NorCal Engineering
SOILS AND GEOTECHNICAL CONSULTANTS
10641 HUMBOLT STREET LOS ALAMITOS, CA 90720
(562)799-9469 FAX (562)799-9459

January 11, 2019 Project Number 20529-18

Molto Properties
18W140 Butterfield Road, Suite 750
Oakbrook Terrace, Illinois 60181

RE: **Response to Riverside County Review Comments dated December 5, 2018** - Proposed Warehouse Development - Located at the Southeast Corner of Perry Street and Seaton Avenue, Mead Valley, in the County of Riverside, California

Dear Sir or Madam:
The following is in response to the above referenced review of our report dated July 23, 2018. Items in the review are addressed in order below.

Item 1
The property is not located in an Alquist-Priolo (AP) earthquake fault zone. Several stereo pair aerial photographs, as referenced below, were reviewed to evaluate for any lineaments or fault-related geomorphic features within, adjacent or trending towards the property. No indications of natural lineaments or other fault-related features indicative of Holocene or older faulting were noted. No indications of faulting were noted during our reconnaissance at and in the vicinity of the site. Based on our evaluation, we conclude that there are no active or potentially active faults trending towards or through the property, and additional fault investigations are not necessary. The potential for surface fault rupture to occur at the site is considered low. As is the case with most of southern California, the property is expected to experience strong ground shaking during the lifetime of the project.

Photos
RC 57-108_1964-89, 90, 91, 95, 96, 97, 101, 102, 103, 104

Source
The Joseph Andrew Rowe Water Resources Archives, Historic Aerial Photo Collection, Water Resources Institute, CSUSB, Moreno Valley Photo Collection, Riverside County Flood Control

Item 2

Geologic Setting

The property is located in the Peninsular Ranges geomorphic province of California. The Peninsular Ranges province extends from the Los Angeles Basin southeast to Baja California and from the Pacific Ocean eastward to the Coachella Valley and Colorado Desert. The province consists of numerous northwest to southeast-trending mountain ranges and valleys that are geologically controlled by several major active faults. The subject site is located in the central part of the Perris block, a generally stable area situated roughly midway between two of these major faults; the Chino /Elsinore and San Jacinto fault zones. More specifically, the property is situated on the western flank of the Perris Valley drainage near the base of the east-facing hillside to the west.

Site Geology

The USGS Open File Report for the Steele Peak 7.5' Quadrangle assigns the soil materials underlying the site as early Pleistocene older alluvial fan deposits. These sediments are, in turn, underlain by Cretaceous granitic rocks of the Val Verde Pluton. The older alluvium is described in general as mostly well-dissected, well-indurated sand deposits. The underlying bedrock is described as relatively homogeneous, massive- to well-foliated, medium- to coarse-grained, biotite-hornblende tonalite. Exploration at the property encountered the alluvium ("Native Soils") to a maximum depth of 11 feet overlying the bedrock. The attached Regional Geologic Map shows the distribution of the alluvial sediments and bedrock in the vicinity of the property.

Item 3

Site elevation of the site ranges from approximately ranges 1521 to 1539 feet.

Item 4

According to the Riverside County Hazards report (Earth Consultants International, 2001), subsidence in Riverside County has been linked to significant fluctuations in groundwater levels within deep alluvial basins, and generally, the subsidence occurs throughout the valley region. Three areas have been identified with documented subsidence; the Elsinore Trough, the San Jacinto Valley, and the southern Coachella Valley. The subject property is situated on shallow alluvium with no groundwater, with historic groundwater levels in the vicinity at depths of greater than 50 feet. Additionally, the property is not situated within any of the three areas of Riverside County associated with documented subsidence. The potential for subsidence to impact the site is considered low.

Item 5
A Regional Geologic Map is attached to this response report.

Item 6
The natural soils encountered in the exploratory excavations are classified as early Pleistocene older alluvium.

Item 7
Soils removals shall be made as recommended in our previous report. "Competent" native soils at bottom of excavations shall be defined as soils exhibiting a relative compaction of 85% or greater. If bedrock is exposed bottom of excavation is made into the dense granitic bedrock, the bottom of excavation shall also be deemed compentent.

Item 8
Approximate Elevations at top of Test Excavations

Excavation	Elevation (ft)
T-1	1521
T-2	1521
T-3	1521
T-4	1527
T-5	1535
T-6	1536
T-7	1532
T-8	1535
T-9	1530
T-10	1524
T-11	1526
T-12	1529
T-13	1531

Item 9
This response has been prepared in conjunction with Andrew Stone, C.E.G., whose signature and stamp are included below.

We appreciate this opportunity to be of service to you. If you have any further questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
NORCAL ENGINEERING

Keith D. Tucker
Project Engineer
R.G.E. 841

Mark A. Burkholder
Project Manager

Andrew Stone
Project Geologist
C.E.G. 1648

RIVERSIDE COUNTY
PLANNING DEPARTMENT

Charissa Leach, P.E.
Assistant TLMA Director

December 5, 2018

NorCal Engineering
Attn: Keith Tucker
Fax: 562-799-9459

RE: Review Comments
County Geologic Report No. 180039
"Geotechnical Investigation, Proposed Warehouse Development, Southeast Corner Perry Street and Seaton Avenue, Mead Valley, County of Riverside, California," dated July 23, 2018.

County Geologic Report GEO No. 180039, submitted for the project PPT180025, APN 314-130-007, was prepared by NorCal Engineering, and is titled; "Geotechnical Investigation, Proposed Warehouse Development, Southeast Corner Perry Street and Seaton Avenue, Mead Valley, County of Riverside, California," dated July 23, 2018.

Prior to scheduling this project for public hearing, the following clarification and/or additional information shall be submitted to the County Geologist for review and approval:

1. The consultant should provide an evaluation of the potential for surface rupture at the site using the positive lines of evidence (aerial photo analysis, site geologic mapping, etc.).

2. Please provide a discussion on the regional geologic setting including geomorphic province description, geomorphology of the project site, and geology of the vicinity.

3. Provide the elevation range for the site.

4. The site is within an area mapped as susceptible to subsidence due to groundwater withdrawal. The consultant should provide an evaluation of the potential for subsidence to occur at the site.

5. Please provide a geologic map with north arrow, a minimum scale of (1in:2,000 ft.), legend, and reference indication.

6. Please provide an estimation of age for the "Natural Soils" encountered the exploratory trenches.

7. Provide the criteria for establishing suitability of soil and/or rock to be left-in-place (removal bottoms), which should be demonstrated using appropriate qualitative and/or quantitative assessments. Qualitative assessments could include criteria such as removing unsuitable soils to expose bedrock, while quantitative assessments could include criteria based on such physical properties as unit weight, degree of saturation, in-situ relative compaction, or hydrocollapse analysis results. These assessments should be tied to site-specific data gathered from the subsurface investigation program, and will ultimately form the basis for determining removal depths during construction. Simply using terms such as "competent", "dense",

"hard", "unyielding", or "undisturbed" without supporting quantitative and/or qualitative data is not sufficient.

8. Provide a Top of Hole elevation for each exploratory trench log.

9. Provide documentation that the site geologic data presented was prepared and reviewed by a Professional Geologist or Certified Engineering Geologist licensed in the State of California, who is familiar with the site. In accordance with California Code of Regulations, Title 16, Division 29 §§3003 (f)(1)(2), the report is to be signed and stamped by a Certified Engineering Geologist.

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

Please email me at dwalsh@rivco.org if you have any questions.

Sincerely,

RIVERSIDE COUNTY PLANNING DEPARTMENT
Charissa Leach, Assistant TLMA Director

Daniel P. Walsh, CEG No. 2413
Associate Engineering Geologist, TLMA-Planning

cc: Planner: John Hildebrand, Riverside Office (jhildebr@rivco.org)
Applicant: LDC Industrial Realty, LLC; Attn: Larry Cochrun (lcochrun@ldcindustrial.com)
Eng/Rep: T&B Planning; Attn: George Atalla (gatalla@tbplanning.com)

File: GEO180039, PPT180025, APN 314-130-007

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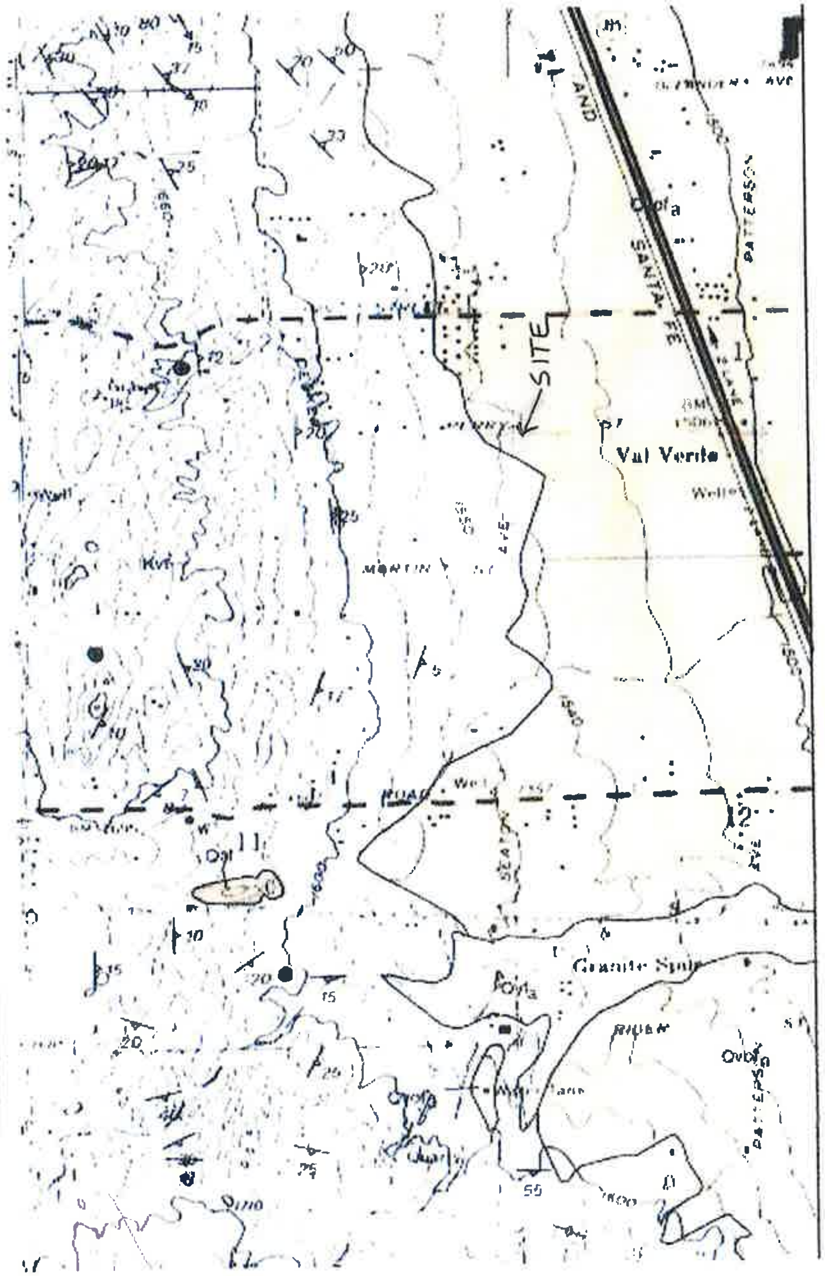
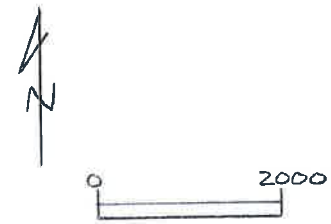


REGIONAL GEOLOGIC MAP



Qvof	**Very old alluvial fan deposits (early Pleistocene)**—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Covers large areas adjacent to U.S. Highway 215 in northeastern part of quadrangle and flanking drainage followed by Cajalco Road
Kvt	**Val Verde tonalite**—Gray-weathering, relatively homogeneous, massive- to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite; principal rock type of Val Verde pluton. Contains subequal biotite and hornblende, quartz and plagioclase. Potassium feldspar generally less than two percent of rock. Where present, foliation typically strikes northwest and dips moderately to steeply northeast. Northern part of pluton contains younger, intermittently developed, northeast-striking foliation. In central part of pluton, tonalite is mostly massive, and contains few segregational masses of mesocratic to melanocratic tonalite. Elliptical- to pancake-shaped, meso-to melanocratic inclusions are common

Portion of: *Geologic Map of the Steele Peak 7.5' Quadrangle, Riverside County, California,* Morton, D.M., 2001, USGS Open-File Report 01-449

GEOTECHNICAL INVESTIGATION
Proposed Warehouse Development
Southeast Corner Perry Street and Seaton Avenue
Mead Valley, County of Riverside, California

Molto Properties
18W140 Butterfield Road, Suite 750
Oakbrook Terrace, Illinois 60181

Project Number 20529-18
July 23, 2018

NorCal Engineering

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SOILS AND GEOTECHNICAL CONSULTANTS
10641 HUMBOLT STREET LOS ALAMITOS, CA 90720
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July 23, 2018

Project Number 20529-18

Molto Properties
18W140 Butterfield Road, Suite 750
Oakbrook Terrace, Illinois 60181

RE: **GEOTECHNICAL INVESTIGATION** - Proposed Warehouse Development - Located at the Southeast Corner of Perry Street and Seaton Avenue, Mead Valley, in the County of Riverside, California

Dear Sir or Madam:

Pursuant to your request, this firm has performed a Geotechnical Investigation for the above referenced project. The purpose of this investigation is to evaluate the geotechnical conditions of the subject site and to provide recommendations for the proposed development. This geotechnical engineering report presents the findings of our study along with conclusions and recommendations for development.

1.0 STRUCTURAL CONSIDERATIONS

1.1 Proposed Development

It is proposed to construct a new warehouse development consisting of a concrete tilt-up structure totaling 208,300 square feet along with associated pavement areas on the 9.15-acre site. Grading for the future development will include cut and fill procedures. Final building plans shall be reviewed by this firm prior to submittal for city approval to determine the need for any additional study and revised recommendations pertinent to the proposed development, if necessary.

2.0 SITE DESCRIPTION

2.1 **Location:** The rectangular shaped subject property is located at the southeast corner of Perry Street and Seaton Avenue, in the Mead Valley area of the County of Riverside, as illustrated on Figure 1, Vicinity Map.

2.2 **Existing Improvements:** The property is currently vacant with some scattered vegetation.

2.3 **Topography/Drainage:** The site topography is generally level and drainage appears to be via sheetflow toward the southwest.

3.0 SEISMICITY EVALUATION

The proposed development lies outside of any Alquist Priolo Special Studies Zone and the potential for damage due to direct fault rupture is considered unlikely.

The following seismic design parameters are provided and are based upon the 2016 California Building Code (CBC) for the referenced project. Seismic design printouts from the USGS website are included in Appendix A.

Seismic Design Parameters

Site Location – Region 1	Latitude	33.8474°
	Longitude	-117.2605°
Seismic Use Group	II	
Site Class	D	
Risk Category	I/II/III	
Maximum Spectral Response Acceleration	S_S	1.500g
	S_1	0.600g
Adjusted Maximum Acceleration	S_{MS}	1.500g
	S_{M1}	0.900g
Design Spectral Response Acceleration Parameters	S_{DS}	1.000g
	S_{D1}	0.600g

The San Jacinto (San Jacinto Valley) Fault zone is located approximately 15 kilometers from the site and is capable of producing a Magnitude 6.9 earthquake. Ground shaking originating from earthquakes along other active faults in the region is expected to induce lower horizontal accelerations due to smaller anticipated earthquakes and/or greater distances to other faults.

4.0 FIELD INVESTIGATION

4.1 Site Exploration

The purpose of the investigation was to explore the subsurface conditions and to provide preliminary geotechnical engineering design parameters for evaluation of the site with respect to the proposed development.

The current investigation consisted of the placement of thirteen excavations by backhoe. The explorations extended to a maximum depth of 15.5 feet below current ground elevations.

The explorations were visually classified and logged by a field engineer with locations of the subsurface borings and excavations shown on the attached Figure 2. Detailed descriptions of the subsurface conditions are listed on the boring/excavation logs in Appendix A. It should be noted that the transition from one soil type to another as shown on the logs is approximate and may in fact be a gradual transition. The soils encountered are described as follows:

Disturbed Topsoils/Fill Soils – Disturbed topsoils and minor amounts of fill soils classifying as sandy, clayey SILTS and clayey SANDS with some gravel, roots and minor other organics were encountered in the explorations to approximately 12 inches below existing surface. These soils were noted to be soft/loose and generally dry.

Native Soils – Native soils also classifying as sandy SILT with some clay to clayey SAND were encountered beneath the upper fill soils. These soils were noted to be medium stiff/dense to stiff/dense and damp. Sand, silt and clay content varied with depth of explorations and slightly decomposed, dense to very dense granitic bedrock was encountered at depths varying from 3 to 11 feet below existing ground surface.

4.2 Groundwater

Groundwater was *not* encountered in any of our subsurface explorations. Historic high groundwater in the vicinity has been recorded deeper than 50 feet below grade, based upon information from the California Department of Water Resources database http://www.water.ca.gov/waterdatalibrary/.

5.0 LABORATORY TESTS

Relatively undisturbed samples of the subsurface soils were obtained to perform laboratory testing and analysis for direct shear, consolidation tests, and to determine in-place moisture/densities. These relatively undisturbed ring samples were obtained by driving a thin-walled steel sampler lined with one-inch long brass rings with an inside diameter of 2.42 inches into the undisturbed soils.

Bulk bag samples were obtained in the upper soils for expansion index tests, corrosion tests and maximum density tests. Wall loadings on the order of 4,000 lbs./lin.ft. and maximum compression loads on the order of 100 kips were utilized for testing and design purposes. All test results are included in Appendix B, unless otherwise noted.

5.1 **Field moisture content** (ASTM:D 2216-10) and the dry density of the ring samples were determined in the laboratory. This data is listed on the logs of explorations.

5.2 **Maximum density tests** (ASTM: D-1557-12) were performed on typical samples of the upper soils. Results of these tests are shown on Table I.

5.3 **Expansion index tests** (ASTM: D-4829-11) were performed on remolded samples of the upper soils to determine the expansive characteristics and to provide any necessary recommendations for reinforcement of the slabs-on-grade and the foundations. Results of these tests are provided on Table II and are discussed later in this report.

5.4 **Direct shear tests** (ASTM: D-3080-11) were performed on undisturbed and/or remolded samples of the subsurface soils. These tests were performed to determine parameters for the calculation of the allowable soil bearing capacity. The test is performed under saturated conditions at loads of 1,000 lbs./sq.ft., 2,000 lbs./sq.ft., and 3,000 lbs./sq.ft. with results shown on Plates A-C.

5.5 **Consolidation tests** (ASTM: D-2435-11) were performed on undisturbed samples to determine the differential and total settlement which may be anticipated based upon the proposed loads. Water was added to the samples at a surcharge of one KSF and the settlement curves are plotted on Plates D-F.

5.6 **Soluble sulfate, pH, Resistivity and Chloride tests** to determine potential corrosive effects of soils on concrete and metal structures were performed in the laboratory. Test results are given in Tables III - VI.

5.7 **Resistance 'R' Value tests** (CA 301) were conducted on a representative soil sample to determine preliminary pavement section design for the proposed pavement areas. Test results are provided in Table VII and recommended pavement sections are provided later within the text of this report.

6.0 LIQUEFACTION EVALUATION

Due to groundwater levels recorded in excess of 50 feet in the vicinity and near surface very dense granitic bedrock, the liquefaction potential at the site is deemed *low* and the design of the proposed construction in conformance with the latest Building Code provisions for earthquake design is expected to provide mitigation of ground shaking hazards that are typical to Southern California.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based upon our evaluations, the proposed development is acceptable from a geotechnical engineering standpoint. By following the recommendations and guidelines set forth in our report, the structures and grading will be safe from excessive settlements under the anticipated design loadings and conditions. The proposed grading and development shall meet all requirements of the City Building Ordinance and will not impose any adverse effect on existing adjacent land or structures.

The following recommendations are based upon soil conditions encountered in our field investigation; these near-surface soil conditions could vary across the site. Variations in the soil conditions may not become evident until the commencement of grading operations for the proposed development and revised recommendations from the soils engineer may be necessary based upon the conditions encountered.

7.1 Site Grading Recommendations

It is recommended that site inspections be performed by a representative of this firm during all grading and construction of the development to verify the findings and recommendations documented in this report. Any unusual conditions which may be encountered in the course of the project development may require the need for additional study and revised recommendations.

Any vegetation shall be removed and hauled from proposed grading areas prior to the start of grading operations. Existing vegetation shall not be mixed or disced into the soils. Any removed soils may be reutilized as compacted fill once any deleterious material or oversized materials (in excess of eight inches) is removed. Grading operations shall be performed in accordance with the attached *Specifications for Placement of Compacted Fill*.

7.1.1 Removal and Recompaction Recommendations

The upper 12 inches of existing fill soils and any other low-density soils encountered shall be removed to competent native materials, the exposed surface scarified to a depth of 8 inches, brought to within 2% of optimum moisture content and compacted to a minimum of 90% of the laboratory standard (ASTM: D-1557-12) prior to placement of any additional compacted fill soils and pavement. The upper 12 inches of soils beneath concrete building slabs and truck traffic slabs shall be compacted to a minimum of 95% relative compaction.

Grading shall extend a minimum of 5 horizontal feet outside the edges of foundations or equidistant to the depth of fill placed, whichever is greater. Care should be taken to provide or maintain adequate lateral support for all adjacent improvements and structures at all times during the grading operations and construction phase. Adequate drainage away from the structures, pavement and slopes should be provided at all times.

It is likely that isolated areas of undiscovered fill, subsurface structures and utility lines not described in this report or materials disturbed during demolition operations will be encountered during site grading; if found, these areas should be excavated and backfilled as discussed earlier. Any existing structures and lines shall be either removed or properly abandoned prior to the proposed construction. Abandonment procedures will be provided if/when underground structures are encountered.

If placement of slabs-on-grade and pavement is not performed immediately upon completion of grading operations, additional testing and grading of the areas may be necessary prior to continuation of construction operations. Likewise, if adverse weather conditions occur which may damage the subgrade soils, additional assessment by the soils engineer as to the suitability of the supporting soils may be needed.

7.1.2 **Fill Blanket Recommendations**

Due to the potential for differential settlement of structures supported on both compacted fill and native materials, it is recommended that all foundations be underlain by a uniform compacted fill blanket at least 2 feet in thickness. The fill blanket shall extend a minimum of 5 horizontal feet outside the edges of foundations or equidistant to the depth of fill placed, whichever is greater.

7.2 Shrinkage and Subsidence

Results of our in-place density tests reveal that the soil shrinkage will be on the order of 5 to 8% due to excavation and recompaction, based upon the assumption that the fill is compacted to 92% of the maximum dry density per ASTM standards. Subsidence should be 0.10 feet due to earthwork operations. The volume change does not include any allowance for vegetation or organic stripping, removal of subsurface improvements or topographic approximations.

Although these values are only approximate, they represent our best estimate of shrinkage values which will likely occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field testing using the actual equipment and grading techniques should be conducted.

7.3 Temporary Excavations and Shoring Design

Temporary unsurcharged excavations including utility trenches less than 4 feet in height may be excavated at vertical inclinations. Excavations over 4 feet in height in the existing site materials may be trimmed at a 1 to 1 (horizontal to vertical) gradient. Any excavation in excess of 10 feet in height should be evaluated further by the soil engineer prior to work. In areas where soils with little or no binder are encountered, where adverse geological conditions are exposed, or where excavations are adjacent to existing structures, shoring, slot-cutting, or flatter excavations may be required.

The temporary cut slope gradients given above do not preclude local raveling and sloughing. All excavations shall be made in accordance with the requirements of the soils engineer, CAL-OSHA and other public agencies having jurisdiction.

Temporary shoring design may utilize an active earth pressure of 25 pcf without any surcharge due to adjacent traffic, equipment or structures. The passive fluid pressures of 250 pcf may be doubled to 500 pcf for temporary design.

7.4 Foundation Design

All foundations may be designed utilizing the following allowable soil bearing capacities for embedded depths of 18 inches into dense compacted fill materials with the corresponding widths. Footings shall be situated on the recommended compacted fill blanket and shall not traverse from compacted fill to native soils due to the potential for differential settlement of structures.

Allowable Soil Bearing Capacity (psf)

Width (ft)	Continuous Foundation	Isolated Foundation
1.5	2200	2700
2.0	2275	2775
4.0	2575	3075
6.0	2875	3375

The bearing value may be increased by 500 psf for each additional foot of depth in excess of the 24-inch minimum depth, up to a maximum of 4,000 psf. Property line screen wall foundations extended a minimum of 18 inches in depth and at least 8 inches into medium stiff/dense native soils may be designed using a reduced allowable soil bearing capacity of 1800 psf. A one-third increase may be used when considering short term loading from wind and seismic forces.

All continuous foundations shall be reinforced with a minimum of two #4 bars top and two bottom. Isolated pad foundations shall be reinforced at the discretion of the project structural engineer. An increase in steel reinforcement due to soil expansion or proposed loadings may be necessary and shall be determined by the project engineers. A representative of this firm shall observe foundation excavations prior placement of reinforcement steel and concrete.

7.5 Settlement Analysis

Resultant pressure curves for the consolidation tests are shown on Plates D-F. Computations utilizing these curves and the recommended allowable soil bearing capacities reveal that the foundations will experience normal (not seismically induced) settlements on the order of 3/4 inch and differential settlements of less than 1/4 inch.

7.6 Lateral Resistance

The following values may be utilized in resisting lateral loads imposed on the structure. Requirements of the California Building Code should be adhered to when the coefficient of friction and passive pressures are combined.

Coefficient of Friction - 0.35
Equivalent Passive Fluid Pressure = 200 lbs./cu.ft.
Maximum Passive Pressure = 2,000 lbs./cu.ft.

The passive pressure recommendations are valid only for approved compacted fill soils or competent native ground.

7.7 Retaining Wall Design Parameters

Active earth pressures against retaining walls will be equal to the pressures developed by the following fluid densities. These values are for **granular backfill material** placed behind the walls at various ground slopes above the walls. If fine-grained soils are exposed behind retaining walls, revised recommendations may be required.

Surface Slope of Retained Materials (Horizontal to Vertical)	Equivalent Fluid Density (lb./cu.ft.)
Level	30
5 to 1	35
4 to 1	38
3 to 1	40
2 to 1	45

Any applicable short-term construction surcharges and seismic forces should be added to the above lateral pressure values. All walls shall be waterproofed as needed and protected from hydrostatic pressure by a reliable permanent subdrain system.

7.8 Floor Slab Design

Concrete floor slabs-on-grade shall be a minimum of 4 and 6 inches in thickness in office and warehouse areas, respectively, and may be placed upon fill soils compacted to a minimum of 95% relative compaction in the upper 12 inches. Steel reinforcement should consist of #3 bars at 18 inch spacing, each way, placed mid-height in the slab. Steel reinforcement may be deleted in 7-inch thick slabs. Additional reinforcement requirements and an increase in thickness of the slabs-on-grade may be necessary based upon soils expansion potential and proposed loading conditions in the structures and should be evaluated further by the project engineers and/or architect.

A vapor retarder should be utilized in areas which would be sensitive to the infiltration of moisture. This retarder shall meet requirements of ASTM E 96, *Water Vapor Transmission of Materials* and ASTM E 1745, *Standard Specification for Water Vapor Retarders used in Contact with Soil or Granular Fill Under Concrete Slabs.* The vapor retarder shall be installed in accordance with procedures stated in ASTM E 1643, *Standard practice for Installation of Water Vapor Retarders used in Contact with Earth or Granular Fill Under Concrete Slabs.*

The moisture retarder may be placed directly upon compacted subgrade, although 1 to 2 inches of sand beneath the membrane is desirable. The subgrade upon which the retarder is placed shall be smooth and free of rocks, gravel or other protrusions which may damage the retarder. Use of sand above the retarder is under the purview of the structural engineer; if sand is used over the retarder, it should be placed in a dry condition.

Subgrade soils shall be moistened to approximately 2% above optimum moisture levels to a depth of 18 inches immediately prior to pouring of concrete, as verified by the soil engineer. All concrete slab areas to receive floor coverings should be moisture tested to meet all manufacturer requirements prior to placement.

7.9 Expansive Soil

The upper on-site soils at the site are low in expansion potential (Expansion Index = 21-50). Sites with expansive soils (Expansion Index >20) require special attention during project design and maintenance. The attached *Expansive Soil Guidelines* should be reviewed by the engineers, architects, owner, maintenance personnel and other interested parties and considered during the design of the project and future property maintenance.

7.10 Utility Trench and Excavation Backfill

Trenches from installation of utility lines and other excavations may be backfilled with on-site soils or approved imported soils compacted to a minimum of 90% relative compaction. All utility lines shall be properly bedded and shaded with clean sand having a sand equivalency rating of 30 or more. This material shall be thoroughly water jetted around the pipe structure prior to placement of compacted backfill soils.

7.11 Corrosion Design Criteria

Representative samples of the surficial soils revealed negligible sulfate concentrations and no special concrete design recommendations are deemed necessary at this time. It is recommended that additional sulfate tests be performed at the completion of rough grading to assure that the as graded conditions are consistent with the recommendations stated in this design. Sulfate test results may be found on the attached Table III.

Tests were also conducted on a random representative sample of soils to determine the potential corrosive effects on buried metallic structures. Tests for pH, resistivity and chloride are included on Tables IV – VI. Soil pH indicates a relatively neutral condition. Resistivity was measured at 2460 ohm-centimeters, a condition which may be considered corrosive to metallic structures. Chloride content tested at 158 ppm.

A corrosion engineer may be consulted regarding protection of buried metallic piping.

7.12 **Preliminary Pavement Design**

The table below provides a preliminary pavement design based upon a tested R-Value of 51 for the proposed pavement areas. Final pavement design may need to be based on R-Value testing of the subgrade soils near the conclusion of rough grading to assure that the as-graded conditions are consistent with those used in this preliminary design.

On-Site Flexible (Asphaltic) Pavement Section Design

Type of Traffic	Traffic Index	Inches Asphalt	Inches Base
Auto Parking/Circulation	5.0	3.0	3.0
Truck	7.0	4.0	5.0

Subgrade soils to receive base material shall be compacted to a minimum of 90% relative compaction; base material shall be compacted to at least 95%. Any concrete slab-on-grade in pavement areas shall be a minimum of 6 inches in thickness and may be placed on subgrade soils compacted to at least 95% relative compaction and moistened to approximately 3% above optimum levels to a depth of 18 inches. An increase in slab thickness and placement of steel reinforcement due to loading conditions and soil expansion may be necessary and should be reviewed by the structural engineer.

The above recommendations are based upon estimated traffic loadings. Client should submit anticipated traffic loadings for the pavement areas to the soils engineer, when available, so that pavement sections may be reviewed to determine adequacy to support the proposed loadings.

8.0 INFILTRATION TESTING

Three test locations (T-1, T-2 and T-3) were excavated to determine the infiltration rate of the proposed infiltration/bio-retention systems. The test locations were excavated by backhoe to depths ranging from 5 to 10 feet below existing ground surface (bgs). Excavations were trimmed at 1:1 (horizontal to vertical) inclinations in order to provide safe entry into the excavations.

The infiltration test consisted of the double ring infiltration test per ASTM Method D 3385. The double ring infiltrometer method consists of driving two open cylinders, one inside the other, into the ground, partially filling the ring with water, and then maintaining the liquid at a constant level. The volume of liquid added to the inner ring, to maintain the liquid level constant is the measure of the volume of liquid that infiltrates into the soil.

The volume infiltrated during timed intervals is converted to an incremental infiltration velocity, usually expressed in centimeters per hour or inches per hour and plotted verses elapsed time. The maximum-steady state or average incremental infiltration velocity, depending on the purpose/application of the test is equivalent to the infiltration rate.

Water levels were maintained at a constant level in both the inner ring and annular space between rings throughout the test, to prevent flow of water from one ring to the other.

The volume of liquid used during each measured time interval was converted into an incremental infiltration velocity of both the inner ring in the annular space using the following equations:

For the inner ring calculated as follows:

Vir=ΔVir/(AirΔt)

where:

Vir = inner ring incremental infiltration velocity, cm/hr

ΔVir = volume of water used during time interval to maintain constant head in the inner ring, cm^3

Air = internal area of the inner ting, cm^2

Δt = time interval, hr

An average of the final readings obtained was used for design purposes in each of the basins. The testing data sheets are attached in Appendix D and summarized in the *Discussion of Results* section below.

The use of on-site disposal system by means of retention/infiltration basins appears to be geotechnically feasible for future development. The field infiltration rates given below may be utilized in the final basin design with a safety factor of 2.0 or greater.

Test No.	Depth (feet bgs)	Soil Type	Infiltration Rate (cm/hr)	(in/hr)
T-1	5.0	sandy Silt w/clay	0.9	0.36
T-2	10.0	clayey Sand	5.4	2.16
T-3	7.5	sandy Silt w/clay	1.7	0.68

It is our opinion that the site is generally suitable for stormwater infiltration without increasing the potential of settlement of proposed and existing structures or adversely affecting retaining/basement walls located either on or adjacent to the subject site. In addition, the potential for hydro-consolidation and the susceptibility for any ground settlements are considered low. All systems shall meet the California Regional Water Quality Control Board (CRWQCB) requirements.

9.0 CLOSURE

The recommendations and conclusions contained in this report are based upon the soil conditions uncovered in our test excavations. No warranty of the soil condition between our excavations is implied. NorCal Engineering should be notified for possible further recommendations if unexpected to unfavorable conditions are encountered during construction phase. It is the responsibility of the owner to ensure that all information within this report is submitted to the Architect and appropriate Engineers for the project.

This firm should have the opportunity to review the final plans (72 hours for review required) to verify that all our recommendations are incorporated. This report and all conclusions are subject to the review of the controlling authorities for the project.

A preconstruction conference should be held between the developer, general contractor, grading contractor, city inspector, architect, and soil engineer to clarify any questions relating to the grading operations and subsequent construction. Our representative should be present during the grading operations and construction phase to certify that such recommendations are complied within the field.

This geotechnical investigation has been conducted in a manner consistent with the level of care and skill exercised by members of our profession currently practicing under similar conditions in the Southern California area. No other warranty, expressed or implied is made.

We appreciate this opportunity to be of service to you. If you have any further questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
NORCAL ENGINEERING

REGISTERED PROFESSIONAL ENGINEER
KEITH D. TUCKER
No. 841
Exp. 12/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

Keith D. Tucker
Project Engineer
R.G.E. 841

Mark A. Burkholder
Project Manager

SPECIFICATIONS FOR PLACEMENT OF COMPACTED FILL

Excavation
Any existing low-density soils and/or saturated soils shall be removed to competent natural soil under the inspection of the Soils Engineering Firm. After the exposed surface has been cleansed of debris and/or vegetation, it shall be scarified until it is uniform in consistency, brought to the proper moisture content and compacted to a minimum of 90% relative compaction (in accordance with ASTM: D-1557-12).

In any area where a transition between fill and native soil or between bedrock and soil are encountered, additional excavation beneath foundations and slabs will be necessary in order to provide uniform support and avoid differential settlement of the structure. Verification of elevations during grading operations will be the responsibility of the owner or his designated representative.

Material For Fill
The on-site soils or approved import soils may be utilized for the compacted fill provided they are free of any deleterious materials and shall not contain any rocks, brick, asphaltic concrete, concrete or other hard materials greater than eight inches in maximum dimensions. Any import soil must be approved by the Soils Engineering firm a minimum of 72 hours prior to importation of site.

Placement of Compacted Fill Soils
The approved fill soils shall be placed in layers not excess of six inches in thickness. Each lift shall be uniform in thickness and thoroughly blended. The fill soils shall be brought to within 2% of the optimum moisture content, unless otherwise specified by the Soils Engineering firm. Each lift shall be compacted to a minimum of 90% relative compaction (in accordance with ASTM: D-1557-12) and approved prior to the placement of the next layer of soil. Compaction tests shall be obtained at the discretion of the Soils Engineering firm but to a minimum of one test for every 500 cubic yards placed and/or for every 2 feet of compacted fill placed.

The minimum relative compaction shall be obtained in accordance with accepted methods in the construction industry. The final grade of the structural areas shall be in a dense and smooth condition prior to placement of slabs-on-grade or pavement areas. No fill soils shall be placed, spread or compacted during unfavorable weather conditions. When the grading is interrupted by heavy rains, compaction operations shall not be resumed until approved by the Soils Engineering firm.

Grading Observations
The controlling governmental agencies should be notified prior to commencement of any grading operations. This firm recommends that the grading operations be conducted under the observation of a Soils Engineering firm as deemed necessary. A 24-hour notice must be provided to this firm prior to the time of our initial inspection.

Observation shall include the clearing and grubbing operations to assure that all unsuitable materials have been properly removed; approve the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished grade and designate areas of overexcavation; and perform field compaction tests to determine relative compaction achieved during fill placement. In addition, all foundation excavations shall be observed by the Soils Engineering firm to confirm that appropriate bearing materials are present at the design grades and recommend any modifications to construct footings.

EXPANSIVE SOIL GUIDELINES

The following expansive soil guidelines are provided for your project. The intent of these guidelines is to inform you, the client, of the importance of proper design and maintenance of projects supported on expansive soils. ***You, as the owner or other interested party, should be warned that you have a duty to provide the information contained in the soil report including these guidelines to your design engineers, architects, landscapers and other design parties in order to enable them to provide a design that takes into consideration expansive soils.***

In addition, you should provide the soil report with these guidelines to any property manager, lessee, property purchaser or other interested party that will have or assume the responsibility of maintaining the development in the future.

Expansive soils are fine-grained silts and clays which are subject to swelling and contracting. The amount of this swelling and contracting is subject to the amount of fine-grained clay materials present in the soils and the amount of moisture either introduced or extracted from the soils. Expansive soils are divided into five categories ranging from "very low" to "very high". Expansion indices are assigned to each classification and are included in the laboratory testing section of this report. *If the expansion index of the soils on your site, as stated in this report, is 21 or higher, you have expansive soils.* The classifications of expansive soils are as follows:

Classification of Expansive Soil*

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

*From Table 18A-I-B of California Building Code (1988)

When expansive soils are compacted during site grading operations, care is taken to place the materials at or slightly above optimum moisture levels and perform proper compaction operations. Any subsequent excessive wetting and/or drying of expansive soils will cause the soil materials to expand and/or contract. These actions are likely to cause distress of foundations, structures, slabs-on-grade, sidewalks and pavement over the life of the structure. ***It is therefore imperative that even after construction of improvements, the moisture contents are maintained at relatively constant levels, allowing neither excessive wetting or drying of soils.***

Evidence of excessive wetting of expansive soils may be seen in concrete slabs, both interior and exterior. Slabs may lift at construction joints producing a trip hazard or may crack from the pressure of soil expansion. Wet clays in foundation areas may result in lifting of the structure causing difficulty in the opening and closing of doors and windows, as well as cracking in exterior and interior wall surfaces. In extreme wetting of soils to depth, settlement of the structure may eventually result. Excessive wetting of soils in landscape areas adjacent to concrete or asphaltic pavement areas may also result in expansion of soils beneath pavement and resultant distress to the pavement surface.

Excessive drying of expansive soils is initially evidenced by cracking in the surface of the soils due to contraction. Settlement of structures and on-grade slabs may also eventually result along with problems in the operation of doors and windows.

Projects located in areas of expansive clay soils will be subject to more movement and "hairline" cracking of walls and slabs than similar projects situated on non-expansive sandy soils. There are, however, measures that developers and property owners may take to reduce the amount of movement over the life the development. The following guidelines are provided to assist you in both design and maintenance of projects on expansive soils:

- Drainage away from structures and pavement is essential to prevent excessive wetting of expansive soils. Grades of at least 3% should be designed and maintained to allow flow of irrigation and rain water to approved drainage devices or to the street. Any "ponding" of water adjacent to buildings, slabs and pavement after rains is evidence of poor drainage; the installation of drainage devices or regrading of the area may be required to assure proper drainage. Installation of rain gutters is also recommended to control the introduction of moisture next to buildings. Gutters should discharge into a drainage device or onto pavement which drains to roadways.

- Irrigation should be strictly controlled around building foundations, slabs and pavement and may need to be adjusted depending upon season. This control is essential to maintain relatively uniform moisture content in the expansive soils and to prevent swelling and contracting. Over-watering adjacent to improvements may result in damage to those improvements. NorCal Engineering makes no specific recommendations regarding landscape irrigation schedules.

- Planting schemes for landscaping around structures and pavement should be analyzed carefully. Plants (including sod) requiring high amounts of water may result in excessive wetting of soils. Trees and large shrubs may actually extract moisture from the expansive soils, thus causing contraction of the fine-grained soils.

- Thickened edges on exterior slabs will assist in keeping excessive moisture from entering directly beneath the concrete. A six-inch thick or greater deepened edge on slabs may be considered. Underlying interior and exterior slabs with 6 to 12 inches or more of non-expansive soils and providing presaturation of the underlying clayey soils as recommended in the soil report will improve the overall performance of on-grade slabs.

- Increase the amount of steel reinforcing in concrete slabs, foundations and other structures to resist the forces of expansive soils. The precise amount of reinforcing should be determined by the appropriate design engineers and/or architects.

- Recommendations of the soil report should always be followed in the development of the project. Any recommendations regarding presaturation of the upper subgrade soils in slab areas should be performed in the field and verified by the Soil Engineer.

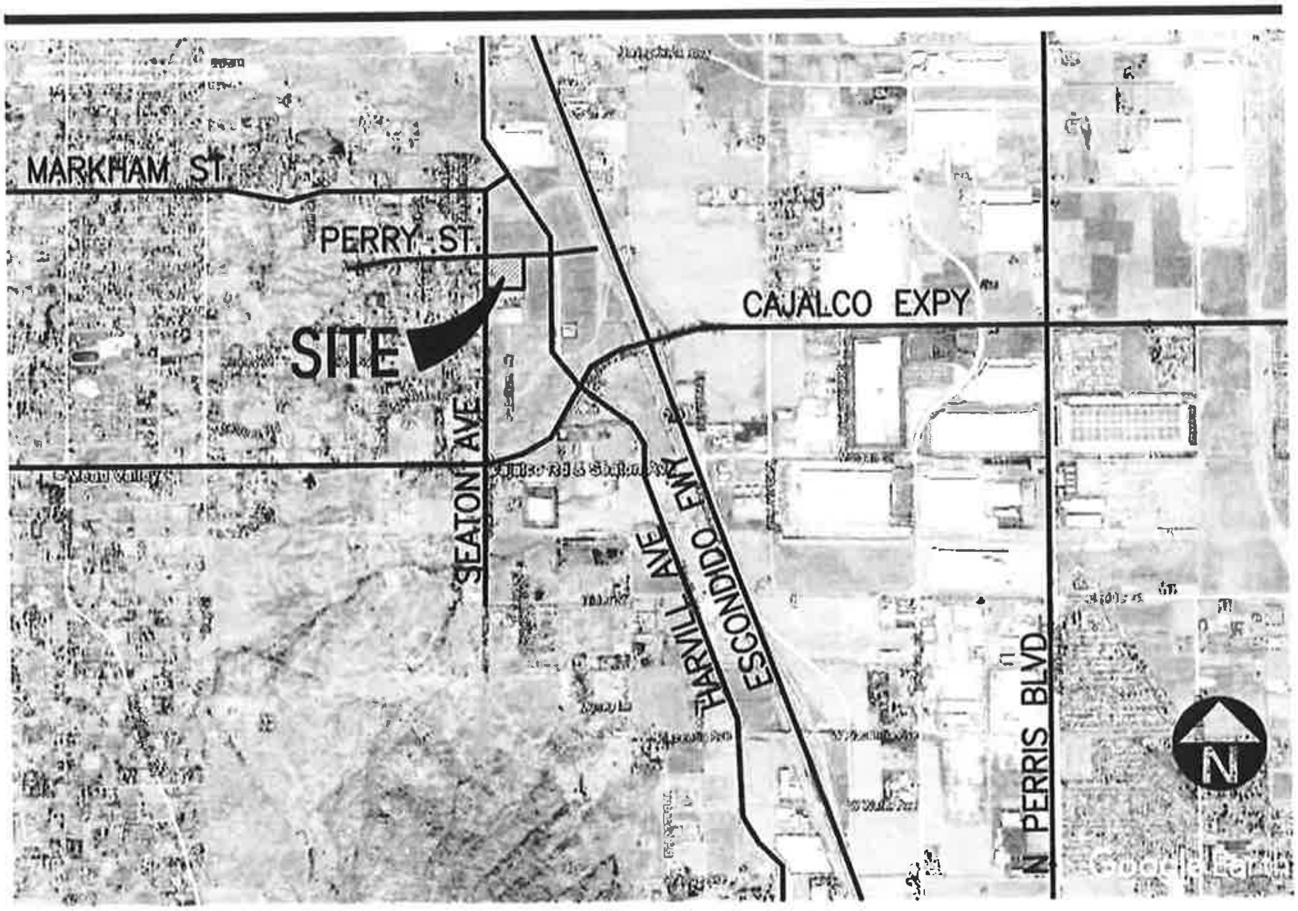
REFERENCES

1. California Building Code, 2013.

2. California Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California: Special Publication 117.

3. International Conference of Building Officials, Uniform Building Code UBC, 2009.

4. ACI Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05), 2005.



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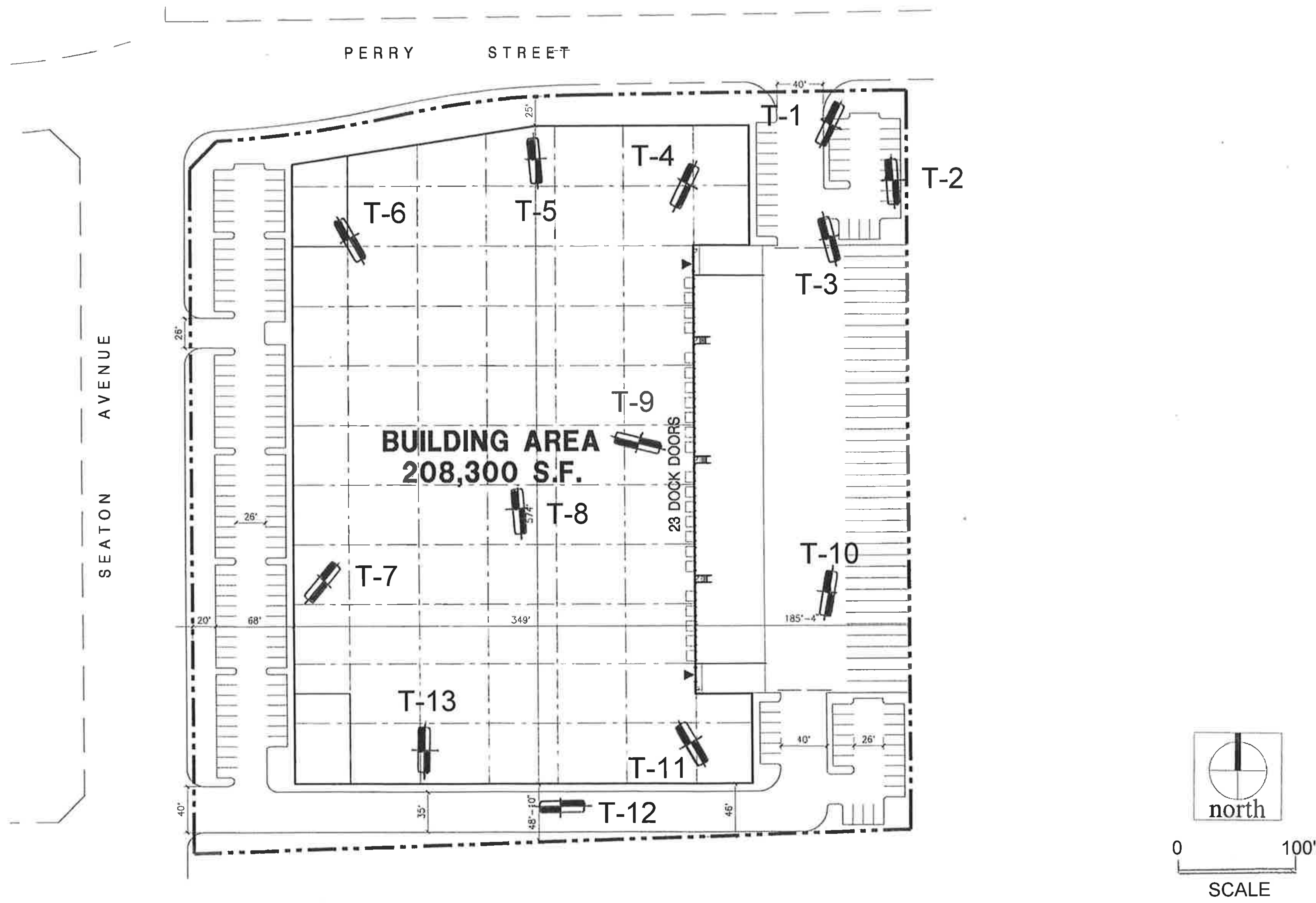
PROJECT 20529-18 DATE 7/2018

VICINITY MAP

FIGURE 1



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PROJECT 20529-18 | DATE 7/2018

APPROXIMATE LOCATIONS OF SOIL BORINGS

FIGURE 2

APPENDICES
(In order of appearance)

APPENDIX A

USGS Design Maps Summary Report

User–Specified Input

Report Title SEC Perry and Seaton, Mead Valley
Wed July 11, 2018 17:10:05 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.8474°N, 117.2605°W

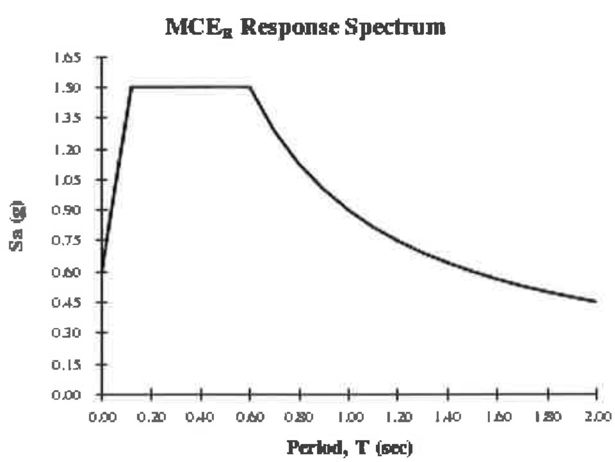
Site Soil Classification Site Class D – "Stiff Soil"

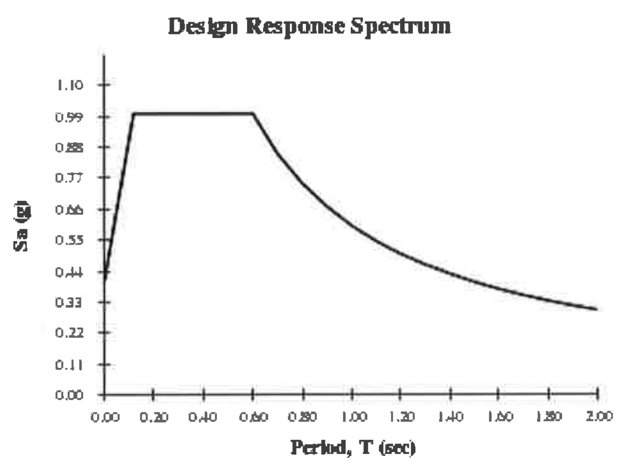
Risk Category I/II/III

USGS–Provided Output

S_S =	1.500 g	S_{MS} =	1.500 g	S_{DS} =	1.000 g
S_1 =	0.600 g	S_{M1} =	0.900 g	S_{D1} =	0.600 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.8474°N, 117.2605°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.500$ g

From Figure 22-2 [2] $S_1 = 0.600$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.500 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.600 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.500 = 1.500$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.600 = 0.900$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

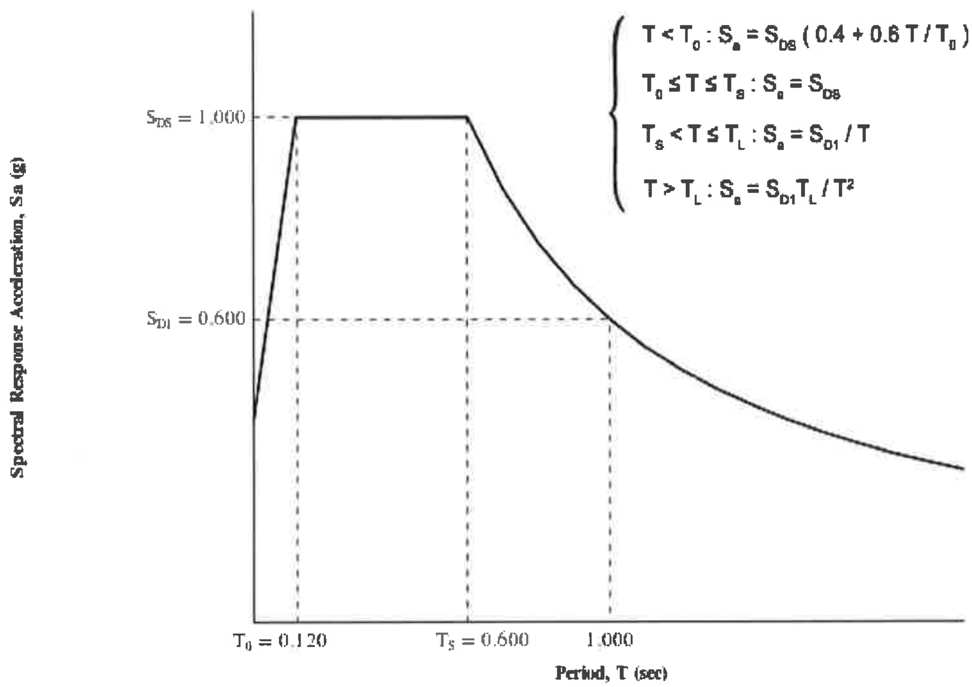
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.500 = 1.000$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.900 = 0.600$ g

Section 11.4.5 — Design Response Spectrum

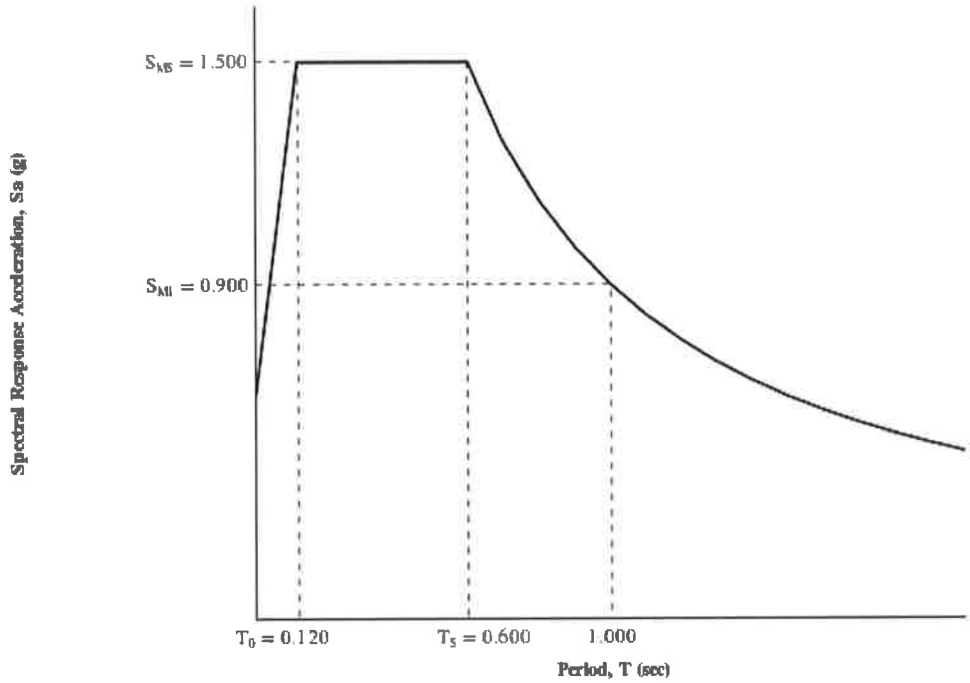
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.500

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.500 = 0.5$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.500 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.076

From Figure 22-18 [6] C_{R1} = 1.045

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.000 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.600 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

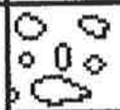
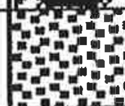
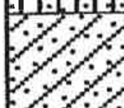
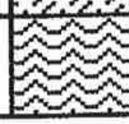
Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX B

MAJOR DIVISION			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL. SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVEL-LY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINE (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

KEY:

- Indicates 2.5-inch Inside Diameter. Ring Sample.
- Indicates 2-inch OD Split Spoon Sample (SPT).
- Indicates Shelby Tube Sample.
- Indicates No Recovery.
- Indicates SPT with 140# Hammer 30 in. Drop.
- Indicates Bulk Sample.
- Indicates Small Bag Sample.
- Indicates Non-Standard
- Indicates Core Run.

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	1 - 5%
Few	5 - 10%
Little	10 - 20%
Some	20 - 35%
And	35 - 50%

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5mm) to No. 200 (0.074mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074 mm)

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
DAMP	Some perceptible moisture; below optimum
MOIST	No visible water; near optimum moisture content
WET	Visible free water, usually soil is below water table.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N -VALUE

COHESIONLESS SOILS		COHESIVE SOILS		
Density	N (blows/ft)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	Very Soft	0 to 2	< 250
Loose	4 to 10	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	Very Stiff	15 to 30	2000 - 4000
		Hard	over 30	> 4000

Molto Properties 20529-18	Log of Trench T-1

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5		Trench completed at depth of 5'			4.8		
10							
15							
20							
25							
30							
35							

GWT not encountered

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT\20529-18.log Date: 7/23/2018

NorCal Engineering

Molto Properties 20529-18	Log of Trench T-2

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0	GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5							
		Decomposed Granitic BEDROCK Brown, dense to very dense, damp					
10		Trench completed at depth of 10'	[sample]		3.4		
15							
20							
25							
30							
35							

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT\20529-18.log Date: 7/23/2018

NorCal Engineering

Molto Properties 20529-18	Log of Trench T-3

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology		Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
			NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5								
			Trench completed at depth of 7.5'					
10								
15								
20								
25								
30								
35								

Molto Properties
20529-18

Log of Trench T-4

Boring Location: Perry and Seaton, Mead Valley

Date of Drilling: 7/9/18 | **Groundwater Depth: None Encountered**

Drilling Method: Backhoe

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Lithology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry	■		2.7	108.7	
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5			■		3.8	131.8	
		Decomposed Granitic BEDROCK Brown, dense to very dense, damp					
10			■		4.3	132.5	
15			■		4.9	135.3	
		Trench completed at depth of 15.5'					
20							
25							
30							
35							

GWT not encountered

NorCal Engineering

Molto Properties 20529-18	Log of Trench T-5

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp	■		4.1	129.6	
5		Decomposed Granitic BEDROCK Brown, dense to very dense, damp					
		Trench completed at depth of 5'					
10							
15							
20							
25							
30							
35							

GWT not encountered

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NorCal Engineering

Molto Properties 20529-18	Log of Trench T-6

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lithology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0	GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry	■		5.2	116.7	
		NATURAL SOILS Sandy SILT with clay to Clayey SAND Brown, medium stiff/dense, damp					
5		Decomposed Granitic BEDROCK Brown, dense to very dense, damp					
		Trench completed at depth of 6'					
10							
15							
20							
25							
30							
35							

NorCal Engineering

Molto Properties 20529-18	Log of Trench T-7

Boring Location: Perry and Seaton, Mead Valley

Date of Drilling: 7/9/18 | Groundwater Depth: None Encountered

Drilling Method: Backhoe

Hammer Weight: | Drop:

Surface Elevation: Not Measured

Depth (feet)	Lith-ology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5		Decomposed Granitic BEDROCK Brown, dense to very dense, damp	■		3.1	128.0	
		Trench completed at depth of 7'					
10							
15							
20							
25							
30							
35							

GWT not encountered

NorCal Engineering

Molto Properties
20529-18

Log of Trench T-8

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lithology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT to Clayey SAND with occasional gravel, roots, other organics Brown, soft/loose, dry	■		4.3	112.6	
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5		Decomposed Granitic BEDROCK Brown, dense to very dense, damp	■		4.6	131.1	
10			■		3.2	125.1	
		Trench completed at depth of 10.5'					

GWT not encountered

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35

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NorCal Engineering

Molto Properties 20529-18	Log of Trench T-9

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology		Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry	■		4.1	117.6	
			NATURAL SOILS Sandy SILT with clay to Clayey SAND Brown, medium stiff/dense, damp					
5				■		6.1	111.3	
				■		4.2	109.6	
10				■		8.3	114.6	
			Decomposed Granitic BEDROCK Brown, dense to very dense, damp			4.6		
			Trench completed at depth of 12'					
15								
20								
25								
30								
35								

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NorCal Engineering

Molto Properties 20529-18	Log of Trench T-10

Boring Location: Perry and Seaton, Mead Valley

Date of Drilling: 7/9/18 | Groundwater Depth: None Encountered

Drilling Method: Backhoe

Hammer Weight: | Drop:

Surface Elevation: Not Measured

Depth (feet)	Lith-ology		Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
			NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp	■		4.3	136.1	
5			Trench completed at depth of 5'					
10								
15								
20								
25								
30								
35								

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NorCal Engineering

Molto Properties **20529-18**	**Log of Trench T-11**

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	**Groundwater Depth: None Encountered**
Drilling Method: Backhoe	
Hammer Weight:	**Drop:**
Surface Elevation: Not Measured	

Depth (feet)	Lithology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
		NATURAL SOILS Sandy SILT with clay Brown, medium stiff to stiff, damp	■		2.7	116.9	
5			■		5.0	116.3	
10							
		Decomposed Granitic BEDROCK Brown, dense to very dense, damp	▯		3.0		
		Trench completed at depth of 12'					
15							
20							
25							
30							
35							

GWT not encountered

Molto Properties
20529-18

Log of Trench T-12

Boring Location: Perry and Seaton, Mead Valley	
Date of Drilling: 7/9/18	Groundwater Depth: None Encountered
Drilling Method: Backhoe	
Hammer Weight:	Drop:
Surface Elevation: Not Measured	

Depth (feet)	Lith-ology		Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
				▽		4.2		
			NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5			Trench completed at depth of 5'					
10								
15								
20								
25								
30								
35								

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Molto Properties
20529-18

Log of Trench T-13

Boring Location: Perry and Seaton, Mead Valley

Date of Drilling: 7/9/18 | Groundwater Depth: None Encountered

Drilling Method: Backhoe

Hammer Weight: | Drop:

Surface Elevation: Not Measured

Depth (feet)	Lithology		Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0		GWT not encountered	FILL SOILS/DISTURBED TOP SOILS Sandy clayey SILT with occasional gravel, roots, other organics Brown, soft, dry					
			NATURAL SOILS Sandy SILT with clay Brown, medium stiff, damp					
5			Decomposed Granitic BEDROCK Brown, dense to very dense, damp					
			Trench completed at depth of 8'					
10								
15								
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT\20529-18.log Date: 7/23/2018

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APPENDIX C

TABLE I
MAXIMUM DENSITY TESTS
(ASTM: D-1557-12)

Sample	Classification	Optimum Moisture	Maximum Dry Density (lbs./cu.ft.)
T-4 @ 2-4'	sandy SILT w/clay	10.0	135.0

TABLE II
EXPANSION INDEX TESTS
(ASTM: D-4829-11)

Sample	Classification	Expansion Index
T-4 @ 2-4'	sandy SILT w/clay	20

TABLE III
SOLUBLE SULFATE TESTS
(CT 417)

Sample	Sulfate Concentration (%)
T-4 @ 2-4'	.0007

TABLE IV
pH TESTS

Sample	pH
T-4 @ 2-4'	7.1

TABLE V
RESISTIVITY TESTS
(CT 643)

Sample	Resistivity (ohm-cm)
T-4 @ 2-4'	2460

TABLE VI
CHLORIDE TESTS
(CT 422))

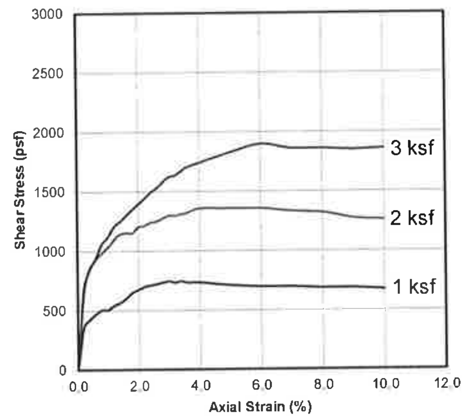
Sample	Concentration (ppm)
T-4 @ 2-4'	158

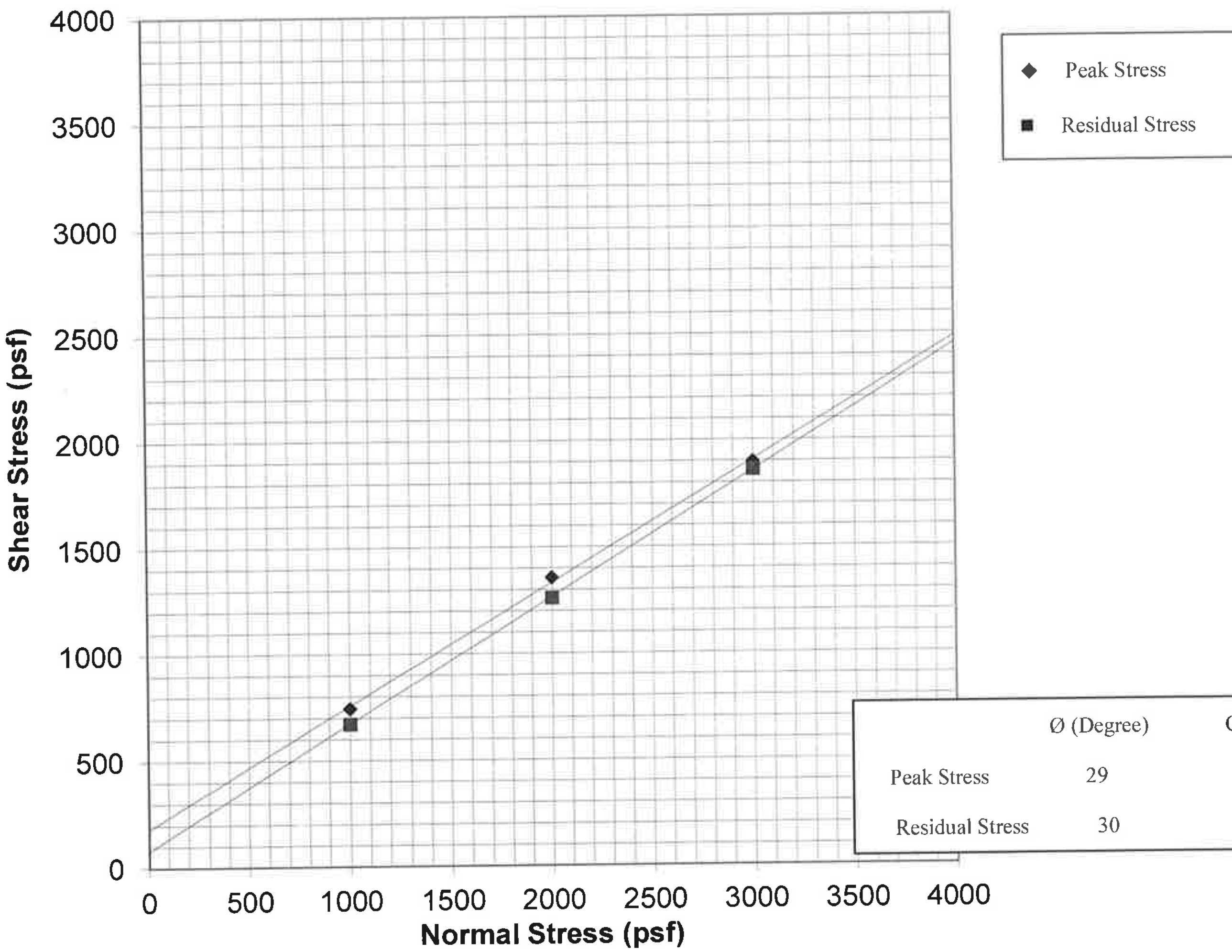
TABLE VII
RESISTANCE 'R' VALUE TESTS
(CA 301))

Sample	'R' Value
T-1 @ 1-2'	51

Sample No. T4@2'
Sample Type: Remolded/Saturated
Soil Description: Silty Sand w/ Some Clay

		1	2	3
Normal Stress	(psf)	1000	2000	3000
Peak Stress	(psf)	744	1356	1896
Displacement	(in)	0.075	0.100	0.150
Residual Stress	(psf)	672	1260	1860
Displacement	(in.)	0.250	0.250	0.250
In Situ Dry Density	(pcf)	122.0	122.0	122.0
In Situ Water Content	(%)	9.5	9.5	9.5
Saturated Water Content	(%)	14.0	14.0	14.0
Strain Rate	(in/min)	0.020	0.020	0.020





	Ø (Degree)	C (psf)
Peak Stress	29	180
Residual Stress	30	80

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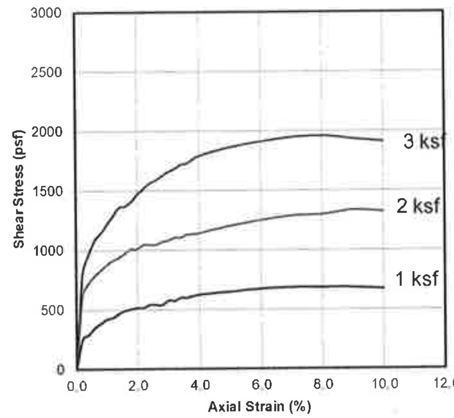
SOILS AND GEOTECHNICAL CONSULTANTS

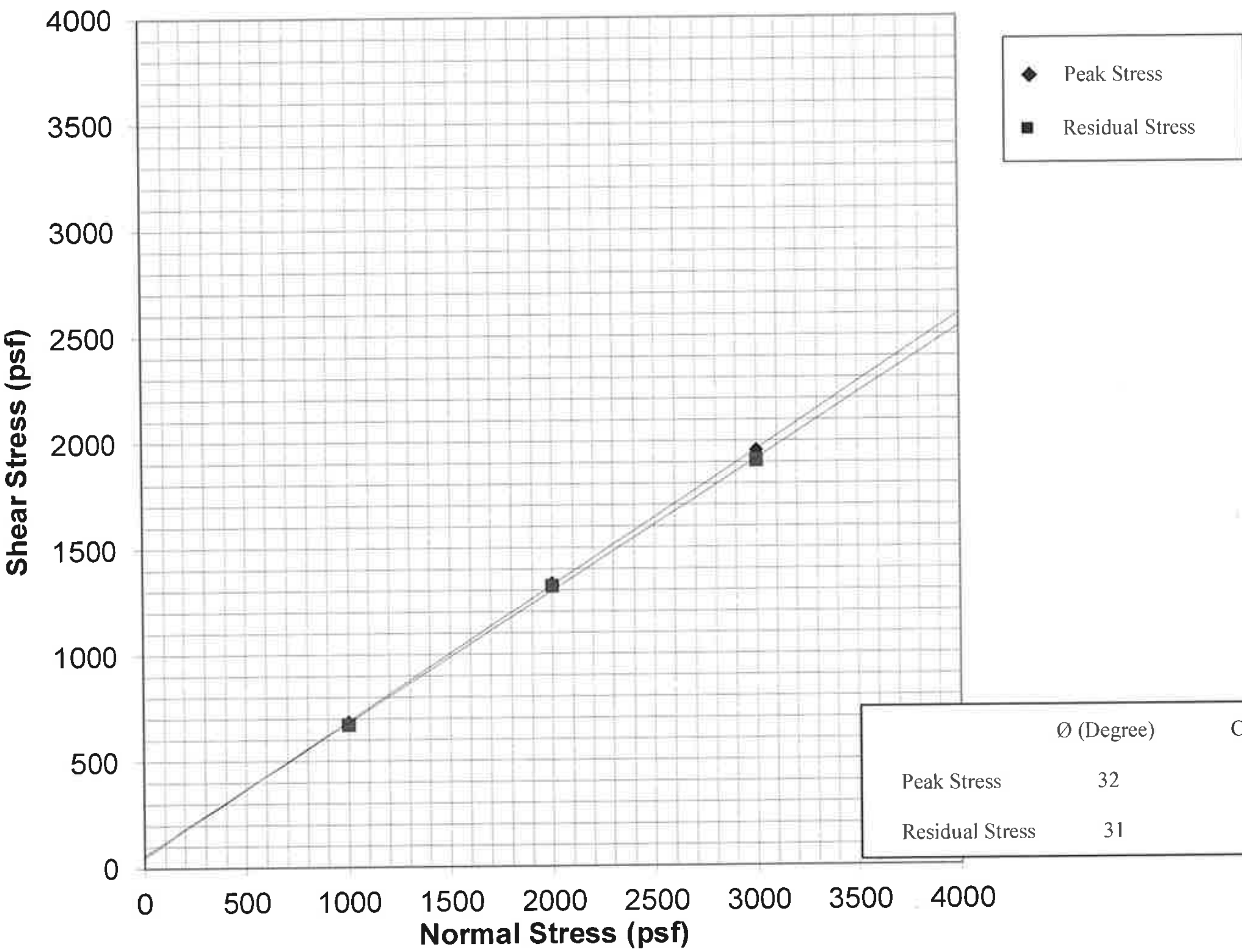
Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

DIRECT SHEAR TEST
ASTM D3080
Plate A

		1	2	3
Sample No.	T6@2'			
Sample Type:	Undisturbed/Saturated			
Soil Description:	Silty Clay w/ Sand			
Normal Stress	(psf)	1000	2000	3000
Peak Stress	(psf)	684	1332	1956
Displacement	(in.)	0.175	0.225	0.200
Residual Stress	(psf)	672	1320	1908
Displacement	(in.)	0.250	0.250	0.250
In Situ Dry Density	(pcf)	116.7	116.7	116.7
In Situ Water Content	(%)	5.2	5.2	5.2
Saturated Water Content	(%)	16.3	16.3	16.3
Strain Rate	(in/min)	0.020	0.020	0.020





	Ø (Degree)	C (psf)
Peak Stress	32	50
Residual Stress	31	60

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SOILS AND GEOTECHNICAL CONSULTANTS

Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

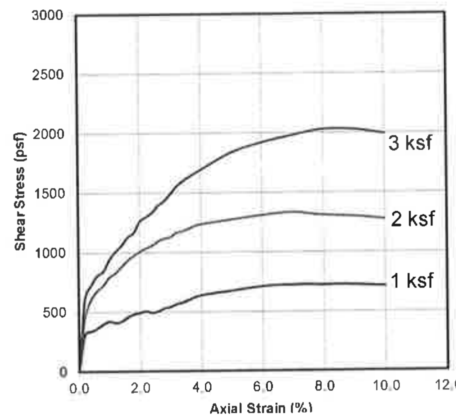
DIRECT SHEAR TEST

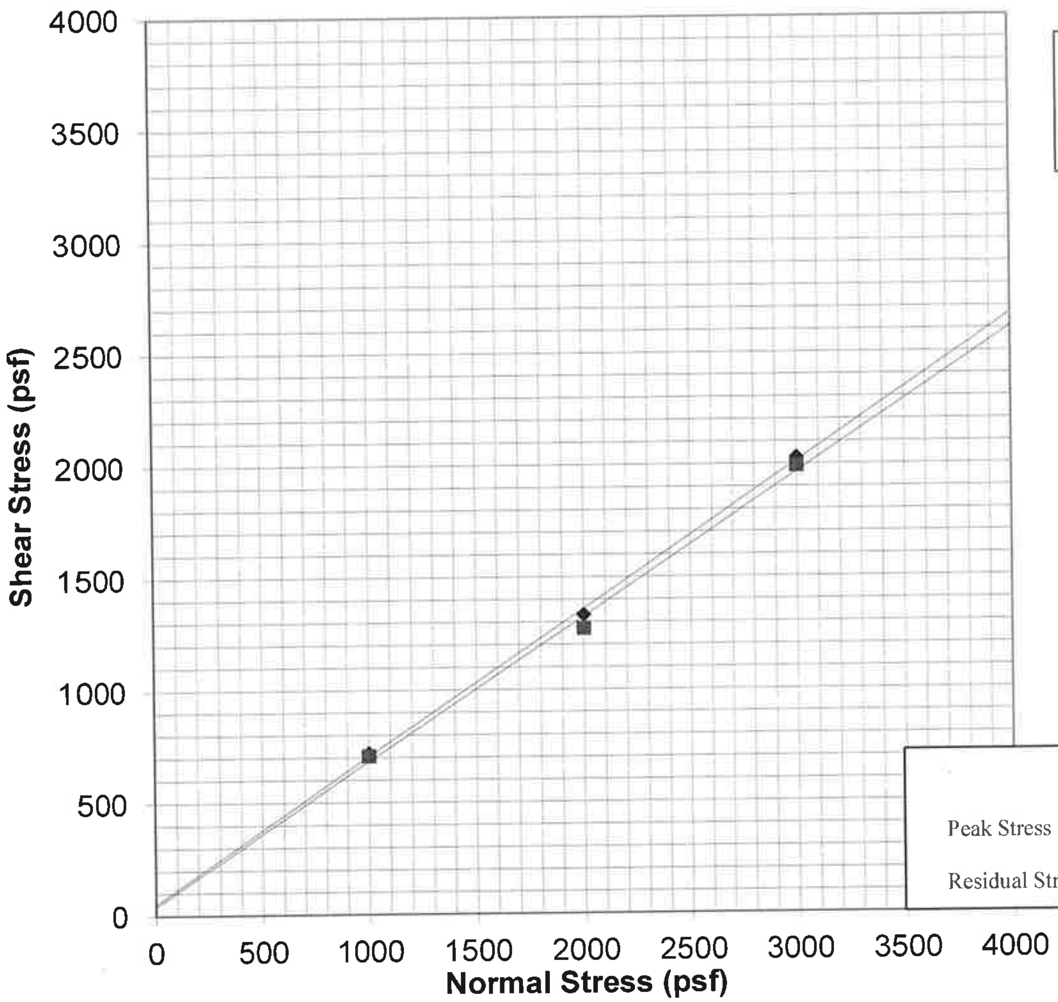
ASTM D3080

Plate B

Sample No.	T11@6'			
Sample Type:	Undisturbed/Saturated			
Soil Description:	Silty Fine-Very Coarse Grained Sand w/ Some Clay			

		1	2	3
Normal Stress	(psf)	1000	2000	3000
Peak Stress	(psf)	720	1332	2028
Displacement	(in.)	0.175	0.175	0.200
Residual Stress	(psf)	708	1272	1992
Displacement	(in.)	0.250	0.250	0.250
In Situ Dry Density	(pcf)	116.3	116.3	116.3
In Situ Water Content	(%)	5.0	5.0	5.0
Saturated Water Content	(%)	16.5	16.5	16.5
Strain Rate	(in/min)	0.020	0.020	0.020





	Ø (Degree)	C (psf)
Peak Stress	33	50
Residual Stress	32	40

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SOILS AND GEOTECHNICAL CONSULTANTS

Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

DIRECT SHEAR TEST

ASTM D3080

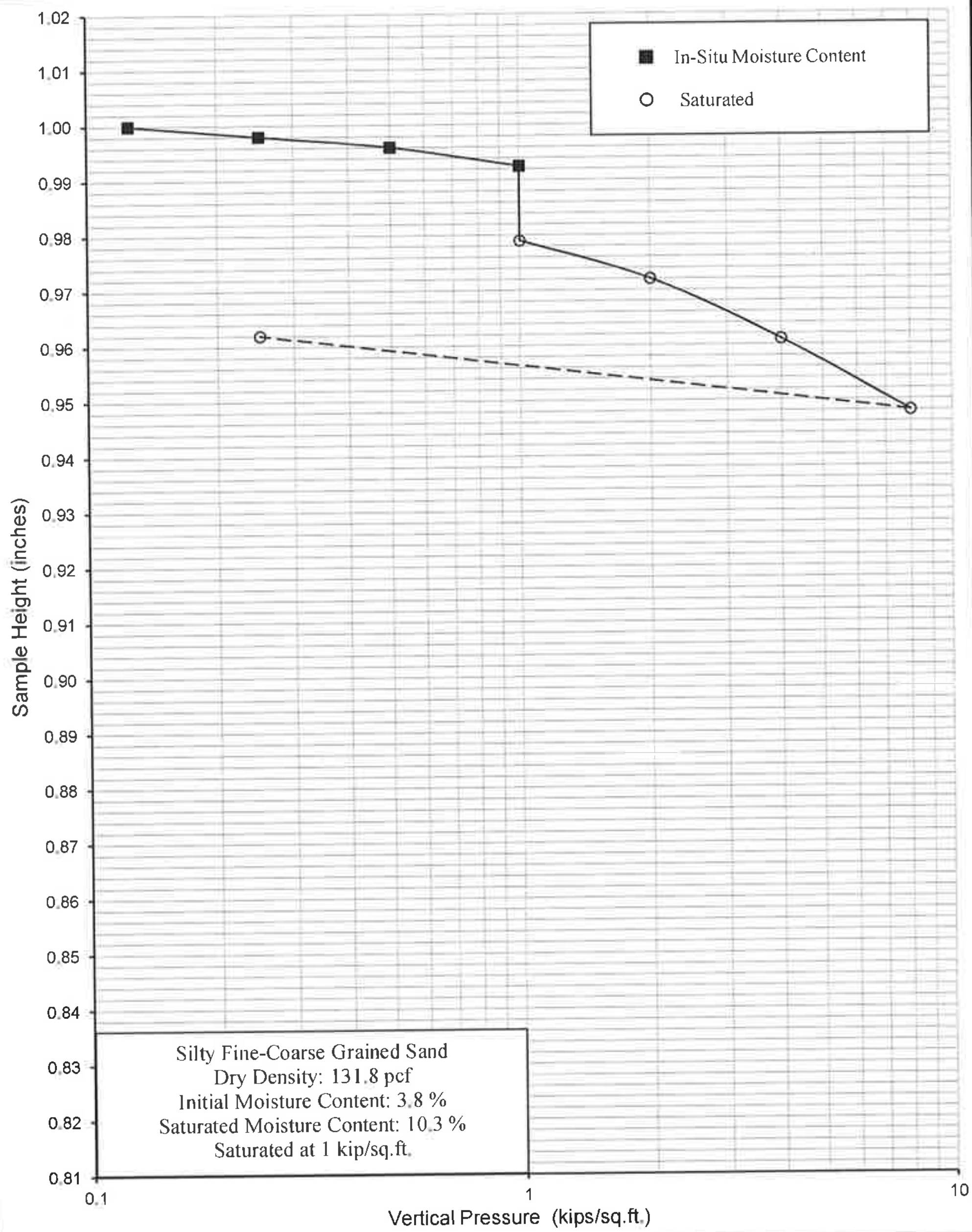
Plate C

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)	Sample No.	T4	Depth 5'	Date 7/20/2018

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)
0.125	1.0000	0.0
0.25	0.9980	0.2
0.5	0.9960	0.4
1	0.9925	0.8
1	0.9790	2.1
2	0.9720	2.8
4	0.9610	3.9
8	0.9480	5.2
0.25	0.9620	3.8

Saturated

Date Tested: 7/17/2018
Sample: T4
Depth: 5'



Silty Fine-Coarse Grained Sand
Dry Density: 131.8 pcf
Initial Moisture Content: 3.8 %
Saturated Moisture Content: 10.3 %
Saturated at 1 kip/sq.ft.

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SOILS AND GEOTECHNICAL CONSULTANTS

Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

CONSOLIDATION TEST

ASTM D2435

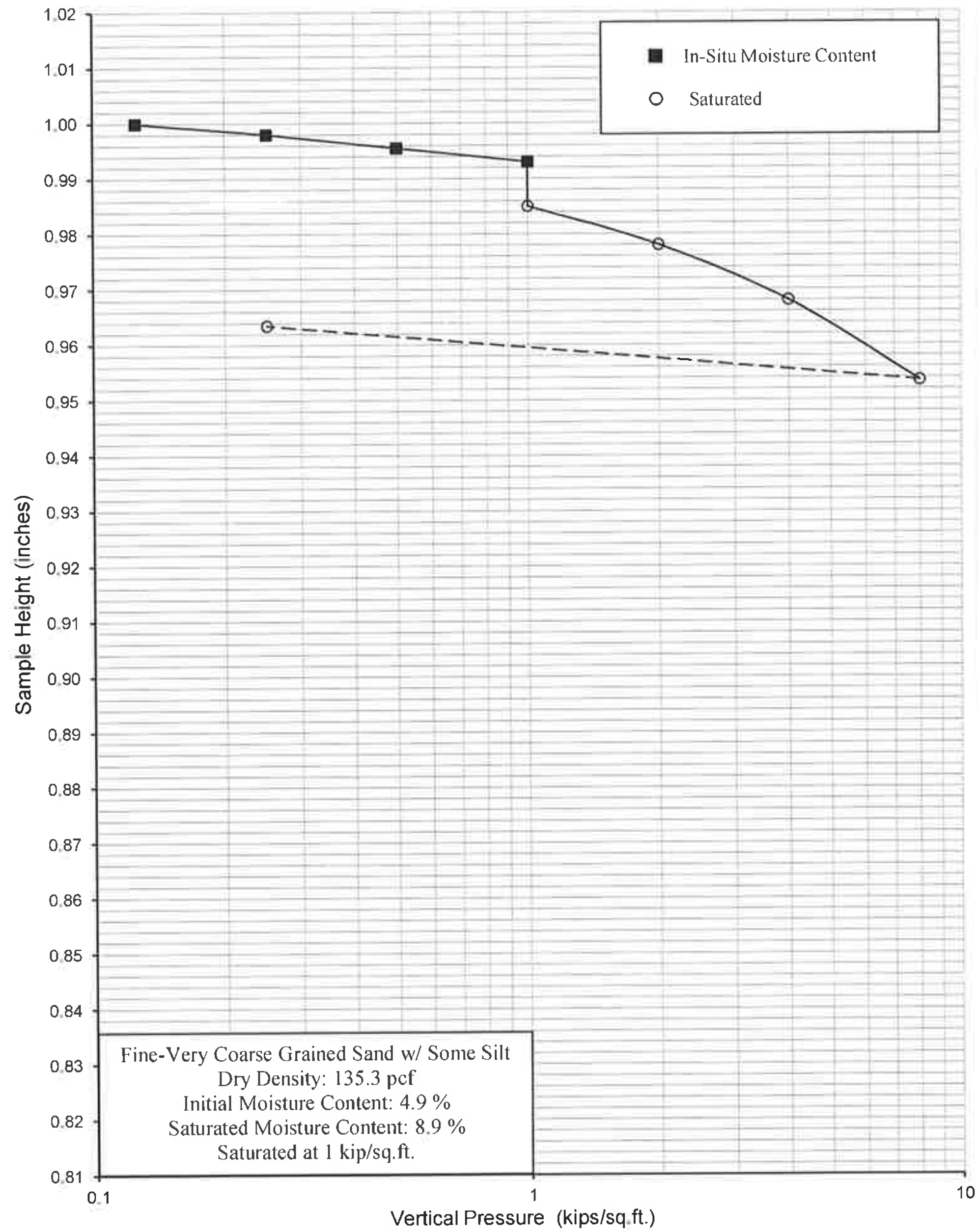
Plate D

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)
0.125	1.0000	0.0
0.25	0.9980	0.2
0.5	0.9955	0.4
1	0.9930	0.7
1	0.9850	1.5
2	0.9780	2.2
4	0.9680	3.2
8	0.9535	4.7
0.25	0.9635	3.7

Saturated

Sample No.	T4	Depth	15'	Date	7/20/2018

Date Tested: 7/17/2018
Sample: T4
Depth: 15'



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SOILS AND GEOTECHNICAL CONSULTANTS

Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

CONSOLIDATION TEST

ASTM D2435

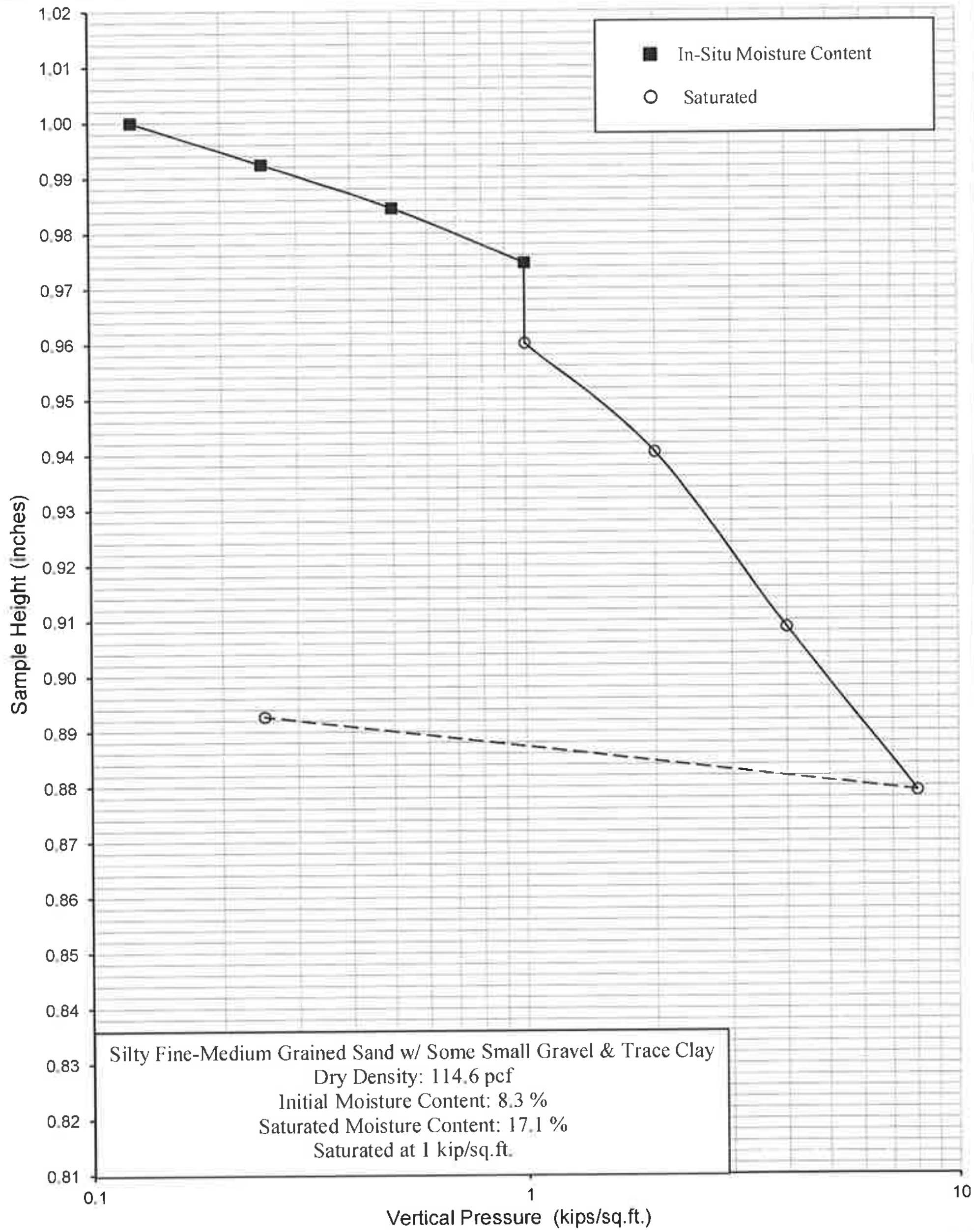
Plate E

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)
0.125	1.0000	0.0
0.25	0.9924	0.8
0.5	0.9845	1.6
1	0.9746	2.5
1	0.9601	4.0
2	0.9404	6.0
4	0.9087	9.1
8	0.8791	12.1
0.25	0.8927	10.7

Saturated

Date Tested: 7/17/2018
Sample: T9
Depth: 10'

Sample No. T9 | Depth 10' | Date 7/20/2018



Silty Fine-Medium Grained Sand w/ Some Small Gravel & Trace Clay
Dry Density: 114.6 pcf
Initial Moisture Content: 8.3 %
Saturated Moisture Content: 17.1 %
Saturated at 1 kip/sq.ft.

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SOILS AND GEOTECHNICAL CONSULTANTS

Molto

PROJECT NUMBER: 20529-18 DATE: 7/20/2018

CONSOLIDATION TEST

ASTM D2435

Plate F

APPENDIX D

NORCAL ENGINEERING

SOILS AND GEOTECHNICAL CONSULTANTS

Project: Molto Properties
Project No: 20529-18
Date: 7/9/18
Test No. T-1
Depth: 5'
Tested By: J.S.

	TIME (hr/min)	CHANGE TIME (min)	CUMULATIVE TIME (min)	INNER RING READING (cm)	INNER RING CHANGE	INNER RING FLOW (cc)	OUTER RING READING (cm)	OUTER RING CHANGE (cm)	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
1	8:05			104.5			49.6					
	8:15	10	10	104.9	0.4		49.7	0.1				
2	8:15			104.9			49.7					
	8:25	10	20	105.3	0.4		49.9	0.2				
3	8:25			105.3			49.9					
	8:35	10	30	105.6	0.3		49.9	0.0				
4	8:35			104.9			48.0					
	8:45	10	40	105.1	0.2		48.0	0.0				
5	8:45			105.1			48.0					
	8:55	10	50	105.2	0.1		48.1	0.1				
6	8:55			105.2			48.1					
	9:05	10	60	105.4	0.2		48.1	0.0				
7	9:05			105.4			48.1					
	9:15	10	70	105.5	0.1		48.3	0.2		0.6	1.2	
8	9:15			105.5			48.3					
	9:25	10	80	105.7	0.2		48.4	0.1		1.2	0.6	
9	9:25			105.7			48.4					
	9:35	10	90	105.8	0.1		48.4	0.0		0.6	0.0	
10	9:35			105.8			48.4					
	9:45	10	100	106.0	0.2		48.5	0.1		1.2	0.6	
11	9:45			106.0			48.5					
	9:55	10	110	106.2	0.2		48.6	0.1		1.2	0.6	
12	9:55			106.2			48.6					
	10:05	10	120	106.3	0.1		48.8	0.2		0.6	1.2	

Average = 0.9 / 0.7cm/hr

NORCAL ENGINEERING

SOILS AND GEOTECHNICAL CONSULTANTS

Project: Molto Properties
Project No: 20529-18
Date: 7/9/18
Test No. T-2
Depth: 10'
Tested By: J.S.

	TIME (hr/min)	CHANGE TIME (min)	CUMULATIVE TIME (min)	INNER RING READING (cm)	INNER RING CHANGE	INNER RING FLOW (cc)	OUTER RING READING (cm)	OUTER RING CHANGE (cm)	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
1	10:30			108.7			50.5					
	10:40	10	10	111.4	2.7		53.1	2.6				
2	10:40			106.5			48.5					
	10:50	10	20	108.5	2.0		50.5	2.0				
3	10:50			108.5			50.5					
	11:00	10	30	109.9	1.4		52.0	1.5				
4	11:00			106.0			48.0					
	11:10	10	40	107.2	0.8		49.2	1.2				
5	11:10			106.0			18.0					
	11:20	10	50	107.1	1.1		49.0	1.0				
6	11:20			107.1			49.0					
	11:30	10	60	108.0	0.9		50.1	1.1				
7	11:30			105.3			47.7					
	11:40	10	70	106.3	1.0		48.3	0.6		6.0	3.6	
8	11:40			106.3			48.3					
	11:50	10	80	107.2	0.9		49.3	1.0		5.4	6.0	
9	11:50			107.2			49.3					
	12:00	10	90	108.4	1.2		50.2	0.9		7.2	5.4	
10	12:00			105.7			47.5					
	12:10	10	100	106.3	0.6		48.4	0.9		3.6	5.4	
11	12:10			106.3			48.4					
	12:20	10	110	107.1	0.8		49.3	0.9		4.8	5.4	
12	12:20			107.1			49.3					
	12:30	10	120	108.0	0.9		50.1	0.8		5.4	4.8	

Average = 5.4 / 5.1 cm/hr

NORCAL ENGINEERING

SOILS AND GEOTECHNICAL CONSULTANTS

Project: Molto Properties
Project No: 20529-18
Date: 7/9/18
Test No. T-3
Depth: 7.5'
Tested By: J.S.

	TIME (hr/min)	CHANGE TIME (min)	CUMULATIVE TIME (min)	INNER RING READING (cm)	INNER RING CHANGE	INNER RING FLOW (cc)	OUTER RING READING (cm)	OUTER RING CHANGE (cm)	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
1	12:41			104.6			47.8					
	12:51	10	10	105.0	0.4		48.0	0.2				
2	12:51			105.0			48.0					
	1:01	10	20	105.3	0.3		48.6	0.6				
3	1:01			105.3			48.6					
	1:11	10	30	105.6	0.3		48.9	0.3				
4	1:11			105.6			48.9					
	1:21	10	40	105.9	0.3		49.4	0.5				
5	1:21			105.9			49.4					
	1:31	10	50	106.1	0.2		49.7	0.3				
6	1:31			106.1			49.7					
	1:41	10	60	106.5	0.4		50.1	0.4				
7	1:41			106.5			50.1					
	1:51	10	70	106.8	0.3		50.6	0.5		1.8	3.0	
8	1:51			106.8			50.6					
	2:01	10	80	107.1	0.3		50.8	0.2		1.8	1.2	
9	2:01			107.1			50.8					
	2:11	10	90	107.4	0.3		51.2	0.4		1.8	2.4	
10	2:11			107.4			51.2					
	2:21	10	100	107.6	0.2		51.5	0.3		1.2	1.8	
11	2:21			107.6			51.5					
	2:31	10	110	107.9	0.3		51.9	0.4		1.8	2.4	
12	2:31			107.9			51.9					
	2:41	10	120	108.2	0.3		52.1	0.2		1.8	1.2	

Average = 1.7 / 2.0 cm/hr