Geotechnical Report

Proposed Dollar General
Palm Canyon Drive
Borrego Springs, California

Prepared for:

NNN Retail Development
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Prepared by:

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January 29, 2021

Mr. David Church
NNN Retail Development
15882 Wakefield Lane
San Diego, CA 92127

Geotechnical Report
Proposed Dollar General Store
Palm Canyon Drive
Borrego Springs, California
LCI Report No. LP20214

Dear Mr. Church:

This geotechnical report is provided for design and construction of the proposed Dollar General store located on the north side of Palm Canyon Drive, west of De Giorgio Road in the unincorporated community of Borrego Springs, County of San Diego, California. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

Based on the geotechnical conditions encountered at the points of exploration, the project site appears suitable for the proposed construction provided the professional opinions contained in this report are considered in the design and construction of this project.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. Please provide our office with a set of the foundation plans and civil plans for review to insure that the geotechnical site constraints have been included in the design documents. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
LandMark Consultants, Inc.

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EXECUTIVE SUMMARY

This executive summary presents ***selected*** elements of our findings and professional opinions. This summary ***may not*** present all details needed for the proper application of our findings and professional opinions. Our findings, professional opinions, and application options are ***best related through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them. The findings of this study are summarized below:

- The findings of this study indicate the site is underlain by interbedded sand and silty sand. The near surface sands are expected to be non-expansive. The subsurface soils are medium dense to very dense in nature.
- Groundwater was not encountered in the borings at the time of exploration.
- Elevated sulfate levels were not encountered in the soil samples tested for this investigation. However, in consideration of the general corrosive environment in the vicinity, it is recommended that concrete should use Type II cement with a maximum water-cement ratio of 0.60 and a minimum compressive strength of 3,000 psi.
- Design soil bearing pressure of 1,800 psf. Differential movement of ½ to ¾ inch can be expected for slab on grade foundations placed on native soils.
- Evaluation of liquefaction potential at the site indicates that it is unlikely that the subsurface soil will liquefy under seismically induced ground-shaking due to the lack of groundwater within the upper 50 feet. No mitigation is required for liquefaction effects at this site.
- Seismic settlements of the dry sands have been calculated and are expected to be approximately ¼ inch at the project site.
- All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 3.0 inches unless epoxy coated (ASTM D3963/A934). Hold-down straps are not allowed at the foundation perimeter. No pressurized water lines are allowed below or within the foundations.
- Pavement structural sections should be designed for subgrade soils (R-Value = 50) and an appropriate Traffic Index (TI) selected by the civil designer.

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed Dollar General store located on vacant parcel (APN 141-370-17-00) on the north side of Palm Canyon Drive approximately 400 feet west of De Giorgio Road in the unincorporated community of Borrego Springs, County of San Diego California (See Vicinity Map, Plate A-1). A site plan for the proposed development was provided by your office

The structure is planned to consist of slabs-on-grade foundations and steel-frame construction. Footing loads at exterior bearing walls are estimated at 2 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 80 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include building pad preparation, underground utility installation including trench backfill, concrete foundation construction, parking lot construction, and concrete driveway and sidewalk placement and on-site storm-water retention basins.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the subsurface soil at selected locations within the site for evaluation of physical/engineering properties and liquefaction potential during seismic events. Professional opinions were developed from field and laboratory test data and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction. The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

< Subsurface soil and groundwater conditions
< Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
< Liquefaction potential and its mitigation
< Expansive soil and methods of mitigation
< Aggressive soil conditions to metals and concrete
< Soil infiltration rates of the native soil for storm-water retention basin design

Professional opinions with regard to the above parameters are provided for the following:

< Site grading and earthwork
< Building pad and foundation subgrade preparation
< Allowable soil bearing pressures and expected settlements
< Concrete slabs-on-grade
< Excavation conditions and buried utility installations
< Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
< Seismic design parameters
< Preliminary pavement structural sections

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions, storm water infiltration, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Mr. David Church of NNN Retail Development provided authorization by written agreement to proceed with our work on November 10, 2020. We conducted our work in general accordance with our written proposal dated November 6, 2020.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on January 6, 2021 using 2R Drilling of Ontario, California to advance seven (7) borings to depths of 10 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A geo-technician observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were classified during drilling according to the Unified Soil Classification System using the visual-manual procedure in accordance with ASTM D2488. Relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler lined with 6-inch stainless-steel sleeves.

After logging and sampling the soil, the exploratory borings in excess of 20 feet below ground surface were backfilled with bentonite and an concrete seal in accordance with the San Diego County Permit requirements for exploratory borings. The remaining borings were backfilled with auger cuttings.

The subsurface logs are presented on Plates B-1 through B-7 in Appendix B. A key to the log symbols is presented on Plate B-8. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil borings to aid in classification and evaluation of selected engineering properties of the site soils.

The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

< Particle Size Analyses (ASTM D422)
< Unit Dry Densities (ASTM D2937)
< Moisture Contents (ASTM D2216)
< Moisture-Density Relationship (ASTM D1557)
< Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods)

The laboratory test results are presented on the subsurface logs (Appendix B) and in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were obtained from the field and laboratory testing program.

Section 3
DISCUSSION

3.1 Site Conditions

The project site is irregularly shaped in plan view, is relatively flat-lying slopes, and consists of approximately 3.7 acres of vacant desert land. The project site is covered with scattered dry brush and weeds. No sand dunes or wind drifts are present. Palm Canyon Drive forms the southern property boundary. J&T Tire Pros business is located to the east and a small gift shop is located to the west. Vacant desert land is located to the north. Adjacent properties are flat-lying and are approximately at the same elevation with this site.

The project site lies at an elevation of approximately 555 feet above mean sea level (AMSL) in the Borrego Springs region of the California low desert. The surrounding properties lie on terrain which slopes downward from west to east. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100°F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located near the boundary between the Salton Trough and the Peninsular Ranges physiographic province. The Salton Trough is a geologic structural depression resulting from large scale regional faulting. The Peninsular Ranges consist of Jurassic to Cretaceous granitic intrusions which extend from Riverside, California to the southern tip of Baja California.

The site is located in the Borrego Sink area in the southern portion of the Borrego Valley. The Vallecito Mountains and Pinyon Ridge are located to the south and are bounded on the east by the San Jacinto Fault Zone (4 miles to the northeast) and to the west by the Elsinore Fault Zone (12 miles to the southwest). The Vallecito Mountains are dominantly composed of granitic and metamorphic rocks.

Tectonic activity that formed the region continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity.

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on January 6, 2021 consist of dry and humid, dominantly medium dense to very dense, interbedded sands (SP) and silty sands (SM) to a depth of 51.5 feet, the maximum depth of exploration. The near surface soils are granular and non-expansive in nature. The subsurface logs (Plates B-1 through B-7) depict the stratigraphic relationships of the various soil types.

3.4 Groundwater

Groundwater was not encountered in the borings during the time of exploration, and it is believed deeper than 50 feet below the ground surface. There is uncertainty in the accuracy of short-term water level measurements, particularly in fine-grained soil. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, drainage, and site grading. The groundwater level noted should not be interpreted to represent an accurate or permanent condition.

3.5 Faulting

The project site is located in the seismically active Borrego Valley of southern California with numerous mapped faults of the San Jacinto fault system traversing the region. We have performed a computer-aided search of known faults or seismic zones that lie within a 45-mile radius of the project site (Table 1). A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults.

The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during Quaternary time is considered to be inactive. ***Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the San Jacinto – Coyote Creek located approximately 4.0 miles northeast of the project site.***

3.6 General Ground Motion Analysis

The project site is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area.

2019 CBC General Ground Motion Parameters: The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 for structures on Site Class D and E sites with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0. ***This project site has been classified as Site Class D and has a S_1 value of 0.66, which would require a site-specific ground motion hazard analysis.*** However, ASCE 7-16 Section 11.4.8 provides three exceptions which permit the use of conservative values of design parameters for certain conditions for Site Class D and E sites in lieu of a site-specific hazard analysis. The exceptions are:

- Exception 1: Structures on Site Class E sites with S_s greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.
- Exception 2: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Equations 12.8-2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_S$ or Equation 12.8-4 for $T > T_L$.
- Exception 3: Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_S and the equivalent static force procedure is used for design.

The project structural engineer should confirm that an exception applies to the project. If none of the exceptions apply, our office should be consulted to perform a site-specific hazard analysis.

The 2019 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps Web Application (SEAOC, 2020) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters.
Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. The Maximum Considered

Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for liquefaction and seismic settlement analysis in accordance with 2019 CBC Section 1803A.5.12 ($PGA_M = F_{PGA}$*PGA) is estimated at 0.83g for the project site. ***Design earthquake ground motion parameters are provided in Table 2.***

3.7 Seismic and Other Hazards

► **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the San Jacinto and Elsinore faults. A further discussion of groundshaking mentioned above.

► **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone. Surface fault rupture is considered to be unlikely at the project site because of the well-delineated fault lines through the Borrego Valley as shown on USGS and CDMG maps. However, because of the high tectonic activity and deep alluvium of the region, we cannot preclude the potential for surface rupture on undiscovered or new faults that may underlie the site.

► **Liquefaction.** Liquefaction is unlikely to be a potential hazard at the site, due to groundwater deeper than 50 feet (the maximum depth that liquefaction is known to occur).

Other Potential Geologic Hazards.

► **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.

► **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.

► **Tsunamis, sieches, and flooding.** The site does not lie near any large bodies of water, so the threat of tsunami, sieches, or other seismically-induced flooding is unlikely.

► **Expansive soil.** The near surface soils at the project site consist of sands which are non-expansive.

3.8 Seismic Settlement

An evaluation of the non-liquefaction seismic settlement potential was performed using the relationships developed by Tokimatsu and Seed (1984, 1987) for dry sands. This method is an empirical approach to quantify seismic settlement using SPT blow counts and PGA estimates from the probabilistic seismic hazard analysis. The soils beneath the site consist primarily of medium dense to very dense silty sands and sands which have been calculated to experience approximately ¼ inch of seismic settlement during strong seismic events.

3.9 Hydro-consolidation

In arid climatic regions, granular soils have a potential to collapse upon wetting. This collapse (hydroconsolidation) phenomena is the result of the lubrication of soluble cements (carbonates) in the soil matrix causing the soil to densify from its loose configuration during deposition.

Based on our experience in the vicinity of the project site and the site soils are medium dense to very dense in nature, there is a slight risk of collapse upon inundation from the site. Therefore, development of building foundation is not required to include provisions for mitigating the hydroconsolidation caused by soil saturation from landscape irrigation or broken utility lines.

3.10 Soil Infiltration Rate

A total of two (2) infiltration tests were conducted on January 11, 2021 at the proposed location for the on-site storm-water retention basin as shown on the Site and Exploration Plan (Plate A-2). The infiltration tests were performed to the guideline from Design Handbook for Low Impact Development Best Management Practices, prepared by Riverside County Flood Control and Water Conservation District, Appendix A, Section 2.3, dated September 2011.

The tests were performed using perforated pipes inside an 8-inch diameter flight auger borehole made to depths of approximately 5.0 feet below the existing ground surface, corresponding to the anticipated bottom depth of the stormwater retention basin. The pipes were filled with water and successive readings of drop in water levels were made every 30 minutes for a total elapsed time of 180 minutes, until a stabilization drop was recorded.

The test results indicate that the stabilized soil infiltration rate for the soil ranges from 1.05 to 1.31 inches per hour. A maximum soil infiltration rate of 1.05 inches per hour may be used for the on-site storm-water retention basin design. An oil/water separator should be installed at inlets to the stormwater retention basin to prevent sealing of the basin bottom with silt and oil residues. The field and conversion calculation worksheets are included in Appendix D. We recommend additional testing should be performed after the completion of rough grading operations, to verify the soil infiltration rate.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Pre-grade Meeting: Prior to site preparation, a meeting should be held at the site with as a minimum, the owner's representative, grading contractor and geotechnical engineer in attendance.

Clearing and Grubbing: All surface improvements, debris and/or vegetation including grass, bushes, and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic stripping should be hauled from the site and not used as fill. ***Any trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign materials and removed. [Abandoned pipes should be traced and removed or filled with concrete.*** Any excavations resulting from site clearing and grubbing should be dish-shaped to the lowest depth of disturbance and backfilled with engineered fill.

Mass Grading: Prior to placing any fills, the surface 12 inches of soil should be removed, the exposed surface uniformly moisture conditioned to a depth of 8 inches by discing and wetting to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density. Native soils may be used for mass grading, placed in 6 to 8 inches maximum lifts, uniformly moisture conditioned to a depth of 8 inches by discing and wetting to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Building Pad Preparation for Foundations: The existing surface soil within the building pad area(s) should be removed to 18 inches below the lowest foundation grade or 36 inches below the original grade (whichever is deeper), extending five feet beyond all exterior wall/column lines (including adjacent concreted areas). The exposed sub-grade should be scarified to a depth of 6 to 8 inches, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have footings extended to a minimum of 18 inches below grade. The existing soil beneath the structure foundation prepared in the manner described for the building pad except the preparation needs only to extend 18 inches below and beyond the footing.

Street and Parking Lot Subgrade Preparation: The native soils in street areas should be removed and recompacted to 12 inches below the design subgrade elevation. Engineered fill in street areas should be uniformly moisture conditioned to at least 2% over optimum moisture, placed in layers not more than 6 to 8 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density.

Sidewalk and Concrete Hardscape Areas: In areas other than the building pad which are to receive concrete slabs, the ground surface should be over-excavated to a depth of 12 inches, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

The on-site soils are suitable for use as compacted fill and utility trench backfill. Imported fill soil (if required) should be similar to onsite soil or non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 6 inches and no less than 5% passing the No. 200 sieve. ***The geotechnical engineer should approve imported fill soil sources before hauling material to the site.*** Native and imported materials should be placed in lifts no greater than 8 inches in loose thickness, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Moisture Control and Drainage: The moisture condition of the building pad should be maintained during trenching and utility installation until concrete is placed or should be rewetted before initiating delayed construction. If soil drying is noted, a 2 to 3 inches depth of water may be used in the bottom of footings to restore footing subgrade moisture and reduce potential edge lift.

Adequate site drainage is essential to future performance of the project. Infiltration of excess irrigation water and stormwaters can adversely affect the performance of the subsurface soil at the site. Positive drainage should be maintained away from all structures (5% for 5 feet minimum across unpaved areas) to prevent ponding and subsequent saturation of the native soil. Gutters and downspouts may be considered as a means to convey water away from foundations.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the geotechnical parameters for site development.

4.2 Utility Trench Backfill

On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill. Backfill within roadways should be placed in layers not more than 6 to 8 inches in thickness, uniformly moisture conditioned to at least 2% over optimum moisture and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density except for the top 12 inches of the trench which shall be compacted to at least 95%. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Pipe envelope/bedding should either be clean sand (Sand Equivalent SE>30). Precautions should be taken in the compaction of the backfill to avoid damage to the pipes and structures.

4.3 Foundations and Settlements

Shallow column footings and continuous wall footings are suitable to support the structures provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 1,800 psf. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 2,800 psf.

All exterior and interior foundations should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Continuous wall footings should have a minimum width of 12 inches. Isolated column footings should have a minimum width of 24 inches. ***Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.***

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static loadings and seismic site conditions are estimated to not exceed ¾ inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Foundation movements under the seismic loading due to dry settlement are provided in Section 3.8 of this report.

4.4 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 5 inches thick. Concrete floor slabs may either be monolithically placed with the foundation or dowelled after footing placement. The concrete slabs may be placed on granular subgrade that has been compacted at least 90% relative compaction (ASTM D1557).

American Concrete Institute (ACI) guidelines (ACI 302.1R-04 Chapter 3, Section 3.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both.

The vapor retarder should extend a minimum of 12 inches into the footing excavations.
The vapor retarder may lie directly on the granular fill with 2 inches of clean sand cover.

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness. For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. ***Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings.*** The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

4.5 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-3). The native soils were found to have low (S0) levels of sulfate ion concentration (180 to 690 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Table 4. Concrete Mix Design Criteria due to Soluble Sulfate Exposure

Sulfate Exposure Class	Water-soluble Sulfate (SO_4) in soil, ppm	Cement Type	Maximum Water-Cement Ratio by weight	Minimum Strength f'c (psi)
S0	0-1,000	–	–	–
S1	1,000-2,000	II	0.50	4,000
S2	2,000-20,000	V	0.45	4,500
S3	Over 20,000	V (plus Pozzolon)	0.45	4,500

Note: From ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1

A minimum of 3,000 psi concrete of Type II Portland Cement with a maximum water-cement ration of 0.60 (by weight) should be placed in contact with native soil on this project (sitework including flatwork, sidewalks, and foundations).

A minimum concrete cover of three (3) inches is recommended around steel reinforcing or embedded components (anchor bolts, hold-downs, etc.) exposed to native soil or landscape water (to 18 inches above grade). The concrete should also be thoroughly vibrated during placement. Thorough concrete consolidation and hard trowel finishes should be used due to the aggressive soil exposure.

The native soil has low levels of chloride ion concentration (8 to 180 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate very potential for metal loss because of electrochemical corrosion processes. Mitigation of the corrosion of steel can be achieved by using steel pipes coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by encapsulating the portion of the pipe lying above groundwater with a minimum of 3 inches of densely consolidated concrete. ***No metallic water pipes or conduits should be placed below foundations.***

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy coated for corrosion protection (in accordance with ASTM D3963/A934) or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

Copper water piping (except for trap primers) should not be placed under floor slabs. All copper piping within 18 inches of ground surface shall be wrapped with two layers of 10 mil plumbers tape or sleeved with PVC piping to prevent contact with soil. The trap primer pipe shall be completely encapsulated in a PVC sleeve and Type K copper should be utilized if polyethylene tubing cannot be used. Pressurized waterlines are not allowed under the floor slab. Fire protection piping (risers) should be placed outside of the building foundation.

Landmark does not practice corrosion engineering. We recommend that a qualified corrosion engineer evaluate the corrosion potential on metal construction materials and concrete at the site to obtain final design recommendations.

4.6 Excavations

All site excavations should conform to CalOSHA requirements for Type C soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration. Excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type C soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope. All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the Elsinore and San Jacinto faults. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.6 and Table 2 of this report.

4.8 Pavements

Pavements should be designed according to the 2020 Caltrans Highway Design Manual or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements.

Based on the current Caltrans method, an estimated R-value of 50 for the subgrade soil and assumed traffic indices, the following table provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

PAVEMENT STUCTURAL SECTIONS

R-Value of Subgrade Soil - 50 (estimated) Design Method - CALTRANS 2020

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
5.0	3.0	4.0	6.0	4.0
6.0	3.5	4.0	6.0	6.0
7.0	4.5	4.0	6.0	8.0
8.0	5.0	5.5	8.0	8.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type A HMA (Hot Mix Asphalt), ¾ inch maximum (½ inch maximum for parking areas), with PG70-10 asphalt concrete, compacted to a minimum of 95% of the Hveem density (CAL 308) or a minimum of 92% of the Maximum Theoretical Density (ASTM D2041).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% (95% if sand subgrade) of the maximum dry density determined by ASTM D1557. Prewetting of subgrade soils (to 3.5 feet) may be required depending on moisture of subgrade at time of aggregate base placement.
4) Portland cement concrete for pavements should have Type II cement, a minimum compressive strength of 3,000 psi at 28 days, and a maximum water-cement ratio of 0.60.

Final pavement sections may need to be determined by sampling and R-Value testing during grading operations when actual subgrade soils are exposed.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed Dollar General store located on the north side of Palm Canyon Drive, west of De Giorgio Road in the unincorporated community of Borrego Springs, County of San Diego, California. The conclusions and professional opinions of this report are invalid if:

- Structural loads change from those stated or the structures are relocated.
- The Additional Services section of this report is not followed.
- This report is used for adjacent or other property.
- Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
- Any other change that materially alters the project from that proposed at the time this report was prepared.

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in San Diego County at the time the report was prepared. No express or implied warranties are made in connection with our services.

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, limited laboratory testing, and our understanding of the proposed project. Our analysis of data and professional opinions presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. The nature and extend of such variations may not become evident until, during or after construction. If variations are detected, we should immediately be notified as these conditions may require additional studies, consultation, and possible design revisions.

Environmental or hazardous materials evaluations were not performed by ***LandMark Consultants, Inc.*** for this project. ***LandMark Consultants, Inc.*** will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.

The client has responsibility to see that all parties to the project including designer, contractor, and subcontractor are made aware of this entire report within a reasonable time from its issuance. This report should be considered invalid for periods after two years from the date of report issuance without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

This report is based upon government regulations in effect at the time of preparation of this report. Future changes or modifications to these regulations may require modification of this report. Land or facility use, on and off-site conditions, regulations, design criteria, procedures, or other factors may change over time, which may require additional work. Any party other than the client who wishes to use this report shall notify ***LandMark Consultants, Inc.*** of such intended use. Based on the intended use of the report, ***LandMark Consultants, Inc.*** may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release ***LandMark Consultants, Inc.*** from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold ***LandMark Consultants, Inc.*** harmless from any claim or liability associated with such unauthorized use or non-compliance.

This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Plan Review

Landmark Consultants, Inc. should be retained during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents. ***Landmark Consultants, Inc.*** should have the opportunity to review the final design plans and specifications for the project prior to the issuance of such for bidding.

Governmental agencies may require review of the plans by the geotechnical engineer of record for compliance to the geotechnical report.

5.3 Additional Services

We recommend that ***Landmark Consultants, Inc.*** be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

Landmark Consultants, Inc. *recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the findings and professional opinions in this report are made contingent upon the opportunity for* **Landmark Consultants, Inc.** *to observe grading operations and foundation excavations for the proposed construction.*

If parties other than **Landmark Consultants, Inc.** *are engaged to provide observation and testing services during construction, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.*

Additional information concerning the scope and cost of these services can be obtained from our office.

Section 6
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TABLES

Table 1
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
San Jacinto - Coyote Creek	4.0	6.5	6.8	41 ± 4	4 ± 2
San Jacinto - Anza	8.7	13.9	7.2	91 ± 9	12 ± 6
San Jacinto - Borrego	11.2	18.0	6.6	29 ± 3	4 ± 2
Earthquake Valley	12.1	19.4	6.5	20 ± 2	2 ± 1
Elsinore - Julian	16.8	26.8	7.1	76 ± 8	5 ± 2
Elsinore - Coyote Mountain	19.3	30.9	6.8	39 ± 4	4 ± 2
Superstition Mountain	30.3	48.5	6.6	24 ± 2	5 ± 3
San Andreas - Coachella	31.2	49.9	7.2	96 ± 10	25 ± 5
Elmore Ranch	34.0	54.4	6.6	29 ± 3	1 ± 0.5
Superstition Hills	34.1	54.5	6.6	23 ± 2	4 ± 2
Indio Hills *	35.1	56.2			
San Andreas - San Bernardino (South)	36.8	58.9	7.4	103 ± 10	30 ± 7
San Andreas - San Bernardino (North)	36.9	59.0	7.5	103 ± 10	24 ± 6
Painted Gorge Wash*	37.0	59.2			
Hot Springs *	37.1	59.3			
Elsinore - Temecula	37.4	59.9	6.8	43 ± 4	5 ± 2
Garnet Hill *	40.8	65.2			
Ocotillo*	40.9	65.5			
Vista de Anza*	43.7	69.9			
Laguna Salada	43.9	70.3	7	67 ± 7	3.5 ± 1.5
Blue Cut *	45.5	72.7			
Yuha Well *	45.5	72.9			

* Note: Faults not included in CGS database.

Table 2
2019 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

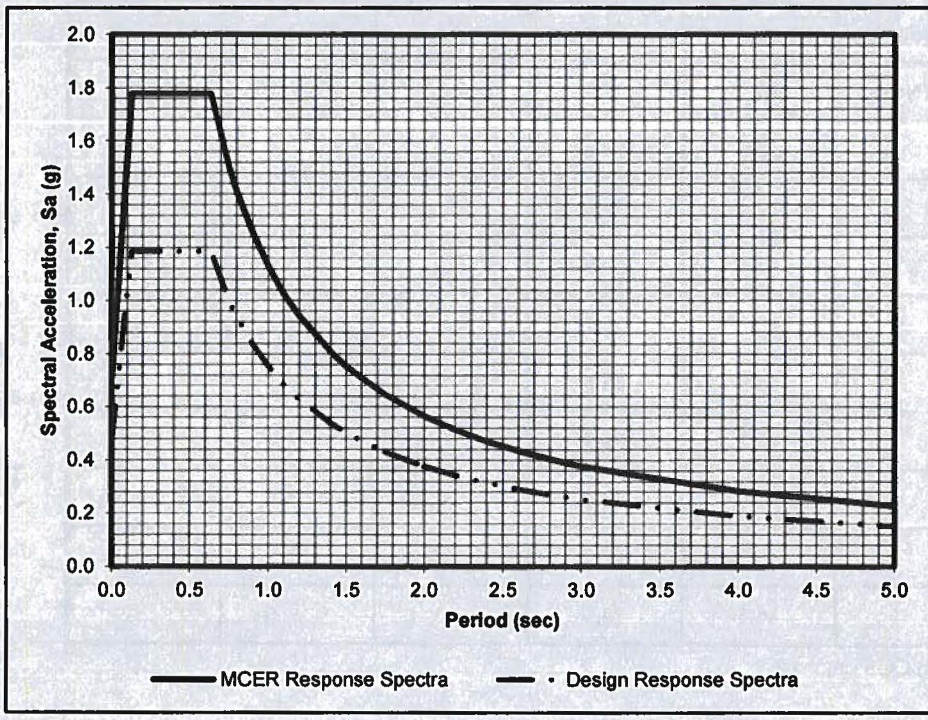
			ASCE 7-16 Reference
Soil Site Class:	D		Table 20.3-1
Latitude:	33.2574	N	
Longitude:	-116.3675	W	
Risk Category:	II		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	S_s	1.778 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	S_1	0.664 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	F_a	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	F_v	1.70	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	1.778 g	$= Fa * S_s$	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.129 g	$= Fv * S_1$	ASCE Equation 11.4-2

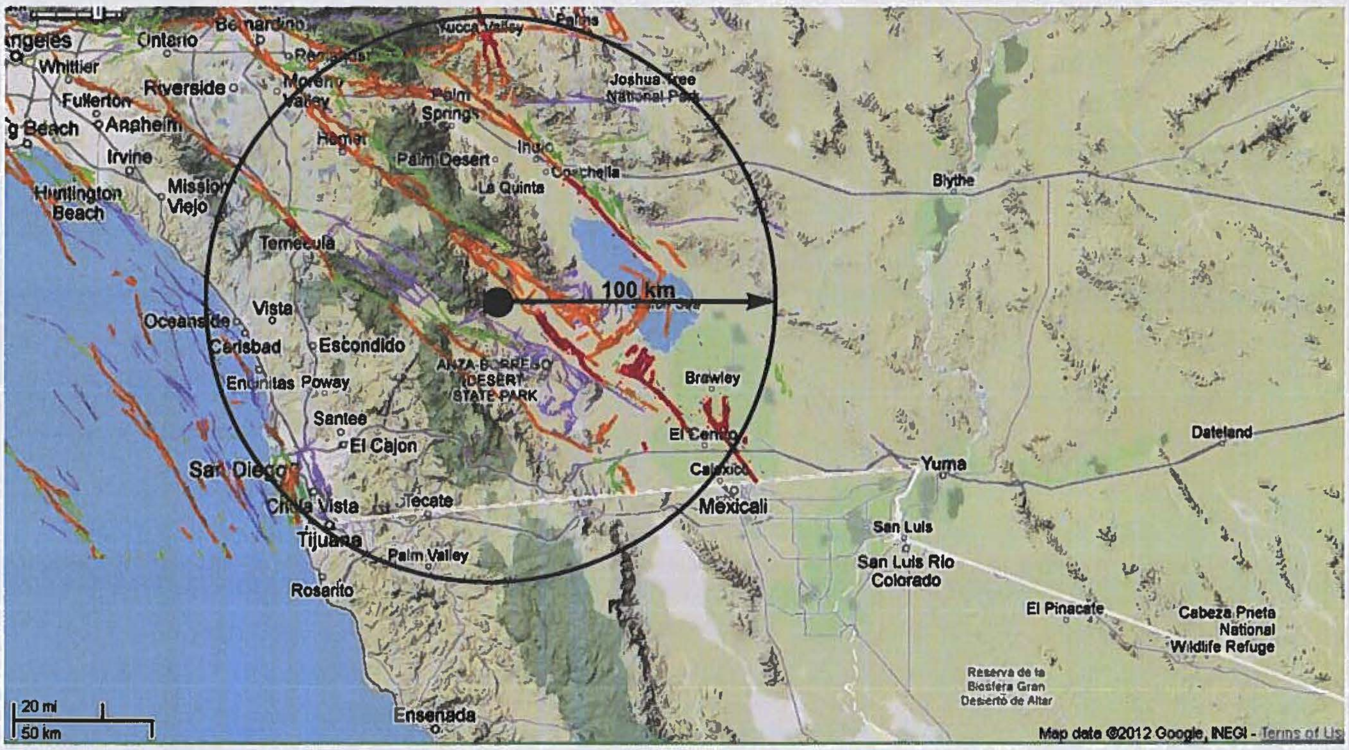
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.185 g	$= 2/3*S_{MS}$	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.753 g	$= 2/3*S_{M1}$	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.916		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.895		ASCE Figure 22-18
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.13 sec	$=0.2*S_{D1}/S_{DS}$	
	T_S	0.63 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	0.83 g		ASCE Equation 11.8-1



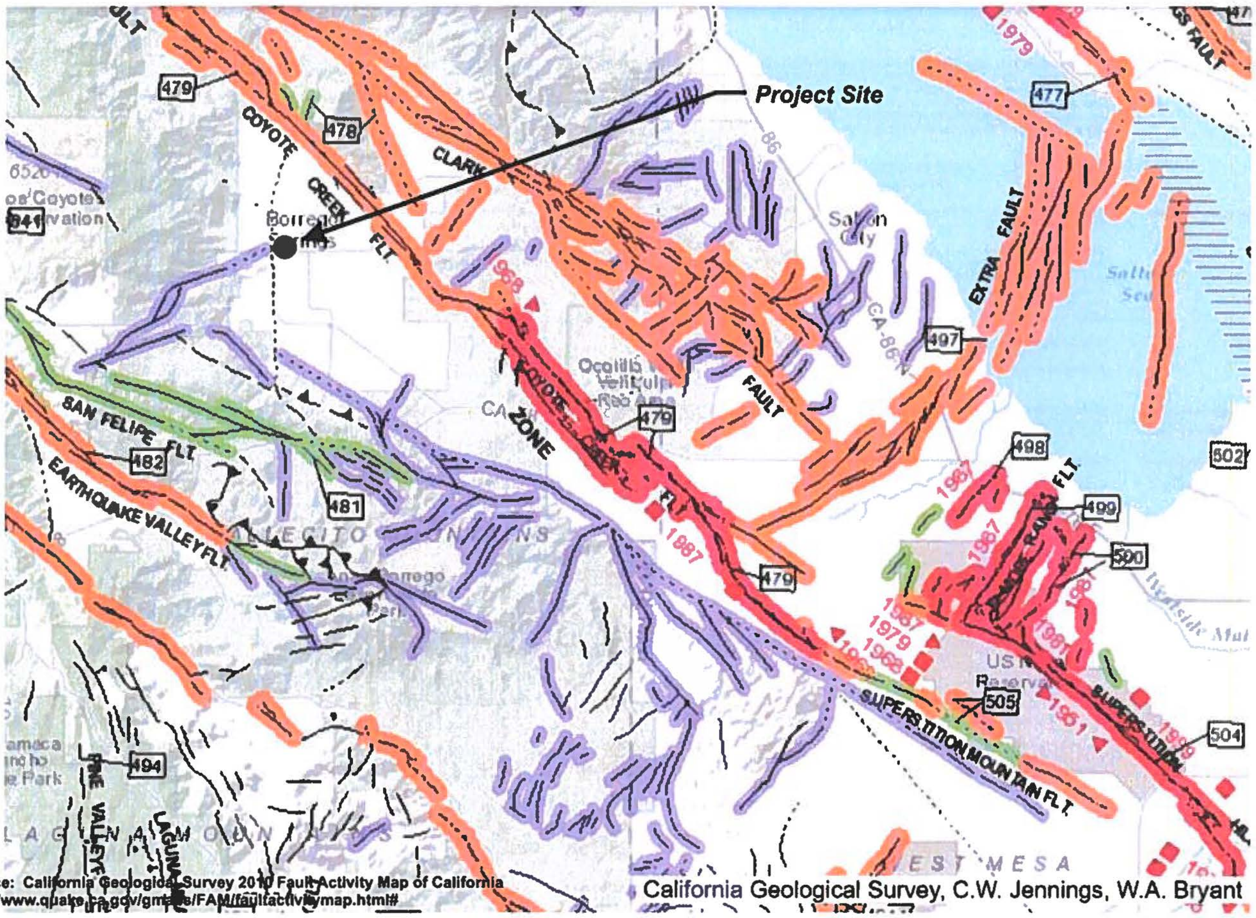
Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.47	0.71
0.13	1.19	1.78
0.63	1.19	1.78
0.75	1.00	1.51
0.80	0.94	1.41
0.90	0.84	1.25
1.00	0.75	1.13
1.10	0.68	1.03
1.20	0.63	0.94
1.20	0.63	0.94
1.40	0.54	0.81
1.50	0.50	0.75
1.75	0.43	0.65
2.00	0.38	0.56
2.20	0.34	0.51
2.40	0.31	0.47
2.60	0.29	0.43
2.80	0.27	0.40
3.00	0.25	0.38
4.00	0.19	0.28
5.00	0.15	0.23

FIGURES



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK Geo-Engineers and Geologists **Project No.: LP20214**	**Regional Fault Map**	**Figure 1**



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

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Map of Local Faults

Figure 2

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

CREEP

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occured (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Fault showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years, possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000*			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
			4.5 billion (Age of Earth)				

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criteria.

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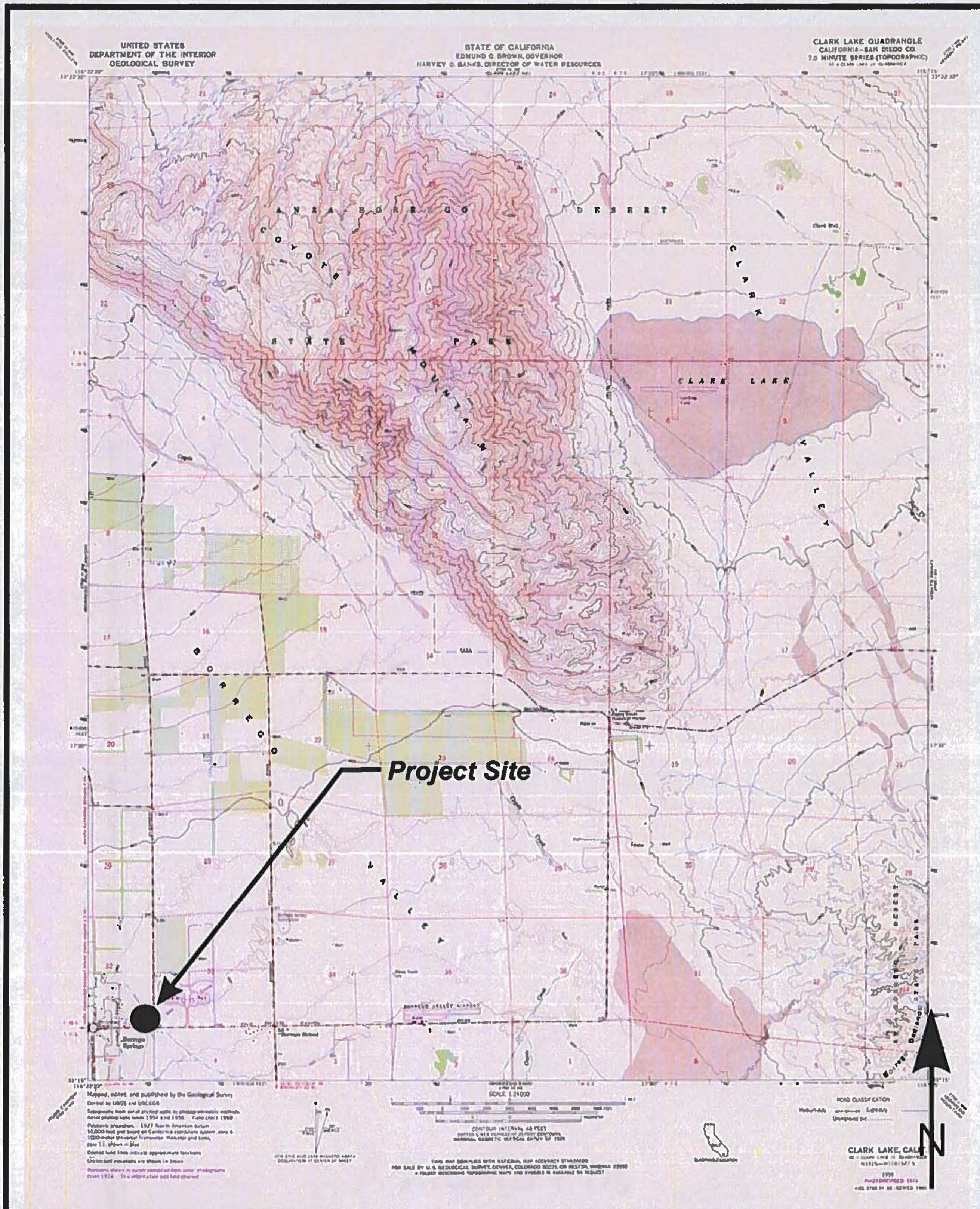
Fault Map Legend

Figure 3

APPENDIX A



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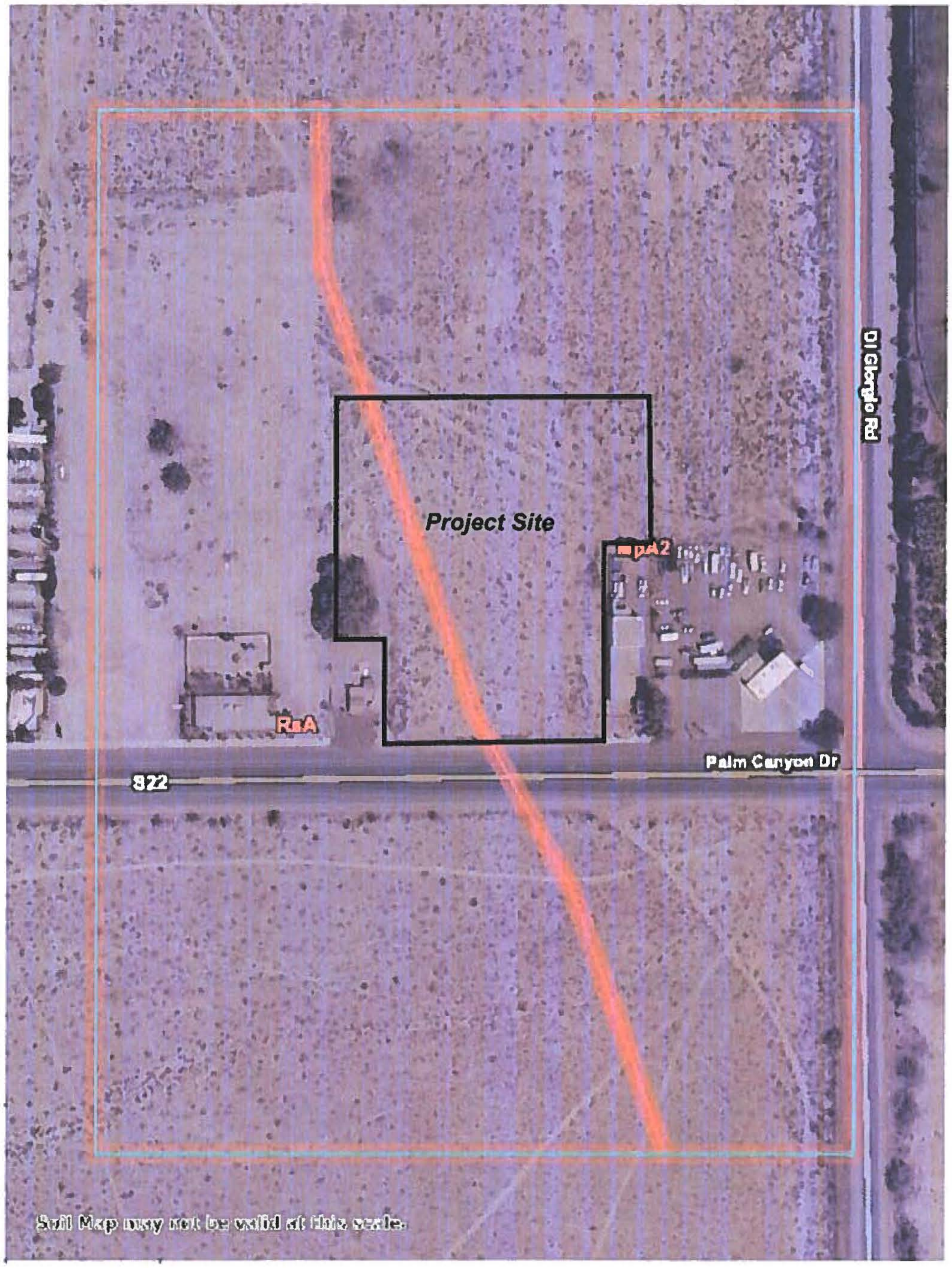
Topographic Map

Plate A-4



Map Scale: 1:2,300 if printed on A portrait (8.5" x 11") sheet.

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84

USDA Natural Resources Conservation Service | Web Soil Survey National Cooperative Soil Survey | 1/29/2021 Page 1 of 3

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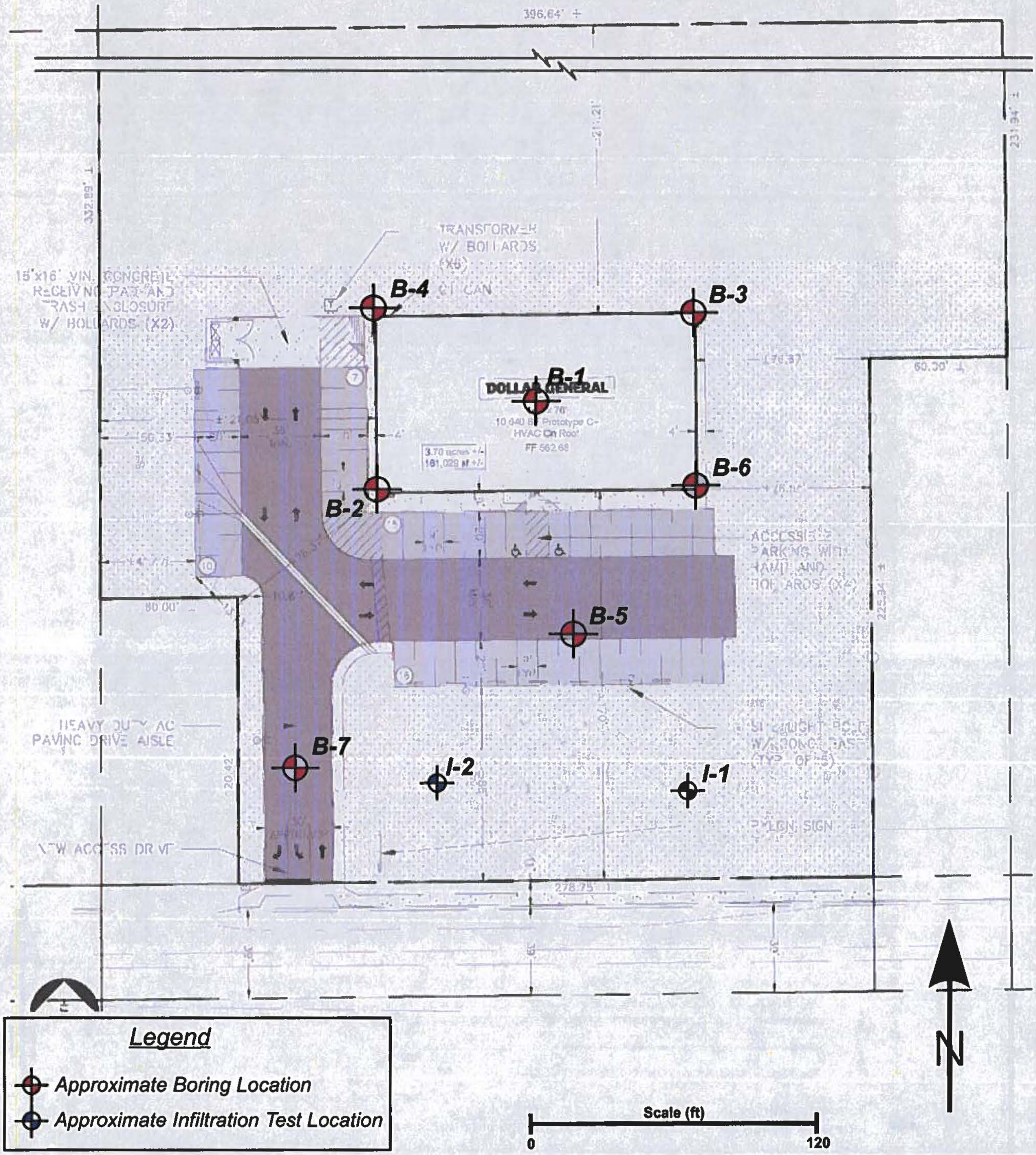
Soil Survey Map

Plate A-3



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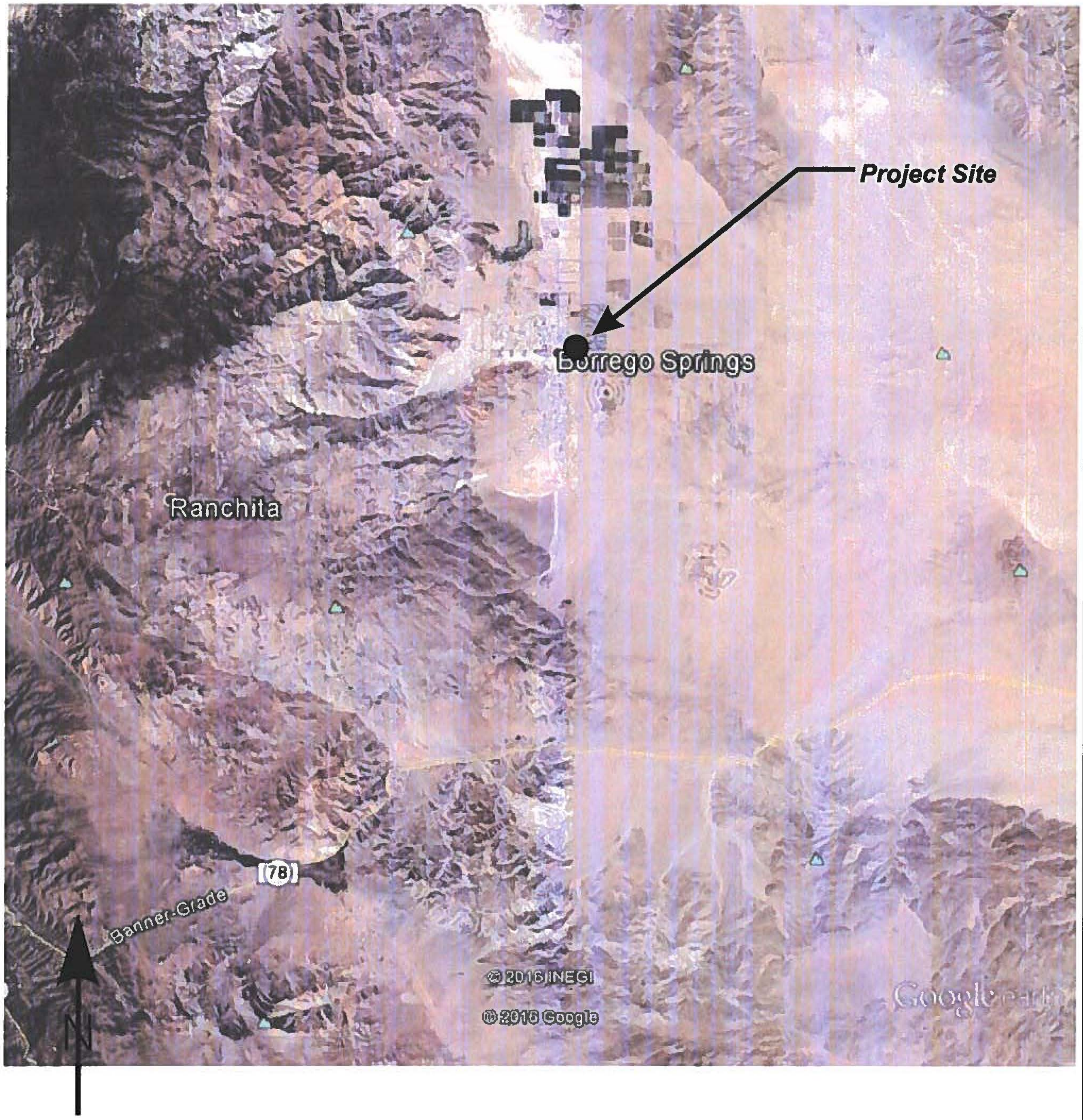
Site and Exploration Map

Plate
A-2



LANDMARK Geo-Engineers and Geologists Project No.: LP20214	Vicinity Map	Plate A-1

APPENDIX B

LOG OF BORING No. B-1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–5		SM			SILTY SAND (SM): Brown, dry, medium dense, fine to coarse grained, some gravel			Passing #200 = 22.7%
5		SM	24			124.6	1.1	
10		SM	38		No recovery			
15		SM	70			114.6	4.5	Passing #200 = 20.2%
20		SP	46		SAND (SP): Gray, dry, dense, medium to coarse grained, some gravel	132.0	1.0	Passing #200 = 4.4%
25		SM	29		SILTY SAND (SM): Brown, dry, medium dense, fine to coarse grained, some gravel		6.5	
30		SP-SM	37		SAND (SP-SM): Brown, dry, dense, fine grained		1.3	Passing #200 = 8.4%
35		SM	36		SILTY SAND (SM): Lt. brown, dry, medium dense to dense, fine grained		1.7	
40		SM	25				2.4	Passing #200 = 20.7%
45		SM	35				7.1	
50		SP-SM	46		SAND (SP-SM): Brown, dry, dense, medium to coarse grained		1.1	Passing #200 = 6.3%
55								
60					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			

DATE DRILLED: 1/6/21

LOGGED BY: A. Arthen

SURFACE ELEVATION:

TOTAL DEPTH: 51.5 Feet

TYPE OF BIT: Hollow Stem Auger

HAMMER WT.: 140 lbs.

DEPTH TO WATER: NA

DIAMETER: 8 in.

DROP: 30 in.

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DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					LOG OF BORING No. B-2 SHEET 1 OF 1			
					SILTY SAND (SM): Lt. brown, dry, medium dense to dense, fine grained, some gravel			Passing #200 = 16.3%
5			27			119.6	1.6	
10			33			106.5	1.5	
15			49		No recovery			
20			85/11"		SAND (SP): Gray, dry, dense to very dense, medium to coarse grained, some gravel	115.0	1.0	
25			33		SILTY SAND (SM): Lt. brown, dry, dense, fine grained		3.2	Passing #200 = 39.9%
30			34				1.0	
35								
40								
45								
50								
55					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60								

DATE DRILLED: 1/6/21
TOTAL DEPTH: 31.5 Feet
DEPTH TO WATER: NA
LOGGED BY: A. Arthen
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION:
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LP20214

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PLATE B-2

LOG OF BORING No. B-3

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
		SM			SILTY SAND (SM): Lt. brown, dry, medium dense, fine grained			
5			22			110.8	2.4	
10		SP	30		SAND (SP): Lt. brown, dry, dense to very dense, medium to coarse grained, some gravel	111.1	1.2	Passing #200 = 2.4%
15			70			125.8	1.5	
20								
25								
30								
35								
40								
45								
50								
55					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60								

DATE DRILLED: 1/6/21 | TOTAL DEPTH: 16.5 Feet | DEPTH TO WATER: NA

LOGGED BY: A. Arthen | TYPE OF BIT: Hollow Stem Auger | DIAMETER: 8 in.

SURFACE ELEVATION: | HAMMER WT.: 140 lbs. | DROP: 30 in.

PROJECT NO. LP20214

LANDMARK Geo-Engineers and Geologists

PLATE B-3

DEPTH	FIELD				LOG OF BORING No. B-4 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY SAND (SM): Brown, dry, medium dense, medium grained, some gravel and cobbles			
5			23				1.5	Passing #200 = 15.5%
10			75		SAND (SP-SM): Lt. brown, dry, dense to very dense, medium to coarse grained, some gravel	124.1	1.3	
15			45			122.7	1.7	Passing #200 = 9.1%
20								
25								
30								
35								
40								
45								
50								
55					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60								

DATE DRILLED: 1/6/21 TOTAL DEPTH: 16.5 Feet DEPTH TO WATER: NA

LOGGED BY: A. Arthen TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT NO. LP20214

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PLATE B-4

LOG OF BORING No. B-5

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–8		SM	21		SILTY SAND (SM): Lt. brown, dry, medium dense, medium grained, some gravel	110.5	2.5	Passing #200 = 31.0%
8–11.5		SP	28		SAND (SP): Lt. brown, dry, medium dense, medium to coarse grained, some gravel	115.4	0.6	Passing #200 = 3.1%

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

Groundwater not encountered at time of drilling.
This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.

DATE DRILLED: 1/6/21	TOTAL DEPTH: 11.5 Feet	DEPTH TO WATER: NA
LOGGED BY: A. Arthen	TYPE OF BIT: Hollow Stem Auger	DIAMETER: 8 in.
SURFACE ELEVATION:	HAMMER WT.: 140 lbs.	DROP: 30 in.

PROJECT NO. LP20214

LANDMARK Geo-Engineers and Geologists

PLATE B-5

LOG OF BORING No. B-6

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			24		SAND (SP): Brown to gray, dry, medium dense to very dense, medium to coarse grained, some gravel and cobbles	110.8	2.4	Passing #200 = 4.9%
10			46			111.1	1.2	
15			58			125.8	1.5	Passing #200 = 4.6%
20								
25								
30								
35								
40								
45								
50								
55					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60								

FIELD / LABORATORY

DATE DRILLED: 1/6/21
TOTAL DEPTH: 16.5 Feet
DEPTH TO WATER: NA
LOGGED BY: A. Arthen
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION:
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LP20214

LANDMARK Geo-Engineers and Geologists

PLATE B-6

LOG OF BORING No. B-7

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY SAND (SM): Lt. brown, humid, medium dense to dense, fine grained			
5			29			103.9	3.3	Passing #200 = 48.9%
10			43			107.2	3.8	Passing #200 = 29.3%
15								
20								
25								
30								
35								
40								
45								
50								
55					Groundwater not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60								

DATE DRILLED: 1/6/21 TOTAL DEPTH: 11.5 Feet DEPTH TO WATER: NA

LOGGED BY: A. Arthen TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT NO. LP20214

LANDMARK Geo-Engineers and Geologists

PLATE B-7

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
	More than half of coarse fraction is larger than No. 4 sieve		GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
	More than half of coarse fraction is smaller than No. 4 sieve		SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays		ML	Inorganic silts, clayey silts with slight plasticity
	Liquid limit is less than 50%		CL	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
			OL	Organic silts and organic clays of low plasticity
	Silts and clays		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	Liquid limit is more than 50%		CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 40 10 4 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples:

Ring Sample   Standard Penetration Test   Shelby Tube   Bulk (Bag) Sample

Drilling Notes:

1. Sampling and Blow Counts
   Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   Standard Penetration Test - Number of blows per foot.
   Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT = Ground Water Table observed @ specified time.

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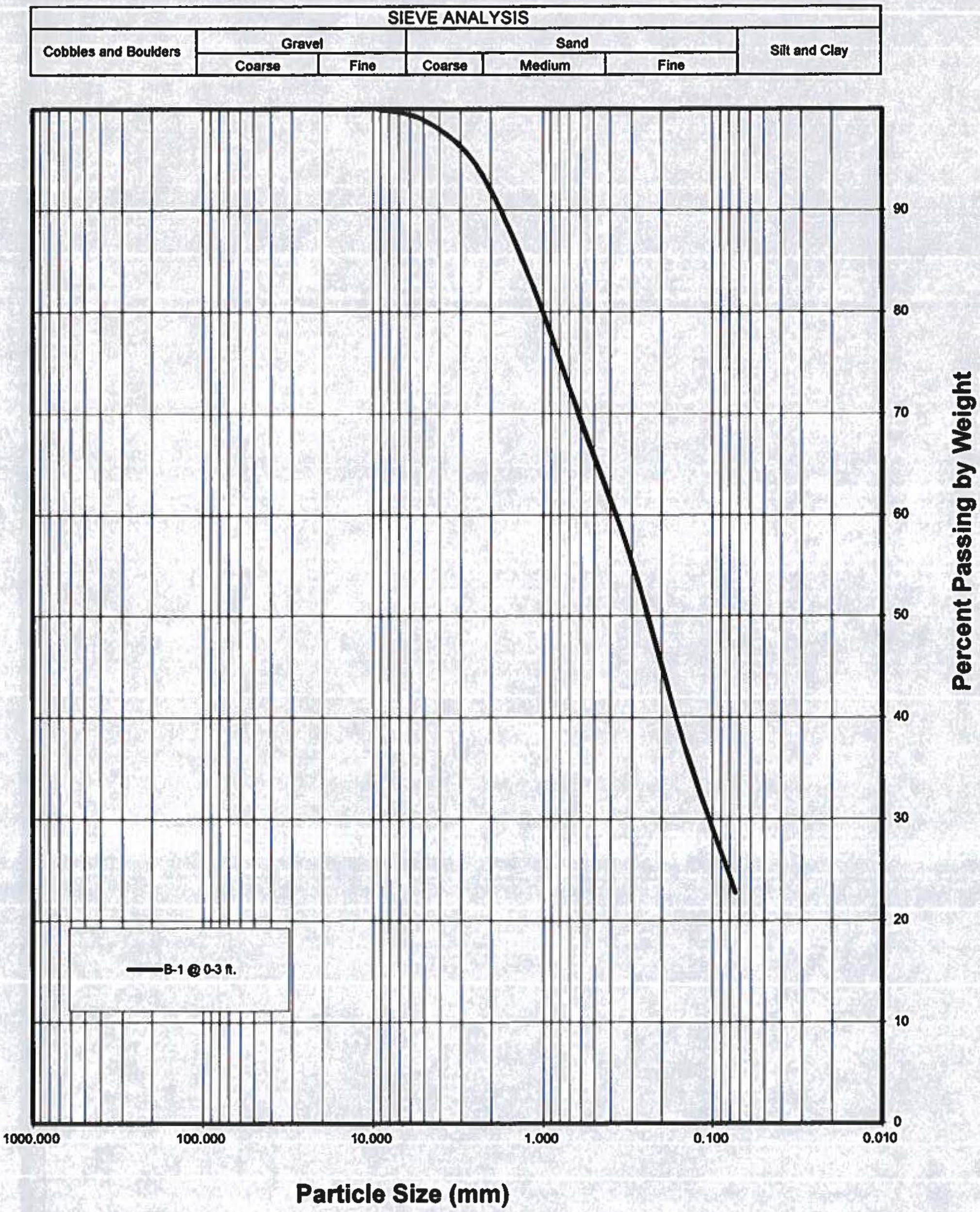
Project No. **LP20214**

Key to Logs

Plate B-8

APPENDIX C



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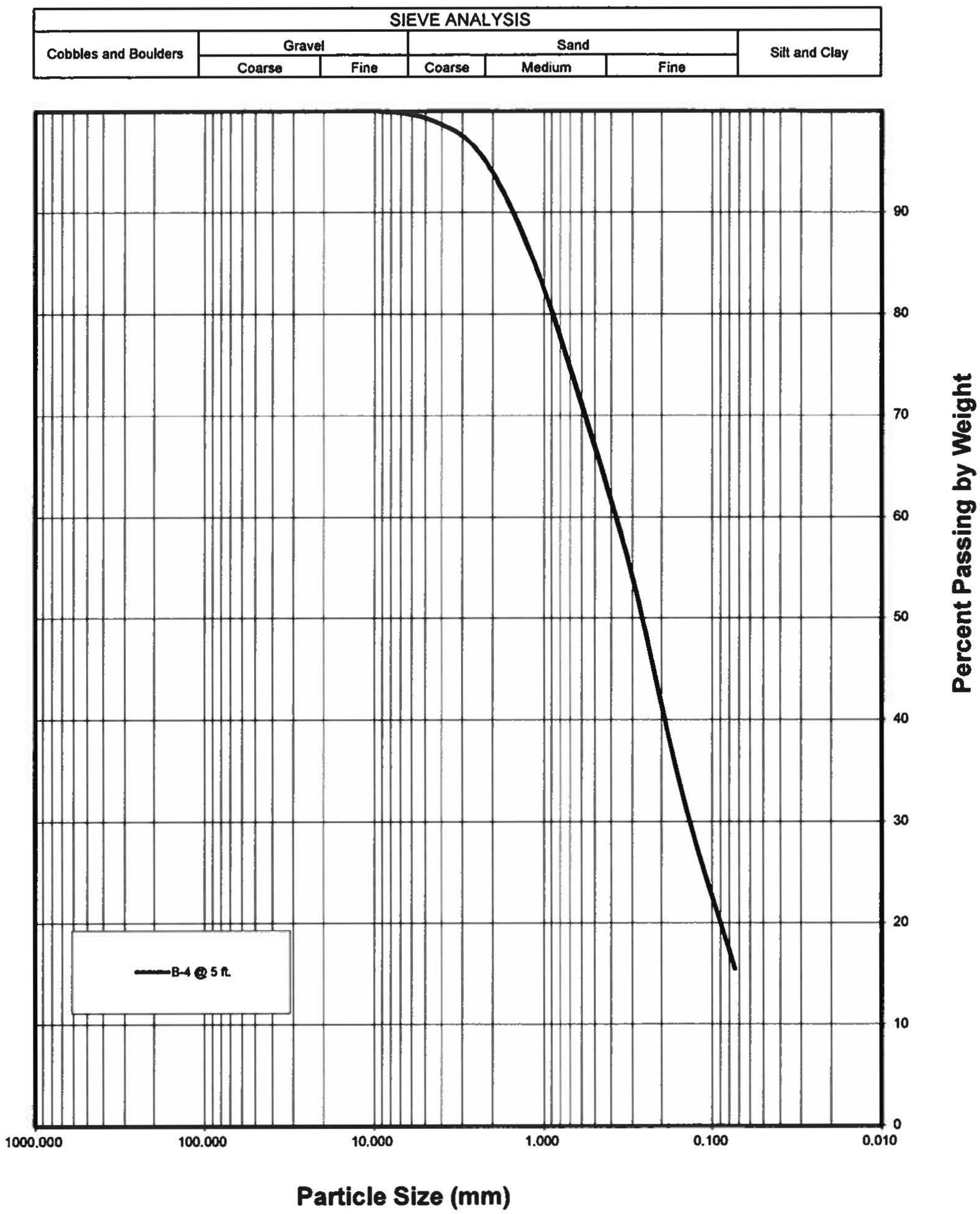
Project No.: LP20214

Grain Size Analysis

Plate
C-1



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Project No.: LP20214

Grain Size Analysis

Plate
C-2

LANDMARK CONSULTANTS, INC.

CLIENT: NNN Development
PROJECT: Dollar General Store -- Borrego Springs, CA
JOB No.: LP20214
DATE: 01/18/21

CHEMICAL ANALYSIS

Boring:	B-1	B-4	**Caltrans**
Sample Depth, ft:	0-3	0-3	**Method**
pH:	8.8	8.5	643
Electrical Conductivity (mmhos):	--	--	424
Resistivity (ohm-cm):	6,500	1,800	643
Chloride (Cl), ppm:	180	80	422
Sulfate (SO4), ppm:	180	690	417

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Amount in Soil (ppm)	Degree of Corrosivity
Concrete	Soluble Sulfates	0 - 1,000	Low
		1,000 - 2,000	Moderate
		2,000 - 20,000	Severe
		> 20,000	Very Severe
Normal Grade Steel	Soluble Chlorides	0 - 200	Low
		200 - 700	Moderate
		700 - 1,500	Severe
		> 1,500	Very Severe
Normal Grade Steel	Resistivity	1 - 1,000	Very Severe
		1,000 - 2,000	Severe
		2,000 - 10,000	Moderate
		> 10,000	Low

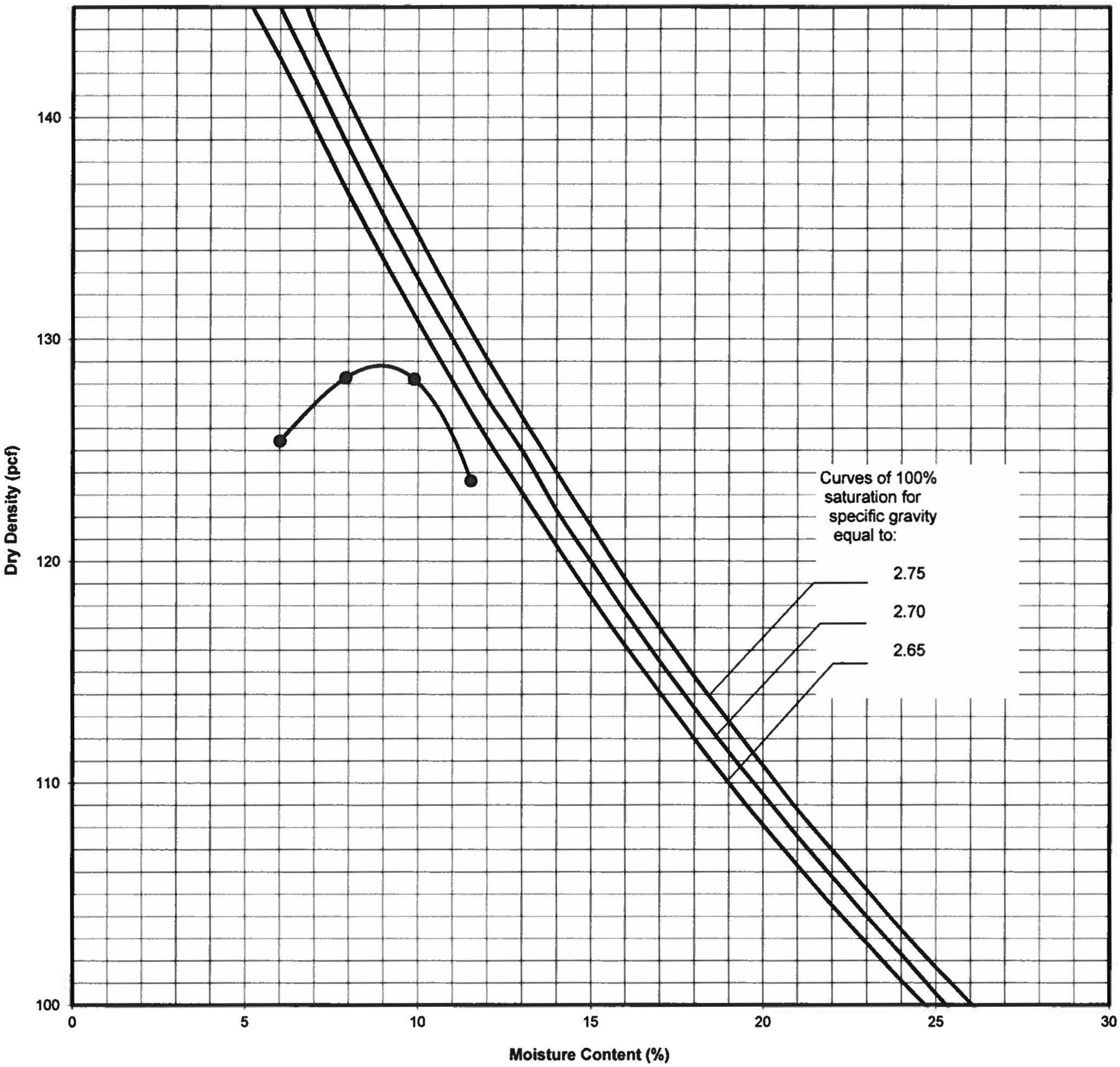
LANDMARK
Geo-Engineers and Geologists

Project No.: LP20214

Selected Chemical Test Results

Plate C-3

Client:	NNN Development	**Soil Description:**	Brown Silty Sand (SM)
Project:	Dollar General Store - Borrego Springs, CA	**Sample Location:**	B-1 @ 0-5 ft.
Project No.:	LP20214	**Test Method:**	ASTM D-1557 A
Date:	1/18/2021	**Maximum Dry Density (pcf):**	**128.8**
Lab. No.:	N/A	**Optimum Moisture Content (%):**	**8.9**



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Project No.: LP20214

Moisture Density Relationship

Plate C-4

APPENDIX D

LANDMARK CONSULTANTS, INC

Project:	Dollar General		Project No:	LP20214		Date:	01/11/21
Test Hole No:		I-1	Tested By:	Alex A			
Depth of Test Hole, D_T:		5'	USCS Soil Classification:				
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		6"	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:50	8:15	25.00	34.00	42.00	8.00	y
2	8:15	8:40	25.00	42.00	48.00	6.00	n

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:02	9:32	30.00	31.00	37.00	6.00	5.00
2	9:32	10:02	30.00	37.00	43.00	6.00	5.00
3	10:02	10:32	30.00	43.00	49.00	6.00	5.00
4	10:32	11:02	30.00	36.50	42.00	5.50	5.45
5	11:02	11:32	30.00	42.00	47.00	5.00	6.00
6	11:32	12:02	30.00	47.00	52.20	5.20	5.78
7							
8							
9							
10							
11							
12							

COMMENTS:

LANDMARK
Geo-Engineers and Geologists

Project No.: LP20214

Percolation Test Results

Plate
D-1

PERCOLATION RATE CONVERSION

CLIENT: NNN Retail Development
PROJECT: Dollar General - Borrego Springs
PROJECT NO.: LP20214
DATE: 11/22/2021

TEST HOLE NO: I-1

Time interval, Δt = 30 minutes

Initial Depth to Water, D_0 = 47 inches

Final Depth to Water, D_f = 52.2 inches

Total Depth of Test Hole, D_T = 60 inches

²Test Hole Radius, r = 3 inches

The conversion equation is used:

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg}}$$

"H_o" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \qquad = 60\text{-}47 = 13 \text{ inches}$$

"H_f" is the final height of water at the selected time interval

$$H_f = D_T - D_f \qquad = 60 - 52.2 = 7.8 \text{ inches}$$

"ΔH" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \qquad = 13\text{-}7.8 = 5.2 \text{ inches}$$

"H_{avg}" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \qquad = (13+7.8)/2 = 10.4 \text{ inches}$$

"I_t" is the tested infiltration rate

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\left(r+2H_{avg}\right)} = \frac{(5.2 \text{ in})(60\text{min/hr})(3\text{in})}{(30 \text{ min})((3 \text{ in}) + 2\ (10.4 \text{ in}))} = 1.31 \text{ in/hr}$$

LANDMARK Geo-Engineers and Geologists

Project No.: LP20214

Percolation Rate Conversion

Plate D-1A

LANDMARK CONSULTANTS, INC

Project:	Dollar General		Project No:	LP20214		Date:	01/11/21
Test Hole No:		I-2	Tested By:	Alex A			
Depth of Test Hole, D_T:		5'	USCS Soil Classification:				
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		6"	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:52	8:17	25.00	24.00	31.00	7.00	y
2	8:17	8:42	25.00	31.00	37.00	6.00	n

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:04	9:34	30.00	34.00	40.00	6.00	5.00
2	9:34	10:04	30.00	40.00	45.50	5.50	5.45
3	10:04	10:34	30.00	26.00	32.00	6.00	5.00
4	10:34	11:04	30.00	32.00	37.50	5.50	5.45
5	11:04	11:34	30.00	37.50	43.00	5.50	5.45
6	11:34	12:04	30.00	43.00	48.50	5.50	5.45
7							
8							
9							
10							
11							
12							

COMMENTS:

LANDMARK
Geo-Engineers and Geologists

Project No.: LP20214

Percolation Test Results

Plate
D-2

PERCOLATION RATE CONVERSION

CLIENT: NNN Retail Development
PROJECT: Dollar General - Borrego Springs
PROJECT NO.: LP20214
DATE: 11/22/2021

TEST HOLE NO: I-2

Time interval, Δt = 30 minutes

Initial Depth to Water, D_0 = 43 inches

Final Depth to Water, D_f = 48.5 inches

Total Depth of Test Hole, D_T = 60 inches

[2]Test Hole Radius, r = 3 inches

The conversion equation is used:

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t(r+2H_{avg}}$$

"H_o" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \quad = 60\text{-}43 = 17 \text{ inches}$$

"H_f" is the final height of water at the selected time interval

$$H_f = D_T - D_f \quad = 60 - 48.5 = 11.5 \text{ inches}$$

"ΔH" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \quad = 17\text{-}11.5 = 5.5 \text{ inches}$$

"H_{avg}" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \quad = (17+11.5)/2 = 14.25 \text{ inches}$$

"I_t" is the tested infiltration rate

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t\,(r+2H_{avg})} = \frac{(5.5\text{ in})(60\text{min/hr})(3\text{in})}{(30\text{ min})((3\text{ in}) + 2\,(14.25\text{ in}))} = 1.05\text{ in/hr}$$

LANDMARK Geo-Engineers and Geologists Project No.: LP20214	Percolation Rate Conversion	Plate D-2A

APPENDIX E

Seismic Dry Settlement Calculation

Project Name: Proposed Dollar General Store - Borrego Springs, CA
Project No.: LP20214
Location: B-1

Maximum Credible Earthquake	6.8	
Design Ground Motion	0.83	g
Water Unit Weight,	62.4	pcf
Depth to Groundwater	60	ft
Hammer Effenciency	85	

9.3

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	D_{50} (mm)	ϕ (°)	Density (pcf)	Total Pressure (tsf)	N1(60)	Relative Density	Fine Content	$N_{1(60)cs}$	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
24		6.00	6	0.25	25	125	0.375	40.6	95	23	48.8	808	1.35E-03	4.64E-04	3.75E-04	0.05	
38		11.00	5	0.25	25	125	0.688	61.2	117	23	71.4	1241	9.85E-04	2.14E-04	1.73E-04	0.02	
70		16.00	5	0.25	25	115	0.920	101.2	151	20	112.9	1670	6.93E-04	8.68E-05	7.02E-05	0.01	
46		21.00	5	0.25	25	132	1.386	60.6	116	4	60.6	1669	1.43E-03	3.78E-04	3.06E-04	0.04	
	29	26.00	5	0.25	25	120	1.560	62.3	118	25	73.8	1889	1.19E-03	2.48E-04	2.01E-04	0.02	
	37	31.00	5	0.25	25	120	1.860	72.8	128	8	74.0	2065	1.22E-03	2.53E-04	2.04E-04	0.02	
	38	36.00	5	0.25	25	120	2.160	69.2	124	21	79.0	2274	1.14E-03	2.20E-04	1.78E-04	0.02	
	25	41.00	5	0.25	25	120	2.460	41.3	96	21	48.6	2068	1.68E-03	5.80E-04	4.69E-04	0.06	
	35	46.00	5	0.25	25	120	2.760	59.5	115	21	68.4	2452	1.15E-03	2.63E-04	2.12E-04	0.03	
	46	51.00	5	0.25	25	120	3.060	74.3	129	6	74.7	2657	9.85E-04	2.03E-04	1.64E-04	0.02	
																	0.29

REFERENCES

(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.
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(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils
(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE
(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.