendix B.

The soils underlying the site to the full depth explored may be grouped into two generalized strata each with similar physical and engineering properties. The lines on the log designating the interface between soil strata represent approximate boundaries. The transition between materials may be gradual. The soil stratigraphy at the boring location is presented in the Boring Logs.

The soils underlying the site were noted to consist mainly of 6-inches of undifferentiated topsoil/fill mantling the natural soils that consisted of (from youngest to oldest):

Artificial fill: The area of study was noted to be mantled by a layer of artificial fill, that was noted to extend to approximately 1.0 foot below the asphalt surface in vicinity of our exploratory excavations. The encountered fill was observed to generally consist of dark reddish-brown to reddish-brown silty sand (SM), this material was observed to have a loose to medium dense consistency and was noted to be moist.

Older Alluvium Qoa: Underlying the artificial fill, we encountered older alluvium deposits consisting of silty sand. This material was observed to be reddish-brown, medium dense to very dense (poorly to moderately cemented) and moist. This material was noted to extend beyond the bottoms of our exploratory excavations 22.0 feet. Table 2 below (following page), presents the main soils stratum encountered during our field exploration and the approximate depth range of each strata.

Table 2

Stratum	Depth Range (feet)	Remarks
ARTIFICAL FILL (af)	0 to 1.0’	No groundwater encountered
Alluvial Fan Deposits (Qa)	1.0’ to >22.0’	

The above description generally highlights the major soil stratification features and soil characteristics. The boring log should be consulted for specific information at the boring location. An excerpt from a regional geologic map is included in Appendix A, as Figure 2.

5.5 Regional Groundwater

Based on our review of the available references (CDWR), there are several state wells located in the vicinity of the subject site with two wells located at approximately 1.2 miles from the site. The first well located at approximately 1.2 miles to the northeast of the site (Station 331356N1170804W001), indicates that groundwater was measured in June of 1987 at an approximate depth of 13.24 feet from the surface; the site's surface elevation is recorded as +664.24 msl. A second well near the site is also located at approximately 1.2 miles to the southwest from the site. Groundwater depth was recorded also on June of 1987 at 8.52 feet from the surface. Surface elevation for this well is reported at approximately +637.24 msl. Site surface elevation is approximately +651 msl. This information suggests that at the site is possible that historic high groundwater may have been approximately at 10 feet from the surface on the same year.

Within the subject property, groundwater was not encountered during the geotechnical investigation conducted on May 20, 2020.

It should be noted that variations in subsurface water (including perched water zones and seepage) may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation and other factors that may not have been evident at the time of our subsurface exploration.

5.6. Laboratory Testing Program

In addition to field exploration, a supplemental laboratory testing program was conducted to determine additional pertinent engineering characteristics of the subsurface materials that are necessary to evaluate the soil parameters. These tests include:

1) Moisture Content & Density (ASTM D2216 & ASTM D2937)
2) Grain Size Distribution (ASTM D422)
3) One-Dimensional Consolidation (ASTM D2435)
4) Corrosion Potential (CT-417, CT-422, CT-532(643))
5) R-Value (Cal 301)

5.6.1 Moisture Content / In-Situ Density

The relationship between the moisture and density of undisturbed soil samples give qualitative information regarding the in-place soil strength characteristics and soil conditions. Results of our in-place moisture and density testing are presented on boring logs, (Appendix B).

The in-place moisture contents of the samples obtained from the upper 5 feet of soils in the vicinity of the proposed building pad areas at the subject site, were observed to range from 10.5 to 19.7 percent. Optimum moisture content was determined to be 8.5 percent. These results indicate a variability of moisture content of the upper soils throughout the site but generally above optimum moisture content and will require moisture conditioning during grading operations.

5.6.2 Maximum Dry Density/Optimum Moisture Content

The maximum dry density and optimum moisture content was obtained in the laboratory from a representative sample of the site soils. Results of our testing indicate that the tested soils yielded a maximum dry density of 131.8 at an optimum moisture content of 8.5 percent.

5.6.3 Grain Size Distribution Analysis

Representative samples of the subsurface materials were subjected to mechanical grain-size analysis by wet-sieving with U.S. Standard brass screens. The results of our grain size distribution analysis indicate that the sample tested contains 2.6 percent of gravel; 61.9 percent sand and 35.5 percent passing the 200 sieve. The percent passing the 200 sieve is presented in Appendix B (Laboratory Testing Results).

5.6.4 Consolidation Test

The consolidation test is used to estimate the consolidation/settlement or expansion that could potentially occur within a soil under specific loadings (such as may be imposed by buildings, walls, piers, etc.) and after saturation. The results of our testing are presented in Appendix B (Laboratory Testing Results).

5.6.5 Expansion Potential

Expansive soils change in volume with change in moisture content. Shrinking and swelling of the clays can cause heaving and cracking on retaining wall, slab-on-grade and structures founded on shallow foundations. The onsite fill and natural alluvial fan deposits soils are mainly sands and silty sands. We did not test for expansivity, our experience with these types of soils suggest that these soils will have a low to very low expansion potential. We recommend that an expansion test be performed on a representative sample when grading starts, and removals take place if significant signs of clays are encountered. Terradyne may provide additional recommendations if expansive soils are encountered during grading construction.

5.6.6 Soil Corrosion Potential

A near surface sample was tested to measure pH, soluble sulfate, soluble chloride and resistivity of the soil. The results are presented on Table No. 3.

Table No. 4

Sample Location/ Depth, (ft)	pH	Soil Resistivity (Ohm-cm)	Soluble Sulfate (PPM)	Soluble Chloride (PPM)
B-1/ 2.0 - 6.0	7.5	4,700	20	210

Sulfate Content

A representative near-surface soil sample was tested during our investigation for soluble sulfate content. The result of this test indicates a soluble sulfate content of (0.0210) percent by weight or negligible sulfate exposure. As such, the soils exposed are not expected to pose a significant potential for sulfate reaction with concrete. Per ACI 318-14 Table 19.3.1.1 the requirement of Exposure Category (S) and Class (S1) is applicable.

Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's pH level, electrical resistivity, and chloride content. As a screening for potentially corrosive soil, a representative soil sample was tested during our investigation to determine soil resistivity, chloride content, and pH level.

In general, soils are considered deleterious to foundation elements when the pH is less than 5.5 and considered to be corrosive and deleterious to metals. A pH of 7 is considered neutral; a pH <7 is considered acidic, a pH >7 is considered to be alkaline. Results of our testing yielded a pH of 7.5 which indicates that the tested soils are slightly alkaline, and that pH is not a significant factor in corrosivity to metals.

Soil with a chloride concentration greater than or equal to 500 ppm or more is considered corrosive to ferrous metals. The Chloride content of the sampled soils measured at approximately 210 ppm, which indicates that Chloride is not a major factor in corrosion to ferrous metals.

The soil resistivity measurement of the sample was approximately 4,700 ohm-cm, which indicates a **corrosive** soil to ferrous metals. Therefore, the corrosion protection measures are advisable to be considered in the design. It should be noted that Terradyne Engineering Inc. does not practice corrosion engineering and our assessment here should be construed as an aid to the owner or owner's representative. A corrosion specialist should be consulted for any specific design requirement.

Concrete

Laboratory test indicated that the subject site contains soil sulfate content in the negligible range (i.e., less than 150 part per million). However, it is recommended that concrete for all construction at the site utilize a widely available, Type-II Portland cement with a maximum 0.50 water/cement ratio and should comply with all the requirements of governing agencies and current applicable Code. The minimum compressive strength of concrete shall be a minimum of 2500 psi at 28 days and maximum slump during placement shall be five inches. The minimum concrete cover should be 1.5-inches. Final selection of the appropriate concrete design should be made by the project structural engineer based on the local laws and ordinances, and desired level of conservatism.

6.0 CONCLUSIONS

6.1 General

Based on the results of our study, it is Terradyne's opinion that the proposed buildings can be constructed as planned, provided that the recommendations presented herein are implemented. It is our opinion that the on-site soils (when properly processed and recompacted as recommended herein) should provide adequate foundation support for the proposed structure.

6.2 Regional Groundwater

Review of the available references (CADWR, 2020), indicate that two state wells located at approximately 1.2 miles from the site one to the northeast and one to the southwest. groundwater was measured at approximately 13.24 and 8.52 feet below ground surface in 1987. These depths suggest that historic groundwater at the site may have been at approximately 10 feet from surface elevation.

Groundwater seepage was not observed during drilling operations. Groundwater levels will fluctuate with seasonal climatic variations and changes in the land use. Soils with low permeability may require several days for groundwater to enter and stabilize in the boreholes. It is not unusual to encounter shallow groundwater during or after periods of rainfall. Surface water tends to percolate through the surface until it encounters a relatively imperious layer.

It should be noted that variations in subsurface water (including perched water zones and seepage) may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation and other factors that may not have been evident at the time of our subsurface exploration.

6.3 Moisture Content

The in-place moisture contents of the samples obtained from the upper 5 feet of soils in the vicinity of the proposed building pad areas at the subject site, were observed to range from 10.5 to 19.7 percent. Optimum moisture content was determined to be 8.5 percent. These results indicate a variability of moisture content of the upper soils throughout the site but generally over optimum moisture content and will require moisture conditioning during grading operations.

6.4 Soil Compressibility

The encountered artificial fills and upper alluvial deposits were noted to be compressible from an engineering standpoint. As such, they are not recommended for foundation or slab support in their current condition. The natural soils were observed to be denser at depth, have a superior bearing capacity and may be used to support future secondary compacted fills.

6.5 Regional and Local Faulting

The principal seismic considerations for improvements at the subject site are surface rupture of fault traces, damage caused by ground shaking during a seismic event, and seismically-induced ground settlement. The potential for any or all of these hazards depends upon the recency of fault activity and the proximity of nearby faults to the subject site. The possibility of damage due to ground rupture is considered unlikely since no active faults are known to cross the site and no evidence of active faulting was noted during our investigation. Our review of the proper literature (CGS 2018) indicates that the subject site lies outside the present Earthquake Fault Zones, which are described in the Alquist-Priolo Earthquake Fault Zoning Act as being placed along active faults.

Our review of the available references indicate that the mapped active fault nearest to the site is the Oceanside Section of the Newport-Inglewood-Rose Canyon Fault Zone, located at approximately 15.3 miles to the southwest of the subject site at the closest point, and described as capable of a Magnitude M_W 6.0 – 7.2 earthquake (SCEDC, 2013). Other nearby active faults are described in Section 5.3 above. As previously stated, the subject property is not within a State of California Fault Zone (CGS, 2018).

6.6 Seismic Design Parameters

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in California. Design of the proposed improvements in accordance with current CBC requirements is intended to reduce the impact of seismic shaking on the proposed improvements. Recommended seismic design acceleration parameters in accordance with the new 2019 California Building Code (CBC) and ASCE 7-16 are presented in Table 4.

Table 5

CBC DESIGN RESPONSE SPECTRUM PARAMETERS	
Latitude	33.126804 degrees north
Longitude	-117.097974 degrees west
Site Class	D – Stiff Soil
MCE_R Ground Motion, Ss (period=0.2s)	0.898 g
MCE_R Ground Motion, S_1(period=1.0s)	0.329 g
Site Amplification Factor at 0.2s, F_a	1.141
Site Amplification Factor at 1.0s, F_v	N/A
Site-modified Spectral Acceleration Value, S_{MS}	1.024 g
Site-modified Spectral Acceleration Value, S_{M1}	N/A
Numeric Seismic Design Value at 0.2s SA, S_{DS}	0.683 g
Numeric Seismic Design Value at 1.0s SA, S_{D1}	N/A
Peak Ground Acceleration	0.388 g
Site Modification Peak Ground Acceleration, PGA_M	0.47g

Note: Ground motion hazard analysis may be required, see ASCE/SEI 7-16 Section 11.4.8
ASCE 7 Hazards Report is attached in Appendix D. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism.

6.7 Seismic Hazards and Other Considerations

Earthquakes or aftershocks may cause secondary ground failures. Ground failures are caused by soil losing its structural integrity. Examples of seismically-induced ground failures are liquefaction, lateral spreading, ground lurching, and subsidence. *Liquefaction* (the rapid transformation of soil to a fluid-like state) affects loose saturated sands. *Lateral spreading* is the horizontal movement of loose, unconfined sedimentary and fill deposits during seismic activity. *Ground-lurching* is the horizontal movement of soil, sediments, or fill located on relatively steep embankments or scarps as a result of seismic activity, forming irregular ground surface cracks. The potential for lateral spreading or lurching is highest in areas underlain by soft, saturated materials, especially where bordered by steep banks or adjacent hard ground. *Subsidence* is vertical downward movement of the ground surface. The review of the available references indicates that the subject site is located in an area underlain by granular and semi-consolidated to consolidated soils. The seismic hazard most likely to impact the site is ground shaking due to a large earthquake on one of the major active regional faults. Because of the proximity to the subject site and the maximum credible event, it appears that Oceanside Section of the Newport-Inglewood-Rose Canyon Fault Zone, located at approximately 15.3 miles to the southwest of the site is most likely to affect the site with severe ground shaking should a significant earthquake occur along this fault.

6.8.1 Liquefaction

Liquefaction of soils can be caused by strong vibratory motion in response to earthquakes. Both research and historical data indicate that loose mostly fine sands or predominantly granular soils are susceptible to liquefaction, while the stability of rock is not as adversely affected by vibratory motion. Liquefaction is generally known to occur primarily in cohesionless silt, sand, and fine-grained gravel deposits of Holocene to late Pleistocene age in areas where the groundwater is shallower than about 50 feet (DMG Special Publication 117A). Is also a function of relative density, soil type and probable intensity and duration of ground shaking. The site is underlain by relatively dense to very dense old alluvium deposits and is not located in an Earthquake Fault Zone. As such, it is our opinion that the potential for liquefaction at the site is very low.

6.8.2 Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. The subject site is generally flat and because of the potential for liquefaction is very low and the underlying soils were noted to be dense to very dense, it is our opinion that the potential for liquefaction induced lateral spreading is also low.

6.8.3 Ground Lurching

As noted previously, ground-lurching is the horizontal movement of soil, sediments, or fill located on relatively steep embankments or scarps as a result of seismic activity, forming irregular ground surface cracks. The subject site is relatively flat, and, as such, the potential for ground lurching is considered to be very low.

6.8.4 Seismic Settlement

Seismic settlement occurs when loose to medium-dense, granular soils consolidate during ground shaking. The materials which underlie the subject site were observed to be primarily dense to very dense sands and silty sands. As such, it appears that the soils underlying the subject site possess a relatively **low potential** for seismically induced settlement in their present condition.

6.8.5 Seismic Induced Landslides

The site topography and the general topography of the area is relatively flat, as such the potential for seismic induced landslides is none.

6.8.6 Design Earthquake Magnitude

The review of readily available references pertinent to the subject site indicates that structures should be designed to resist moderate earthquakes with a low probability of structural damage. Such design shall resist major or severe earthquakes with some structural damage, but with a low probability of collapse.

The moderate and major earthquakes have been interpreted to represent the maximum probable and maximum credible earthquakes, respectively. The maximum credible earthquake is defined as the largest event that a specific fault is theoretically capable of producing within the presently known tectonic framework and is established based on mechanical relationships of the fault and fault mechanisms and does not consider rate of recurrence or probability of occurrence. The seismic design parameters at the site were obtained using the USGS seismic design maps site. The subject site is located at latitude of 33.126804° north and longitude 117.097974°. The peak horizontal ground acceleration at the site was calculated to be 0.388g.

6.9 Other Potential Site Hazards

6.9.1 Flood

The project site is located within FEMA Map Number 06073C0813G, effective on 05/16/2012. This map shows that the project site is located within FEMA-designated Flood “Zone X” described as “zone of minimal risk of flood hazard” (FEMA, 2016). Zone X is designated as Areas determined to be outside the 0.2% annual chance floodplain. This statement should be verified with the City of Escondido and/or County of San Diego.

7.0 RECOMMENDATIONS

7.1 General

Based on our geotechnical study at the site, our review of readily available reports and literature pertinent to the site (Attached), and our understanding of the proposed final grades, it is our opinion that development and/or improvement of the site is feasible from a geotechnical standpoint, provided the conclusions and recommendations included in this report are properly incorporated into the design and construction of any proposed structures. There appear to be no significant geotechnical constraints on-site that cannot be mitigated by proper planning, design, and utilization of sound construction practices. The engineering properties of the underlying materials, surface drainage, and anticipated degree of seismic risk offer conditions comparable to the other sites surrounding the subject project. The following sections provide geotechnical recommendations that should be incorporated into the design of the proposed improvements at the site.

7.2 Earthwork

Grading and earthwork should be performed in accordance with the recommendations presented herein and the 2019 California Building Code (CBC, 2019. In case of conflict, the following recommendations shall supersede those presented in the 2019 California Building Code (CBC, 2019).

7.2.1 General

Grading should conform to the guidelines presented in the 2019 California Building Code (CBC, 2019), as well as the requirements of the City of Escondido and County of San Diego.

During earthwork construction, removals and reprocessing of fill materials, as well as general grading procedures of the contractor should be observed, and the fill placed selectively tested by representatives of the geotechnical engineer. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the geotechnical engineer and if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

The recommendations presented herein have been completed using the information provided to us regarding site development. If information concerning the proposed development is revised, or any changes in the design and location of the proposed property modified or approved in writing by this office.

7.2.2. Site Preparation

Prior to earthwork or construction operations, the site should be cleared of surface and subsurface obstructions and stripped of any vegetation in the areas proposed for development. Removed vegetation and debris should then be properly disposed of off-site. Holes resulting from removal of buried obstructions which extend below finish site grades should be backfilled with suitable fill soils compacted to a minimum 92 percent relative compaction (based on ASTM Test Method D1557).

7.2.3. Removal of Unsuitable Soils

As noted above, the existing fill soils and upper alluvium soils are considered to be potentially compressible in their current condition. As a result, we recommend the reprocessing of these existing soils in all areas to receive building additions or new buildings (where not anticipated to be removed during proposed grading operations). Based on the results of our subsurface investigation, it is anticipated that the removal depths in the vicinity of the proposed buildings will be a minimum of **3 feet** below existing grade elevations in the areas of the proposed building structure. For the area of the proposed pump island we recommend removals of 2-feet below existing grades. The removals should extend to a minimum distance of 5 feet outside the building footprint. Following removal of the upper soils, the bottom of the excavation(s) should be observed and approved by a representative of this office to verify that these potentially compressible materials have been properly removed.

Prior to fill placement, all areas to receive fill and/or other surface improvements, shall be scarified to a minimum depth of 8 inches below removal grade elevations, be moisture conditioned to **2 percent over optimum** moisture content and compacted to minimum **92** percent relative compaction, based on ASTM Test Method D1557. After this procedure is completed, backfill of the removal excavation should take place by moisture conditioning the removed soils prior to placement to at least optimum to 2 percent over optimum moisture content and recompaction of these soils to a minimum **92** percent relative compaction (based on ASTM Test Method D1557). These operations should be performed under the observation and testing of a representative of this office. It should be understood that based on the observations of our field representative, localized deeper or shallower removals may be recommended. Any removed soils shall be moisture conditioned as necessary to achieve a moisture content of at least optimum to 2 percent over optimum moisture content and be recompacted to a minimum **92** percent relative compaction (based on ASTM Test Method D1557). This earthwork should extend a minimum of 5 feet beyond the proposed footing limits.

7.2.4. Fill Placement and Compaction

If necessary, the on-site soils are suitable for reuse as compacted fill, provided they are free of organic materials and debris and material larger than 6 inches in diameter. Should import soils be utilized for near-surface fills, these soils should be predominately granular, possess a low or very low expansion potential, and be approved by the geotechnical engineer prior to their transportation to the site. Lift thicknesses will be dependent upon the size and type of equipment used. <u>In general, fill should be placed in uniform lifts not exceeding 8 inches</u>. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant.

We recommend that if encountered, oversize materials (materials greater than 6 inches in maximum dimension) be removed from the upper 4 feet of fill.

7.2.5. Trench Excavations Underground Tanks and Backfill

Utility trenches and underground tank excavation are anticipated to be excavated with moderate effort using conventional construction equipment in good operating condition. Deep trenches may require the use of heavy equipment operations. The encountered soils at the site consisted of medium dense to dense, moderately to well consolidated sands and silty sands. These soils may be subject to collapse and or cave-ins. To satisfy OSHA requirements and for workmen's safety, it will be necessary to shore excavations deeper than 5 feet. The proposed trenches deeper than 5 feet may also be laid back in a 1:1 horizontal to vertical (45 degrees). Because of the potential for shallow groundwater during a wet year, the underground fuel tanks may require to be anchored to minimize or prevent buoyancy.

The on-site soils may be used as trench backfill provided they are screened of rock sizes over 6 inches in maximum dimension and organic matter. Trench backfill should be compacted in uniform lifts (not exceeding 8 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (based on ASTM D1557).

7.2.6 Temporary Drainage and Excavation Measures

Temporary drainage provisions should be established to minimize water runoff into construction areas. If standing water does accumulate, it should be removed by pumping as soon as possible. Adequate protection against sloughing of soils should be provided for workers and inspectors entering the excavations. This protection should meet OSHA and other applicable building codes. Temporary excavations that could potential be a safety hazard are not anticipated for this project. However, the following recommendations should be followed based on anticipated and/or exposed conditions for continuous foundation and trench excavations:

- Vertical cuts (if proposed) exposing artificial fill shall have a maximum height of 4 feet. Upper portions of excavations that are deeper than 4 feet shall be laid back at a 1.5:1 (horizontal: vertical) slope gradient.

- Excavations above these maximum allowable heights and excavations that expose creep-prone soils or unsupported bedding planes should be trimmed back to the bedding plane / slip angle or shored.

- Excavation walls in sands and dry soils should always be kept moist (but not saturated).

All excavations deeper than 5 feet should conform to safety requirements for excavations as set forth in the State Construction Safety Orders enforced by the State Division of Industrial Safety, CAL OSHA.

7.2.7. Shrinkage and Bulking

Several factors will impact earthwork balancing on the site, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compaction effort achieved during construction. For planning purposes, the shrinkage factor is estimated to be on the order of 10 to 15 percent for the onsite natural soils to be utilized as fill. This shrinkage factor may vary with methods employed by the contractor. Subsidence is estimated to be on the order of 0.1 feet. Losses from site clearing and removal of existing site improvements may affect earthwork quantity calculation and should be considered.

The previous estimates are intended as an aid for the project engineers in estimating earthwork quantities. It is recommended that the site development be planned to include an area that could be raised or lowered to accommodate final site balancing.

7.2.8 Control Testing and Field Observation

Subgrade preparation and structural fill placement should be monitored by the project geotechnical engineer or his representative. Field-tests for moisture content and relative compaction of the fill soils shall be performed by Terradyne, Inc. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction. Any areas not meeting the required compaction should be re-compacted and retested until compliance is met.

7.3 Foundations and Slab Design

Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that soils exposed at finish pad grade will have a low potential for expansion. These recommendations may be verified by performing additional expansion tests after grading is completed. Localized areas of higher expansion may be possible.

7.3.1 Foundation Design

All proposed building and non-building improvements that are anticipated to constitute a structural load may be supported by an appropriate foundation system designed by the project structural engineer in accordance with the guidelines of the Uniform Building Code and/or all applicable local building codes. Footings adequately founded in firm natural soils or properly compacted fill soils should be a minimum 12 inches wide by 18 inches deep for a one-story building structure and 24 inches deep by 15 inches wide for a two-story building or in accordance with the project structural engineer requirements. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces of the requirements of 2019 CBC. A coefficient of friction of 0.35 of dead load may be used. At these dimensions, footings adequately founded in properly compacted fill soil may be designed for an allowable soil bearing value of **2000** pounds per square foot. These values may be increased by one-third for loads of short duration including wind or seismic forces. The allowable bearing value may be increased by 250 pounds per square foot per foot increase in depth or width to a maximum of 3500 psf. Foundations should be properly reinforced in accordance with the project structural engineer's recommendations. Minimum reinforcement shall consist of two No. 4 rebar at the top and two No. 4 rebar at the bottom of the footing or in accordance with the structural engineers’ requirements, whichever is greater. We estimate that the total and differential settlement for the proposed improvements will be on the order of 1-inch and approximately ½ -inch between structural elements.

All foundation excavations should be observed and tested by a representative of Terradyne Engineering Inc., prior to placement of steel and concrete.

Table 6		
Earth Material and Foundation Design Parameters		
Earth Material Parameters	Foundation Bearing Material	Certified Fill/Approved Soil
	Foundation Bearing Pressures [3]	2,000 psf
	Coefficient of Friction [1]	0.35
	Passive Earth Pressure (EFP) [3]	200 pcf
	Maximum Passive Earth Pressure	3,500 psf
Continuous Footing Design	Minimum Width	12-inches for one-story 15-inches for two-story
	Min. Embedment Depth into Bearing Material [2]	18-inches for one-story 24-inches for two-story
	Minimum Reinforcement	2 No.4 Rebars at Top and 2 No.4 Rebars at Bottom
Independent Pad Design	Minimum Foundation Dimensions	24" x 24" square
	Min. Embedment Depth into Bearing Material [2]	24-inches

Notes:
[1] When combining frictional resistance and passive pressure, the passive pressure component should be reduced by one-third.
[2] Foundation depths subject to increase per the project structural engineer's design.
[3] One-third increases on the bearing and passive pressures for wind and seismic loads are allowed.

7.3.2 Concrete Slabs

Interior concrete slabs should have a minimum thickness of 4.5 inches and be underlain by a 10-mil visqueen moisture barrier, underlain with a 2-inch layer of clean sand (sand equivalent of at least 30). The visqueen moisture barrier should be overlain by a 2-inch layer of clean sand to aid in concrete curing. All slabs should be constructed with preferred minimum reinforcement consisting of No. 3 bars placed mid-height in the slab and spaced on 18-inch centers in both directions. Welded wire mesh is not an acceptable alternative. Crack control joints should be provided in accordance with the recommendations of the project structural engineer. For the proposed site, a modulus of subgrade reaction k_1 of 100 psi/in is recommended.

Exterior concrete flatwork (sidewalks, etc.) should have a minimum thickness of 4 inches, be underlain by a 2-inch layer of clean sand and reinforced with a minimum. No. 3 bars placed mid-height in the slab and spaced on 18-inch centers in both directions. Care should be taken by the contractor to ensure that the reinforcement is placed and maintained at slab midheight. We recommend that crack control joints for exterior flatwork be provided with a minimum spacing of 12 feet and a maximum of 15 feet, or in accordance with the structural engineer's recommendations. We also recommend that every third control joint be converted to an expansion joint.

Some slab cracking due to shrinkage should be anticipated. The potential for this slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if crack-sensitive flooring is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

7.3.3 Moistening of Foundation Soils

Footing excavations and slab subgrades should be thoroughly moistened prior to placement of concrete.

7.3.4 Cement Type

Our laboratory testing of a representative sample of the near surface indicated a **negligible** concentration of soluble sulfates. Based on the guidelines presented in the current edition of the Uniform Building Code, a minimum Type II cement may be utilized in concrete that will be in direct contact with the near-surface soils. Based on the guidelines presented in the current edition of the International Building Code (IBC 2018, CBC 2019). Terradyne Engineering Inc. can provide additional recommendations.

7.4 Retaining Walls Lateral Earth Pressures (if proposed)

For design purposes, the following lateral earth pressure values for level and free-draining backfill are recommended for retaining walls (**if proposed**) backfilled with on-site soils, and for those backfilled with select soils (possessing an internal friction angle of at least 30 degrees and extending at least 0.5H from the upslope face of the wall, where H is the wall height).

Table 7 Equivalent Fluid Weight (pcf)

Conditions	On-Site Backfill	Select Backfill (PHI≥30 Degrees)
Active	33	35
At-Rest	70	55
Passive (Fill Soils)	250	350

Unrestrained (yielding) cantilever walls should be designed for an active equivalent pressure value provided above. In the design of walls restrained from movement at the top (nonyielding), such as basement walls or re-entrant corners, the at-rest pressures should be used. For areas of re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner. The above values assume backfill soils will have a low expansion potential and free-draining condition. If conditions other than those covered herein are anticipated, the equivalent fluid pressures should be provided on an individual basis by the geotechnical engineer. Retaining wall structures should be provided with appropriate drainage. Typical drainage design is illustrated in Appendix C. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM Test Method D1557). Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight of 8 feet.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, a friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

7.5 Wall Backdrain and Waterproofing

In order to reduce the potential for water build-up and an increase in hydrostatic pressure behind a retaining wall (if proposed), a wall backdrain shall be installed (see attached Figure 1). The wall backdrain shall consist of a 4-inch minimum diameter Schedule 40 PVC perforated pipe (perforations oriented down) inclined at a minimum 1-percent gradient. The backdrain pipe shall be encased in a 1-foot wide by 1-foot tall envelope (minimum) of 3/4" to 1-1/2 inch crushed rock, wrapped in a filter fabric envelope consisting of Mirafi 140N or an approved equivalent with a minimum 6-inch fabric overlap. The new wall backdrain shall either be discharged via 4-inch diameter Schedule 40 PVC pipes. The water should be directed into a suitable surface drainage system or catch basin. If a catch basin is used the collected water should be pumped out by means of a sump pump to a suitable drainage system. The wall backdrain system construction (clean bottom, subdrain pipe installation gravel wrap and backfill), shall also be observed and confirmed by a representative of this office to document compliance with these recommendations.

In order to reduce the infiltration of water through the wall face, we recommend the back of the new retaining wall be cleaned, dried, prepared, and waterproof sealed (with products such as Vulkem 201, or BT Type-2 by Pacific Polymer, and Amaco PB-4 (green foam board) or approved equivalent) installed in accordance with the manufacturer's recommendations. A minimum of a two-foot-thick compacted fill cap shall be placed over the gravel backfill to reduce the potential for infiltration of moisture into the subsurface. For below grade walls, a concrete swale may be constructed to direct flow of water away from the surface and prevent ponding and soil saturation.

One of the most common post-construction problems is moisture affecting below grade walls. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Special care should be taken in the design and installation of waterproofing to avoid these types of moisture problems, or actual water seepage into the structure through any shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints.

The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

The backfill behind the wall should be drained properly. The drainage system should consist of a 4-inch PVC down perforated drainpipe located near the bottom of the wall. The drain collects the water that enters the backfill and this may be disposed of through solid pipe outlets along the base of the wall (See Appendix A, Figure F).

7.6 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and a preliminary design R-value of 40. The design R-value was chosen based on laboratory testing of a representative sample and considering the sandy soil conditions at near the surface. The preliminary flexible pavement sections may consist of the following for the Traffic Indices (TI) indicated and the calculations are in the Appendix B. The Asphalt Cement (AC) and Class II Aggregate Base (AB) thickness are presented below for different Traffic Indices. Final pavement design where needed should be based on the Traffic Index determined by the project civil engineer.

Table 8 Pavement Sections

Traffic Index (TI)	Minimum Section Thickness (inches)		
	Asphalt Concrete (AC)	Class II Aggregate Base* (AB)	Compacted Subgrade to 95%
5 or less (auto parking)	3.0	4.0	12.0-inches
7 (truck access)	4.0	6.0	12.0-inches

*Caltrans Class 2 aggregate base, minimum R-value of 78

The final pavement design also should be verified during actual site grading and the above sections may be revised accordingly per actual representative R-value. The minimum required compaction of aggregate base and the subgrade is 95% of maximum dry density.

In areas where rigid concrete pavement is planned, at a minimum, concrete should be 4000 psi with fiber mesh, 5 inches thick in parking areas (light duty) and 6 inches thick (heavy duty) in loading areas. Concrete paving to be placed over a minimum 4-inch thick granular base on prepared subgrade soil. Reinforcement should be specified by the structural engineer but should be a minimum of #3 rebar at 18 inches on center each way. The PCC pavement sections should be provided with crack-control joints spaced no more than 14 feet on center each way. If saw cuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that sections be as nearly square as possible.

8.0 SITE DRAINAGE AND MAINTENANCE

Final drainage is important for the performance of the proposed construction. Landscaping, plumbing, and downspout drainage is also important. It is vital that all roof drainage be transported away from the building so that water does not pond around it, which can result in a soil volume change underneath the building. Plumbing leaks (if any) should be repaired as soon as possible in order to minimize the magnitude of a moisture change under the slab. Large trees and shrubs should not be planted in the immediate vicinity of the structures, since root systems can cause a substantial reduction in soil volume in the vicinity of the trees during dry periods.

Adequate drainage should be provided to reduce seasonal variations in moisture content of foundation soils. All pavement and sidewalks within 10-feet of the structures should be sloped away from the structures to prevent ponding of water around the foundations. Final grades within 10-feet of the structure should be adjusted to slope away from structures preferably at a minimum slope of 2 percent. Maintaining positive surface drainage throughout the life of the structure is essential.

In areas with pavement or sidewalks adjacent to the new structure, a positive seal must be provided and maintained between the structures and the pavement or sidewalk to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and flat-work is not uncommon. Maximum grades practical should be used for paving and flatwork to prevent areas where water can pond. In addition, allowances in final grades should take into consideration post construction movement of flatwork, particularly if such movement would be critical. Normal maintenance should include inspection of all joints in paving and sidewalks, etc. as well as re-sealing where necessary.

Trench backfill for utilities should be properly placed and compacted, as outlined in this report, and in accordance with the requirements of local City, County and/or State Standards. Since granular bedding backfill is used for most utility lines, the backfilled trench should be prevented from becoming a conduit and allowing an access for surface or subsurface water to travel toward the new structures. Concrete cut-off collars or clay plugs should be provided where utility lines cross building lines to prevent water from traveling in the trench backfill and entering beneath the structures.

9.0 STORMWATER INFILTRATION

Percolation testing for design of stormwater infiltration BMPs was completed after guidance contained in the *Appendix D of the Escondido Storm Water Design Manual (BMP Design Manual, 2016).*

Based upon the indications of the field exploration and laboratory testing reported herein, Terradyne Engineering Ing., has evaluated the site as summarized below.

- Based on our review and site reconnaissance it appears that there is no visible evidence of areas of contaminated soil or contaminated groundwater known to be within the site or the immediate surroundings of the site.

- There are no 'brownfield' sites within 1,000 feet of the site.

- There are no slopes steeper than 25% within the area of the proposed BMP.

- There are no known water supply wells, permitted UST's (GeoTracker, 2016) or permitted graywater systems within 1,000 feet of locations contemplated for retention/biofiltration/BMPs.

Section 5.1.2 provides a description of the field procedure performed to complete the testing. Figures 3 and 4 (Appendix A) depicts the location of the testing. This section provides the results of that testing and related recommendations for management of stormwater in conformance with the *BMP Design Manual*.

As is well-established by the *BMP Design Manual*, the feasibility of stormwater infiltration is principally dependent on geotechnical and hydrogeologic conditions at the project site. This section provides Terradyne Engineering Inc. assessment of the feasibility for stormwater infiltration BMPs utilizing the information developed by the field exploration described in Section 5.1.2, as well as other elements of the site assessment.

9.1 Soil and Geologic Conditions

The engineering borings and percolation tests borings completed for this assessment disclose the sequence of artificial fill and Quaternary alluvium deposits described below.

- <u>Unit 1, Artificial Fill (Qaf).</u> An approximately 1.0-foot-thick layer of undocumented fill was encountered within all of our exploratory borings. The fill may be deeper or shallower at other locations. The fill is of loose to medium dense consistency, comprised of sand and silty sand.

- Unit 2, Quaternary Older Alluvium Deposits is the primary sedimentary deposit that underlies the site. These deposits were observed to consist of medium dense to very dense sand and silty sand. The sands were observed to range from fine to medium grained.

  The United States Department of Agriculture Natural Resources Conservation Service (USDA NRCS) provides soil data and information for the entire United States. Data available from the USDA NRCS include a description of the soils, their location on the landscape, and tables that show soil properties and limitations affecting various uses.

  Review of USDA NRCS data indicates that 100 percent of the subject site is underlain by PeC, “Placentia Sandy Loam”; The profiles are described as alluvial fan and is classified as hydrologic Soil Unit Group C, described as “Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. (USDA 2007). See Appendix C for Soil Resource Report.

9.2 Percolation/Infiltration Testing

During our subsurface exploration at the site, Terradyne conducted percolation testing in our exploratory borings P-1 through P-4, which were drilled within the proposed BMP area (see Figures 3 and 4 in Appendix A for locations), to the south and southwest of the property. Our testing was performed at an approximate depth of 5 feet below the existing ground surface. Upon conclusion of testing, the perforated pipe was removed, and the test excavation was backfilled.

We note that a soil profile’s percolation rate is not the same as its infiltration rate. Therefore, the measured/calculated field percolation rate was converted to an estimated infiltration rate utilizing a reduction factor known as the Porchet method (Ritzema, 1974). Results of percolation testing and infiltration rate are presented in the following table, Table 9.

Table 9 Percolation-Infiltration Test Results

Test	Depth of Test (feet below existing grade)	Infiltration Rate (in/hour)	Infiltration Rate (in/hour, F=3*) (in./hr)
P-1	5.0	2.094	0.698
P-2	5.0	1.021	0.34
P-3	5.0	0.783	0.261
P-4	5.0	1.193	0.398

“F” Indicates Factor of Safety

9.3 Design Infiltration Rate

As may be seen by review of Table 9, the infiltration rates measured in the area of study (proposed BMP location) vary but are higher than 0.5 in/hr and are considered to be suitable for infiltration from a Geotechnical perspective. In consideration of the nature and variability of infiltration materials, as well as the natural tendency of infiltration structures to become less efficient with time, the infiltration rates presented in Table 9 should be modified to use at least a factor of safety (F) of F= 3.0 for design purposes. Measured infiltration across the proposed BMP location range from I = 0.261 to I = 0.698 (inches per hour using a factor of safety (F) of F= 3.0), which indicates that the tested areas present infiltration rates that range from below 0.5 in/hr (Tests P-2 through P-4) and above 0.5 in/hr (Test P-1).

9.4 Suitability of the Site for Stormwater Infiltration

It is TEI's opinion based on the results of the field study, that the area of the proposed infiltration system **is not suitable** for stormwater infiltration BMPs. This judgment is based upon the factors listed below.

1. Relatively Low Infiltration Rate. The design infiltration rate determined from the site-specific percolation testing in the property is less than 0.5 inches per hour in three of the four sites tested.

2. Relatively High Historic Groundwater Conditions. Review of the available references (CADWR, 2020), indicate that two state wells located at approximately 1.2 miles from the site: one to the northeast and one to the southwest. groundwater was measured at approximately 13.24 and 8.52 feet respectively below ground surface in 1987. These depths suggest that historic groundwater at the site may have been at approximately 10 feet from surface elevation in the same year since the site is mapped very close to the midpoint between those two wells. As such, there is not enough distance (minimum 10 feet) between the base of the proposed BMP system to the possible high historic groundwater to meet the requirements of the City of Escondido BMP Manual.

In consideration of the site evaluation- it is TEI's opinion that the site **is not suitable** for application of full stormwater infiltration BMP's, and alternative methods of retention systems should be considered.

10.0 REVIEW and SERVICES

All soil, geologic, and structural aspects of the proposed Project are subject to the review and approval of the governing agency(s). It should be recognized that the governing agency(s) can dictate the manner in which the project proceeds. They could approve or deny any aspect of the proposed improvements and/or could dictate which foundation and grading options are acceptable.

10.1 Plan Review

Upon completion, we should review the project plans and specifications to check that they conform to the intent of our recommendations.

10.2 Additional Geotechnical Services

Additional geotechnical services will be required subsequent to the investigation report. Additional fees will accrue for the additional services. The additional fees will depend on the scope of the additional work. A separate proposal and agreement will be prepared for the additional services. The following services are considered additional services.

- Response to questions from the reviewing agencies.
- Once plans for the proposed development are completed, the geotechnical consultant will need to review and approve the drawings.
- During construction, the geotechnical consultant will need to observe and test earthwork and observe foundation excavations for the proposed development.

11.0 LIMITATIONS

The analysis and recommendations submitted in this report are based upon the data obtained from seven (7) borings drilled at the site.

This report may not reflect the exact variations of the soil conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. If variations appear evident, it will be necessary to re-evaluate our recommendations after performing on-site observations and tests to establish the engineering significance of these variations. The project geotechnical engineer should review the final plans for the proposed structures so that he may determine if changes in the foundation recommendations are required. The project geotechnical engineer declares that the findings, recommendations, or professional advice contained herein have been made and this report prepared in accordance with generally accepted professional engineering practice in the fields of geotechnical engineering and engineering geology. No other warranties are implied or expressed.

This report is valid until site conditions change due to disturbance (cut and fill grading) or changes to nearby drainage conditions or two (2) years from the date of this report, whichever occurs first. Beyond this expiration date, Terradyne shall not accept any liability associated with the engineering recommendations in the report, particularly if the site conditions have changed. If this report is desired for use for design purposes beyond this expiration date, we highly recommend an update of this report with the possibility of drilling additional borings so that we can verify the subsurface conditions and validate the recommendations in this report.

This report has been prepared for the exclusive use of the owner, owner's representative and the design team for the specific application to the proposed 7-11 convenience store and gasoline station located 900 W. Mission Avenue in the City of Escondido, San Diego County, California.

REFERENCES

ASCE Hazard Tool https://asce7hazardtool.online/

American Society of Civil Engineers (ASCE), Minimum Design Loads for Buildings and other Structures, Standard 7-16

American Society for Testing and Materials (ASTM), 2011, Annual Book of ASTM Standards, Volume 04.08, Construction: Soil and Rock (I), Standards D 420 - D 5876.

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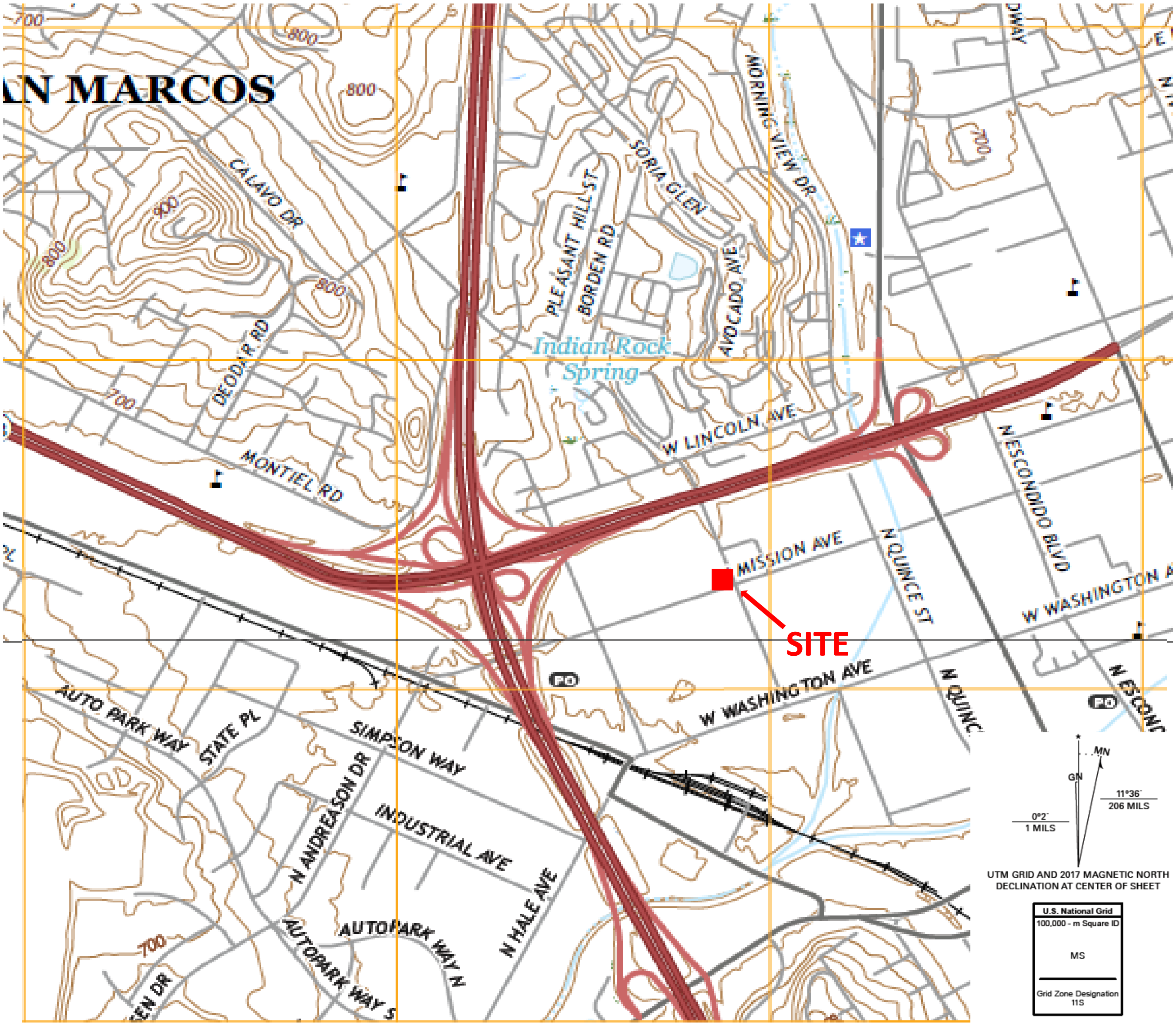
APPENDIX A



SCALE 1:24 000

1 0.5 0 KILOMETERS 1 2

1000 500 0 METERS 1000 2000

1 0.5 0 1

MILES

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

FEET

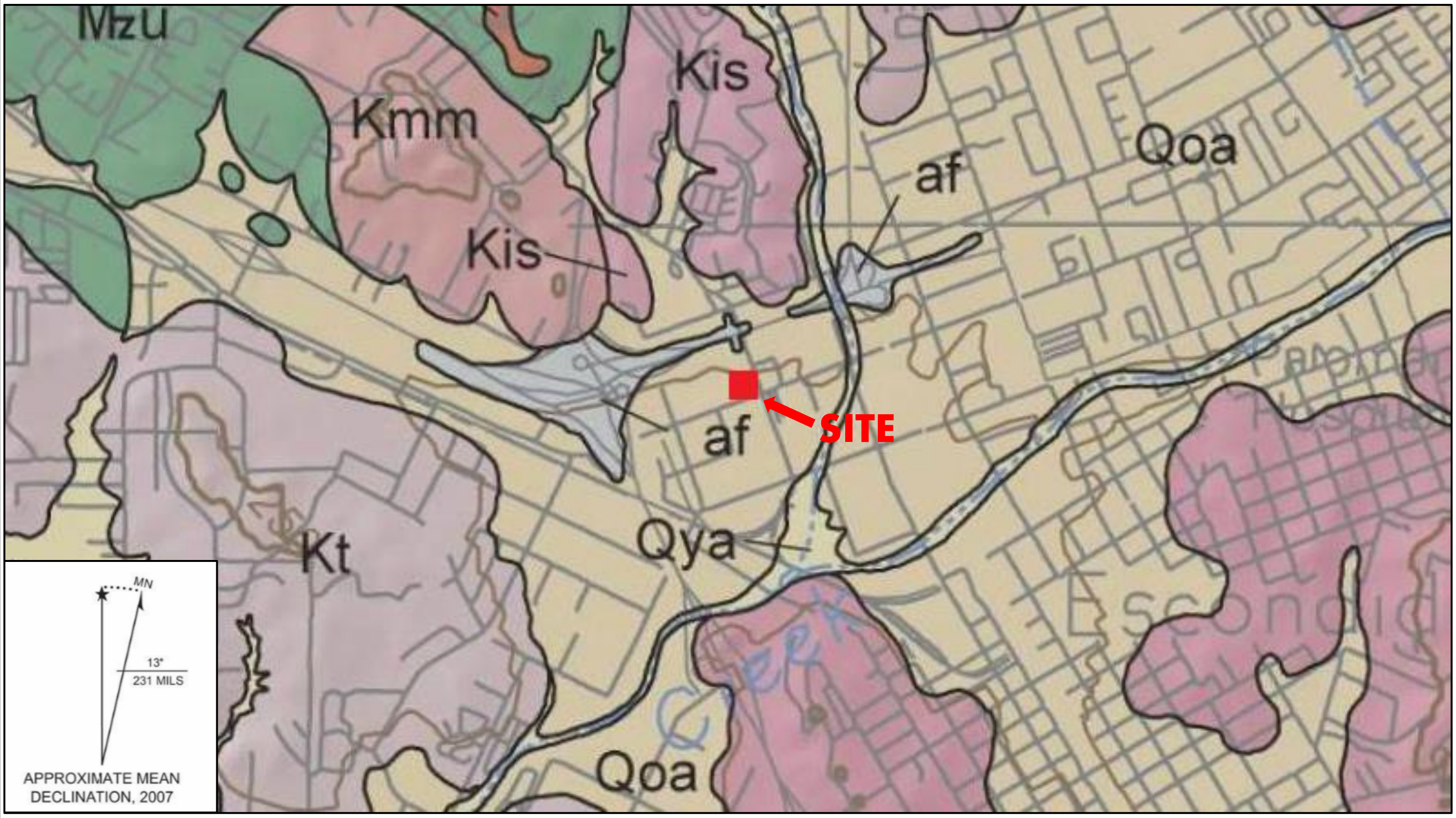
CONTOUR INTERVAL 20 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

This map was produced to conform with the
National Geospatial Program US Topo Product Standard, 2011.
A metadata file associated with this product is draft version 0.6.18

Map Reference: United States Geological Survey (USGS), 2018 edition, 7.5' Minute Topographic Map of the Valley Center Quadrangle, San Diego County, California, Scale 1:24,000

United States Geological Survey (USGS), 2018 edition, 7.5' Minute Topographic Map of the Escondido Quadrangle, San Diego County, California, Scale 1:24,000

Proposed New Structure 900 West Mission Ave, Escondido, California 92025	**Terradyne, Inc.** Regional Topographic Map Terradyne Project No: L201027 — Figure: 1



Unit Explanation

af	Artificial fill (late Holocene)
Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qoa	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)
Kis	Granite of Indian Springs (mid-Cretaceous)

Scale

0 1 2 Miles

Map Reference: Kennedy, M.P., Tan, S.S., Bovard, K.R., Alvarez, R.M., Watson, M.J., and Gutierrez, C.I., 2007, Geologic map of the Oceanside 30x60-minute quadrangle, California: California Geological Survey, Regional Geologic Map No. 2, scale 1:100,000

Proposed New Structure **900 Mission Blvd,** **Escondido, California 92025**	**Terradyne, Inc.** **Regional Geologic Map** **Terradyne Project No: L201027** — **Figure: 2**

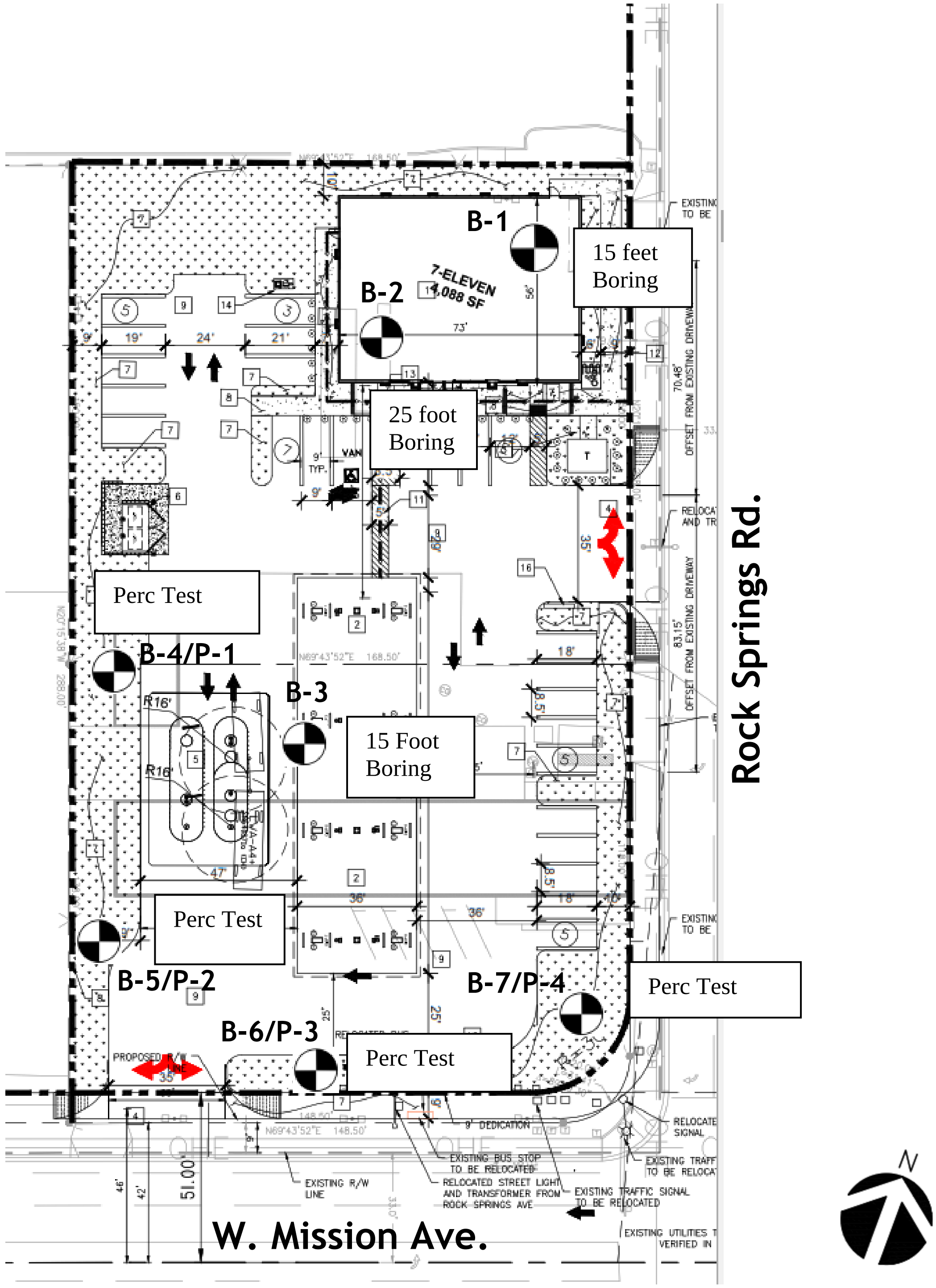
Approximate Boring and Percolation Locations

Approximate Subject Lot Boundary

Map Reference: Google Earth Pro®, 2019, version 7.3.1

Boring Location Map Aerial 900 West Mission Ave, Escondido, CA 92025	**Terradyne, Inc.**	
	Google Earth Pro®, 2019	
	Terradyne Project No: L201027	Figure: 3

Boring locations are approximate.



Boring Location Plan
7-Eleven
900 West Mission Avenue
Escondido CA 92025

TERRADYNE ENGINEERING

Prepared By: KCAB	Scale: Not to Scale	Project # L201027
Base Plan By: Golcheh Group	Date: May 15, 2020	Figure # 4

APPENDIX B
Excavation Logs

Project: **7-Eleven Escondido**	# Log of Boring B-1
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **22.25 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **Modified California, SPT**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Northeast Corner in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" of Asphalt Concrete, No Base.			
					Fill		3"-3' FILL			
			B-1 1.5'	3/4/5	SM		Silty SAND, reddish brown, dense to medium dense, moist	13.21	116.3	
			B-1 3'	7/12/28	SM		3' - 22.25' Old Alluvial Flood-Plain Deposit Silty SAND, Reddish Brown, Moist, dense. Not Plastic. Friable. Uniform.	14.57	118.9	
	5		B-1 5'	10/16/31	SM			19.68	109.4	
			B-1 8'	37/37/50	SM		Consistency becomes very dense			
	10									
			B-1 12'	50 for 5"	SM					
	15		B-1 15'	50 for 6"	SM					
	20		B-1 20'	50 for 5"	SM					
							End Boring at 22.25' Auger Refusal to Advance No Groundwater No Caving Filled with Cut Materials and Asphalt Patch			
	25									
	30									

Project: **7-Eleven Escondido**	Log of Boring B-2
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **16.5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **Modified California**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		0-3" Asphalt No Base			
					Fill		3"-1' FILL Silty SAND, light reddish brown, moist, medium dense			
					SM					
			B-2 2'	14/25 50 for 6"	SM		1'-16.5'Old Alluvial Flood-Plain Deposit Silty SAND, light brown (Red Hue), moist, very dense, fine grained sand, few coarse to medium grained	11.5	113.2	
	5		B-2 5'	8/14/30	SM		Consistency becomes medium dense	15.06	119.2	
			B-2 8'	9/18/27	SM					
	10		B-2 10'	9/18/27	SM					
	15		B-2 15'	50 for 6"	SM		Consistency becomes very dense			
							End Boring at 16.5' No Groundwater No Caving Filled with Cut Material and Asphalt Patch			
	20									

Project: **7-Eleven Escondido**	**Log of Boring B-3**
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **16.5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **Modified California, SPT**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" Asphalt Concrete No Base			
					Fill		3"-1' FILL Silty SAND, reddish brown, medium dense, moist			
					SM					
			B-3 2'	15/35/50	SM		1'-16.5' Old Alluvial Flood-Plain Deposit Silty Sand Reddish Light Brown. Moist. Very Dense. Fine Grained Sand. Not Plastic.	13.56	117.9	
	5		B-3 5'	7/10/35	SM		Consistency becomes medium dense	10.45	111.0	
			B-3 8'	10/17/27	SM					
	10		B-3 10'	15/27/27	SM		Consistency becomes dense			
	15		B-3 15'	50 for 6"	SM		Consistency becomes very dense	5.68		
							End Boring 16.5' No Groundwater No Caving Filled with Cut Materials and Asphalt Patch			
	20									

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Project: **7-Eleven Escondido**	Log of Boring B-4/P-1
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **SPT**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" Asphalt No Base			
					Fill		3"-1' FILL Silty SAND, reddish brown, moist, medium dense			
					SM		1'-5.0' Old Alluvial Flood-Plain Deposit Silty SAND, dark reddish brown, moist, dense, uniform			
	2		P-1 2'	11/25/33	SM		Consistency becomes very dense			
	5						End Boring at 5.0' No Groundwater No Caving Filled with Cut Material and Asphalt Patch			
	10									
	15									

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Project: **7-Eleven Escondido**	# Log of Boring B-5/P-2
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **Modified California**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" Asphalt No Base			
					Fill		3"-1' FILL Silty SAND, dark reddish brown, medium dense, moist			
					SM		1'-5.0' Old Alluvial Flood-Plain Deposit Silty SAND, dark reddish brown, moist, dense			
			P-2 2'	11/25/33	SM		Consistency becomes very dense	11.91		
	5						End Boring at 5.0' No Groundwater No Caving Filled with Cut Material and Asphalt Patch			
	10									
	15									

Project: **7-Eleven Escondido**	Log of Boring B-6/P-3
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **SPT**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" Asphalt No Base			
					Fill		3"-1' FILL Silty SAND, dark reddish brown, medium dense, moist			
					SM		1'-5.0' Old Alluvial Flood-Plain Deposit Silty SAND, dark reddish brown, moist, dense			
			P-3 2'	7/31/45	SM		Consistency becomes very dense	16.24		
	5						End Boring at 5.0' No Groundwater No Caving Filled with Cut Material and Asphalt Patch			
	10									
	15									

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Project: **7-Eleven Escondido**	# Log of Boring B-7/P-4
Project Location: **900 Mission Ave, Escondido**	**Sheet 1 of 1**
Project Number: **L201027**	

Date(s) Drilled **5-22-2020**	Logged By **CR**	Checked By **HE**
Drilling Method **CME-75**	Drill Bit Size/Type **8.75"**	Total Depth of Borehole **5 feet bgs**
Drill Rig Type	Drilling Contractor **Geoboden**	Approximate Surface Elevation
Groundwater Level and Date Measured **NE**	Sampling Method(s) **SPT**	Hammer Data **75 lbs**
Borehole Backfill **Cut Material and Patch**	Location **Center in Parking Lot.**	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
	0				Asphalt		3" Asphalt No Base			
					Fill		3"-1' FILL Silty SAND, dark reddish brown, medium dense, moist			
					SM		1'-5.0' Old Alluvial Flood-Plain Deposit Silty SAND, dark reddish rrown, moist, dense			
			P-4 2'	5/21/32	SM		Consistency becomes very dense	9.21		
	5						End Boring at 5.0' No Groundwater No Caving Filled with Cut Material and Asphalt Patch			
	10									
	15									

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Project: **7-Eleven Escondido** Project Location: **900 Mission Ave, Escondido** Project Number: **L201027**	Key to Log of Boring **Sheet 1 of 1**

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 Elevation (feet): Elevation (MSL, feet).
2 Depth (feet): Depth in feet below the ground surface.
3 Sample Type: Type of soil sample collected at the depth interval shown.
4 Sample Number: Sample identification number.
5 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
6 Material Type: Type of material encountered.
7 Graphic Log: Graphic depiction of the subsurface material encountered.
8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
9 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.
10 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.
11 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Asphaltic Concrete (AC)

AF

Silty SAND (SM)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Auger sampler

Bulk Sample

3-inch-OD California w/ brass rings

CME Sampler

Grab Sample

2.5-inch-OD Modified California w/ brass liners

Pitcher Sample

2-inch-OD unlined split spoon (SPT)

Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)

Water level (after waiting)

Minor change in material properties within a stratum

Inferred/gradational contact between strata

? Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

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APPENDIX C
Laboratory Tests

Moisture Anaysis Results

Project No.: L201008

Project : 900 W Mission Avenue, Escondido, San Diego County, CA 92025

Depth	B1, 1.5'-2'	B1, 3'-3.5'	B1, 5'-5.5'	B2, 2'-2.5'	B2, 5'-5.5'	B3, 2-2.5'	B3, 5'-5.5'	B3, 15-15.5'	P2, 2-2.5'	P3, 2-2.5'	P4, 2-2.5'			
Tare	118.0	119.4	109.0	114.7	115.3	118.0	116.4	117.8	118.8	114.8	114.6			
Wet Material + Tare	425.6	288.5	264.1	314.4	302.5	289.7	406.0	298.3	300.1	301.6	303.2			
Dry Material + Tare	389.7	267.0	238.6	293.8	278.0	269.2	378.6	288.6	280.8	275.5	287.3			
Dry Material	271.7	147.6	129.6	179.1	162.7	151.2	262.2	170.8	162.0	160.7	172.7			
Wet Material	307.6	169.1	155.1	199.7	187.2	171.7	289.6	180.5	181.3	186.8	188.6			
Moisture Loss	35.9	21.5	25.5	20.6	24.5	20.5	27.4	9.7	19.3	26.1	15.9			
Moisture Content %	**13.2**	**14.6**	**19.7**	**11.5**	**15.1**	**13.6**	**10.5**	**5.7**	**11.9**	**16.2**	**9.2**			
Wet Density	131.69	136.25	130.87	126.23	137.11	133.91	122.5							
Dry Density	116.3	118.9	109.4	113.2	119.2	117.9	111.0							
Optimum Moisture	8.5	8.5	8.5	8.5	8.5	8.5	8.5							
Maximum Dry Density	131.8	131.8	131.8	131.8	131.8	131.8	131.8							
Percent of Maximum Dry Density	0.88	0.90	0.83	0.86	0.90	0.89	0.84							
Ring+ Soil	**614.6**	**1052.8**	**1219.9**	**198.3**	**1264.8**	**1038.7**	**774.8**							
Number of Rings	**3**	**5**	**6**	**1**	**6**	**5**	**4**							
Blow Counts														

PH Measured	**7.5**	
Resistsivity p	**4700**	Ωm
Resistsivity Rg		kΩ

Sulfate Content		ppm
Chloride Content		ppm
Nitrate Content		ppm

Tested By: ZJ

Date Tested: 6 / 4 / 2020

Notes:

Laboratory Compaction Test Results

(Proctor # 1 :)

TERRADYNE
Engineers, Geologists & Environmental Scientists

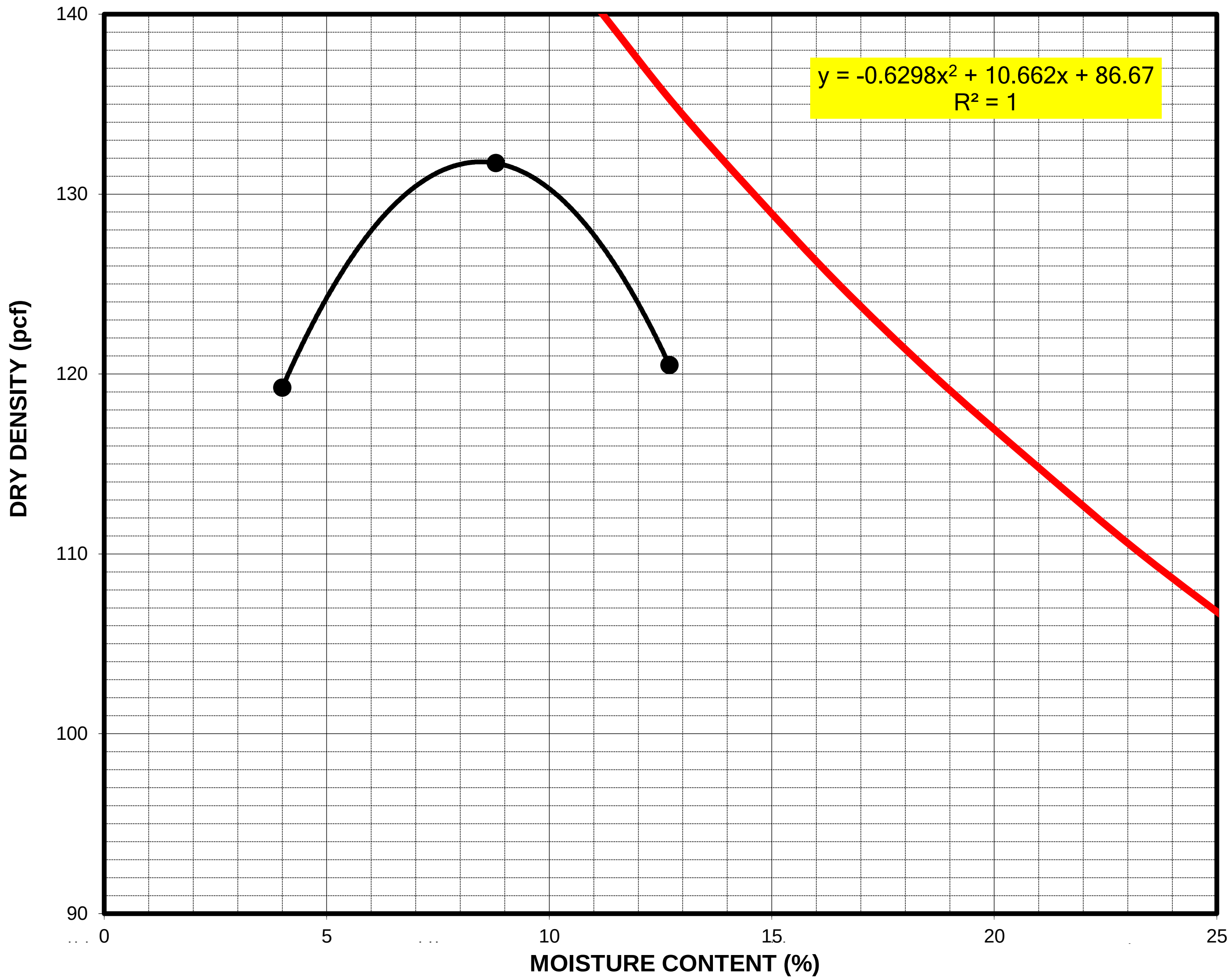
PROJECT:	900 W Mission Ave. Escondido SD, CA
PROJECT #:	L201027
PROCTOR #:	1
SAMPLE #:	1
DATE COMPLETED:	6/10/2020
MAXIMUM DRY DENSITY:	131.8
OPTIMUM MOISTURE CONTENT:	8.5 %
CLIENT:	

Gradation:

Sieve Size:	Cumulative % Retained
1 3/4"	
1 1/4"	
7/8"	
3/4"	
3/8"	
#4	
#40	
#200	

Correction for oversize rock:
(using assumed values for bulk specific gravity and absoption of oversize particles)

% oversize	Max D.D.	Opt M.C.
5	133.8	8.3
10	135.8	8.1
15	137.9	7.9
20	140.1	7.7
25	142.3	7.5
30	144.7	7.2



Sample Description: USCS: SM
Silty Sand, Dark Brown

Location: 900 W Mission Ave. Escondido SD, CA

Compaction Test Procedure: ASTM D 1557 Method A

Lab and QC by: FS/ZJ

Other Test Procedures:
ASTM D4318

LIQUID LIMIT (LL)
PLASTI. INDEX (PI)
GRAVEL %
SAND %
SILT AND CLAY %

CLIENT:

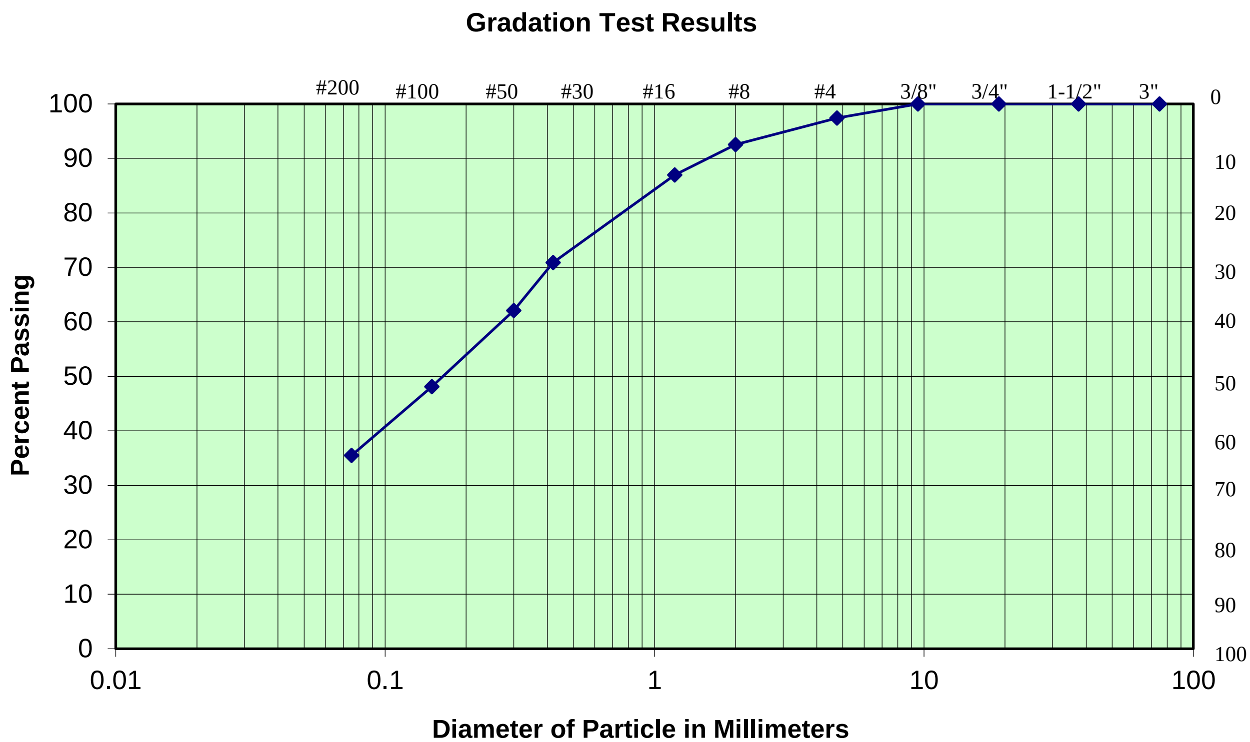
PROJECT: 900 W Mission Ave.

Escondido, CA 92025

Terradyne Project #: L201027

TEST DATA

BORING #: 1 **DEPTH:** 2-6'

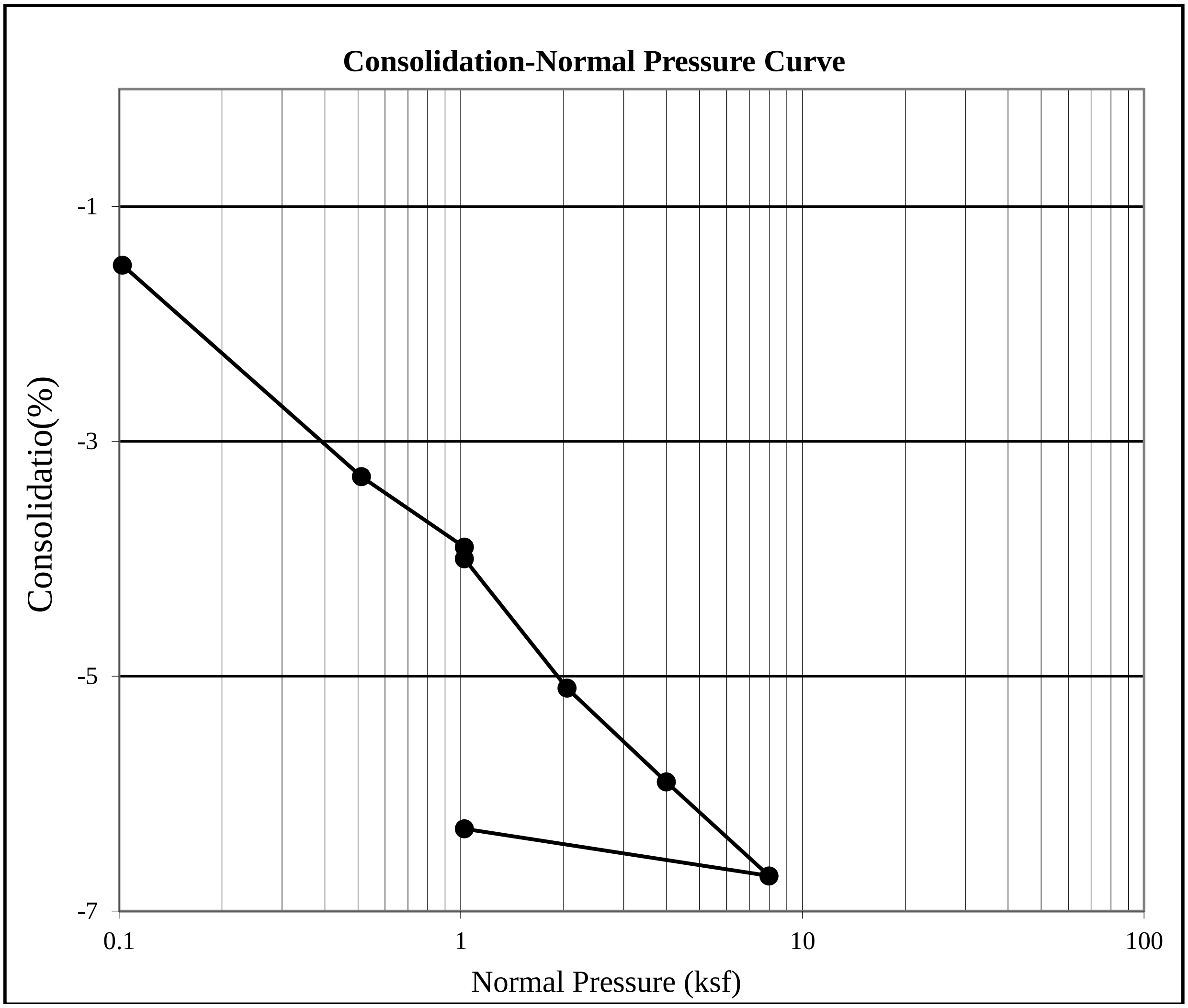


DESCRIPTION:	--

Gravel	2.6%
Sand	61.9%
Fines	35.5%
Moisture	--

Classification	
USCS:	--
AASHTO:	--

TERRADYNE DEN, Inc.
15403 E 17th Ave Ste E, Aurora, CO 80011 PH: 303-463-9317 FAX: 303-463-9321 Email: denver@terradyne.com



Boring Info		Before Test		After Test		Record	
Boring No.	B-1	Moisture (%)	13.21	Moisture (%)	12.12	Test by	ZJ
Depth	1.5-2	Total Weight (g)	216.3	Total Weight (g)	214.3	Check by	
Soil Classification	Silty Sand	Ring Weight (g)	44.6	Ring Weight (g)	44.6	Start Date	6/9/2020
		Wet Density (pcf)	142.7	Wet Density (pcf)	141.0	End Date	6/11/2020

Geotechnical Investigation Report:
900 W Mission Ave., Escondido, San Diego County, CA 92025

Terradyne Engineering, Inc.

Consolidation Test Diagram

Terradyne Project No: L201027

Plate:

APPENDIX D
Infiltration Worksheet
And
USDA Soils Report