GEOTECHNICAL EVALUATION
PROPOSED PHOTOVOLTAIC
SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL
SAN DIEGO, CALIFORNIA

PREPARED FOR:
Zagrodnik + Thomas Architects
3956 30th Street
San Diego, California 92104

PREPARED BY:
Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
5710 Ruffin Road
San Diego, California 92123

November 24, 2015
Project No. 108029003

November 24, 2015
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Mr. Scott Thomas
Zagrodnik + Thomas Architects
3956 30th Street
San Diego, California 92104

Subject: Geotechnical Evaluation
Proposed Photovoltaic Solar Generation System
Morse High School
San Diego, California

Dear Mr. Thomas:

In accordance with your request and authorization, we are providing this geotechnical evaluation for the proposed photovoltaic solar generation system project at Morse High School located at 6905 Skyline Drive in San Diego, California. This report has been prepared in accordance with our proposal dated August 20, 2015 and presents our findings, conclusions, and recommendations regarding the geotechnical aspects of the project. We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

PROFESSIONAL GEOLOGIST
JASON M. MOORE
No. 2434
CERTIFIED ENGINEERING GEOLOGIST
Exp. 12/31/16
STATE OF CALIFORNIA

Jason M. Moore, PG, CEG
Senior Geologist

Gabriel Smith, PE
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
Gabriel Marshall Smith
No. C82857
CIVIL
STATE OF CALIFORNIA

Jeffrey T. Kent, PE, GE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
JEFFREY T. KENT
No. 2817
EXP. 6/30/16
GEOTECHNICAL
STATE OF CALIFORNIA

NMM/JMM/GS/JTK/gg

Distribution: (1) Addressee

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1. INTRODUCTION

In accordance with your request and authorization, we are providing this geotechnical evaluation report for the proposed photovoltaic solar generation system to be constructed at the Morse High School campus located at 6905 Skyline Drive in San Diego, California (Figure 1). Presented in this report are the results of our background review, field explorations, geotechnical laboratory testing, our conclusions regarding the geotechnical conditions at the site, and our recommendations for the design and earthwork construction aspects of this project.

2. SCOPE OF SERVICES

Ninyo & Moore's scope of services for this project included review of pertinent background data, performance of a geologic reconnaissance, subsurface exploration, and engineering analysis with regard to the proposed construction. These services generally follow the scope outlined our proposal dated August 20, 2015.

3. SITE AND PROJECT DESCRIPTION

The site is located at 6905 Skyline Drive in the city of San Diego, California (Figure 1). The site coordinates are approximately 32.70107°N latitude and 117.04853°W longitude. The site is occupied by Morse High School, which is developed with various classroom and administrative buildings, athletic fields, landscaping, parking lots, and access roads. Prior to construction of the existing improvements, the site was graded to its current configuration by cutting and filling operations. The proposed project will include construction of canopy-mounted photovoltaic (PV) arrays in the dirt lots adjacent to the athletic fields located in the southern portion of the school campus (Sage, 2015). Electrical conductors linking the proposed canopies to the main electrical service will also be placed in excavated trenches.

4. SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration was conducted on October 23, 2015, and included the drilling, logging, and sampling of five small-diameter borings (designated B-1 through B-5). Prior to commencing the subsurface exploration, Underground Service Alert and San Diego Unified School District personnel were notified for marking of the existing site utilities. In addition, we considered the utility maps for the school provided by Zagrodnik + Thomas Architects when marking our boring locations. The purpose of the borings was to evaluate subsurface conditions and to collect soil samples for laboratory testing.

The borings were drilled to depths up to approximately 20 feet using a truck-mounted drill rig equipped with 8-inch diameter, continuous-flight, hollow-stem augers. Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2488 by observing cuttings and drive samples. Representative bulk and in-place soil samples were collected at selected depths from within the exploratory borings and then returned to our in-house geotechnical laboratory for analysis. Boring logs are presented in Appendix A. The approximate locations of the borings and proposed PV arrays are presented on Figure 2.

Geotechnical laboratory testing was performed on representative soil samples collected during the subsurface exploration. This testing included an evaluation of in-place moisture content and dry density, gradation, Atterberg limits, shear strength, expansion index, and soil corrosivity. The results of the in-situ dry density and moisture content tests are presented on the boring logs in Appendix A. Descriptions of the geotechnical laboratory test methods and the results of the other geotechnical laboratory tests performed are presented in Appendix B.

5. GEOLOGY AND SUBSURFACE CONDITIONS

Geologic units encountered during our subsurface exploration included fill and materials of the Otay Formation (Figure 3) (Kennedy and Tan, 2008). Generalized descriptions of the earth units encountered during our subsurface exploration are provided below. Additional descriptions are provided on the boring logs in Appendix A.

5.1.1. Fill

Fill materials were encountered from the ground surface in each of our borings. The thickness of the fill varied from approximately 3 feet in Boring B-3 to more than 20 feet in Borings B-2 and B-5. As encountered, the fill materials generally consisted of various shades of brown, dry to moist, stiff to hard, silty to sandy clay and loose to medium dense, clayey to silty sand. Scattered gravel and pieces of claystone were encountered in the fill materials. Documentation regarding placement of these fills was not available for review. Based on review of historic aerial photographs and topographic maps, grading operations at the site were performed in the late 1950's and included placement of fills up to approximately 45 feet in thickness.

5.1.2. Otay Formation

The Oligocene-age Otay Formation was encountered beneath the fill materials in Borings B-1, B-3, and B-4. As encountered, these materials were observed to consist of light brown to brown and grayish brown, dry to moist, moderately indurated, sandy claystone with gravel and weakly to moderately cemented, clayey and silty sandstone.

5.2. Groundwater

Groundwater was not encountered in our borings. Based on the elevation and topography of the area and our review of records available through Geotracker (2015), we anticipate groundwater is present at a depth of more than 30 feet below the ground surface. It should be noted that fluctuations in groundwater level and local perched conditions may occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, and other factors. Furthermore, existing utility trenches act as conduits for subsurface water and perched water conditions may be present within these areas.

5.3. Faulting and Seismicity

Based on our review of the referenced geologic maps, as well as on our geologic field mapping, the subject site is not located within a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). The closest known active fault, the Rose Canyon fault, has been mapped approximately 6 miles west of the site; no active faults are known to cross the project vicinity (Figures 4 and 5). While the potential for ground rupture due to faulting at the site is considered low, lurching or cracking of the ground surface as a result of nearby seismic events is possible. The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structures.

The 2013 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.41g using the United States Geological Survey (USGS, 2015) seismic design tool (web-based). Spectral response acceleration parameters, consistent with the 2013 CBC, are also provided in Section 7.5 for the evaluation of seismic loads on buildings and other structures.

The 2013 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration

with adjustment for site class effects (PGA_M) was calculated as 0.42g using the USGS (USGS, 2015) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.37g for the site and a site coefficient (F_{PGA}) of 1.13 for Site Class D.

6. CONCLUSIONS

Based on our review of the referenced background data, the subsurface exploration, and geotechnical laboratory testing, it is our opinion that construction of the proposed photovoltaic solar generation system is feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into the design and construction of the project. In general, the following conclusions were made:

- The areas of the proposed canopy-mounted photovoltaic solar arrays are underlain by fill soils and materials of the Otay Formation.
- The fill materials underlying the proposed construction should be generally excavatable with heavy-duty earth moving equipment in good working condition.
- Excavations within the existing Otay Formation materials underlying the proposed construction may encounter cemented sandstone and/or indurated claystone layers. Therefore, very difficult excavation conditions should be anticipated and the use of heavy ripping or coring may be needed.
- Groundwater was not encountered in the borings performed as part of our subsurface exploration. However, perched water conditions may be encountered in such areas as existing utility trenches.
- Due to the potential for perched water conditions at the site, the contractor should be prepared to take appropriate measures during construction to address wet hole conditions. These measures may include, but are not limited to, placement of concrete by tremie method.
- Results of our geotechnical laboratory testing of the materials encountered during our subsurface evaluation indicated in-situ moisture contents above optimum moisture content. Therefore, materials derived from on-site excavations are anticipated to be wet and the contractor will need to moisture condition the excavated materials prior to reuse.
- Where shallow, spread footings are utilized, the resulting excavation bottom should be prepared by scarifying, moisture conditioning, and recompacting the upper 12 inches.

- Based on the results of our limited geotechnical laboratory testing and our previous experience at the site, the on-site soils exhibit a medium to very high expansion potential. Our recommendations presented herein include consideration for the expansive nature of the on-site soils.

- The closest known active fault, the Rose Canyon fault, has been mapped approximately 6 miles west of the site. No active faults are reported underlying the subject site. Therefore, potential for ground rupture due to faulting at the site is considered low.

- Based on the results of our limited geotechnical laboratory testing presented in Appendix B, as compared to the Caltrans (2012) corrosion guidelines and ACI 318, we consider the on-site soils to be corrosive. Specifically, the on-site soils are considered corrosive to ferrous metals.

7. RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed project. These recommendations are based on our evaluation of the site geotechnical conditions and our assumptions regarding the planned development. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

7.1. Site Preparation

Site preparation should begin with the removal of asphalt, concrete, utility lines, vegetation, and other deleterious debris from areas where improvements are to be constructed. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite.

7.2. Excavation Characteristics

Based on our subsurface exploration and our experience with similar materials, the on-site fill soils are generally excavatable with heavy-duty earthmoving equipment in good working condition. Strongly cemented sandstone and/or strongly indurated claystone may be encountered within the Otay Formation and the contractor should be prepared to utilize heavy ripping, rock wheel/saw, and/or blasting. The presence of cemented/indurated materials can also be problematic in narrow

trenches and should be anticipated. Excavations (including utility trenches) will generate oversize material and additional processing and handling of these materials including screening and/or crushing should be anticipated. Drilling of holes within the Otay Formation can also be expected to be difficult and the use of specialized equipment (such as core barrels) may be needed to advance to design depths. Though primarily cohesive soils were encountered in our exploratory borings, caving of open excavations (e.g., trenches, drilled excavations) should be anticipated, particularly where cohesionless soils are encountered, or where excavations are not promptly backfilled.

7.3. Excavation Bottom Stability

Due to the variability of the on-site soils, some of the bottoms of the excavations may be soft and/or unstable, especially if exposed to wetting. In general, soft and/or unstable bottom conditions may be mitigated by scarifying and recompacting the exposed bottom or overexcavating the excavation bottom to suitable depths (as evaluated in the field by Ninyo & Moore's representative) and replacing with granular fill or aggregate base materials. Specific recommendations for stabilizing excavation bottoms should be based on evaluation in the field by Ninyo & Moore at the time of construction.

7.4. Utility Trench Zone Backfill

On-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are generally suitable for use as trench zone backfill. On-site soils classified as silts or clays should not be used for backfill in the pipe zone. These materials should be replaced with an imported granular fill. Trench zone backfill material should also generally not contain rocks or lumps over approximately 3 inches, and generally not more than approximately 30 percent larger than ¾-inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of off site. Materials generated from on-site excavations should be moisture-conditioned (wetted or dried) to a moisture content generally above the laboratory optimum to allow compaction. We understand that sufficient volumes of on-site soils will be produced by foundation and utility trench excavations that import soils will not be needed.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods to a relative compaction of 90 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until finished grades are achieved.

7.5. Seismic Design Parameters

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with the CBC (2013) guidelines and adjusted MCE spectral response acceleration parameters (USGS, 2015).

Table 1 – 2013 California Building Code Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Site Coefficient, F_a	1.136
Site Coefficient, F_v	1.704
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.910g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.348g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.034g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.593g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.689g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.395g

7.6. Foundations

The proposed canopy-mounted PV arrays may be supported on shallow, spread footings or cast-in drilled hole (CIDH) pile foundations in accordance with the recommendations presented in this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

7.6.1. Shallow Spread Footings

Where shallow, spread footings are utilized, the foundation excavation should be cut to grade and the upper 12 inches of the resulting excavation bottom should be scarified, moisture conditioned to generally above optimum moisture, and recompacted to 90 percent relative compaction as evaluated by ASTM D 1557. Shallow, spread footings supported on compacted fill may be designed using an allowable bearing capacity of 2,000 pounds per square foot (psf). This is based on the bottom of the foundation excavation being compacted at 90 percent relative compaction as evaluated by ASTM D 1557. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. To mitigate for the expansive nature of the on-site soils, we recommend that shallow, spread footings be founded 36 inches below the lowest adjacent grade and be 36 inches in width. The spread footings should be reinforced in accordance with the recommendations of the project structural engineer.

7.6.1.1. Lateral Resistance – Shallow Foundations

For the analysis of the shallow foundations using lateral resistance pressures, we recommend an allowable passive pressure of 300 pounds per cubic foot (pcf) of depth be used with a value of up to 3,000 pounds per square foot (psf). This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.25 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance.

The lateral resistance values presented above may be increased by one-third when considering loads of short duration such as wind or seismic forces

7.6.2. Cast-in-Drilled-Hole Piles

We understand that the proposed canopy-mounted PV arrays may be supported on CIDH pile foundations. Geotechnical recommendations for CIDH foundations are provided below. These types of structures typically impose relatively light axial loads on the foundations. We anticipate that the foundation dimensions will be generally controlled by the lateral load or uplift demand. The type, depth, and size of the foundations should be evaluated by the structural engineer based on the loading conditions and these geotechnical recommendations. We recommend that the foundation plans and design submittal be reviewed by this office for general conformance to these recommendations prior to finalizing.

Although we anticipate that pile dimensions will be generally controlled by the lateral load demand, we recommend that CIDH pile foundations have a diameter of 2 feet or more. Furthermore, to mitigate the expansive nature of the on-site soils, we recommend that the CIDH pile foundations extend to a depth of 6 feet or more. We recommend that the downward axial capacities of CIDH piles be designed using a side frictional resistance value of 160 pounds per square foot of area along the perimeter of the pile based on a factor of safety of 2. Uplift capacities should be designed using a frictional resistance value of 100 pounds per square foot of area along the perimeter of the pile based on a factor of safety of 1.5. The pile dimensions (i.e., diameter and embedment) should be evaluated by the project structural engineer.

The axial and uplift pile capacities presented are based on side friction. We recommend that end bearing not be incorporated into the pile design capacities. The allowable bearing pressures and lateral capacities may be increased by one-third when considering loading of short duration such as wind or seismic forces.

The drilled pile construction should be observed by the geotechnical consultant during construction to evaluate if the piles have been extended to the design depths. The drilled holes should be cleaned of loose soil and gravel. It is the contractor's responsibility to (a) take appropriate measures for maintaining the integrity of the drilled holes, (b) see that the holes are cleaned and straight, and (c) see that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by 1 percent of the depth of the pile. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile.

We recommend that special measures, such as placement of concrete by tremie method, are implemented to see that the aggregate and cement do not segregate during concrete placement.

7.6.2.1. Lateral Resistance – CIDH Piles

For the analysis of the CIDH pile foundations using lateral resistance pressures, we recommend an allowable passive pressure of 600 pounds per cubic foot (pcf) of depth be used with a value of up to 6,000 pounds per square foot (psf). For this case, an effective width of one times the diameter of the piles may be considered when evaluating lateral resistance. The recommended passive pressure assumes the piles are not adversely affected by approximately ½-inch of movement at the top of the pile. Furthermore, this value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.25 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance.

The lateral resistance values presented above may be increased by one-third when considering loads of short duration such as wind or seismic forces.

For lateral loading, piles in a group may be considered to act individually when the center-to-center spacing is greater than 4D (where, D is the diameter of the pile) in the direction normal to loading and greater than 8D in the direction parallel to loading. Table 2 presents the lateral load reduction factors to be applied for various pile spacing for in-line loading.

Table 2 – Lateral Load Reduction Factors

Center-To-Center Pile Spacing for In-Line Loading	Reduction Factor*		
	Row 1	Row 2	Row 3 and higher
2D	0.60	0.35	0.25
3D	0.75	0.55	0.40
5D	1.00	0.85	0.70
7D	1.00	1.00	0.90

Note:
* Based on California Amendments to AASHTO LRFD Bridge Design Specifications, 4th Edition, November 2011

7.6.3. Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½-inch over a horizontal span of 40 feet should be expected.

7.7. Corrosion

Laboratory testing was performed on a representative sample of the on-site earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and

the sulfate and chloride content tests were performed in accordance with CT 417 and CT 422, respectively. These laboratory test results are presented in Appendix B.

The results of the soil corrosivity testing from this evaluation indicated an electrical resistivity of 380 ohm-cm and a soil pH of 8.9. The chloride content of the tested sample was approximately 700 parts per million (ppm). The sulfate content of the tested sample was approximately 0.041 percent (i.e. 410 ppm). Based on the laboratory test results, American Concrete Institute (ACI) 318, Caltrans (2012) criteria, and our experience with similar soils, the site is classified as corrosive, which is defined as soil with an electrical resistivity less than 1,000 ohm-cm, a chloride content more than 500 ppm, more than 0.10 percent sulfates, and/or a pH less than 5.5. Specifically, the on-site soils are considered corrosive to ferrous metals. We recommend that a corrosion engineer be consulted with during design to address mitigation recommendations for the proposed project.

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Laboratory testing as part of this and the previous evaluations indicated a sulfate content of approximately 0.041 percent. In accordance with ACI 318, we consider the site soils to possess a negligible potential for sulfate attack on concrete. Due to the potential for variability of the on-site soils, we recommend consideration be given to using Type II/V cement for concrete in contact with soil.

7.8. Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.

7.9. Plan Review and Construction Observation

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

8. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

9. REFERENCES

American Concrete Institute, 2011, ACI 318-11, Building Code Requirements for Structural Concrete and Commentary: dated August.

American Society of Civil Engineers (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, ASCE 7-10.

California Building Standards Commission (CBSC), 2013, California Building Code, Title 24, Part 2, Volumes 1 and 2.

California Department of Transportation (Caltrans), 2012, Corrosion Guidelines (Version 2.0), Division of Engineering and Testing Services, Corrosion Technology Branch: dated November.

City of San Diego, 1954, Topographic Survey, Map Sheet 194-1749, Scale 1:2,400.

City of San Diego, 1976, Ortho-topographic Survey, Map Sheet 194-1749, Scale 1:2,400.

City of San Diego, 2008, Seismic Safety Study, Geologic Hazards and Faults, Grid Tile 18: dated April 3.

Geotracker, 2015, www.geotracker.com; accessed in October.

Google Inc., 2015, www.googleearth.com.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Geological Survey, Special Publication 42, with Supplements 1 and 2 added in 1999.

Historic Aerials, 2015, 6905 Skyline Drive, San Diego, California, www.historicaerials.com.

Kennedy, M.P. and Tan, S.S, 2008, Geology of the San Diego 30' X 60' Quadrangle, California, Scale 1:100,000.

Ninyo & Moore, in-house proprietary information.

Ninyo & Moore, 2015, Revised Proposal for Geotechnical Evaluation, Proposed Photovoltaic Solar Generation Systems, Eight San Diego Unified School District Sites, San Diego, California: dated August 20.

Public Works Standards, Inc., 2012, "Greenbook," Standard Specifications for Public Works Construction.

Sage Renewables, 2015, DRAFT Proposed Site Plan, Morse HS, San Diego USD Solar PV Project, dated September.

Tan, S.S., 1995, Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, San Diego County, California; Landslide Hazard Identification Map No. 33, CGS Open-File Report 95-03.

United States Geological Survey, 2015, U.S. Seismic Design Maps, World Wide Web, http://earthquake.usgs.gov/designmaps/us/application.php.

AERIAL PHOTOGRAPHS				
Source	**Date**	**Flight**	**Numbers**	**Scale**
USDA	April 14, 1953	AXM-10M	114 and 115	1:20,000

N
SITE
Wunderlin Ave
Brooklyn Ave
Akins Ave
Herrick St
Encanto Park
Lisbon St
Leppert St
Cadman St
Cahill Dr
Anza Dr
San Vicente St
Knollwood Dr
San Felipe
Black Oak Rd
Shorewood Dr
Shaules Ave
Sullivan Ave
Eleanor Dr
Benson Ave
Tuther Way
Peter Pan Ave
Gribble St
Woodbine Way
Skyline Dr
Alderley St
Martin Luther King Jr Memorial Park
Pittsburgh Ave
Varney Dr
Plaza Cuernavaca
Plaza Blvd
Terra Cotta Rd
Aquamarine Rd
S Siena St
Woodshawn Dr
Coralwood Dr
Deep Dell Rd
Brookhaven Rd
Bullock Dr
Sunswept St
Windward St
Brookhaven Rd
Gatewood Ln
Paradise Valley Rd
Parkwood Dr
Kingswood St
Zest St
Bignell Dr
Nebraska Ave
Crabtree St
Division St
Elsberry St
Jouglard St
Plaza Ridge Rd
Paradise Hills Park
Bougainville Rd
Oriskany Rd
Sandy Shore St
Kenwell St
Alcona St
Honey Dr
Latrobe Cir
Spring Oak Way
Grove View
Montcliff Rd
Fowler Dr
Leighton Ct
Fujii St
Hayward Way
Parkland Way
Gilmartin Dr
Paradise Hills Rd
Rexview Dr
Woodman St
Sweetwood St
Treewood St
Henson St
Omeara St
Leghorn Ave
Arroyo Seco Dr
Bluewood St
Anton Ln
Los Sonetto Dr
Dolo St
Lausanne Dr
Sychar Rd
Welling Way
Siena St
Deerock Pl
Theodore Dr
Daisy Ave
Daisy Pl
Petal Dr
Dillard St
Kauser St
Fergus St
Stork St
Flicker St
Pidgeon St
Joanna Dr
Beacon Dr
Bonsall St
Jacumba St
Goode St
Brookhaven Rd
Coralwood Dr
Ridgecrest Dr
Lisbon St
Aviation Dr
Eleanor Dr
Skyline Dr
Alderley St
Varney Dr
Plaza Blvd
Plaza Cuernavaca
Plaza Ridge Rd
Paradise Valley Rd
Bougainville Rd
Oriskany Rd
Sandy Shore St
Kenwell St
Angelo Dr
Munda Rd
Hayward Way
Fowler Dr
Leighton Ct
Fujii St
Alcona St
Honey Dr
Latrobe Cir
Spring Oak Way
Grove View
Montcliff Rd
Nebraska Ave
Bignell Dr
Zest St
Kingswood St
Paradise Hills Rd
Parkland Way
Gilmartin Dr
Jouglard St
Crabtree St
Elsberry St
Division St
Kauser St
Bullock Dr
Windward St
Sunswept St
Ridgecrest Dr
Deep Dell Rd
Brookhaven Rd
Coralwood Dr
Woodshawn Dr
S Siena St
Aquamarine Rd
Terra Cotta Rd
Peter Pan Ave
Tuther Way
Benson Ave
Gribble St
Woodbine Way
Montcliff Rd
Spring Oak Way
Grove View
Latrobe Cir
Honey Dr
Alcona St
Leighton Ct
Fowler Dr
Fujii St
Hayward Way
Bougainville Rd
Oriskany Rd
Sandy Shore St
Kenwell St
Angelo Dr
Munda Rd
Paradise Hills Park
Paradise Valley Rd
Plaza Ridge Rd
Jouglard St
Elsberry St
Petal Dr
Dillard St
Daisy Pl
Daisy Ave
Theodore Dr
Varney Dr
Plaza Cuernavaca
Plaza Blvd
Pittsburgh Ave
Martin Luther King Jr Memorial Park
Omeara St
Skyline Dr
Alderley St
S 61st St
Henson St
Eleanor Dr
Aviation Dr
Shaules Ave
Sullivan Ave
Treewood St
Sweetwood St
Woodman St
Rexview Dr
Leghorn Ave
S 66th St
S 65th St
S 69th St
S Woodman St
Arroyo Seco Dr
Bluewood St
Anton Ln
Los Sonetto Dr
Dolo St
Lausanne Dr
Sychar Rd
Welling Way
Siena St
Deerock Pl
Gribble St
Woodbine Way
Peter Pan Ave
Tuther Way
Benson Ave
Terra Cotta Rd
Aquamarine Rd
S Siena St
Woodshawn Dr
Coralwood Dr
Deep Dell Rd
Brookhaven Rd
Bullock Dr
Sunswept St
Ridgecrest Dr
Windward St
Brookhaven Rd
Gatewood Ln
Paradise Valley Rd
Parkwood Dr
Kingswood St
Zest St
Bignell Dr
Nebraska Ave
Goode St
Paradise Hills Rd
Parkland Way
Gilmartin Dr
Jouglard St
Crabtree St
Elsberry St
Division St
Kauser St
Dillard St
Petal Dr

SOURCE: GOOGLE EARTH, 2015.
SAGE, 2015.

LEGEND

B-5 TD=20.0' — BORING
TD=TOTAL DEPTH IN FEET

PROPOSED PHOTOVOLTAIC ARRAYS

N

SCALE IN FEET

0 100 200 400

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	2
108029003	11/15		

2_108029003_BL_93.mxd AOB

SOURCE: KENNEDY, M.P., AND TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA

LEGEND

Qls	LANDSLIDE DEPOSITS, UNDIVIDED
Qya	YOUNG ALLUVIAL FLOODPLAIN DEPOSITS
Qvop	VERY OLD PARALIC DEPOSITS
Tsdcg	SAN DIEGO FORMATION, TRANSITIONAL MARINE AND NONMARINE
Tsdss	SAN DIEGO FORMATION, MARINE SANDSTONE
To	OTAY FORMATION
Tmv	MISSION VALLEY FORMATION
U D	FAULT - SOLID WHERE ACCURATELY LOCATED, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED. ARROW AND NUMBER INDICATE DIRECTION AND ANGLE OF DIP OF FAULT PLANE
5	STRIKE AND DIP OF BEDS, INCLINED
75	STRIKE AND DIP OF METAMORPHIC FOLIATION

SCALE IN FEET

0 2,000 4,000

N

Ninyo & Moore		GEOLOGY	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	3
108029003	11/15		

3_108029003_G_93.mxd AOB

SOURCE: JENNINGS, C.W., AND BRYANT, W.A., 2010, FAULT ACTIVITY MAP OF CALIFORNIA, CALIFORNIA GEOLOGICAL SURVEY.

LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

SCALE IN MILES

0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore	FAULT LOCATIONS	FIGURE
PROJECT NO. 108029003 / DATE 11/15	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	4

4_108029003_FL_93.mxd AOB

LEGEND

GEOLOGIC HAZARDS

FAULT

INFERRED FAULT

12 POTENTIALLY ACTIVE, INACTIVE, PRESUMED INACTIVE, OR ACTIVITY UNKNOWN

LANDSLIDES

22 — 22 POSSIBLE OR CONJECTURED

SLIDE-PRONE FORMATIONS

27 — 27 OTAY, SWEETWATER, AND OTHERS

LIQUEFACTION

32 — 32 LOW POTENTIAL -- FLUCTUATING GROUNDWATER MINOR DRAINAGES

OTHER TERRAIN

52 — 52 OTHER LEVEL AREAS, GENTLY SLOPING TO STEEP TERRAIN, FAVORABLE GEOLOGIC STRUCTURE, LOW RISK

53 — 53 LEVEL OR SLOPING TERRAIN, UNFAVORABLE GEOLOGIC STRUCTURE, LOW TO MODERATE RISK

AREA NOT MAPPED

SOURCE: SANGIS, 2008, CITY OF SAN DIEGO SEISMIC SAFETY STUDY GEOLOGIC HAZARDS AND FAULTS.

SCALE IN FEET

0 2,000 4,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		SAN DIEGO SAFETY ELEMENT	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	5
108029003	11/15		

5_108029003_GH.mxd AOB

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1⅜ inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the Modified Split-Barrel Drive Sampler. The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a 140-pound hammer, in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0	■							Bulk sample.
		■						Modified split-barrel drive sampler.
		▲						2-inch inner diameter split-barrel drive sampler.
		☒						No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
		▌						Sample retained by others.
5		◤						Standard Penetration Test (SPT).
		╱						No recovery with a SPT.
		▭	XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
		╱						No recovery with Shelby tube sampler.
		▤						Continuous Push Sample.
10				♀				Seepage.
				▽				Groundwater encountered during drilling.
				▼				Groundwater measured after drilling.
						▦	SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
						▨	CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

SOIL CLASSIFICATION CHART PER ASTM D 2488

PRIMARY DIVISIONS			SECONDARY DIVISIONS	
			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

PLASTICITY CHART

APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

DATE DRILLED 10/23/15 BORING NO. B-1

GROUND ELEVATION 340' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY NMM LOGGED BY NMM REVIEWED BY JMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	FILL: Light reddish brown to brown, moist, stiff, sandy CLAY; trace gravel.
		■	87/10"	16.5				
								OTAY FORMATION: Brown, moist, moderately indurated, sandy CLAYSTONE; trace fine gravel; carbonate deposits.
10		◤	44					Light brown, moist, weakly to moderately cemented, silty fine-grained SANDSTONE; few clay; scattered lenses of clay.
		■	84/9"	17.7	108.6			Clayey fine-grained sandstone.
20								Total Depth = 16.3 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 10/23/15. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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BORING LOG

PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
108029003	11/15	A-1

DATE DRILLED 10/23/15 BORING NO. B-2

GROUND ELEVATION 340' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY NMM LOGGED BY NMM REVIEWED BY JMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, moist, loose to medium dense, clayey fine SAND.
			13				CL	Light reddish brown to brown, moist, stiff, sandy CLAY; scattered layers of silty sand.
10			26	25.6	92.2			Very stiff; increased sand.
								Brown; trace gravel.
			16					
			27	19.2	105.0			
20								Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 10/23/15. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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BORING LOG

PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
108029003	11/15	A-2

DATE DRILLED 10/23/15 BORING NO. B-3

GROUND ELEVATION 355' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY NMM LOGGED BY NMM REVIEWED BY JMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brownish gray, dry to moist, loose to medium dense, silty fine to medium SAND; trace gravel.
							SC	Yellowish to reddish brown, moist, medium dense, clayey fine to medium SAND.
		■	50/5"	10.7	111.9			OTAY FORMATION: Light brown to light grayish brown, dry to moist, weakly to moderately cemented, silty fine-grained SANDSTONE; scattered lenses of clayey sandstone and claystone.
10		◥	62					
	■							Grayish brown, moist, moderately indurated, sandy CLAYSTONE; manganese-oxide staining.
	■	■	50/2"	13.4	119.1			
								Total Depth = 15.7 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 10/23/15.
20								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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BORING LOG

PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
108029003	11/15	A-3

DATE DRILLED 10/23/15 BORING NO. B-4

GROUND ELEVATION 355' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY NMM LOGGED BY NMM REVIEWED BY JMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	FILL: Reddish brown to brown, moist, stiff, silty CLAY; trace sand; scattered chunks of well indurated claystone.
		◤	22					Very stiff to hard.
10		■	43	17.1	107.9			
		◤	65					OTAY FORMATION: Light brown to brown, moist, weakly cemented, silty fine-grained SANDSTONE; scattered layers of moderately cemented sandstone.
		◤	50/6"					Manganese-oxide staining.
20								Total Depth = 19 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 10/23/15.
30								
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Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING LOG

PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
108029003	11/15	A-4

DATE DRILLED 10/23/15 BORING NO. B-5

GROUND ELEVATION 355' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY NMM LOGGED BY NMM REVIEWED BY JMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Light reddish brown to brown, moist, loose to medium dense, clayey SAND.
							CH	Light reddish brown to grayish brown, moist, very stiff, sandy CLAY.
	■		42	20.5	94.5			Very stiff to hard; claystone pieces; scattered layers of clayey sand.
10		◥	22				SC	Yellowish brown and light reddish brown, moist, medium dense, clayey SAND; scattered pieces of claystone.
		■	38	20.6	100.9			Reddish brown.
		◥	23				CL	Reddish brown, moist, very stiff to hard, sandy CLAY; many chunks of claystone and siltstone.
20								Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 10/23/15. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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Ninyo & Moore

BORING LOG

PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM
MORSE HIGH SCHOOL, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
108029003	11/15	A-5

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. The test results were utilized in evaluating the soil classification in accordance with the USCS.

Atterberg Limits

Tests were performed on a selected representative fine-grained soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classification are shown on Figure B-2.

Direct Shear Test

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-3.

Expansion Index Tests

The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-4.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on a representative sample in general accordance with CT 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-5.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	5.0-10.0	--	--	--	--	--	--	--	--	68	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	B-1
108029003 / 11/15		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-5	5.0-6.5	53	28	25	CH	CH

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. 108029003 / DATE 11/15	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	B-2

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Sandy CLAY	—●—	B-2	10.0-11.5	Peak	100	48	CL
Sandy CLAY	- - x - -	B-2	10.0-11.5	Ultimate	100	38	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	B-3
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SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	1.0-5.0	13.0	100.2	27.5	0.074	75	Medium
B-2	5.0-10.0	12.5	99.8	25.1	0.088	87	Medium
B-4	0.0-4.0	15.5	90.1	32.4	0.068	67	Medium

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

Ninyo & Moore		EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	B-4
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SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-4	0.0-4.0	8.9	380	410	0.041	700

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROPOSED PHOTOVOLTAIC SOLAR GENERATION SYSTEM MORSE HIGH SCHOOL SAN DIEGO, CALIFORNIA	B-5
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