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February 1, 2023 Project Number 22228-20

Molto Properties, LLC
1 Lincoln Centre
18W140 Butterfield Road, Suite 750
Oakbrook Terrace, Illinois 60181

Attn: Mr. Carlos Cornejo

RE: **Updated Geotechnical Investigation** - Proposed Warehouse Building Development – Located at the Southwest Corner of Bay Avenue and Day Street, in the City of Moreno Valley, California

Dear Mr. Cornejo:

Pursuant to your request, this firm has prepared an update letter based on review of our previous reports titled "Geotechnical Investigation" dated January 22, 2021, "Supplemental Infiltration Testing" report dated March 16, 2021 and our "Second Supplemental Infiltration Testing" report dated March 31, 2021 in regards to the proposed site development at the above referenced property. Overall, the geotechnical conditions of the subject site are currently representative of the conditions described in our previous referenced report.

Proposed Development

It is now proposed to develop the 9.57-acre subject site with a 194,775 square-feet concrete tilt-up warehouse building with associated pavement, hardscape and landscaping.

Updated Seismic Design Criteria

The seismic design parameters have been revised and are provided on the following page and based upon the 2022 California Building Code (CBC) Standard ASCE/SEI 7-22. The data was obtained from the American Society of Civil Engineers (ASCE) website, https://asce7hazardtool.online/ and the ASCE 7 Hazards Report.

Seismic Design Acceleration Parameters

Latitude	33.919
Longitude	-117.279
Site Class	D
Risk Category	II
Peak Ground Acceleration	$PGA_M = 0.66$
Adjusted Maximum Acceleration	$S_{MS} = 1.91$ $S_{M1} = 1.66$
Design Spectral Response Acceleration Parameters	$S_{DS} = 1.27$ $S_{D1} = 1.11$
Mapped Spectral Response Acceleration	$S_S = 1.75$ $S_1 = 0.64$

Use of these values is dependent on requirements of Section 11-4.8 ASCE 7, exception 2 that requires the value of the seismic response coefficient C_s be determined by Equation 12.8.2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either 12.8-3 for $T_L \geq T \geq 1.5T_s$ or Equation 12.8-4 for $T > T_L$. Computations and verification of these conditions is referred to the structural engineer.

Updated Settlement Estimates

Our liquefaction evaluation based upon results from deep boring (B-1) which extended to a depth of 51.5 feet below grade. The boring encountered stiff/dense to very stiff/dense clays and sands at 5 feet and below. The SPT blowcounts were 31 blows/foot or greater from 10 to 50 feet.

Assuming a conservative historic high groundwater of 20 feet below grade in the area, the stiff/dense soil layers below that level are judged to be non-liquefiable and the seismic settlement would be less than ½ inch with a PGA_M of 0.66g. These settlements should occur rather uniformly across the lot with differential settlements on the order of less than ¼ inch over a 30 feet (horizontal) distance in the building pad area. Our liquefaction calculations are included in Appendix A.

Conclusions

It is our opinion that the planned construction will be geotechnically feasible provided that all of the recommendations presented in our referenced report are implemented and conditions of latest California Building Code (CBC) are adhered. This firm shall have the opportunity to review building plans when they are made available to verify that all recommendations are incorporated and if additional information or revisions will be required.

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. It is recommended that site inspections be performed by a representative of this firm during all construction to verify the findings and recommendations documented in this report.

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

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

REGISTERED PROFESSIONAL ENGINEER
KEITH D. TUCKER
No. 841
Exp. 12/31/2024
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Barone
Project Manager

Appendix A

SITE LOCATION: ______

GEOTECHNICAL REPORT: ______

GEOLOGY REPORT: ______

DEPTH TO WATER TABLE = 20'

EARTHQUAKE MAGNITUDE = 6.9

PEAK GROUND ACCELERATION = 0.64g,

DEPTH BELOW FINAL GRADE (FEET)	MOIST DENSITY (PCF)	σ_0 TOTAL STRESS (PSF)	$\overline{\sigma}_0$ EFFECTIVE STRESS (PSF)	$\sigma_0/\overline{\sigma}_0$ (-)	r_d (-)	① $\tau_h/\overline{\sigma}_0$ (-)	N VALUE (BLOWS/FT)	RELATIVE DENSITY (%)	C_N (-)	C_E (-)	C_B (-)	C_R (-)	C_S (-)	$(N_1)_{60}$ (Blows/ft)	FINES (%)	CRR M=7.5	MSF (-)	CRR M=6.9	LIQ. F.S.
5	125	625	Same	1.00	0.99	0.42	>50	>90	>1.6	1.00	1.05	0.70	1.20	>72	38	>0.50	1.3	>0.65	>1.5
10	130	1275	↓	↓	0.96	0.40	>50	↓	1.2	↓	↓	0.75	↓	>57	31	↓	↓	↓	>1.6
15	115	1850	↓	↓	0.92	0.38	>50	↓	1.04	↓	↓	0.85	↓	>56	41	↓	↓	↓	>1.7
20	120	2450	↓	↓	0.87	0.36	>50	↓	0.90	↓	↓	0.90	↓	>51	32	↓	↓	↓	>1.8
25	↓	3050	2738	1.11	0.80	0.37	52	↓	0.88	↓	↓	0.95	↓	55	21	↓	↓	↓	>1.8
30	↓	3650	3026	1.21	0.74	0.37	31	80	0.84	↓	↓	1.00	↓	33	37	↓	↓	↓	>1.8
35	↓	4250	3314	1.28	0.68	0.37	59	>90	0.81	↓	↓	↓	↓	60	20	↓	↓	↓	>1.8
40	↓	4850	3602	1.35	0.64	0.36	42	85	0.78	↓	↓	↓	↓	41	47	↓	↓	↓	>1.8
45	↓	5450	3890	1.40	0.61	0.36	57	>90	0.76	↓	↓	↓	↓	55	56	↓	↓	↓	>1.8
50	↓	6050	4178	1.45	0.58	0.35	44	85	0.74	↓	↓	↓	↓	41	55	↓	↓	↓	>1.9

① INDUCED CYCLIC STRESS RATIO = $\tau_{ave}/\overline{\sigma}_0 = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_0}{\overline{\sigma}_0} \cdot r_d$

- C_E = Corr. - Energy Ratio = Energy Ratio/60%
- C_B = Corr. - Borehole Dia. = 1.15 for 8" dia. borehole
- C_R = Corr. - Rod Length
- C_S = Corr. - Sampling Method

Actual Energy Ratio = 0.67-1.17 (Safety Hammer)
= 0.50-1.00 (Donut Hammer)

Sampling Method = 1.0 Standard sampler
~~= 1.2 Sampler W/o liners~~

NorCal Engineering
SOILS AND GEOTECHNICAL CONSULTANTS

PROJECT | DATE

EVALUATION OF LIQUEFACTION POTENTIAL

GEOTECHNICAL INVESTIGATION
Proposed Warehouse Building Development
Southwest Corner of Bay Avenue and Day Street
Moreno Valley, California
PEN21-0123

LDC Molto Edgemont, LLC
555 N. El Camino Real, A456
San Clemente, California 92672

Attn: Matthew Snyder

Project Number 22228-20
January 22, 2021
Revised March 17, 2022

NorCal Engineering

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January 22, 2021
Revised March 17, 2022

Project Number 22228-20

LDC Molto Edgemont, LLC
555 N. El Camino Real, A456
San Clemente, California 92672

Attn: Matthew Snyder

RE: **GEOTECHNICAL INVESTIGATION** - Proposed Warehouse Building Development - Located at the Southwest Corner of Bay Avenue and Day Street, in the City of Moreno Valley, California PEN21-0123

Dear Mr. Snyder:

Pursuant to your request, this firm has performed this Geotechnical Investigation for the above referenced project. The purpose of this investigation is to evaluate the geotechnical conditions of subject property 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 currently proposed to construct a new concrete tilt-up structures totaling 210,710 square feet on the 9.2-acre property. Asphaltic and concrete pavement areas and landscaping will also be installed. Grading for the 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 property is located at the southwest corner of Bay Avenue and Day Street, in the City of Moreno Valley, as shown on the Vicinity Map, Figure 1.

2.2 **Existing Improvements:** The property is occupied by several residential structure in the east portion of the site but is otherwise vacant and covered with light vegetation growth. A storm drain inlet is located near the east property line along Day Street.

2.3 **Drainage:** The site topography is generally flat and drainage pattern is not readily discernible. Land at the southeast corner of the property slopes down to the north gently on the order of 12 vertical feet.

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 in accordance with the 2019 California Building Code (CBC) as determined using the ASCE 7 Hazard Tool (https://asce7hazardtool.online/) for the referenced project. Complete printout from the source is included in Appendix A.

Seismic Design Parameters

Site Location	Latitude	33.921186°
	Longitude	-117.279323°
Site Class		D
Risk Category		II
Maximum Spectral Response Acceleration	S_S	1.500g
	S_1	0.600g
Adjusted Maximum Acceleration	S_{MS}	1.500g
Design Spectral Response Acceleration Parameters	S_{DS}	1.000g

The San Jacinto (San Jacinto Valley) Fault zone is located approximately 10 kilometers from the site and is capable of producing a Magnitude 6.9 earthquake and a PGA_M of 0.638g. 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 investigation consisted of the placement of eight (8) subsurface exploratory borings by hollow-stem auger drill rig and hand auger and six (6) excavations by backhoe. Explorations extended to a maximum depth of 51.5 feet below current ground elevations. The explorations were placed at accessible locations throughout the site; existing improvements somewhat limited the placement of explorations.

The explorations were visually classified and logged by a field engineer with locations of the subsurface excavations are shown on the attached Figure 2. Detailed descriptions of the subsurface conditions are listed on the logs in Appendix B. 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:

Fill/Disturbed Top Soils– Fill and disturbed top soils classifying as sandy CLAY with gravel, some minor debris and roots were encountered in the explorations to depths ranging from 1 to 4 feet. These soils were noted to be soft to firm and damp to moist.

Native Soils – Native soils classifying as sandy CLAY were encountered beneath the upper fill soils. These soils were noted to be medium stiff to stiff and damp to moist. Sand, silt and clay content varied with depth of exploration.

4.2 Groundwater

Groundwater was encountered at a depth of approximately 25 feet at the site.

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, resistance value 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 C, 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 **Sieve analyses** and the percent by weight of soil finer than the No. 200 sieve (ASTM: 1140-00) were performed on selected soil samples. These results are detailed later in this report along with discussion of the liquefaction potential at the site.

5.5 **Atterberg Limits** (ASTM: D 4318-10) consisting of liquid limit, plastic limit and plasticity index were performed on selected soil samples. Results are shown on Table III.

5.6 **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 - B.

5.7 **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 C - E.

5.8 **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 IV – VII and are discussed later in this report.

5.9 **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 VIII and recommended pavement sections are provided later within the text of this report.

6.0 LIQUEFACTION EVALUATION

The property lies within areas mapped as potentially liquefiable by the County of Riverside Safety Element. The site is expected to experience ground shaking and earthquake activity that is typical of Southern California area. It is during severe ground shaking that loose, granular soils below the groundwater table can liquefy. Therefore, the liquefaction potential of the underlying soils has been evaluated with findings from our deep boring (B-1) which extended to a depth of 51.5 feet below grade. The boring encountered stiff/dense to very stiff/dense clays and sands at 5 feet and below. The SPT blowcounts were 31 blows/foot or greater from 10 to 50 feet.

Assuming a conservative historic high groundwater of 20 feet below grade in the area, the stiff/dense soil layers below that level are judged to be non-liquefiable and the seismic settlement would be less than ½ inch with a PGA_M of 0.638g. These settlements should occur rather uniformly across the lot with differential settlements on the order of less than ¼ inch over a 30 feet (horizontal) distance in the building pad area.

Our liquefaction calculations are included in Appendix D.

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 and organic laden soils shall be removed and hauled from proposed grading areas prior to and during the grading operations if encountered. 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 existing fill soils (1 to 4 feet) shall be removed to competent native materials, the exposed surface scarified to a depth of 8 inches, brought to approximately 3% above 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 building pad and concrete paving shall be compacted to a minimum of 95%.* 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 not described in this report or materials disturbed during demolition operations will be encountered on site; if found, these areas should be treated as discussed earlier. A diligent search shall also be conducted during grading operations in an effort to uncover any underground structures, irrigation or utility lines. If encountered, these structures and lines shall be either removed or properly abandoned prior to the proposed construction. Abandonment procedures will be provided once 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 soils, 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.

Building floor slabs should also be underlain by a minimum of 2 feet of compacted fill soils.

7.1.3 **Shrinkage and Subsidence**

Results of our in-place density tests reveal that the soil shrinkage will be on the order of 4 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 0.08 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.

7.2 **Temporary Excavations and Shoring Design**

Temporary unsurcharged excavations 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 for the entire height of the cut. 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.

7.3 Foundation Design

All foundations may be designed utilizing the following allowable soil bearing capacities for an embedded depth of 18 inches into approved compacted fill materials with the corresponding widths. Footings 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	2000	2500
2.0	2100	2600
4.0	2400	2900
6.0	2800	3300

The bearing value may be increased by 500 psf for each additional foot of depth in excess of the 18-inch minimum depth, up to a maximum of 4500 psf. Property line screen wall foundations where proper overexcavation and recompaction is not possible due to property line restrictions may be designed using a reduced allowable soil bearing capacity of 1,800 psf for foundations a minimum of 18 inches in depth *and* at least 8 inches into the underlying competent native soils. 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 2 #4 steel bars top and bottom. Additional reinforcement may be necessary due to soil expansion or proposed loadings and shall be further evaluated by the project engineers and/or architect. A representative of this firm shall observe foundation excavations prior placement of steel reinforcement and concrete.

7.4 **Settlement Analysis**

Resultant pressure curves for the consolidation tests are shown on Plates C-E. Computations utilizing these curves and the recommended allowable soil bearing capacities reveal that the foundations will experience normal settlements on the order of ¾ inch and differential settlements of less than ¼ inch.

7.5 **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.

7.6 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.

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.

During a local Magnitude 6.9 earthquake along the San Jacinto fault zone, additional lateral pressures will occur along the back of walls retaining more than 6 feet of soil. The seismic-induced lateral soil pressure may be computed using a triangular pressure distribution with the maximum value at the top of the wall. The maximum lateral pressure of (20 pcf) H where H is the height of the retained soils above the wall footing should be used in final design of retaining walls.

Sliding resistance values and passive fluid pressure values given in our previous report may be increased by 1/3 during short-term wind and seismic loading conditions.

7.7 **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 and brought to 3% above optimum moisture contents to a depth of 18 inches, as verified by the soil engineer. Slabs should be reinforced with a minimum of #3 steel bars, placed at 18 inches on-center in each direction, positioned mid-height in the slab. Concrete slabs (4000 psi) 8 inches in thickness with dowel baskets may delete the above reinforcement requirement. 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 upon 4 inches of sand or gravel. The surface 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.

All concrete slab areas to receive floor coverings should be moisture tested to meet all manufacturer requirements prior to placement.

7.8 Expansive Soil

The upper soils at the site are low (Expansion Index = 21-50) in expansion potential. 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.

7.9 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.

7.10 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 IV.

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 V – VII. Soil pH indicates a slightly acidic condition. Resistivity is representative of moderately corrosive soils and metallic structures should be protected as necessary. Chloride content measured 150 ppm.

7.11 Preliminary Pavement Design

The table below provides a preliminary pavement design based upon a tested R-Value of 43 for the proposed pavement areas. Final pavement design should 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	3.5
Truck	7.0*	4.5	6.0
Truck	8.0**	5.5	6.5

* Design assumes 26 80,000 lb. trucks per week over 20 years.
** Design assumes 80 80,000 lb. trucks per week over 20 years.

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 7 inches in thickness and may be placed on subgrade soils compacted to at least 95% relative compaction and brought to 3% above optimum moisture levels to a depth of 18 inches, as verified by the soil engineer. 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 of 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. No significant caving occurred to the depths of these test 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:
$Vir=\Delta Vir/(Air\Delta t)$
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 E and summarized below.

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)	Infiltration Rate (in/hr)
T-1	5.0	sandy Clay	0.7	0.28
T-2	7.5	sandy Clay	0.6	0.24
T-3	10.0	sandy Clay	0.1	0.04

Soils at all locations are not suitable for infiltration due to very low tested rates. 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

Keith D Tucker

REGISTERED PROFESSIONAL ENGINEER
KEITH D. TUCKER
No. 841
Exp. 12/31/2022
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.

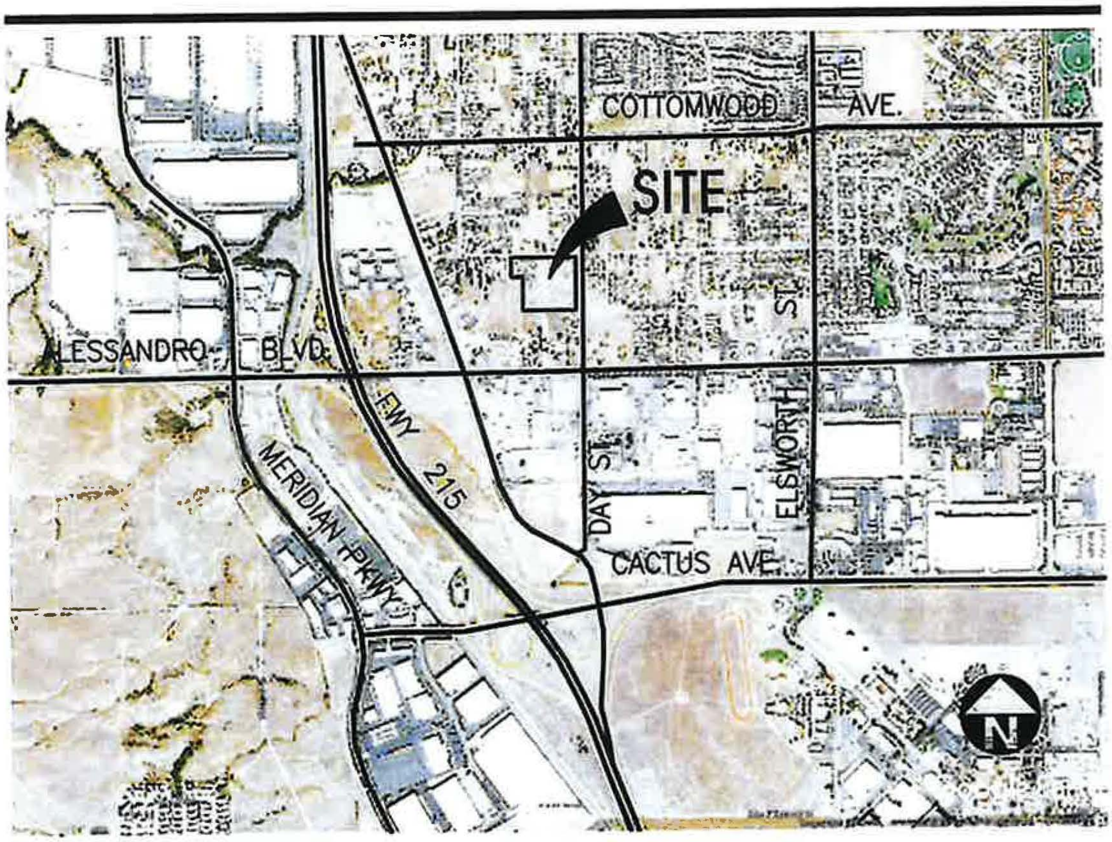
- Irrigation should be strictly controlled around building foundations, slabs and pavement and may need to be adjusted depending upon season. This control is essential to maintain a relatively uniform moisture content in the expansive soils and to prevent swelling and contracting. Over-watering adjacent 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.



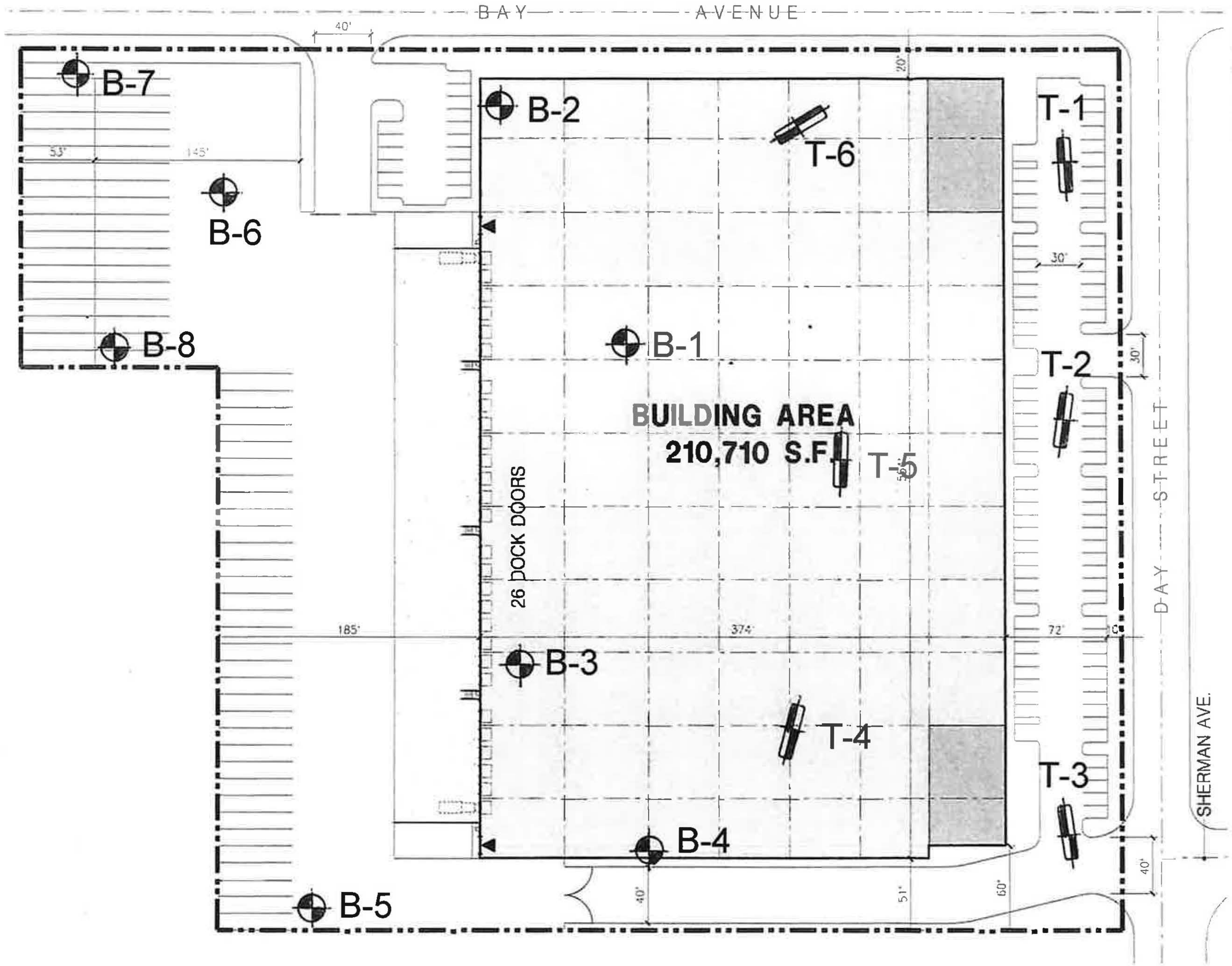
NorCal Engineering

SOILS AND GEOTECHNICAL CONSULTANTS

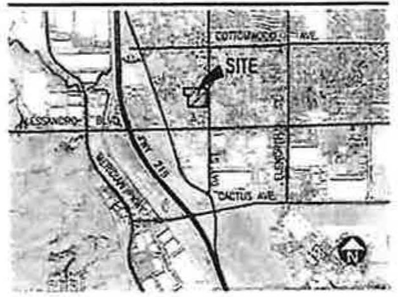
VICINITY MAP

PROJECT 22228-20 DATE 1/2021

FIGURE 1



Aerial Map



Legend

▨ POTENTIAL OFFICE WITH 2ND FLOOR

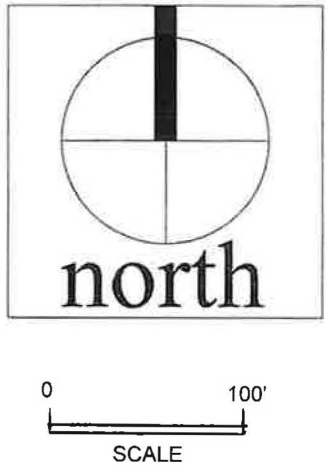
☐ WAREHOUSE

▼ DRIVE THRU DOOR

Tabulation

SITE AREA	
In s.f.	432,125 s f
In acres	9.92 ac
BUILDING AREA	
Office - 1st floor	4,000 s.f.
Office - 2nd floor	4,000 s.f.
Warehouse	202,710 s.f
TOTAL	210,710 s.f
COVERAGE	48.8%
AUTO PARKING REQUIRED	
Office: 1/250 s.f.	32 stalls
Whse: 1st 20,000 @ 1/1.000 s.f.	20 stalls
2nd 20,000 @ 1/2.000 s f.	10 stalls
above 40,000 @ 1/4,000 s f.	41 stalls
TOTAL	103 stalls
AUTO PARKING PROVIDED	
Standard (9' x 18')	104 stalls
TRAILER PARKING REQUIRED	
Trailer: 1 per dock door	26 doors
TRAILER PARKING PROVIDED	
Trailer (12' x 53')	67 stalls
ZONING ORDINANCE FOR CITY	
Zoning Designation - Business park (BP)	
MAXIMUM BUILDING HEIGHT ALLOWED	
Height - no height requirement	
MAXIMUM FLOOR AREA RATIO	
FAR - to be verified	
SETBACKS	
Building	Landscape
Front - 20'	Front / street - 10'
Side/Rear - 3'	
Street - 20'	

Note: This is a conceptual plan. It is based on preliminary information which is not fully verified and may be incomplete. It is meant as a comparative aid in examining alternate development strategies and any quantities indicated are subject to revision as more reliable information becomes available.



Conceptual Site Plan

Bay Ave. & Day Street

Moreno Valley, CA

NorCal Engineering
SOILS AND GEOTECHNICAL CONSULTANTS

PROJECT 22228-20
DATE 1/2021

GEOTECHNICAL MAP

FIGURE 2

APPENDICES

(In order of appearance)

APPENDIX A

AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE 7 Hazards Report

Address:
No Address at This
Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Elevation: 0 ft (NAVD 88)
Latitude: 33.921186
Longitude: -117.279323

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.5	S_{D1} :	N/A
S_1 :	0.6	T_L :	8
F_a :	1	PGA :	0.58
F_v :	N/A	PGA_M :	0.638
S_{MS} :	1.5	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	1	C_v :	1.4

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Fri Jan 22 2021

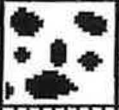
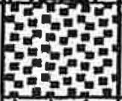
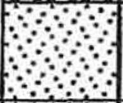
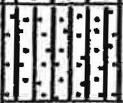
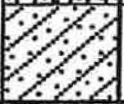
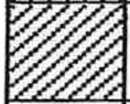
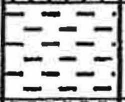
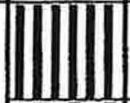
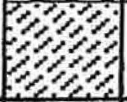
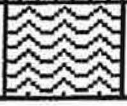
Date Source: USGS Seismic Design Maps

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.

APPENDIX B

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

LDC Molto Edgemont, LLC
22228-20

Log of Boring B-1

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/2020 | **Groundwater Depth:** 25

Drilling Method: Drill Rig

Hammer Weight: 140 lbs | **Drop:** 30"

Surface Elevation: Not Measured

Depth (feet)	Lithology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨		FILL SOILS Sandy CLAY with rootlets, occasional gravel and organics Brown, soft, damp					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, moist					
5	▨		Clayey SAND with gravel Brown, very dense, moist	☒	25/35/48	8.9		38
10	▨			☒	25/50+	12.6		31
	▥		Sandy SILT to Silty SAND with some clay Brown, very stiff, moist					
15	▥			☒	25/42/50+	9.9		41
20	▥		Silty SAND Brown, very dense, moist	☒	21/30/38	10.2		32
25	▥	▽	Seeping groundwater @ 25'	☒	23/25/27	14.1		21
30	▥			☒	5/10/21	21.6		37
35								

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LDC Molto Edgemont, LLC
22228-20

Log of Boring B-1

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/2020 | **Groundwater Depth: 25**

Drilling Method: Drill Rig

Hammer Weight: 140 lbs | **Drop: 30"**

Surface Elevation: Not Measured

Depth (feet)	Lith-ology	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
35	▤	Silty SAND Brown, very dense, moist	☒	19/27/32	13.2		20
	▨	Sandy CLAY Brown, very stiff, very moist					
40	▨		☒	16/18/24	16.0		47
45	▨		☒	13/20/37	19.3		56
50	▨		☒	11/16/28	18.3		55
		Boring completed at depth of 51.5'					
55							
60							
65							
70							

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22228-20

Log of Boring B-2

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/2020 **Groundwater Depth:** None Encountered

Drilling Method: Drill Rig

Hammer Weight: 140 lbs **Drop:** 30"

Surface Elevation: Not Measured

Depth (feet)	Lith-ology	Material Description	Samples		Laboratory		
			Type	Blow Counts	Moisture	Dry Density	Fines Content %
0		FILL SOILS Clayey SAND with rootlets and organics Brown, soft, damp to moist					
		NATURAL SOILS Sandy CLAY Brown, stiff, moist	☐				
5			■	35/50+	9.1	114.8	
		Clayey SAND with some small gravel Brown, very dense, moist					
10			■	28/48	11.3	120.7	
		Silty SAND Brown, very dense, moist					
15			■	30/50+	10.9	105.8	
20			■	30/38	10.3	110.2	
		Boring completed at depth of 21'					
25							
30							
35							

GWT not encountered

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NorCal Engineering

LDC Molto Edge[illegible]ont, LLC
22228-20

Log of Boring B-3

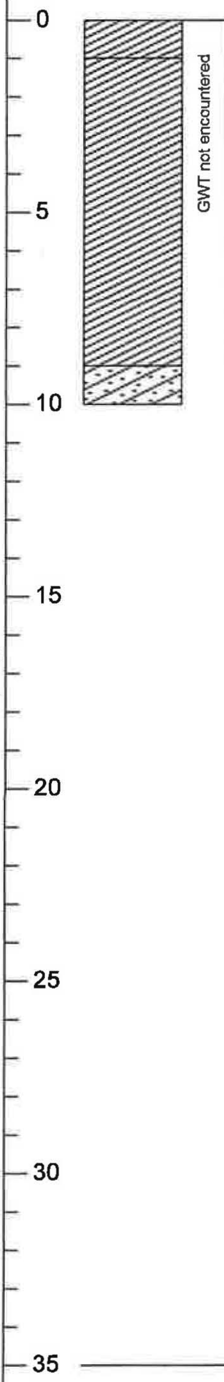
Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020 | **Groundwater Depth: None Encountered**

Drilling Method: Hand Auger

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured



Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
FILL SOILS Sandy CLAY with occasional gravel and rootlets Brown, soft, damp	■ ▽		6.2	112.8	
NATURAL SOILS Sandy CLAY Brown, stiff, damp	■		9.3	115.3	
Clayey SAND Brown, very dense, damp	▽		6.6		
Refusal at depth of 10'					

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22228-20

Log of Boring B-4

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020 | **Groundwater Depth: None Encountered**

Drilling Method: Hand Auger

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Lith- ology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨	GWT not encountered	FILL SOILS Sandy CLAY with occasional gravel, rootlets and organics Brown, stiff, moist					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, moist	■		9.0	111.7	
5			Boring completed at depth of 5'					
10								
15								
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT\22228-20.log Date: 1/22/2021

LDC Molto Edge..ont, LLC
22228-20

Log of Boring B-5

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020

Groundwater Depth: None Encountered

Drilling Method: Hand Auger

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 Sandy CLAY with rootlets Brown, soft, damp					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, moist	■		8.2	114.2	
5	▨							
	▨			■		7.4	118.6	
			Boring completed at depth of 7.5'					
10								
15								
20								
25								
30								
35								

LDC Molto Edge[illegible]ont, LLC
22228-20

Log of Boring B-6

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020 | **Groundwater Depth: None Encountered**

Drilling Method: Hand Auger

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 Sandy CLAY with occasional gravel and rootlets Brown, soft, damp					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, moist					
5			Boring completed at depth of 5'					
10								
15								
20								
25								
30								
35								

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LDC Molto Edgemont, LLC
22228-20

Log of Boring B-7

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020 **Groundwater Depth:** None Encountered

Drilling Method: Hand Auger

Hammer Weight: **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Material Description	Samples: Type	Samples: Blow Counts	Laboratory: Moisture	Laboratory: Dry Density	Laboratory: Fines Content %
0	FILL SOILS Sandy CLAY with rootlets Brown, soft, damp					
	NATURAL SOILS Sandy CLAY Brown, stiff, damp					
5		■		6.3	117.3	
	Clayey SAND Brown, dense, damp	■		9.4	122.1	
10	Boring completed at depth of 10'					
15						
20						
25						
30						
35						

GWT not encountered

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LDC Molto Edgemont, LLC
22228-20

Log of Boring B-8

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/28/2020

Groundwater Depth: None Encountered

Drilling Method: Hand Auger

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 Sandy CLAY with occasional gravel and rootlets Brown, soft, moist					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, moist					
5			Boring completed at depth of 5'					
10								
15								
20								
25								
30								
35								

NorCal Engineering

LDC Molto Edgemont, LLC
22228-20

Log of Trench T-1

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20

Groundwater Depth: None Encountered

Drilling Method: Backhoe

Hammer Weight:

Drop:

Surface Elevation: Not Measured

Depth (feet)	Lith-ology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨	GWT not encountered	FILL SOILS Sandy CLAY with occasional gravel, rootlets and asphalt pieces Brown, firm, damp					
5	▨		NATURAL SOILS Sandy CLAY Brown, stiff, damp	⊠		6.8		
			Trench completed at depth of 5'					
10								
15								
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT\22228-20.log Date: 1/22/2021

LDC Molto Edgemont, LLC
22228-20

Log of Trench T-2

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20 | **Groundwater Depth: None Encountered**

Drilling Method: Backhoe

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Lith- ology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨	GWT not encountered	FILL SOILS Sandy CLAY with rootlets and organics Brown, soft, damp	☒				
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, damp					
5	▨							
			Trench completed at depth of 7.5'	☒		7.4		
10								
15								
20								
25								
30								
35								

LDC Molto Edgemont, LLC
22228-20

Log of Trench T-3

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20 | **Groundwater Depth: None Encountered**

Drilling Method: Backhoe

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Lith-ology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨	GWT not encountered	FILL SOILS Sandy CLAY with rootlets Brown, soft, moist					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff to very stiff, moist					
5	▨							
10			Trench completed at depth of 10'	M		8.3		
15								
20								
25								
30								
35								

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LDC Molto Edgemont, LLC
22228-20

Log of Trench T-4

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20

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 Sandy CLAY with rootlets and organics Brown, soft, damp	■		4.3	110.7	
	▨		NATURAL SOILS Sandy CLAY Brown, very stiff, damp to moist					
5	▨			■		9.5	127.6	
			Trench completed at depth of 6'					
10								
15								
20								
25								
30								
35								

LDC Molto Edgemont, LLC

22228-20

Log of Trench T-5

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20

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 Sandy CLAY with rootlets and organics Brown, soft, moist					
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, damp	■		5.1	113.4	
5	▨			■		7.6	120.1	
			Trench completed at depth of 7'					
10								
15								
20								
25								
30								
35								

LDC Molto Edgemont, LLC
22228-20

Log of Trench T-6

Boring Location: SWC Bay Ave & Day St, Moreno Valley

Date of Drilling: 12/29/20 | **Groundwater Depth:** None Encountered

Drilling Method: Backhoe

Hammer Weight: | **Drop:**

Surface Elevation: Not Measured

Depth (feet)	Lith-ology		Material Description	Samples		Laboratory		
				Type	Blow Counts	Moisture	Dry Density	Fines Content %
0	▨	GWT not encountered	FILL SOILS Sandy CLAY with occasional gravel and asphalt pieces Brown, soft, damp	■		2.9	112.6	
	▨		NATURAL SOILS Sandy CLAY Brown, stiff, damp					
5	▨			■		4.7	117.2	
10	▨			■		8.6	115.1	
			Trench completed at depth of 11'					
15								
20								
25								
30								
35								

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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.)
B-2 @ 2-4'	sandy CLAY	9.5	131.0

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

Sample	Classification	Expansion Index
B-2 @ 2-4'	sandy CLAY	32

TABLE III
ATTERBERG LIMITS
(ASTM: D-4318-10)

Sample	Liquid Limit	Plastic Limit	Plasticity Index
B-2 @ 2-4'	23	17	6

TABLE IV
SOLUBLE SULFATE TESTS
(CT 417)

Sample	Sulfate Concentration (%)
B-3 @ 2-3'	0.0002

TABLE V
pH TESTS

Sample	pH
B-3 @ 2-3'	6.7

TABLE VI
RESISTIVITY TESTS
(CT 643)

Sample	Resistivity (ohm-cm)
B-3 @ 2-3'	4204

TABLE VII
CHLORIDE TESTS
(CT 422))

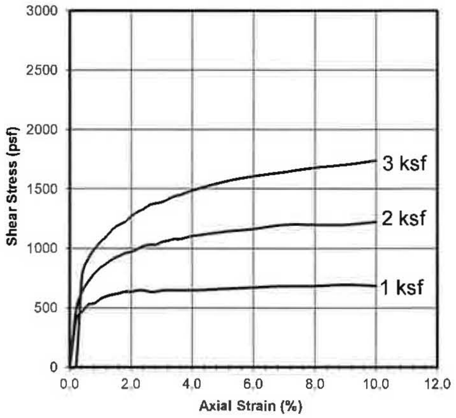
Sample	Concentration (ppm)
B-3 @ 2-3'	150

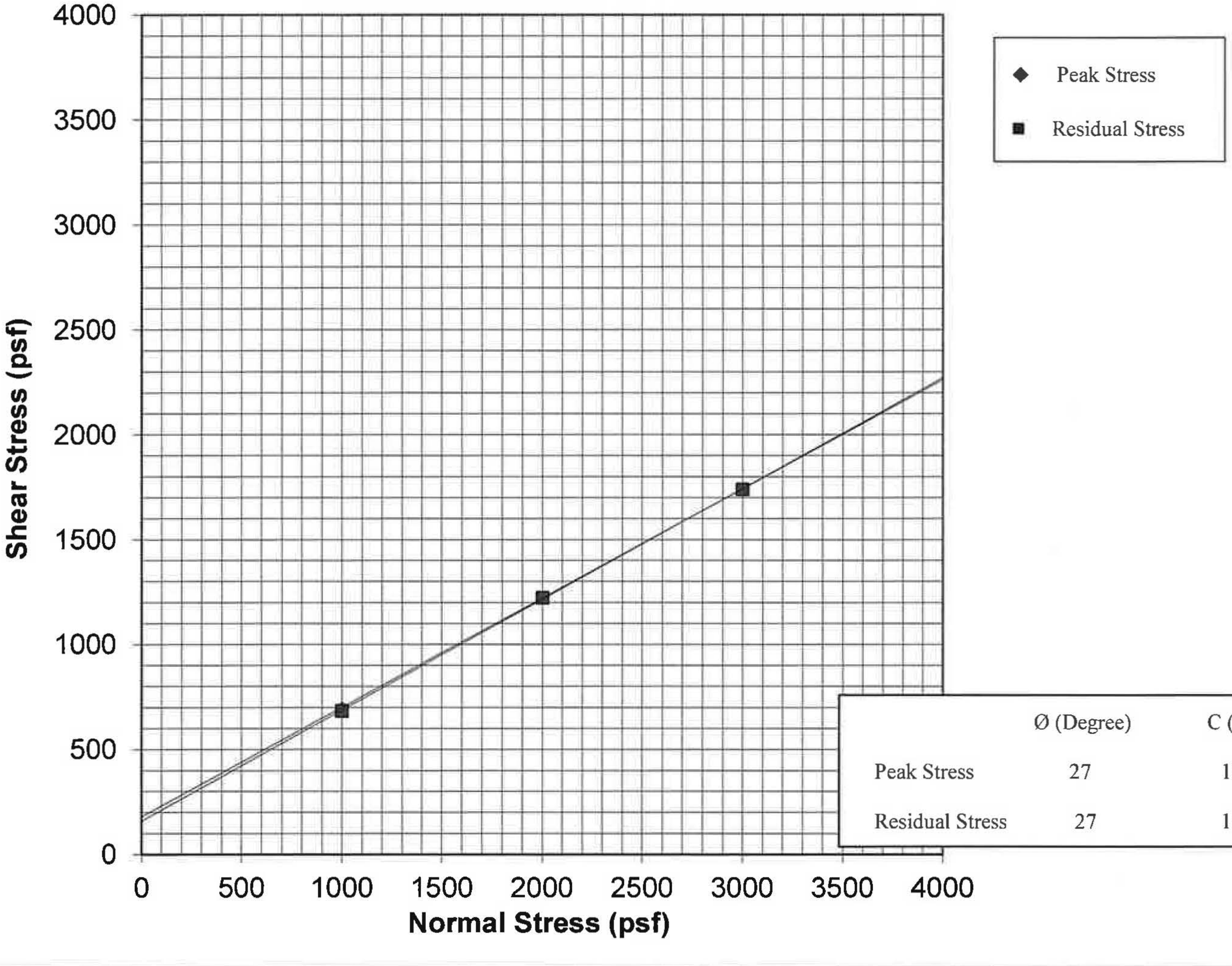
TABLE VIII
RESISTANCE 'R' VALUE TESTS
(CA 301))

Sample	'R' Value
T-2 @ 1-2'	43

Sample No.	B2@2-5'
Sample Type:	Remolded/Saturated
Soil Description:	sandy Clay

		1	2	3
Normal Stress	(psf)	1000	2000	3000
Peak Stress	(psf)	696	1224	1740
Displacement	(in)	0.225	0.250	0.250
Residual Stress	(psf)	684	1224	1740
Displacement	(in.)	0.250	0.250	0.250
In Situ Dry Density	(pcf)	117.9	117.9	117.9
In Situ Water Content	(%)	9.5	9.5	9.5
Saturated Water Content	(%)	15.8	15.8	15.8
Strain Rate	(in/min)	0.020	0.020	0.020





	Ø (Degree)	C (psf)
Peak Stress	27	180
Residual Stress	27	160

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SOILS AND GEOTECHNICAL CONSULTANTS

LDC Molto Edgemont, LLC

PROJECT NUMBER: 22228-20 DATE: 1/21/2021

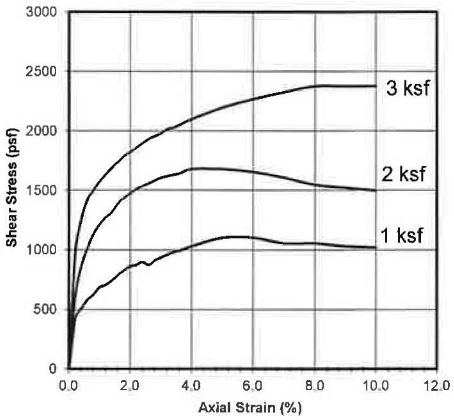
DIRECT SHEAR TEST

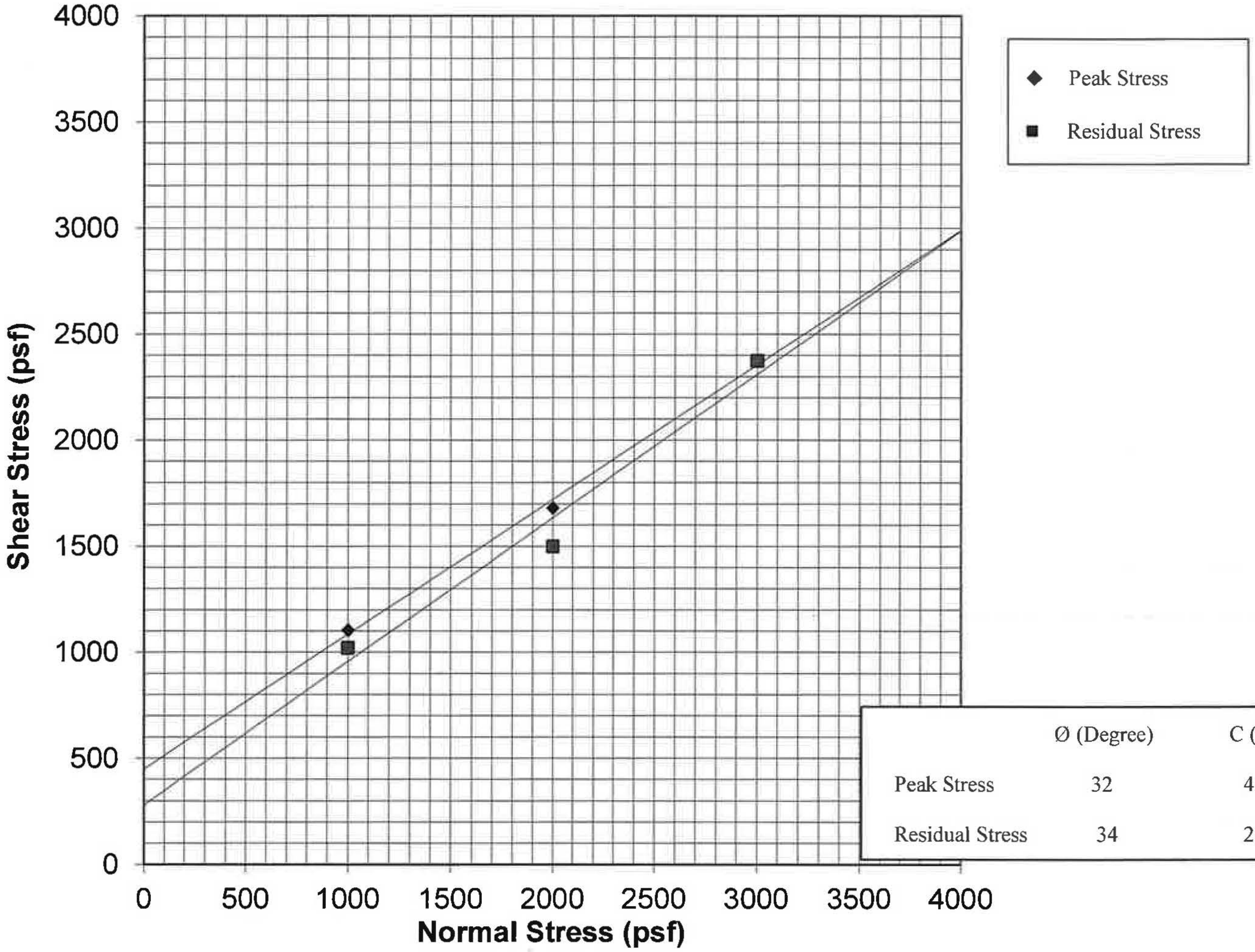
ASTM D3080

Plate A

Sample No. T4@5'
Sample Type: Undisturbed/Saturated
Soil Description: sandy Clay w/some small gravel

		1	2	3
Normal Stress	(psf)	1000	2000	3000
Peak Stress	(psf)	1104	1680	2376
Displacement	(in.)	0.125	0.100	0.200
Residual Stress	(psf)	1020	1500	2376
Displacement	(in.)	0.250	0.250	0.250
In Situ Dry Density	(pcf)	127.6	127.6	127.6
In Situ Water Content	(%)	9.5	9.5	9.5
Saturated Water Content	(%)	11.8	11.8	11.8
Strain Rate	(in/min)	0.020	0.020	0.020





	Ø (Degree)	C (psf)
Peak Stress	32	450
Residual Stress	34	280

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PROJECT NUMBER: 22228-20 DATE: 1/21/2021

DIRECT SHEAR TEST

ASTM D3080

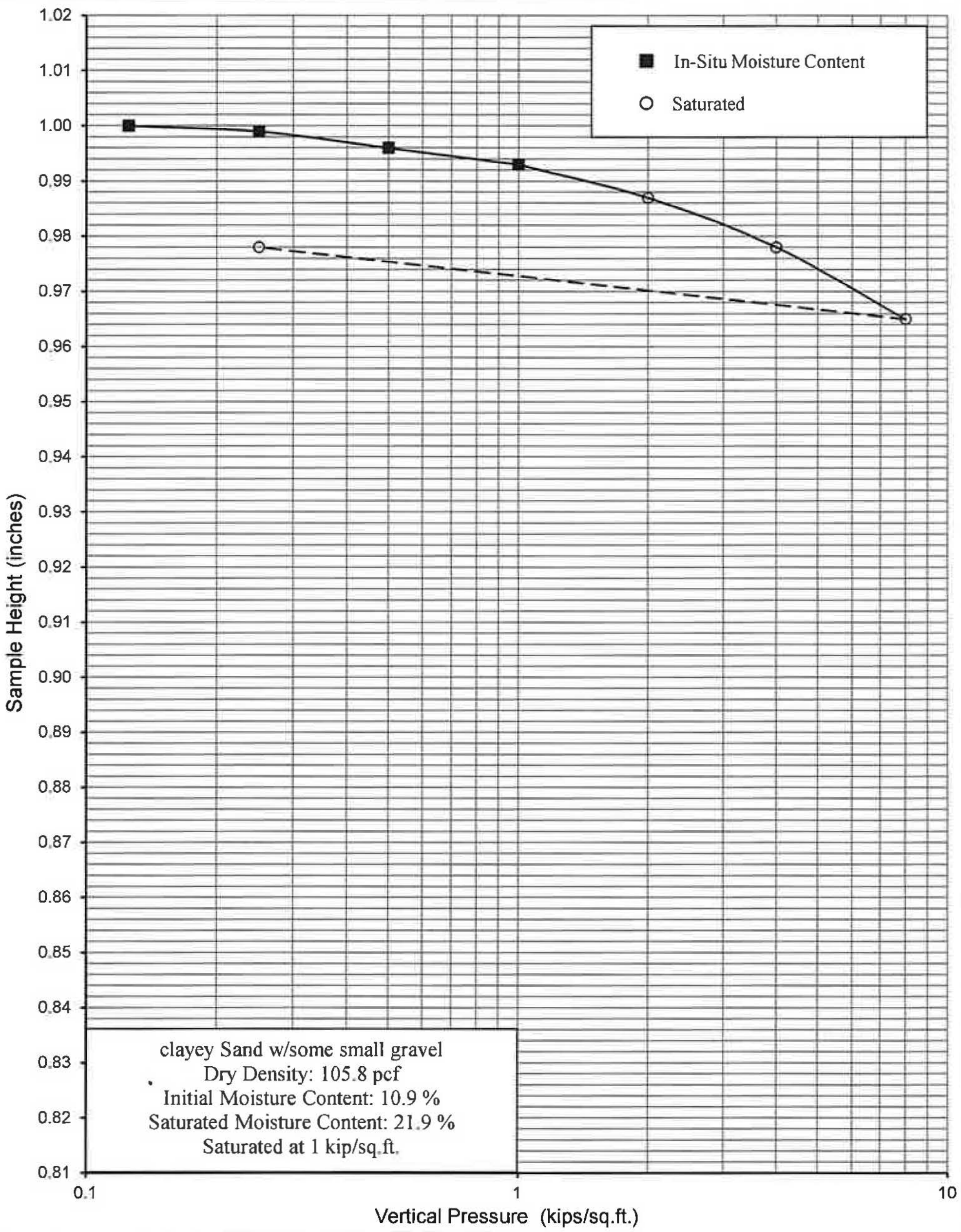
Plate B

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)
0.125	1.0000	0.0
0.25	0.9990	0.1
0.5	0.9960	0.4
1	0.9930	0.7
1	0.9930	0.7
2	0.9870	1.3
4	0.9780	2.2
8	0.9650	3.5
0.25	0.9780	2.2

Saturated

Sample No. B2 **Depth** 15' **Date** 1/21/2021

Date Tested: 1/11/2021
Sample: B2
Depth: 15'



clayey Sand w/some small gravel
Dry Density: 105.8 pcf
Initial Moisture Content: 10.9 %
Saturated Moisture Content: 21.9 %
Saturated at 1 kip/sq.ft.

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SOILS AND GEOTECHNICAL CONSULTANTS

LDC Molto Edgemont, LLC

PROJECT NUMBER: 22228-20 DATE: 1/21/2021

CONSOLIDATION TEST

ASTM D2435

Plate C

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)	
0.125	1.0000	0.0	
0.25	0.9970	0.3	
0.5	0.9940	0.6	
1	0.9885	1.2	
1	0.9822	1.8	Saturated
2	0.9671	3.3	
4	0.9456	5.4	
8	0.9287	7.1	
0.25	0.9412	5.9	

Sample No. B4

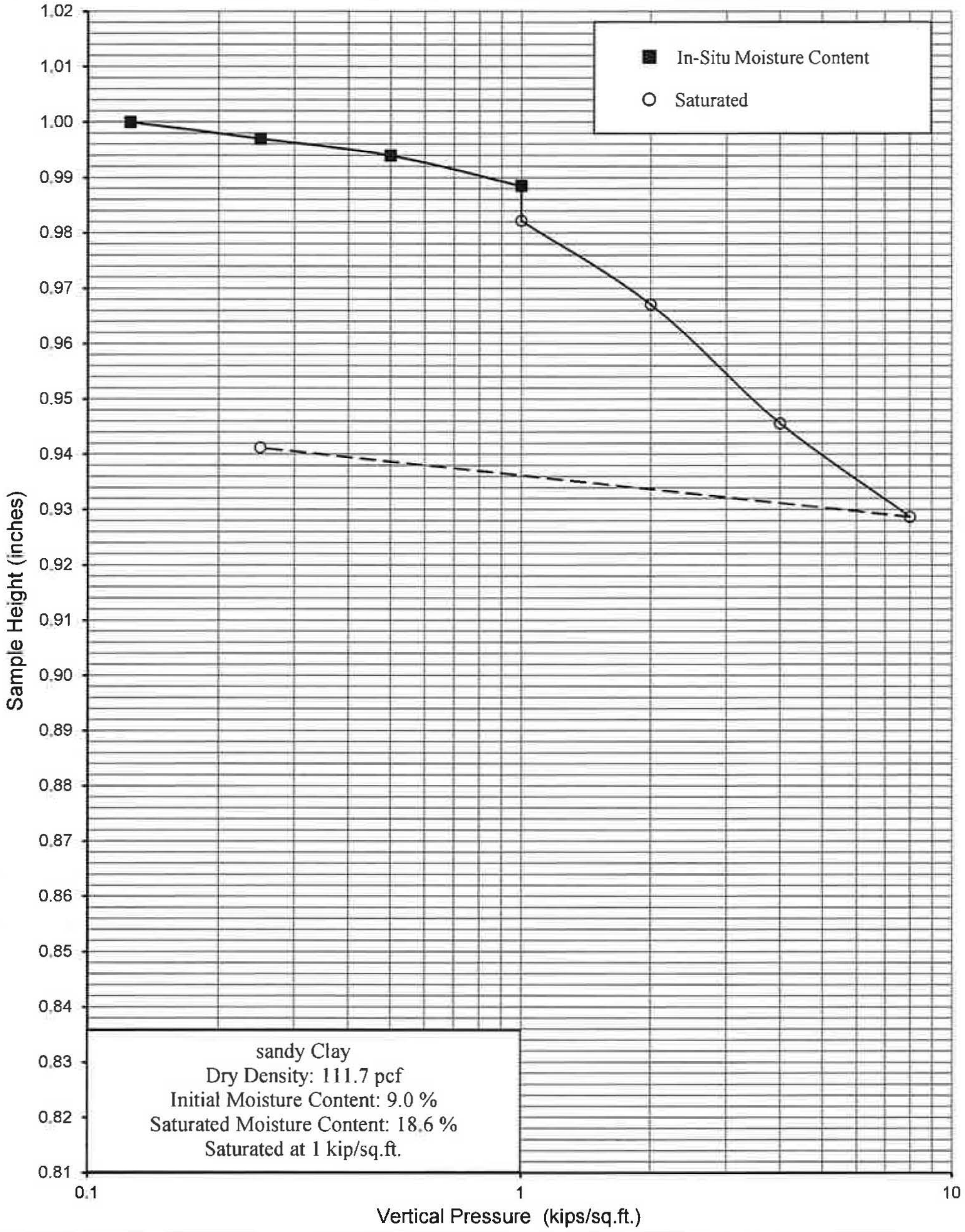
Depth 4'

Date 1/21/2021

Date Tested: 1/11/2021

Sample: B4

Depth: 4'



sandy Clay
Dry Density: 111.7 pcf
Initial Moisture Content: 9.0 %
Saturated Moisture Content: 18.6 %
Saturated at 1 kip/sq.ft.

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SOILS AND GEOTECHNICAL CONSULTANTS

LDC Molto Edgemont, LLC

PROJECT NUMBER: 22228-20 DATE: 1/21/2021

CONSOLIDATION TEST

ASTM D2435

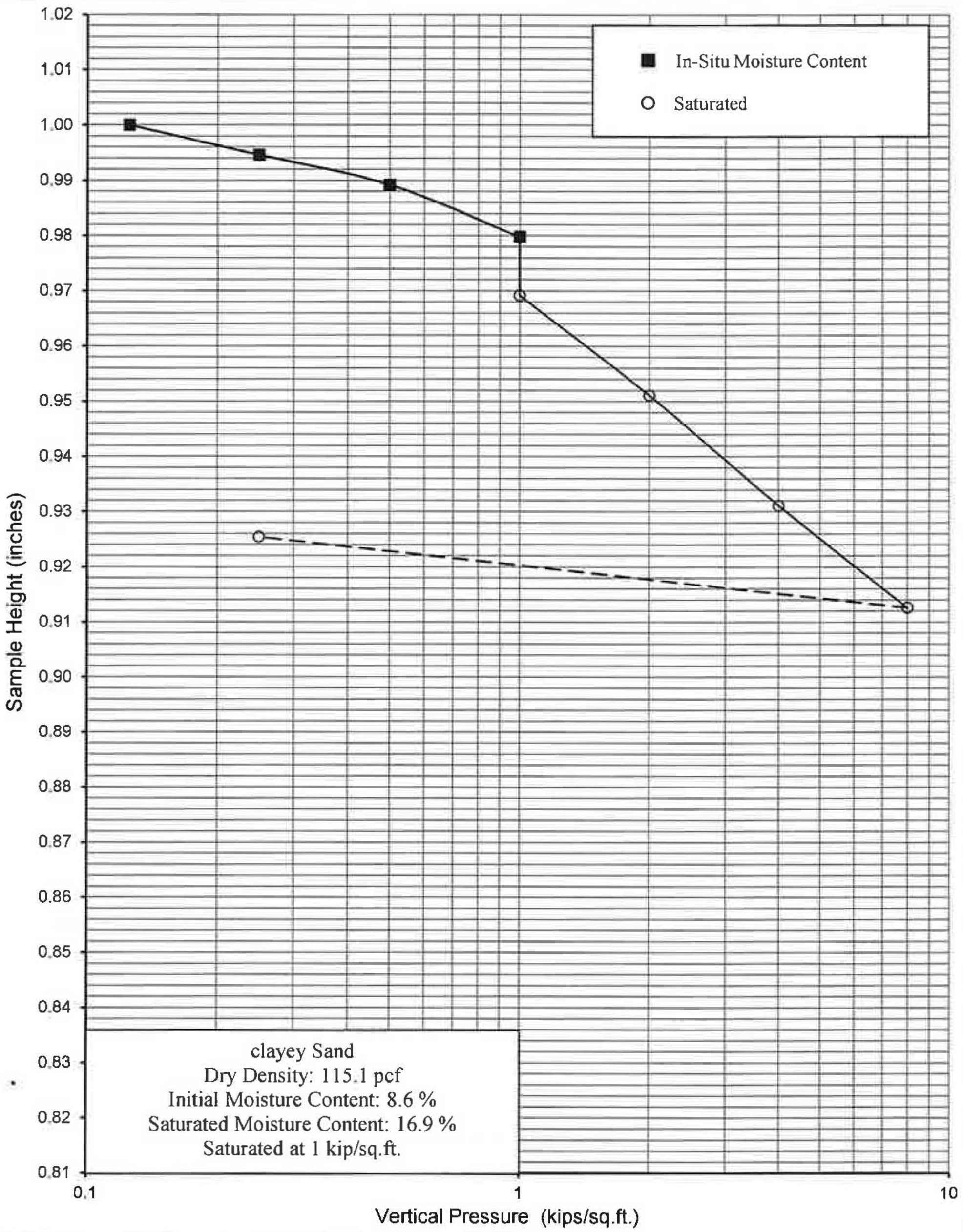
Plate D

Vertical Pressure (kips/sq.ft.)	Sample Height (inches)	Consolidation (percent)	Sample No.	T6	Depth 10'	Date 1/21/2021

0.125	**1.0000**	**0.0**
0.25	**0.9946**	**0.5**
0.5	**0.9892**	**1.1**
1	**0.9798**	**2.0**
1	**0.9692**	**3.1**
2	**0.9511**	**4.9**
4	**0.9311**	**6.9**
8	**0.9126**	**8.7**
0.25	**0.9254**	**7.5**

Saturated

Date Tested: 1/12/2021
Sample: T6
Depth: 10'



clayey Sand
Dry Density: 115.1 pcf
Initial Moisture Content: 8.6 %
Saturated Moisture Content: 16.9 %
Saturated at 1 kip/sq.ft.

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SOILS AND GEOTECHNICAL CONSULTANTS

LDC Molto Edgemont, LLC

PROJECT NUMBER: 22228-20 DATE: 1/21/2021

CONSOLIDATION TEST

ASTM D2435

Plate E

APPENDIX D

SITE LOCATION:

GEOTECHNICAL REPORT:

GEOLOGY REPORT:

DEPTH TO WATER TABLE = 20'

EARTHQUAKE MAGNITUDE = 6.9

PEAK GROUND ACCELERATION = 0.64g

DEPTH BELOW FINAL GRADE (FEET)	MOIST DENSITY (PCF)	σ_o TOTAL STRESS (PSF)	$\bar{\sigma}_o$ EFFECTIVE STRESS (PSF)	$\sigma_o/\bar{\sigma}_o$ (-)	r_d (-)	① $\tau_h/\bar{\sigma}_o$ (-)	N VALUE (BLOWS/FT)	RELATIVE DENSITY (%)	C_N (-)	C_E (-)	C_B (-)	C_R (-)	C_S (-)	$(N_1)_{60}$ (Blows/ft)	FINES (%)	CRR M=7.5	MSF (-)	CRR M=6.9	LIQ. F.S.
5	125	625	Same	1.00	0.99	0.42	>50	>90	>1.6	1.00	1.05	0.70	1.20	>72	38	>0.50	1.3	>0.65	>1.5
10	130	1275	↓	↓	0.96	0.40	>50	↓	1.2	↓	↓	0.75	↓	>57	31	↓	↓	↓	>1.6
15	115	1850	↓	↓	0.92	0.38	>50	↓	1.04	↓	↓	0.85	↓	>56	41	↓	↓	↓	>1.7
20	120	2450	↓	↓	0.87	0.36	>50	↓	0.90	↓	↓	0.90	↓	>51	32	↓	↓	↓	>1.8
25	↓	3050	2738	1.11	0.80	0.37	52	↓	0.88	↓	↓	0.95	↓	55	21	↓	↓	↓	>1.8
30	↓	3650	3026	1.21	0.74	0.37	31	80	0.84	↓	↓	1.00	↓	33	37	↓	↓	↓	>1.8
35	↓	4250	3314	1.28	0.68	0.37	59	>90	0.81	↓	↓	↓	↓	60	20	↓	↓	↓	>1.8
40	↓	4850	3602	1.35	0.64	0.36	42	85	0.78	↓	↓	↓	↓	41	47	↓	↓	↓	>1.8
45	↓	5450	3890	1.40	0.61	0.36	57	>90	0.76	↓	↓	↓	↓	55	56	↓	↓	↓	>1.8
50	↓	6050	4178	1.45	0.58	0.35	44	85	0.74	↓	↓	↓	↓	41	55	↓	↓	↓	>1.9

① INDUCED CYCLIC STRESS RATIO = $\tau_{ave}/\bar{\sigma}_o = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\bar{\sigma}_o} \cdot r_d$

- C_E = Corr. - Energy Ratio = Energy Ratio/60%
- C_B = Corr. - Borehole Dia. = 1.15 for 8" dia. borehole
- C_R = Corr. - Rod Length
- C_S = Corr. - Sampling Method

Actual Energy Ratio = 0.67-1.17 (Safety Hammer)
= 0.50-1.00 (Donut Hammer)

Sampling Method = 1.0 Standard sampler
= 1.2 Sampler W/o liners

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SOILS AND GEOTECHNICAL CONSULTANTS

PROJECT

DATE

EVALUATION OF LIQUEFACTION POTENTIAL

APPENDIX E

Project: LDC Molto Edgemont, LLC
Project No.: 22228-20
Date: 12/28/2020
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	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
8:30			69.3			36.4					
8:40	10	10	69.7	0.4		36.5	0.2				
8:40			69.7			36.5					
8:50	10	20	69.9	0.2		36.7	0.4				
8:50			69.9			36.7					
9:00	10	30	70.1	0.2		36.9	0.4				
9:00			70.1			36.9					
9:10	10	40	70.3	0.2		37.0	0.2				
9:10			70.3			37.0					
9:20	10	50	70.4	0.1		37.0	0.1				
9:20			70.4			37.0					
9:30	10	60	70.6	0.2		37.1	0.2				
9:30			70.6			37.1					
9:40	10	70	70.7	0.1		37.1	0.1		0.6	0.6	
9:40			70.7			37.1					
9:50	10	80	70.9	0.2		37.2	0.2		1.2	1.2	
9:50			70.9			37.2					
10:00	10	90	71.0	0.1		37.4	0.2		0.6	0.6	
10:00			71.0			37.4					
10:10	10	100	71.1	0.1		37.5	0.1		0.6	0.6	
10:10			71.1			37.5					
10:20	10	110	71.2	0.1		37.6	0.1		0.6	0.6	
10:20			71.2			37.6					
10:30	10	120	71.3	0.1		37.6	0.0		0.6	0.0	

Average = 0.7 / 0.6 cm/hr

Project: LDC Molto Edgemont, LLC
Project No.: 22228-20
Date: 12/28/2020
Test No. T-2
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	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
9:03			101.2			39.6					
9:13	10	10	101.5	0.3		39.8	0.2				
9:13			101.5			39.8					
9:23	10	20	101.6	0.1		40.0	0.2				
9:23			101.6			40.0					
9:33	10	30	101.6	0.0		40.0	0.0				
9:33			101.6			40.0					
9:43	10	40	101.6	0.0		40.0	0.0				
9:43			101.6			40.0					
9:53	10	50	101.6	0.0		40.0	0.0				
9:53			101.6			40.0					
10:03	10	60	101.7	0.1		40.0	0.0				
10:03			101.7			40.0					
10:13	10	70	101.7	0.0		40.1	0.1		0.0	0.6	
10:13			101.7			40.1					
10:23	10	80	101.9	0.2		40.2	0.2		1.2	1.2	
10:23			101.9			40.2					
10:33	10	90	101.9	0.0		40.2	0.0		0.0	0.0	
10:33			101.9			40.2					
10:43	10	100	102.0	0.1		40.3	0.1		0.6	0.6	
10:43			102.0			40.3					
10:53	10	110	102.2	0.2		40.4	0.1		1.2	0.6	
10:53			102.2			40.4					
11:03	10	120	102.3	0.1		40.5	0.1		0.6	0.6	

Average = 0.6 / 0.6 cm/hr

NORCAL
ENGINEERING
SOILS AND GEOTECHNICAL CONSULTANTS

Project: LDC Molto Edgemont, LLC
Project No.: 22228-20
Date: 12/28/2020
Test No. T-3
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	OUTER RING FLOW (cc)	INNER RING INF RATE (cm/hr)	OUTER RING INF RATE (cm/hr)	INNER RING INF RATE (ft/hr)
10:50			78.2			46.9					
11:00	10	10	78.2	0.0		46.9	0.0				
11:00			78.2			46.9					
11:10	10	20	78.2	0.0		46.9	0.0				
11:10			78.2			46.9					
11:20	10	30	78.2	0.0		46.9	0.0				
11:20			78.2			46.9					
11:30	10	40	78.2	0.0		46.9	0.0				
11:30			78.2			46.9					
11:40	10	50	78.2	0.1		46.9	0.0				
11:40			78.2			46.9					
11:50	10	60	78.3	0.1		46.9	0.0				
11:50			78.3			46.9					
12:00	10	70	78.3	0.0		47.0	0.1		0.0	0.6	
12:00			78.3			47.0					
12:10	10	80	78.3	0.0		47.0	0.0		0.0	0.0	
12:10			78.3			47.0					
12:20	10	90	78.3	0.0		47.0	0.0		0.0	0.0	
12:20			78.3			47.0					
12:30	10	100	78.4	0.1		47.0	0.0		0.6	0.0	
12:30			78.4			47.0					
12:40	10	110	78.4	0.0		47.1	0.1		0.0	0.6	
12:40			78.4			47.1					
12:50	10	120	78.4	0.0		47.1	0.0		0.0	0.0	

Average = 0.1 / 0.2 cm/hr