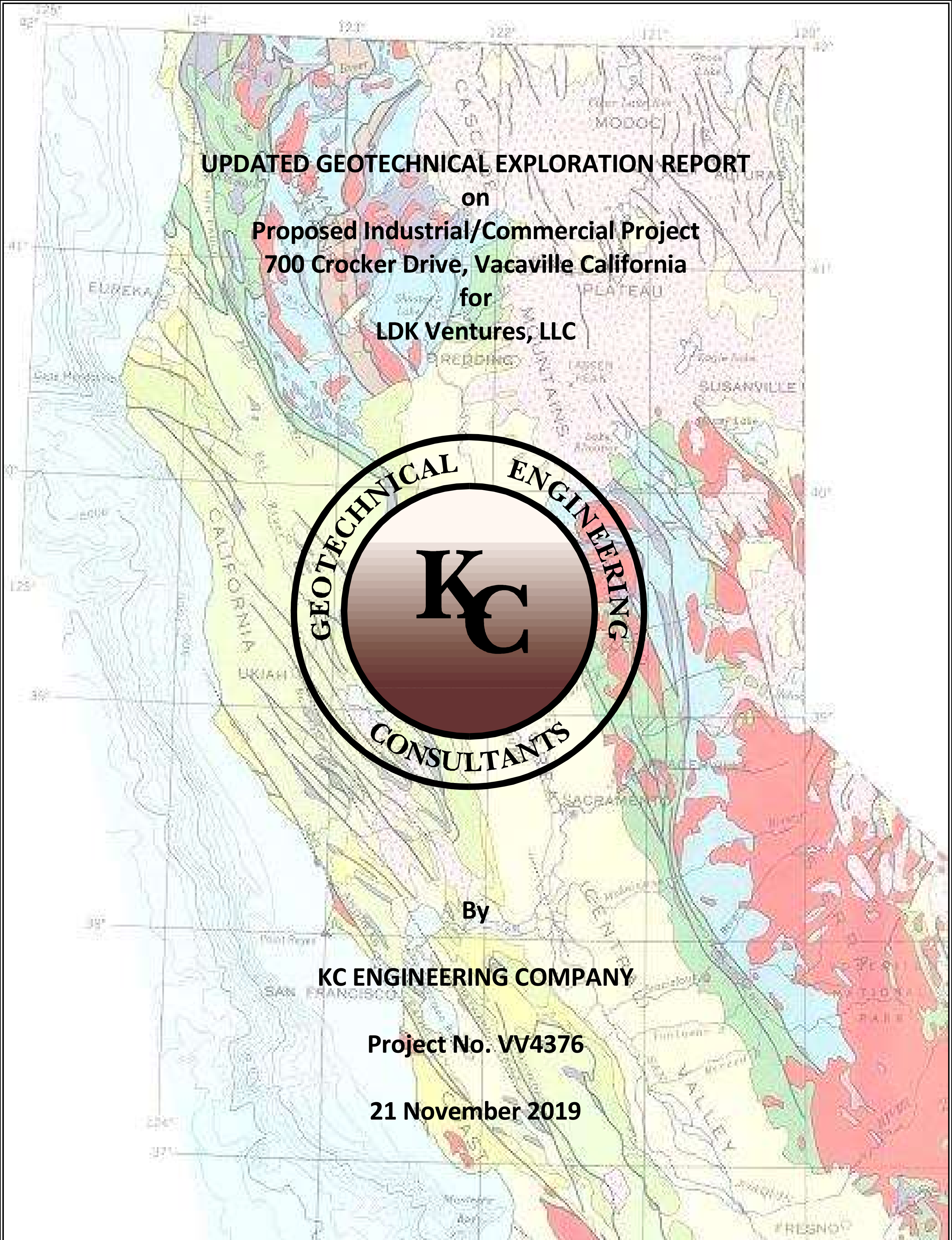
Appendix D

Geotechnical Exploration Report



UPDATED GEOTECHNICAL EXPLORATION REPORT
on
Proposed Industrial/Commercial Project
700 Crocker Drive, Vacaville California
for
LDK Ventures, LLC

GEOTECHNICAL ENGINEERING CONSULTANTS

KC

By

KC ENGINEERING COMPANY

Project No. VV4376

21 November 2019

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Project No. VV4376
21 November 2019

Mr. Bud Schiveley
LDK Ventures, LLC
3140 Peacekeeper Way
McClellan, CA 95652

Subject: Proposed Industrial/Commercial Project
700 Crocker Drive
Vacaville, California
UPDATED GEOTECHNICAL EXPLORATION REPORT

Dear Mr. Schiveley:

In accordance with your authorization, **KC ENGINEERING COMPANY** has explored the geotechnical conditions of the surface and subsurface soils at the subject site for the proposed Industrial/Commercial project to be constructed at the subject site. This report updates our prior Geotechnical Exploration Report dated 11/3/17.

The accompanying report presents our conclusions and recommendations based on our exploration. Our findings indicate that the proposed Industrial/Commercial project is geotechnically feasible for construction on the subject site provided the recommendations of this report are carefully followed and are incorporated into the project plans and specifications.

Should you have any questions relating to the contents of this report or should you require additional information, please contact our office at your convenience.

Reviewed by,

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Respectfully Submitted,
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UPDATED GEOTECHNICAL EXPLORATION

Purpose and Scope

The purpose of the updated geotechnical exploration for the proposed Industrial/Commercial project at 700 Crocker Drive in Vacaville, California, was to determine the surface and subsurface soil conditions for the proposed improvements at the subject site. Based on the results of the exploration, geotechnical criteria were established for the grading of the site, the design of foundations, slabs-on-grade, retaining walls, pavement sections, box-culvert creek crossings, and the construction of other related facilities on the property.

In accordance with your authorization, our exploration services included the following tasks:

a. A review of available geotechnical and geologic literature concerning the site and vicinity;
b. Site reconnaissance by the Geotechnical Engineer to observe and map surface conditions;
c. Drilling of a total of 18 exploratory borings, excavating 14 test pits, and sampling of the subsurface soils;
d. Laboratory testing of the samples obtained to determine their classification and engineering characteristics;
e. Analysis of the data and formulation of conclusions and recommendations; and
f. Preparation of this written report.

Site Location and Description

The subject site is located at 700 Crocker Drive in the City of Vacaville, California as shown on Figure 1 “Aerial Vicinity Map” included in the Appendix of this report. The property includes the existing old Lucky/Savemart grocery warehouse structure and related site improvements. The property is bounded on the north by Midway Road, on the east by Interstate 505, on the southeast by the termination of Crocker Drive, on the south by an open field and the Mariani Nut facility, and on the west by an open field and a newly constructed warehouse building. Gibson Canyon Creek crosses the site from the west to the east.

The proposed development areas are shown on Figure 2, “Site Plan”. The proposed Phase 1 industrial building structure is planned to be located in the field north of the existing warehouse building. This field area is relatively flat and covered with weeds and a few scattered trees. This area has been recently disked. Stockpiled fill areas are located on the northwest, north central and southcentral areas of the field as shown on Figure 3, “Undocumented Fill & Creek Washout Exhibit”. An existing electrical transformer is located on the east side of the field area. Concrete pavement and a loading dock structure is present on the southeast.

The proposed Phase 2 industrial buildings are planned north of Gibson Canyon Creek and south of Midway Road as shown on Figure 2. A future commercial development is also planned on the northeast quadrant of the property. Within this development area north of Gibson Canyon Creek, the ground surface topography is relatively flat to gently sloping as shown on Figure 3. A low height ridge fans downward from the north to the south in the central portion of this area. Stockpiled fill materials are present on the northwest adjacent to a drainage ditch. A Solano Irrigation District (SID) earthen canal crosses the site north of Gibson Canyon Creek. Underground SID pipelines and siphon water pipelines convey water to and between the canal sections. A recently installed City of Vacaville water pipeline crosses the site from Hartley Road south into the property. We understand that an old railroad also entered the site from the vicinity of Hartley Road

We understand that Gibson Canyon Creek formerly meandered through the property and was formerly realigned/straightened during development of the existing 700 Crocker facility. The current creek alignment was dry during our exploration and appeared to be performing well, except for three obvious creek bank washouts as shown on Figure 3. The washouts ranged in width from about 10 feet on the west to about 100 feet on the east near the future creek road crossing. The creek banks have inclinations ranging from about 2H:1V to near vertical conditions along the flow line. Rip-Rap rock slope protection materials were noted along portions of the creek banks.

The above description is based on a reconnaissance of the site by the Geotechnical Engineer, a review of an “Overall Site Plan” prepared by Phillippi Engineering dated 10/25/19 and a review of a Google Earth image dated 9/1/18. The Google Earth image was used as the basis for our “Aerial Vicinity Map” included as Figure 1, and the Overall Site Plan was used as our “Site Plan” included as Figure 2 in the Appendix.

Proposed Development

We understand that the proposed development will be performed in phases and will include construction of a large industrial building structure north of the existing 700 Crocker warehouse, followed by construction of two smaller industrial/warehouse buildings south of Midway Road as shown on Figure 2, “Site Plan” in the Appendix. The main industrial/warehouse building is planned to be approximately 617,760 square feet and constructed with a reinforced concrete slab floor and tilt-up walls with interior structural steel columns and roof truss system. The two smaller industrial/warehouse buildings south of Midway Road are planned to be approximately 33,696 square feet and 93,240 square feet. The northeastern quadrant at the southwest corner of Midway Road and the I-505 on-ramp is expected to be a future commercial project with potentially a gas station and fast-food restaurant. We anticipate structural loading for the industrial warehouse structures to have wall loads of about 5,000 to 7,500 p.l.f. with column loads on the order of 25 to 100 kips. Truck loading docks are planned for the structures. The future commercial buildings are expected to be one or two-stories in height and constructed of conventional wood or light-gauge metal framing.

Site improvements are planned to consist of demolishing the transformer, loading dock and metal building structures on the east, as well as removal of the SID canal and pipelines. New truck driveways and parking lot construction is planned for the proposed industrial buildings as shown on Figure 2. Two roadway crossings are planned over Gibson Canyon Creek and will require the use of pre-cast double box culverts and cast-in-place concrete headwalls. Rock slope protection is also planned to provide erosion and scour protection for the creek crossings, as well as to stabilize the existing creek bank washouts. Midway Road is also expected to be widened along the project frontage. Mass grading for the project is expected to consist of various cuts and fills of about 10 vertical feet or less to achieve design pad and road way grades. Low height cut and fill slopes of 10 feet or less may also be incorporated in the project. Additional site improvements will consist of underground utilities, lighting, garbage enclosures, storm water bio-filtration swales or basins and landscaping.

Field Exploration

Our field explorations were performed in October 2017 and October 2019, and included a reconnaissance of the site and the drilling of eighteen exploratory test borings at the approximate locations shown on Figure 2, “Site Plan”. Additionally, fourteen test pits were excavated across the site to evaluate the stockpiled materials and general soil conditions. The test pit locations are shown on Figure 3, “Undocumented Fill & Creek Washout Exhibit”. Bulk samples of the building pads and proposed parking lot/drive lane subgrade were also obtained.

The borings were drilled to a maximum depth of 25 feet below the existing ground surface. The drilling was performed with a Mobile B-24 drill rig using power-driven, four-inch diameter solid flight augers. Visual classifications were made from auger cuttings and the samples in the field. As the drilling proceeded, relatively disturbed tube samples were obtained by driving a 3-inch O.D., California split-tube sampler, containing thin brass liners, into the boring bottom in accordance with ASTM D3550. Disturbed samples were also obtained by driving a 2-inch O.D., split-barrel SPT sampler into the boring bottom in accordance with ASTM D1586. The samplers were driven into the in-situ soils at various depths under the impact of a 140-pound hammer having a free fall of 30 inches. The number of blows required to advance the sampler 12 inches into the soil, after seating the sampler 6 inches, were adjusted to the standard penetration resistance (N-Value). The raw blow counts obtained using the California sampler were corrected to equivalent N-Values using Burmister's (1948) energy and diameter correction formula. When the sampler was withdrawn from the boring bottom, the samples were removed, examined for identification purposes, labeled and sealed to preserve the in-situ moisture content, and transported to our laboratory for testing.

Classifications made in the field were verified in the laboratory after further examination and testing. The stratification of the soils, descriptions, location of disturbed soil samples and standard penetration resistance are shown on the respective "Log of Test Boring" and "Test Pit Log" contained within the Appendix.

Laboratory Testing

The laboratory testing program was directed towards providing sufficient information for the determination of the engineering characteristics of the site soils so that the recommendations outlined in this report could be formulated. The laboratory test results are presented on the respective Boring Logs and data sheets in the Appendix.

Moisture content and dry density tests (ASTM D2937) were performed on representative relatively disturbed soil samples in order to determine the consistency of the soil and the moisture variation throughout the explored soil profile as well as estimate the compressibility of the underlying soils.

The strength parameters of the foundation soils were determined from direct shear tests (ASTM D3080) and unconfined compression tests (ATSTM D2166) performed on selected relatively disturbed soil samples. Standard field penetration resistance (N-Values) and pocket penetrometer readings also assisted in the determination of strength and bearing capacity. The standard penetration resistances and pocket penetrometer readings are recorded on the respective "Log of Test Boring".

In order to assist in the identification and classification of the subsurface soils, sieve analysis tests (ASTM D6913) and Atterberg Limits tests (ASTM D4318) were performed on selected soil samples. The Atterberg Limits test results were used to estimate the expansion potential of the near surface soils. An expansion Index test (ASTM D4829) was also performed on a sample representative of the pad soils. The sieve analysis results also aided in our liquefaction analysis.

R-Value tests (Cal Test 301) were performed on composite bulk samples representative of the proposed subgrade to assist in pavement section design. The location of the R-Value sample locations are shown on Figure 2.

Representative bulk samples of the near surface soils were obtained from the building pads to evaluate the presence and concentration of water soluble sulfates in accordance with ASTM C1580. These test results were used to identify the corrosion potential of the soils to at or below grade concrete. Additional soil corrosion potential tests (pH, Resistivity & Chlorides) were also performed. The corrosivity sample locations are also shown on Figure 2. A discussion is presented in the "Soil Corrosivity" section of this report.

Subsurface Conditions

Based on our field exploration and laboratory testing, the surface and subsurface soil conditions generally consist of isolated areas of undocumented fills and poorly stratified alluvial deposits of variable thickness across the property. Very dense and hard soils and bedrock of the Tehama Formation were encountered below the alluvial deposits.

In general, the upper 12 inches across the entire site was dry and loose due to recent disking. The upper 1 to 12 feet consists of moderately to very highly expansive, firm to very stiff sandy and silty clays and clayey silts mixed with variable amounts of sands and gravels. The underlying materials then consist of variable layers and thicknesses of medium dense to dense clayey and silty sands, and very stiff to hard silty and sandy clays to the maximum depth explored of 25 feet. It is noted that the upper 4 to 6 feet in Borings 1 and 8 consist of dry to moist and firm to stiff fill materials. Test Pits 1, 2, 3, 5, 7, 8, 10, 11 and 13 revealed relatively clean sandy clay fill, with some gravels and minor concrete debris in the upper 1.5 to 7 feet. Friable and hard claystone, siltstone and sandstone bedrock was encountered at 15 to 19 feet in Borings 12 and 13 and at 3 feet in Test Pit 8.

The Geologic Map, included herein as Figure 4, indicate that an old railroad and Gibson Canyon Creek crossed the eastern portion of the site. Boring 1 encountered fill very stiff and dense fill materials in the upper 6 feet which may be within the old creek channel.

Groundwater was not encountered in any of the borings or test pits at the time of drilling. Fluctuations in the groundwater conditions can occur with variations in seasonal rainfall, site irrigation and variations in subsurface stratification.

A more thorough description and stratification of the soils encountered along with the results of the laboratory tests are presented on the respective “Log of Test Boring” and “Test Pit Log” in the Appendix. The approximate locations of the borings are shown on Figure 2, and the test pits on Figure 3.

Soil Corrosivity

A representative composite sample of the near surface building pad soil (upper 5 feet) was collected and transported to Sunland Analytical in Rancho Cordova for testing of water soluble sulfates, pH, minimum resistivity and chlorides per California Test Methods.

The testing indicates a sulfate contents ranging from 11.6 to 38 ppm (mg/kg), chloride contents of 3.2 to 16.1 ppm, minimum resistivity’s of 880 to 3,480 ohm-cm, and soil pH’s of 5.82 to 6.57 for the sample collected. It is noted that the sulfate test results indicate “not-applicable” or “S0” sulfate exposure to concrete as identified in the Durability Requirements, Section 1904 of the 2016 and 2019 California Building Code, and Tables 19.3.1.1 of ACI 318-14 Building Code Requirements for Structural Concrete. No cement type restriction is required, however, we do recommend that a Type I/II cement be utilized in concrete mixes for additional sulfate and corrosion resistance.

The Caltrans Corrosion Guidelines[1] defines a corrosive site as one where the soil and/or water has a sulfate concentration of 1,500 ppm or more, a chloride concentration of 500 ppm or more, a pH of 5.5 or less, and a minimum resistivity less than 1,100 ohm-cm. Based on these criteria and the low resistivity value, the soils at the site are considered to have a severe corrosion potential to buried metal.

KC Engineering Company is not a corrosion engineering firm. Therefore, to further define the soil corrosion potential and interpret the above test results, or to design cathodic protection or grounding systems, a licensed Corrosion Engineer should be consulted.

[1] California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, Materials Engineering and Testing Services, *Corrosion Guidelines*, Version 3, March 2018.

Site Geology

According to the Geologic Map of the Lodi Quadrangle[2], the site exists in a transitional zone with geologic deposits underlying the northeast portion of the site as the Pliocene-aged Tehama Formation and the remainder of the site as Pleistocene-aged Alluvial Fan deposits. A partial geologic map showing the site is included in the Appendix as Figure 4, "Geologic Map". The Tehama Formation is noted to consist of poorly consolidated, nonmarine, gray to maroon siltstone, quartzarenite sandstone, tuff and pebble to cobble conglomerate. The alluvial fan deposits consist of varying layers of sands, silts, clays and gravels that are poorly sorted and bedded. The subsurface deposits encountered during our exploration generally correlate with the mapped geology.

Geo-Hazards

Seismicity & Ground Motion Analysis

The site is not located within an Alquist-Priolo Earthquake Fault Zone[3]. There are no known active faults crossing the site as mapped and/or recognized by the State of California. However, Vacaville is located in a seismically-active region and earthquake related ground shaking should be expected during the design life of structures constructed on the site. The California Geological Survey has defined an active fault as one that has had surface displacement in the last 11,000 years, or has experienced earthquakes in recorded history.

Based on our review of the Fault Activity Map of California[4] and the USGS National Seismic Hazard Maps-Source Parameters[5], the nearest major active faults are the Great Valley 4b Vaca Fault segment, the Kirby Hills Fault, the Green Valley Fault and the Hunting Creek-Berryessa Fault, located approximately 3.4 miles southwest, 9 miles south, 12.7 miles southwest, and 13.8 miles northwest of the site, respectively. Numerous other active faults in the Bay Area may also produce significant seismic shaking at the site.

The 2016 & 2019 CBC specifies that the potential for liquefaction and soil strength loss should be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with an adjustment for site class effects in accordance with American

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[2] T. E. Dawson, 2009, *Preliminary Geologic Map of the Lodi 30'x60' Quadrangle, California*, California Geological Survey

[3] Parish, J.G., 2018 *Earthquake Fault Zones,* California Geological Survey, Special Publication 42, Revised 2018.

[4] Jennings, C.W. and Bryant, W.A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6, scale 1:750,000

[5] U.S. Geological Survey, 2008 National Seismic Hazards Maps – Source Parameters, accessed 11/5/19, from USGS web site: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

Society of Civil Engineer (ASCE 7-10 & 7-16)[6]. The MCE_G is peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. Based on ASCE 7-10, the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated to be 0.584g using the SEAOC/OSHPD seismic design maps web-based tool with a site coefficient (F_{PGA}) of 1.0 for Site Class D. Based on ASCE 7-16, the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated to be 0.574g for the property using SEAOC/OSHPD U.S. Seismic Design Maps web-based tool with a site coefficient (F_{PGA}) of 1.1 for Site Class D.

The structures at the site should be designed to withstand the anticipated ground accelerations. Based on the SEAOC/OSHPD U.S Seismic Design Maps[7] website and ASCE 7-10, the 2016 CBC earthquake design values are as follows. The US seismic design summary report is included in the Appendix.

Site Class:	D
Mapped Acceleration Parameters:	S_S = 1.605g; S_1 = 0.55g
Design Spectral Response Accelerations:	S_{DS} = 1.07g; S_{D1} = 0.55g

Based on the SEAOC/OSHPD U.S Seismic Design Maps website and ASCE 7-16, the 2019 CBC earthquake design values are as follows. The US seismic design summary report is included in the Appendix.

Site Class:	D
Mapped Acceleration Parameters:	S_S = 1.248g; S_1 = 0.448g
Design Spectral Response Accelerations:	S_{DS} = 0.833g; S_{D1} = 0.553g

The provided values are based on a stiff clay soil profile or Site Class D for the upper 100 feet. In our opinion, a ground motion hazard analysis is not necessary per ASCE 7-16, Section 11.4.8, Exception 2. The seismic response coefficient Cs should be determined by Eq. (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 Eq. (12.8-3) for $T_L \geq T > 1.5T_S$ or Eq. (12.8-4) for $T > T_L$. This must be evaluated and verified by the Structural Engineer.

Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on our review of geologic maps, no known active or inactive faults cross or project toward the subject site. In

[6] American Society of Civil Engineer (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, Standard 7-10 & 7-16.
[7] https://seismicmaps.org/, accessed 11/5/19

addition, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that there is no potential for fault-related surface rupture at the subject site.

Landsliding

The subject site and immediate vicinity is relatively flat and therefore, not subject to seismically-induced landslide hazards. However, it is noted that a few erosion scar/creek bank washouts are located on the southern and northern banks of Gibson Canyon Creek as described above and as shown on Figure 3. These areas may be subject to additional movement during a seismic event. These washout/erosion scar areas should be mitigated with rip-rap rock slope protection. The eastern most washout is located along the northern creek bank and is outside of the current proposed development. In our opinion, the eastern most washout is not considered a hazard to the currently proposed development. This eastern washout should be further evaluated when development plans for the northeastern parcel is designed.

Liquefaction

Soil liquefaction is a phenomenon in which loose and saturated cohesionless soils are subject to a temporary, but essentially total loss of shear strength, due to pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. Soils typically found most susceptible to liquefaction are saturated and loose, fine to medium grained sand having a uniform particle range and less than 35% fines passing the No. 200 sieve, and a corrected standard penetration blow count $(N_1)_{60}$ less than 30. According to Special Publication 117A by the California Geological Survey, the assessment of hazards associated with potential liquefaction of soil deposits at a site must consider translational site instability (i.e. lateral spreading, etc.) and more localized hazards such as bearing failure and settlement. The acceptable factor of safety against liquefaction is recommended in SP117 to be 1.3 or greater.

Based on our site exploration and laboratory test data, the soil profile within the upper 25 feet was found to consist of stiff to very stiff fine-grained cohesive clays, and dense to very dense silty and clayey sands with no groundwater. Materials deeper than the 25 feet depth explored are expected to be even denser and harder. Due to the dense nature of the sands and lack of groundwater, it is our opinion that liquefaction potential at the site is considered very low.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical point of view, the proposed Industrial/Commercial project and associated parking lot, roadway and creek crossing improvements are considered to be feasible for construction on the subject site provided the recommendations presented in this report are incorporated into the project plans and specifications.

All grading and foundation plans for the development must be reviewed by the Geotechnical Engineer prior to contract bidding or submittal to governmental agencies to ensure that the geotechnical recommendations contained herein are properly incorporated and utilized in design.

KC ENGINEERING CO. should be notified at least two working days prior to site clearing, grading, and/or foundation operations on the property. This will give the Soil Engineer ample time to discuss the problems that may be encountered in the field and coordinate the work with the contractor.

Field observation and testing during the grading and/or foundation operations must be provided by representatives of ***KC ENGINEERING CO.*** to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to the grading and/or foundation operations performed without the full knowledge and under the direct observation of the Soil Engineer will render the recommendations of this report invalid.

Geotechnical Considerations

The primary geotechnical considerations for the industrial/commercial project are the presence of near surface un-documented fills, loose surficial soils, highly expansive soils, and the potential for scour and erosion such as the creek washout/erosion scars along the banks of Gibson Canyon Creek. With respect to the un-documented fills, we recommend that the undocumented fills and stockpiled materials be over-excavated and debris removed. In general, the stockpiled and fill materials, as shown on Figure 3, were found to be relatively clean of debris and may be used on the project.

With respect to the loose surficial native soils, the upper 2 feet of the native soil materials across the property were found to be relatively loose from disking and soft to firm and porous. Therefore, we recommend that these native materials will need to be over-excavated, processed and compacted to minimize the potential for settlement due to structure and pavement improvements.

With respect to the highly expansive soils, the site soils are prone to heave and shrink movements with changes in moisture content and, consequently, must be carefully considered in the design of grading, foundations, and drainage. Under the building pads and surrounding/adjacent concrete flatwork, such as loading docks and entry flatwork, we recommend that the existing materials be lime treated to mitigate the expansive nature of the materials, as well as to provide a structural fill pad. Specific recommendations are presented in the "Grading" section of this report. The recommendations provided in the following sections will minimize the detrimental effects of expansive soil movement. Also, we are providing alternative recommendations should the roadway subgrade be lime treated to provide a more competent subgrade prior to aggregate base placement.

Assuming the proposed building pad areas will be processed, compacted and lime treated as recommended herein, it is the opinion of **KC ENGINEERING COMPANY** that the proposed industrial/warehouse and future commercial buildings structure be supported on a well-reinforced and inter-connected spread footing foundation system with a thickened floor slab. Grading, foundation design, drainage, and slab-on-grade recommendations are presented herein.

With respect to the potential for scour and erosion, such as the washout/erosion scars within Gibson Canyon Creek, we recommend that the washout areas near the east and west roadway crossings be mitigated by removal of loose materials followed by placement of rip-rap rock slope protection (RSP) per the Caltrans manual "California Bank and Shore Rock Slope Protection Design". Due to the potential for future scour and creek bank erosion, we recommend that the up and downstream sides of the box-culvert creek crossings be armored with RSP for a distance of 15 to 20 feet. In addition, we expect 1 to 3 feet of loose creek bed soils in the area of the double box-culvert road crossings. These materials will need to be over-excavated and replaced with compacted aggregate base or a lean mix cement slurry. Specific recommendations are provided herein.

<u>Demolition</u>

Prior to building pad grading, demolition of the existing concrete pavements, underground pipelines, and any building foundations must be performed under the proposed pavements and building footprint, plus a 5 foot lateral over-build. Demolition should include the complete removal of all surface and subsurface structures. Where any of the following are encountered: concrete, storm drain systems, foundations, asphalt, buried pipelines, tanks, etc.; these should be removed with the exception of items specified by the owner for salvage. In addition, all underground structures must be located on the grading plans so that proper removal may be carried out. It is vital that ***KC ENGINEERING CO***., intermittently observe the demolition operations and be notified in ample time to ensure that subsurface structures are not covered.

Excavations made by the removal of any structure should be left open by the demolition contractor for backfill in accordance with the requirements for engineered fill. The removal of any underground structures or utility pipelines should be done under the observation of the Soil Engineer to assure adequacy of the removal and that subsoils are left in proper condition for placement of engineered fills. Any soil exposed by the demolition operations, which are deemed soft or unsuitable by the Soil Engineer, shall be excavated as uncompacted fill soil and be removed as required by the Soil Engineer during grading. The demolition operation should be approved by the Soil Engineer prior to commencing building pad grading operations. Any resulting excavations should be properly backfilled with engineered fill under the observation of the Soil Engineer. Should the location of any localized excavation be found to underlie any structure, backfill should be compacted to a minimum relative compaction of 95% or the excavation widened to extend 5 feet beyond the footprint of the structure and backfilled to the specifications for engineered fill as recommended in the “Grading” section below.

Temporary Excavations

Applicable safety standards require that excavations in excess of 4 feet must be properly shored or that the walls of the excavation be sloped back to provide safety for installation of pipelines and structure construction. We expect that the excavations will be open cut with no shoring. All temporary excavations used in the construction of underground pipelines and the box-culvert crossings should be designed, planned, constructed and maintained by the Contractor and should conform to all state and federal safety regulations and requirements.

Based on the subsurface soils encountered during our field exploration, it is our opinion that the soils may be considered to be Type B per OSHA Standards. Based on our findings, it is our opinion that the proposed temporary excavations will perform adequately at a maximum 1H:1V slope. This is provided that the following comments and recommendations are included in the Contractor work plan:

1) The excavations must be monitored by competent personnel for stability and safety. We also recommend that the upper embankment areas be walked daily and visually inspected for signs of lateral displacement and/or tension cracks. Should any cracking or displacement be observed, the excavation must be backfilled immediately, or the upper slope laid back and flattened and our office notified for further recommendations.
2) The cut slope and adjacent surface should be covered with visqueen during rain events and to minimize rainfall infiltration.
3) Dewatering must be provided to maintain the level at 3 feet below the base of the excavation. We should note that the boring logs indicate sandy deposits at both sites which may generate the most amount of seepage. Our Geotechnical Engineer should be contacted to evaluate the excavations and provide supplemental recommendations as necessary.
4) Stockpiling of spoils and equipment should be setback 10 feet minimum from the top of cut as shown on the attached cross section.
5) Excavation stability and dewatering is the responsibility of the Contractor.

Grading

As mitigation to minimize differential heave and shrink movements from the highly expansive clays and to minimize expected total and differential settlement across the building footprints, we recommend that the upper 3 feet of the industrial and commercial building pads be processed and compacted as well compacted structural fill by lime treatment. We recommend that the structural fill pad be a minimum of 3 feet thick, extend 5 feet beyond the building footprint, and consist of the existing on-site materials treated with high-calcium quicklime as described below. Alternatively, select import may be used to construct the structural fill pads. The upper 12 inches of the driveways and parking lots may also be lime treated as described herein and in the "Pavement" section below.

Grading activities may be performed during the rainy season, however, achieving proper compaction may be difficult due to excessive moisture; and delays may occur. Grading performed during the dry months will minimize the occurrence of the above problems. When project grading plans become available for our review, supplemental grading recommendations may be required.

The surface of the site in areas to be graded should be stripped to remove all existing vegetation and/or other deleterious materials. It is estimated that stripping depths of 1 to 2 inches may be necessary, unless the site has been disked. Trees and all roots should be removed. Any material that is deemed to be topsoil and requiring stripping may not be used as engineered fill but may be stockpiled and used later for landscaping purposes.

After demolition, stripping and over-excavating any undocumented fills and stockpiles, it is recommended that the upper 12 inches of the native existing grades be over-excavated 1 foot, followed by scarifying the exposed bottom 12 inches, and uniformly mixing and compacting to a minimum degree of relative compaction of 90% at least 3 percent above optimum moisture content as determined by ASTM D1557 Laboratory Test Procedure. After processing the lower 12 inches and compacting the over-excavated bottom, the site may be brought to the desired finished grades by placing engineered fill in lifts of 8 inches in un-compacted thickness and compacting to a relative compaction of 90% at 3 percent over optimum in accordance with the aforementioned test procedure. All soils encountered during our investigation are suitable for use as engineered fill when placed and compacted at the recommended moisture content.

Any loose or soft soil materials must be excavated to undisturbed native ground. Excavated soil materials may be used as engineered fill with the approval of the Soil Engineer provided they do not contain debris or excessive organics. As noted above, we expect 1 to 3 feet of loose creek bed soils in the area of the double box-culvert road crossings and at the earthen SID canal. At the box culvert creek crossings, the loose and soft materials will need to be over-excavated and replaced with compacted aggregate base to 92% or a lean mix cement slurry or CLSM having a minimum compressive strength of 500 psi. We recommend a 2 feet minimum over-excavation depth. This will also serve as the foundation and working platform for the box culverts.

Under the building pads and surrounding/adjacent concrete flatwork, such as loading docks and entry flatwork, we recommend that the existing materials be lime treated to mitigate the expansive nature of the materials, as well as to provide a structural fill pad. It is recommended that the upper 3 feet of the building pads and adjacent concrete flatwork areas comprise the on-site materials modified with high calcium quicklime meeting ASTM C977. It is noted that the structural fill should extend at least 5 feet beyond the building footprint and to the edge of surrounding flatwork, whichever is greater. The building pad and adjacent flatwork area to be treated should be graded to a depth of 1.5 feet below design pad grade. The bottom 1.5 feet of the excavation can then be processed and compacted in- place with lime, followed by placing the upper 1.5 feet and treating the upper lift.

The lime treatment should consist of a 5% mixture by dry weight. Based on a unit weight of 120 p.c.f., a spread rate of 9.0 p.s.f. each for the 18-inch mixing depths. In the pavement areas, the upper 12 inches of subgrade should also be lime treated with a minimum spread rate of 6.0 p.s.f. The lime treated soils should be compacted to at least 95% relative compaction of the maximum wet density per ASTM D1557 at a moisture content at least 4% above optimum. The lime treatment must be performed by a qualified soil stabilization contractor in general conformance with Caltrans Standard Specification Section 24. The product specification and quality control test results must be provided to us by the contractor for review and acceptance prior to the treatment operations. The lime should be spread and mixed with equipment capable of providing relatively uniform conditions. The lime treated sections must be mixed at least twice prior to compaction which must be performed within 24 hours after final mixing. After compaction, it is important to moist cure the lime treated soils until placement of the subsequent slab subbase materials (i.e. do not let pad dry out and desiccate).

Should select import material be used to establish the upper 36 inches of the structural fill pad or be required for general fill, the import material should be approved by the Soil Engineer before it is brought to the site. Where select import soil is used within the upper 36 inches of the pad, it should meet the following requirements:

a. Have an R-Value of not less than 25;
b. Have a Plasticity Index not higher than 12;
c. Not more than 15% passing the No. 200 sieve;
d. No rocks larger than 3 inches in maximum size;

The fill materials shall be placed in uniform lifts of not more than 8 to 12 inches in uncompacted thickness depending on size and weight of equipment used. Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to obtain uniformity of material in each layer. Before compaction begins, the fill shall be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry.

Compaction shall be by footed rollers or other types of acceptable compacting rollers. Rollers shall be of such design that they will be able to compact the fill to the specified density. Rolling shall be accomplished while the fill material is within the specified moisture content range. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to ensure that the required density has been obtained. No ponding or jetting shall be permitted.

The standard test used to define maximum densities and optimum moisture content of all compaction work shall be the Laboratory Test procedure ASTM D1557 and field tests shall be expressed as a relative compaction in terms of the maximum dry density and optimum moisture content obtained in the laboratory by the foregoing standard procedure. Field density and moisture tests shall be made in each compacted layer by the Soil Engineer in accordance with ASTM D6938, respectively. When footed rollers are used for compaction, the density and moisture tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements for any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

Slopes

Should any fill slope grading be required, we recommend that the toe of fill slopes be properly keyed into competent material before filling. Prior to placement of fill slopes and after stripping of vegetation, a toe of slope keyway must be constructed into competent soil materials prior to placement of engineered fill as required by the 2016 and 2019 CBC Appendix J. A toe key excavation should be placed at the base of all such fills. This key should be a minimum of 12 feet in width, cut into competent non-yielding material a minimum of 2 vertical feet, and sloped into the hillside at a gradient of no less than 5%. Subsequent keyed benches should be excavated as the fill progresses upslope. Subdrainage in keyways surrounding structures will also be required. A typical fill slope, keyway and subdrain detail is presented in the Appendix.

Unsupported cut and fill slopes should not be steeper than 2H:1V (horizontal to vertical). Fill slopes must be compacted as the filling operation progresses upslope, and include over-constructing the fill slope face and cutting back the looser surface soils to a firm and adequately compacted designed slope grade. Track-walking of slope surfaces does not provide adequate soil densities and is an unacceptable method of slope compaction.

Cut and fill slopes in soil may experience severe erosion when grading is halted during rainy weather. Before work is stopped, a positive gradient away from the slopes must be established to carry the surface runoff water away from the slopes to areas where erosion and sediment can be controlled. After the completion of the slope grading, erosion protection and hydro-seeding must be provided on all soil surfaces. Slope planting, preferably with deep-rooted native plants requiring minimal irrigation, should be completed on all exposed surfaces of cut and fill slopes. Graded slopes should not be left exposed through a winter season without the completion of erosion control measures and slope planting.

Rock Slope Protection (RSP)

As discussed above, we recommend that the creek bank washout/erosion scars located in the area of the western and eastern driveway creek crossings be cleaned of loose materials and filled with rip-rap RSP or compacted engineered fill. The eastern most washout is located along the northern creek bank and is outside of the current proposed development. In our opinion, the eastern most washout is not considered a hazard to the currently proposed development. This eastern washout should be further evaluated when development plans for the northeastern parcel is designed.

At the two washouts in the area of the two driveway creek crossings, toe keyway of 10 feet wide should be excavated at the re-established creek toe prior to placement of RSP. In addition, the finished slopes around the backfilled box-culvert structures on the up and downstream sides be armored with rip-rap RSP per Caltrans specifications and guidelines to prevent erosion and/or landsliding. The RSP should extend a minimum of 20 feet laterally past the structures and be placed from toe to top of embankment slope. RSP material and installation should be in accordance with 2015 Caltrans Standard Specification 72, and the California Bank and Shore Rock Slope Protection Design manual. Filter fabric material and installation should be per Caltrans Section 72-2.02C, Class 8 and Section 96-1.02I with a 3 feet wide lap at seams. In our opinion, the base and toe material should be ¼ ton RSP per Caltrans Specification 72-2.202B. Installation of RSP should be per Section 72, Method "A" placement. RSP sizing should be confirmed by the Civil Engineer based on maximum creek flow velocities.

Surface & Subsurface Drainage

A very important factor affecting the performance of structures and pavements is the proper design, implementation, and maintenance of surface drainage, as well as maintaining uniform moisture conditions around the structures. Ponded water will cause swelling and/or loss of soil strength and may also seep under structures. Should surface water be allowed to seep under the structures, differential foundation movement resulting in structural damage and/or standing water under the slab will occur. This may cause dampness to the floor which may result in mildew, staining, and/or warping of floor coverings. To minimize the potential for the above problems, dampproofing and waterproofing should be provided as required by Section 1805 of the 2016 & 2019 CBC. In addition, the following surface drainage measures are recommended and must be maintained by the property owner in perpetuity:

a) Positive building pad slopes and surface drainage must be provided by the project Civil Engineer to remove all storm water from the pad and to prevent storm and/or irrigation water from ponding adjacent to the structure foundations. The finished pad grade around the structures should be compacted and sloped 5% away from the exterior foundations and as required in Section 1804.4 of the 2016 or 2019 CBC and directed to appropriate drainage inlets. Surface swales should be sloped a minimum of 2% as required by the CBC.

b) Enclosed or trapped planter areas adjacent to the structure foundations should be avoided if possible. Where enclosed planter areas are constructed, these areas must be provided with adequate measures to drain surface water (irrigation and rainfall) away from the foundation. Positive surface gradients and/or controlled drainage area inlets should be provided. Care should be taken to adequately slope surface grades away from the structure foundations and into area inlets. Drainage area inlets should be piped to a suitable discharge facility.

c) Adequate measures for storm water discharge from the roof gutter downspouts must be provided by the project Civil Engineer and maintained by the property owners at all times, such that no water is allowed to pond next to the structure. Closed pipe discharge lines should be connected to downspouts and discharged into a suitable drainage facility. It is important not to allow concentrated discharge on the surface of any slope so as to prevent erosion.

d) Site drainage should be designed by the project Civil Engineer. Civil engineering, hydraulic engineering, and surveying expertise is necessary to design proper surface drainage to assure that the flow of water is directed away from the foundations.

e) Over-irrigation of plants is a common source of water migrating beneath a structure. Consequently, the amount of irrigation should not be any more than the amount necessary to support growth of the plants. Foliage requiring little irrigation (drip system) is recommended for the areas immediately adjacent to the structures.

f) Landscape mounds or concrete flatwork should not be constructed to block or obstruct the surface drainage paths. The Landscape Architect or other landscaper should be made aware of these landscaping recommendations and should implement them as designed. The surface drainage facilities should be constructed by the contractor as designed by the Civil Engineer.

With respect to any proposed bio-retention swales or basins, we anticipate that bio-swales will be located relatively close to the proposed structures. We recommend a minimum separation of 10 horizontal feet. The bottom of the swales should be sloped away from the structure foundation. In addition, we recommend that a subsurface drain be provided below the select treatment soils at the low side of the swale/basin. The subdrain should be connected to the nearest storm drain catch basin. A 4 inch SDR35 perforated pipe surrounded by Caltrans Class 2 Permeable Material should be provided to discharge collected water into the nearest catch basin. An impermeable liner may also be required in the bottom of the swales. Additional details can be provided when plans are available.

Building Foundations

Provided that the upper 3 feet of the building pad soils are lime treated or constructed with select import as recommended in the “Grading” section above to minimize the effects of the estimated total and differential settlements noted above, the proposed structures may be supported by utilizing a deepened, well-reinforced and inter-connected spread footing foundation system with a thickened slab floor.

A continuous spread footing should be placed around the perimeter of the structure and be a minimum of 18 inches wide. All interior and exterior column footings should be interconnected to the perimeter with reinforced concrete tie-beams or by continuous slab floor reinforcing extending through the interior column footings. Isolated footings should not be utilized unless connected with reinforced tie-beams or through reinforced slab connections. The continuous and pad/column footings should extend to a minimum depth of 24 inches below the interior slab subgrade soil elevation. The tie beams where used should extend to a minimum depth of 18 inches below the interior soil pad grade. The recommended design allowable bearing pressure for footings is 2,500 p.s.f. due to dead plus live loads. The allowable pressure may be increased by 1/3 due to all transient loads which include wind and seismic. All foundations must be adequately reinforced to provide structural continuity and resist the anticipated loads as determined by the project Structural Engineer. The final footing design and reinforcement should be determined by the project Structural Engineer. However, continuous footings and tie-beams are recommended to be reinforced with a minimum of four No. 6 bars, two at the top and two near the bottom of the footing. Additional reinforcement will be as required by the structural engineer and in accordance with structural building code requirements. Foundations designed in accordance with the above criteria are expected to experience a total settlement of less than 1 of an inch with less than ½ inch of an inch in 50 feet.

To accommodate lateral building loads, the passive resistance of the foundation soil can be utilized. The passive soil pressures can be assumed to act against the front face of the footing below a depth of 1 foot below the ground surface. It is recommended that a passive pressure equivalent to that of a fluid weighing 250 p.c.f. be used. For design purposes, an allowable friction coefficient of 0.32 can be assumed at the base of the spread footings. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since the mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

Slab-on-Grade Construction

Interior floor slabs, and exterior concrete slabs, including loading docks, sidewalks, driveways, non-structural detached general flatwork will likely experience some cracking due to finishing, curing methods, drying shrinkage, as well as moisture variations and related soil movements within the underlying clay soils. We should note that City or County maintained curbs, gutters, sidewalks and driveway aprons should be designed and constructed per the City of Vacaville or Solano County Standards, Specifications and Plans. To reduce the potential cracking of the slabs-on-grade, the following recommendations are made:

a) It is important to moist cure the lime treated soils on the building pad until placement of the subsequent materials. All areas to receive slabs should be thoroughly wetted and soaked to seal any desiccation or shrinkage cracks prior to placing concrete. This work should be done under the observation of the Soil Engineer.

b) Interior building slabs should be underlain by a minimum of 4 inches of Caltrans Class II Aggregate Base placed and compacted to a minimum of 90% between the finished subgrade soils and the slabs to serve as subbase support. Three-quarter inch crushed rock may be used under pedestrian flatwork.

c) Interior warehouse slabs and loading dock/pavement areas should be a minimum of 6 inches thick and reinforced with a minimum of No. 4 rebar spaced 16 inches center to center, each way. Exterior pedestrian flatwork and general slabs should be a minimum of 5 inches thick and reinforced with either flat stock welded wire reinforcement or No. 4 rebar spaced at 18 inches on center. The actual slab thickness and reinforcement should be determined by the project Structural Engineer in accordance with the structural requirements and the anticipated loading conditions. The reinforcement shall be placed in the center of the slab unless otherwise designated by the design engineer.

d) A vapor retarder membrane should be installed between the prepared building pad aggregate base and the interior slabs to minimize moisture condensation under the floor coverings and/or upward vapor transmission. The vapor barrier membrane should be a minimum 15-mil extruded polyolefin plastic that complies with ASTM E1745 Class A and have a permeance of less than 0.01 perms per ASTM E96 or ASTM F1249. It is noted that polyethylene films (visqueen) do not meet these specifications. The vapor barrier must be adequately lapped and taped/sealed at penetrations and seems in accordance with ASTM E1643 and the manufacturer's specifications. The vapor retarder must be placed continuously across the slab area.

e) Water vapor migrating to the surface of the concrete can adversely affect floor covering adhesives. Provisions should be provided in the concrete mix design to minimize moisture emissions. This should include the selection of a water-cement ratio which inhibits water permeation (0.45 max) and/or the addition of suitable admixtures to limit water transmission. We also recommend the use of Type I/II cement for additional corrosion resistance.

f) Slabs for driveways, and exterior flatwork should be placed structurally independent of the foundations. Driveway slab recommendations are presented in the "Pavement" section of the report. A 30-pound felt strip, expansion joint material, or other positive separator should be provided around the edge of all floating slabs to prevent bonding to the foundation. However, rebar doweling is recommended to minimize vertical movements between exterior slabs and building foundations. Doweling details should be determined by the Structural Engineer.

g) To minimize moisture infiltration under exterior slabs and to add edge rigidity, we recommend that slabs be thickened at the edges to extend below the aggregate base layer to the soil subgrade for a minimum width of 6 inches.

h) Slabs should be provided with crack control saw cut joints or tool joints to allow for expansion and contraction of the concrete. In general, contraction joints should be spaced no more than 20 times the slab thickness in each direction. The layout of the joints should be determined by the project Structural Engineer and/or Architect.

i) We recommend that appropriate provisions be provided by the Structural Engineer and Contractor to minimize slab cracking, such as curing measures and/or admixtures to minimize concrete drying-shrinkage and curling. American Concrete Institute methods and guidelines of curing, such as wet curing or membrane curing, are recommended to minimize drying shrinkage cracking.

Retaining Walls & Box Culverts

Any retaining walls, including loading docks, elevator pits and the box culvert wing walls, that are to be incorporated into the project should be designed to resist lateral pressures exerted from a media having an equivalent fluid weight as follows:

Gradient of Back Slope	Equivalent Fluid Weight (p.c.f.)			Coefficient of Friction
	Unrestrained Condition (Active)	Restrained Condition (At Rest)	Passive Resistance	
Horizontal	65 (native soils) 40 (select import)	75 (native soils) 55 (select import	250	0.32

It should be noted that the effects of any vehicle or other surcharge or compaction loads behind the walls must be accounted for in the design of the walls. In addition, an earthquake load of $13H^2$ applied at 1/3H where H = wall height, from the bottom of the wall is applicable. Restrained conditions should be used where framing or other structural members rests on top or is connected to the top of walls.

We expect the supporting materials at the box culverts and wing walls to be over-excavated and replaced with compacted aggregate base or a lean slurry mix as recommended in the Grading section above. In this case, an allowable bearing capacity of 3,000 psf may be utilized in the design. A one-third increase may be used for transient loads.

The above criteria are based on fully drained conditions. Groundwater is not expected for the elevator pits and loading docks, however we do recommend water-proofing for elevator pits. In order to achieve fully-drained conditions, a gravel drainage filter blanket should be placed behind the wall. The gravel blanket should be a minimum of 12 inches thick and should extend to within 12 inches of the surface and capped with compacted soil. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket should consist of compacted engineered fill or blanket material. The drainage blanket material may consist of either granular crushed rock or drain pipe fully encapsulated in geotextile filter fabric (Mirafi 140N or equivalent) or Class II permeable material that meets CalTrans Specification, Section 68. A 4-inch diameter SDR35 perforated drain pipe should be installed in the bottom of the drainage blanket and should be underlain by 4 inches of filter type material. Piping with a minimum gradient of 2% shall be provided to discharge water that collects behind the walls to an adequately controlled discharge system away from the structure foundations or to sump pit. Weep holes may alternatively be utilized.

Wall backfill should be placed in thin lifts, moisture conditioned and compacted per the Grading recommendations above.

Pavement Areas

The driveways and parking areas will be paved with either asphalt concrete (AC) or Portland cement concrete (PCC) surfaces. Recommendations for these pavement surfaces are presented below. We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, as well as engineered fill and utility trench backfill within the limits of pavements. Pavements will typically have poor performance and shorter life where water is allowed to migrate into the aggregate base and subgrade soils. The main sources of water into pavement materials are landscape planters constructed within or adjacent to pavement areas. Where this is planned, it is suggested to extend the curbs into the soil subgrade at least 2 inches. The construction of all pavements should conform to the requirements set forth by the latest Standard Specifications of the Department of Transportation of the State of California (Caltrans) and/or the City of Vacaville.

R-Value: Three composite bulk samples were obtained of the near surface soils within the planned parking lot and driveways that is relatively representative of the anticipated subgrade soils. The samples were tested in accordance with the California Test Method 301 to determine the R-Value for the site soils. R-Values of 6, 8 and 20 were determined for the samples as shown in the Appendix. Due to the expansive clays on site, we recommend an R-value of 6 be used for the design of pavement sections. However, we understand that new pavement areas may be constructed during the winter months and will be lime treated. Therefore, an alternate lime treated R-value of 35 is also considered herein.

Preparation of Subgrade: After underground utilities have been placed in the areas to receive pavement and removal of excess material has been completed, the upper 12 inches of the subgrade soil shall be scarified, moisture conditioned and compacted to a minimum relative compaction of 95% at a moisture content at 3% or more above optimum in accordance with the grading recommendations specified in this report. As recommended in the "Grading" section above, the upper 12 inches of the subgrade may alternatively be lime treated. Prior to placement of aggregate baserock, it is recommended that the subgrade be proof rolled and observed for deflection by the Soils Engineer. Should deflection and/or pumping conditions be encountered, stabilization recommendations will be provided based on field conditions. Geotextile fabric is required to be placed over the City street subgrades per section CS 7-03 in the City of Vacaville Standards.

Aggregate Base: All aggregate base material placed subsequently should also be compacted to a minimum relative compaction of 95% based on the ASTM Test Procedure D1557. Aggregate base should meet the minimum requirements of Caltrans ¾" Class 2 per Section 26 and be crushed and angular. The recommended aggregate base thicknesses for asphalt concrete pavements are noted in the table below. The minimum aggregate base thickness for Portland cement concrete PCC roadway pavements is 6 compacted inches.

Asphalt Concrete: Asphalt concrete shall conform with Section 39 of Caltrans Standard Specifications and shall be per the City of Vacaville Standards. Based on an R-Value of 6, and traffic indices typical for industrial/commercial developments, the recommended pavement sections for asphalt concrete surfaces are summarized in the table below. Should the parking lot or drive lanes soils be lime treated, we are providing an alternate section based on a minimum R-value of 35. The appropriate traffic index (TI) and any minimum pavement sections should be determined by the Civil Engineer in conformance with the City of Vacaville.

Traffic Condition	**Traffic Index (TI)**	**Asphalt Concrete (inches)**	**Class II Aggregate Base[1] (inches)**
Auto Parking Stalls	4.5	3.0 3.0	8.0 4.0*
Truck Parking and Drive Lanes	7.0	4.0 4.0	15.0 8.0*
Collector	8.0	4.5	18.0
Arterials	10.0	6.0	23.0

NOTES:
(1) Minimum R-Value = 78
(2) All layers in compacted thickness to CalTrans Standard Specifications.
* Lime Treated Subgrade (R-Value = 35 min)

Portland Cement Concrete: Where PCC pavement areas are utilized, such as for drive isles and truck areas or at trash enclosures, the concrete should be poured on the compacted aggregate base layer described above of 6 inches. The concrete section should be designed by the project Civil or Structural Engineer per Chapter 620 of the Highway Design Manual or City Standards. We recommend a minimum of 6 inches thick PCC reinforced with a minimum of No. 4 rebar spaced at 16 inches on center, each way, underlain by 6 inches of compacted Class 2 aggregate base. Additional reinforcement may be required by the Structural Engineer. Pavement joints shall be per the HDM and City Standards.

General Construction Requirements

Utility trenches extending underneath all traffic areas must be backfilled with native or import soil materials and compacted to relative compaction of 90% to within 12 inches of the subgrade. The upper 12 inches should be compacted to 95% relative compaction in accordance with Laboratory Test Procedure ASTM D1557. Backfilling and compaction of these trenches must also meet the requirements set forth by the City of Vacaville, Department of Public Works.

Applicable safety standards require that trenches in excess of 5 feet must be properly shored or that the walls of the trench slope back to provide safety for installation of lines. If trench wall sloping is performed, the inclination should vary with the soil type and applicable OSHA Safety Standards. The soils at the site are considered to be Type B, except where groundwater is encountered Type C should be used.

With respect to state-of-the-art construction or local requirements, utility lines are generally bedded with granular materials. These materials can convey surface or subsurface water beneath the structures. It is, therefore, recommended that all utility trenches which possess the potential to transport water be sealed with a compacted impervious cohesive soil material or lean concrete where the trench enters/exits the building perimeter. This impervious seal should extend a minimum of 2 feet away from the building perimeter.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his representative to notify ***KC ENGINEERING CO.***, in writing, a minimum of two working days before any clearing, grading, or foundation excavation operations can commence at the site.

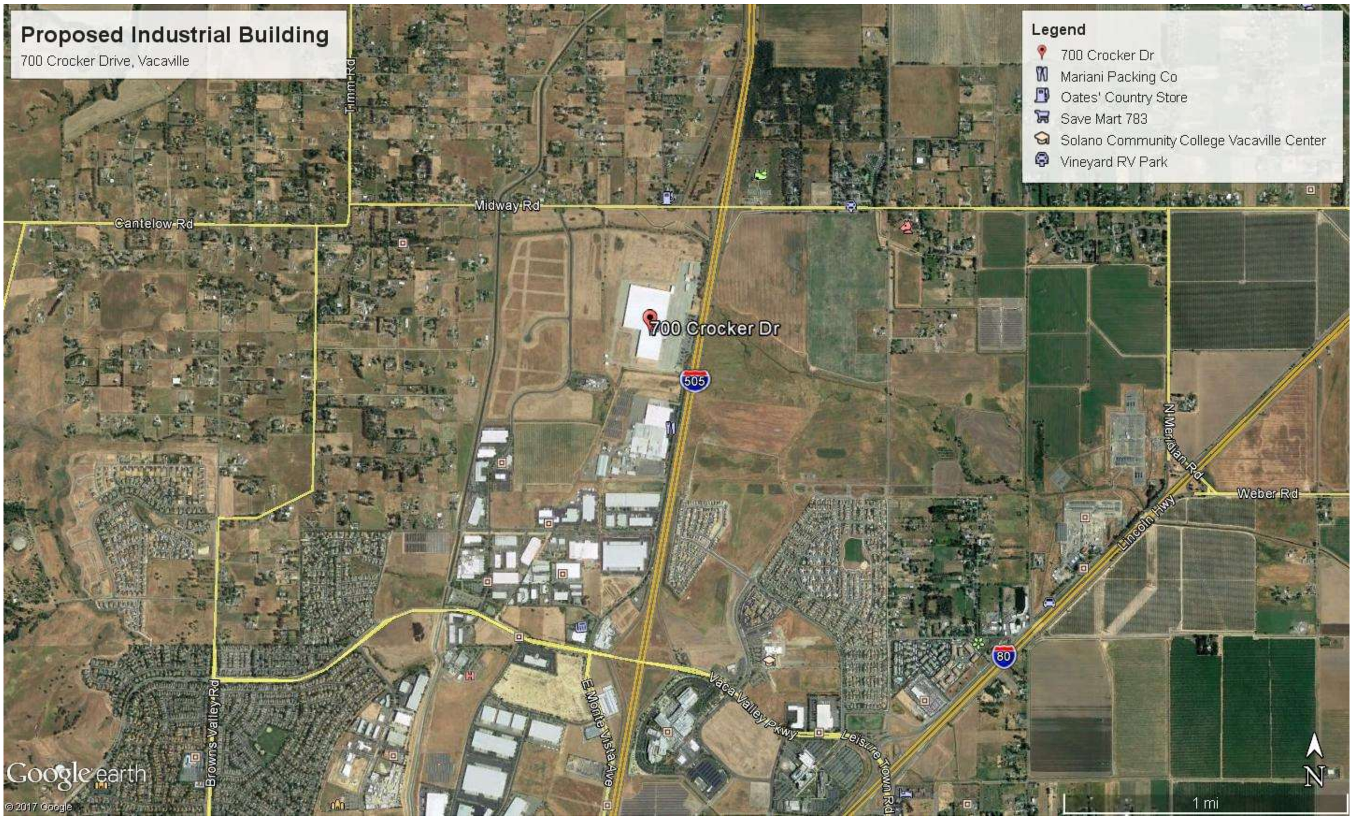
2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings and from a reconnaissance of the site. Should any variations or undesirable conditions be encountered during the development of the site, ***KC ENGINEERING CO.***, will provide supplemental recommendations as dictated by the field conditions.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

4. At the present date, the findings of this report are valid for the property investigated. With the passage of time, significant changes in the conditions of a property can occur due to natural processes or works of man on this or adjacent properties. In addition, legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may render this report invalid, wholly or partially. Therefore, this report should not be considered valid after a period of two (2) years without our review, nor should it be used, or is it applicable, for any properties other than those investigated.

5. Not withstanding, all the foregoing applicable codes must be adhered to at all times.

APPENDIX



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Project No. VV4376
Proposed Industrial/Commercial Project
700 Crocker Drive, Vacaville, CA
Figure 1 – AERIAL VICINITY MAP

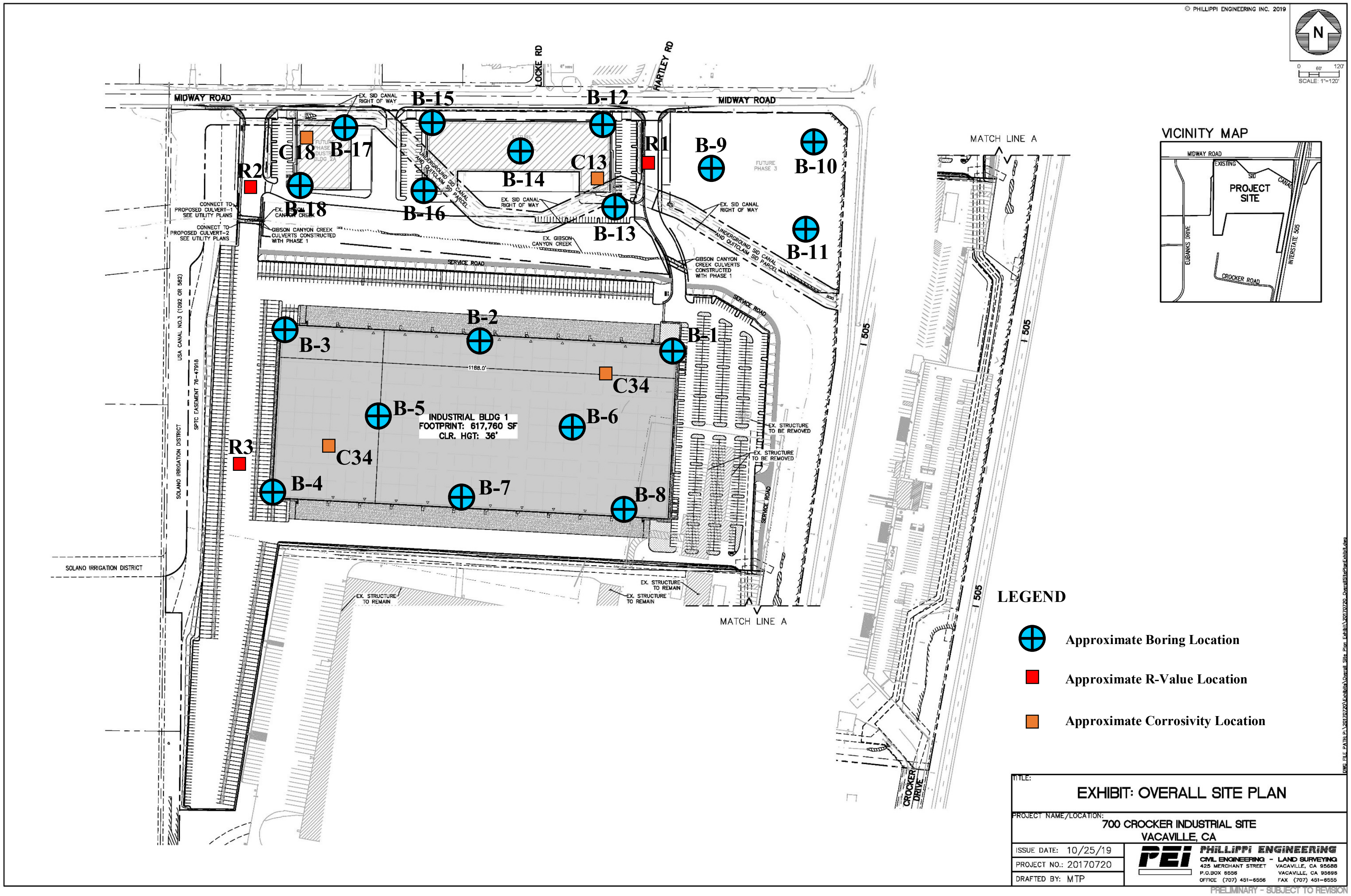

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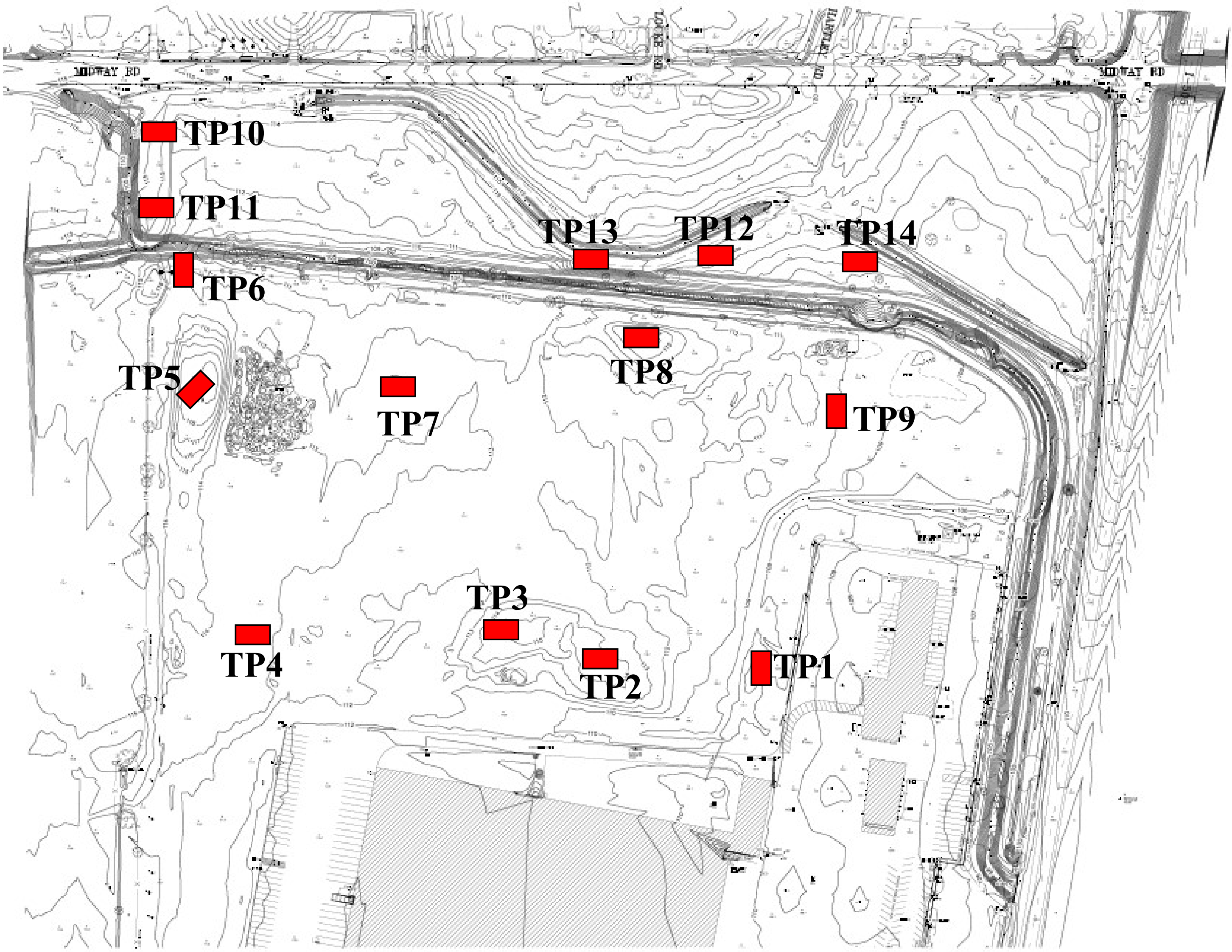
Figure 2 – SITE PLAN



LEGEND

Approximate Test Pit Location

Approximate Areas of Undocumented Fills & Stockpiles

Approximate Creek Bank Washout Areas

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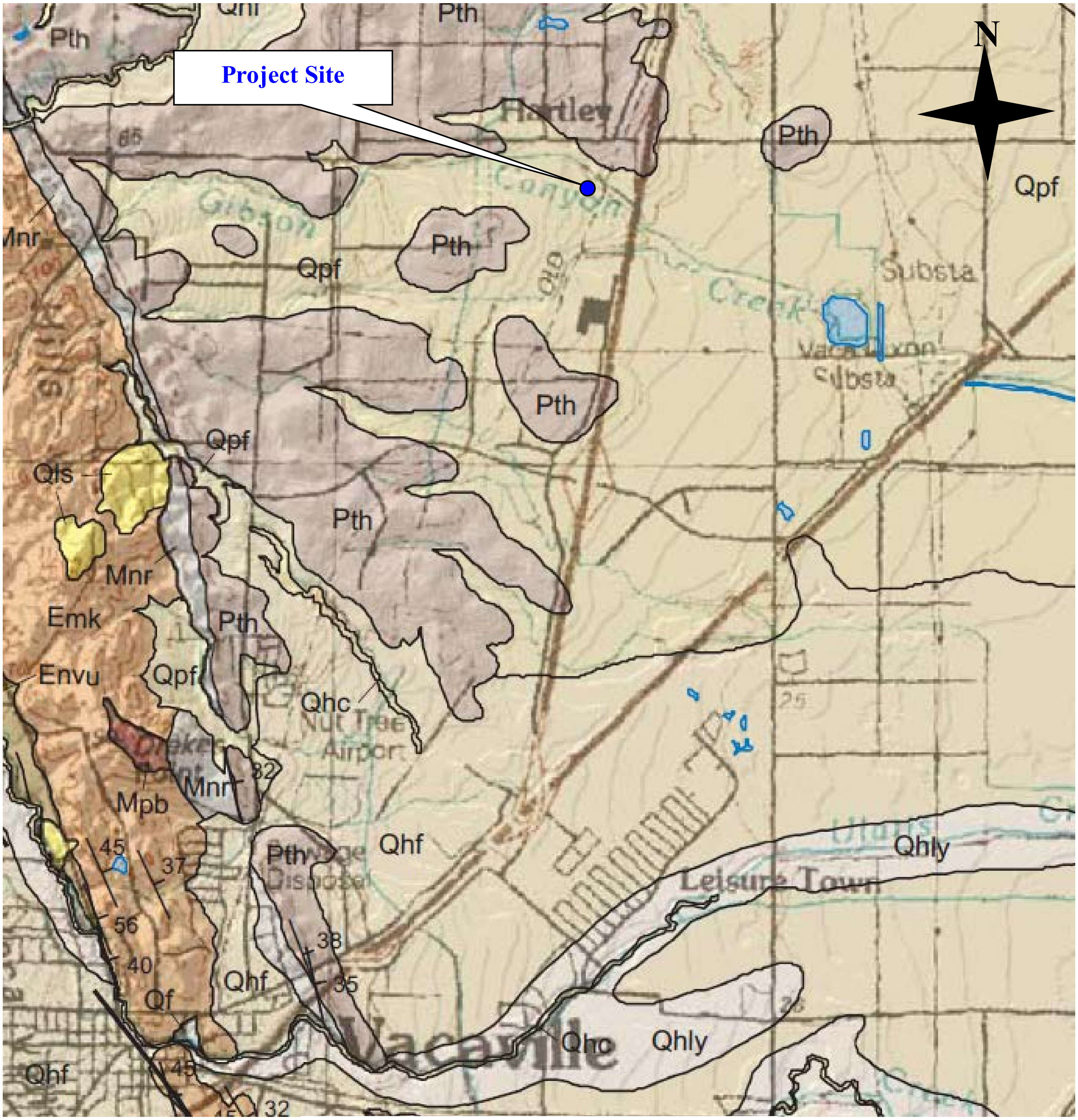
Figure 3 – UNDOCUMENTED FILL & CREEK WASHOUT EXHIBIT



PRELIMINARY GEOLOGIC MAP OF
THE LODI 30' x 60' QUADRANGLE, CALIFORNIA

Compiled by
Timothy E. Dawson
2009

Qpf	Alluvial fan deposits
Pth	Tehama Formation

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Project No. VV4376
Proposed Industrial/Commercial Project
700 Crocker Drive, Vacaville, CA
Figure 4 – GEOLOGIC MAP

LOG OF TEST BORING
BORING NO.: 1

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Olive Brown Clayey SILT with Some Sand & Gravel; top 12" dry & loose then moist & very stiff. (FILL)	ML					
		1-1					23	103.9	6.5	>4.5	%<200=69%
					Brown Sandy GRAVEL; dry, dense. (FILL?)	GP					
	5										
					Yellowish Brown Silty SAND; moist, very dense. (NATIVE)	SM					
		1-2					50-4"	93.7	18.4		
	10				Brown Clayey SAND, moist; very dense.	SC					
		1-3					59				
	15										
					Mottled Gray & Brown Clayey SAND; moist, dense.	SC					
		1-4					42	103.9	23.0		
	20				Boring Terminated @ 20'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

KC ENGINEERING CO.

Figure 5

LOG OF TEST BORING
BORING NO.: 2

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Dark Brown Silty CLAY; moist, top 12" loose then firm.	CL/CH					
					Brown Clayey SILT with Sand; moist, firm to stiff.	ML					
		2-1					8	115.4	10.6	2.0	ϕ=23° c=447 psf
	5										
					Gravels @ 7 to 7.5'	GP					
					Mottled Gray & Brown Sandy CLAY; moist, hard.	CL					
		2-2					36	103.3	23.5	>4.5	
	10										
					Mottled Brown Clayey SAND; moist, very dense.	SC					
		2-3					55	113.1	16.0	>4.5	
	15				Boring Terminated @ 15'. No Groundwater Encountered.						
	20										
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 3

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ≚ :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Brown Sandy CLAY with Some Gravel; dry, loose (top 12") then very stiff. (FILL?)	CH					
		3-1					33	106.9	6.2	>4.5	LL=60 PI=41 UCC=5,829 psf
					Light Brown Sandy SILT; moist, hard.	ML					
	5	3-2					55	119.3	12.7	>4.5	
					Gravel	GP					
					Mottled Gray & Brown Fine Sandy CLAY; moist, very stiff.	CL					
	10	3-3					26	102.6	24.3	3.5	%<200=85%
	15				Brown Clayey SAND; moist, dense.	SC					
					Brown Silty SAND; moist, very dense.	SM					
	20	3-4					50-6"				
					Boring Terminated @ 20.5'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

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Figure 7

LOG OF TEST BORING

BORING NO.: 4

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION:
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Brown Fine Sandy CLAY; dry & loose (top 12"), then moist & stiff.	CL					
		4-1					11	113.1	15.7	1.25	LL=30 PI=16 UCC=1,906 psf
	5				Mottled Brown Clayey SAND; moist, very dense.	SC					
		4-2					50-5"				
	10										
		4-3			Mottled Gray & Yellowish Brown Sandy SILT; moist, hard.	ML	63	82.5	16.6	>4.5	
	15										
	20										
		4-4			Mottled Gray & Yellow Sandy CLAY; moist, hard.	SC	59	107.4	21.1		
	25				Boring Terminated @ 25'. No Groundwater Encountered.						

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 5

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Gray SILT with Gravel; dry & loose (top 12") then moist & stiff.	ML					
					Dark Brown Sandy CLAY; moist, stiff.	CL					
		5-1					9	101.5	16.4	2.0	LL=38 PI=23
	5				Mottled Brown Sandy CLAY with some Gravel; moist, medium dense.	CL					
		5-2					16	114.3	16.2	3.5	
	10										
					Mottled Brown Sandy CLAY; moist, medium dense.	CL					
		5-3					26	104.1	22.9	3.5	
	15				Mottled Gray & Yellowish Brown Sandy SILT; moist, hard.	ML					
		5-4					63			>4.5	
	20				Boring Terminated @ 20'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 6

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL :

PROJECT NO.: VV4376
DATE: 10/11/17
ELEVATION:
LOGGED BY: DVC
BORING DIAMETER: 4"
FINAL : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Brownish Gray Clayey SILT with Some Gravel, dry & loose (top 12"), then moist & stiff.	ML					
		6-1			Brown Sandy CLAY with Some Gravel, moist, medium dense.	CL	25	16.4	111.9		%<200=67%
	5										
		6-2			Brown Sandy CLAY with Fine Gravel, moist, hard.	CL	43	118.1	11.4		
	10				Pale Brown Gray Sandy SILT, moist, very hard.	ML					
		6-3									
	15				Boring Terminated @ 15'. No Groundwater Encountered.						
	20										
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING
BORING NO.: 7

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ≚ :

PROJECT NO.: VV4376
DATE: 10/12/17
ELEVATION: n/a
LOGGED BY: ALK
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				(Disked) Dark Brown Sandy CLAY; dry, loose.	CL					
					Light Brown Clayey SAND; moist, medium dense.	SC					
		7-1					20	117.7	12.1	4.0	%<200=39%
	5	7-2			As Above.		9	106.4	15.6	2.25	UCC=1,038 psf
					Light Brown CLAY with Sand; moist, stiff.	CL					
					Reddish Brown Sandy CLAY; moist to wet, hard.	CL					
	10	7-3					36	121.4	15.0	3.0	
					Mottled Orange & Gray Sandy CLAY; moist, hard.	CL					
	15	7-4					38	106.4	22.7		
	20	7-5			As Above.		39			>4.5	
					Boring Terminated @ 21.5'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING
BORING NO.: 8

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: Hillside Geotechnical Drilling, Inc.
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/12/17
ELEVATION: n/a
LOGGED BY: ALK
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				(Disked w/ Concrete Debris) Dark Brown Sandy SILT with Gravel; dry, loose. (FILL)	ML					
		8-1					8	100.5	9.7	1.5	
					Brown Sandy CLAY; moist, very stiff. (NATIVE)	CL					
	5	8-2					28	112.1	17.9		
					Mottled Brown & Gray Sandy CLAY; moist, very stiff.	CL					
	10				Reddish Brown Clayey Coarse SAND; moist to wet, dense.	SC					
		8-3					30	107.0	16.2		
	15	8-4					42				
					Mottled Brown & Gray Clayey Fine SAND; moist, dense.	SC					
					Mottled Brown & Gray Fine Sandy CLAY; moist, hard.	CL					
	20	8-5					52				
					Boring Terminated @ 21.5'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

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Figure 12

LOG OF TEST BORING

BORING NO.: 9

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ☲ :

PROJECT NO.: VV4376
DATE: 09/30/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty CLAY; dry to moist, very stiff.	CL					
					Mottled Tan & Orange Fine Sandy Clayey SILT; dry to moist, hard.	ML					
		9-1					50-5.5"	81.5	30.1	4.5+	ϕ=37° c=435 psf
	5										
		9-2			As Above; dry to moist, hard.		50-4"	88.6	30.8	4.5+	
	10				Tan & Orange Silty CLAY; dry to moist, hard.	CL					
		9-3a					41	93.4	7.9		
		9-3b			Tan Silty Fine SAND; dry to moist, medium dense to dense.	SM					
					Tan & Orange Silty CLAY; dry to moist, hard.	CL					
	15										
					Tan Silty SAND; dry to moist, very dense.	SM					
		9-4					50-6"	89.4	12.1		
	20				Boring Terminated @ 19.5'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 10

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 09/30/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Tan Fine Sandy Silty CLAY; dry to moist, hard.	CL					
		10-1					50-5"			4.5+	
					Light Olive Brown Silty SAND; dry to moist, dense.	SM					
		10-2					31				
					Light Olive Brown Clayey Sandy SILT; dry to moist, hard.	ML					
		10-3					50-6"				%<200=73%
		10-4			As Above; dry to moist, hard. Boring Terminated @ 18'. No Groundwater Encountered.		50-5"				

This information pertains only to this boring and is not necessarily indicitive of the whole site.

KC ENGINEERING CO.

Figure 14

LOG OF TEST BORING
BORING NO.: 11

PROJECT: Proposed Industrial/ Commercial Project
PROJECT NO.: VV4376
CLIENT: LDK Ventures, LLC
DATE: 09/30/19
LOCATION: 700 Crocker Drive, Vacaville, CA
ELEVATION: n/a
DRILLER: California Geotech
LOGGED BY: DS
DRILL RIG: Mobile B-24
BORING DIAMETER: 4"
DEPTH TO WATER: INITIAL ▽ :
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0		Reddish Brown Sandy CLAY; dry to moist, very stiff.	CL					
		11-1			22	107.0	15.9		UCC=13,780 psf
	5		Tan & Orange Sandy SILT; dry to moist, hard.	ML					
		11-2			50-3"	83.0	22.0		
	10		Tan & Orange Clayey SILT; dry to moist, hard.	ML					
		11-3			54	83.1	22.5		%<200=77%
	15								
			Tan Clayey SAND; dry to moist, dense.	SC					
	20	11-4			47	96.5	10.1		
			Boring Terminated @ 20'. No Groundwater Encountered.						
	25								

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 12

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 09/30/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty CLAY; dry to moist, very stiff.	CL					
					Tan & Orange Silty CLAY; dry to moist, very stiff.	CL					
		12-1					29			4.5+	
	5										
					Tan Sandy SILT; dry to moist, dense.	ML					
		12-2					41	88.3	13.7		%<200=69%
	10										
					Grey Silty SAND; dry, very dense.	SM					
		12-3					52	100.1	6.0		%<200=13%
	15				Grey & Orange SILTSTONE/ SANDSTONE; completely weathered, friable.	Rx					
		12-4					50-4"	87.2	12.8		
					Boring Terminated @ 17.5'. No Groundwater Encountered.						
	20										
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

KC ENGINEERING CO.

LOG OF TEST BORING
BORING NO.: 13

PROJECT: Proposed Industrial/ Commercial Project
PROJECT NO.: VV4376
CLIENT: LDK Ventures, LLC
DATE: 09/30/19
LOCATION: 700 Crocker Drive, Vacaville, CA
ELEVATION: n/a
DRILLER: California Geotech
LOGGED BY: DS
DRILL RIG: Mobile B-24
BORING DIAMETER: 4"
DEPTH TO WATER: INITIAL ∇ :
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Sandy CLAY; moist, stiff to very stiff.	CH					
		13-1					15	104.9	21.5	1.75	LL=59 PI=46 UCC=4,240 psf
					Light Olive Brown Silty Sandy CLAY; moist, very stiff.	CL					
	5										
		13-2					17	113.4	17.0	2.0	
	10										
		13-3			As Above; moist, hard.		50-5.5"	108.4	22.0		
	15										
					Olive CLAYSTONE; highly weathered, friable.	Rx					
	20	13-4					50-5"				
					Boring Terminated @ 21'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

KC ENGINEERING CO.

LOG OF TEST BORING

BORING NO.: 14

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/01/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty Sandy CLAY; dry to moist, stiff to hard.	CL					
		14-1					50-5.5"	125.2	9.1	4.5+	ϕ=29.8° c=1,028 psf
	5				Reddish Brown Silty Sandy CLAY w/ Gravels up to 1/4"; moist, hard.	CL					
		14-2			Yellowish Red Silty Sandy CLAY; moist, hard.	CL	50'4"	95.5	15.2	4.5+	
	10										
		14-3			As Above; moist, hard.		46				
					Boring Terminated @ 13.5'. No Groundwater Encountered.						
	15										
	20										
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 15

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/01/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty CLAY; moist, very stiff.	CH					
		15-1			Olive Yellow Silty CLAY; dry to moist, very stiff.	CH	26	117.4	18.2	4.5+	LL=67 PI=46 UCC=14,382 psf
	5										
		15-2			As Above; dry to moist, hard.		50-5"	104.1	17.9	4.5+	
	10										
		15-3			As Above w/ Sand; moist, hard.		73				
					Boring Terminated @ 13.5'. No Groundwater Encountered.						
	15										
	20										
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING

BORING NO.: 16

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV4376
DATE: 10/01/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty CLAY; dry to moist, firm to stiff.	CL					
					Olive Yellow Silty Sandy CLAY; moist, stiff.	CL					
		16-1					14			4.0	
	5										
		16-2			As Above; moist, hard.		50-4"	100.1	21.7		
	10										
	15										
	20	16-3			As Above; moist, hard.		62				
					Boring Terminated @ 20'.						
					No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

KC ENGINEERING CO.

Figure 20

LOG OF TEST BORING

BORING NO.: 17

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ☵ :

PROJECT NO.: VV4376
DATE: 10/01/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Silty CLAY w/ Trace Gravel up to 1/4"; moist, firm to stiff.	CL					
		17-1			Olive Silty Sandy CLAY w/ Trace Gravel up to 1/4"; moist, firm to stiff.	CL	8	97.7	25.1		UCC=2,571 psf
	5										
		17-2			As Above; moist, hard.		35	94.8	25.4		
	10				Olive Sandy CLAY; moist, hard.	CL					
		17-3					47	93.2	25.5		
	15										
		17-4			As Above; moist, hard.		50-5.5"	108.1	18.4		
	20				Boring Terminated @ 19.5'. No Groundwater Encountered.						
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

LOG OF TEST BORING
BORING NO.: 18

PROJECT: Proposed Industrial/ Commercial Project
CLIENT: LDK Ventures, LLC
LOCATION: 700 Crocker Drive, Vacaville, CA
DRILLER: California Geotech
DRILL RIG: Mobile B-24
DEPTH TO WATER: INITIAL ∇ :

PROJECT NO.: VV4376
DATE: 10/01/19
ELEVATION: n/a
LOGGED BY: DS
BORING DIAMETER: 4"
FINAL ▼ : AFTER: hrs.

ELEVATION	DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	0				Reddish Brown Sandy CLAY; moist, very stiff.	CL					
		18-1					18	109.8	18.4	2.25	LL=45 PI=29 UCC=5,476 psf
	5				Olive Yellow Sandy CLAY; moist, hard.	CL					
		18-2					31	102.6	23.7	2.25	
	10										
		18-3			As Above; moist, hard.		42	105.2	17.9		
	15										
	20	18-4			As Above; moist, hard. Boring Terminated @ 20'. No Groundwater Encountered.		43	97.3	25.2		
	25										

This information pertains only to this boring and is not necessarily indicitive of the whole site.

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KC ENGINEERING COMPANY
A SUBSIDIARY OF MATERIALS TESTING, INC.

TEST PIT LOG

Client: LDK Ventures, LLC
3140 Peacekeeper Way
McClellan, CA 95652

Project No: VV4376
Date of Test Pits: 18 October 2019

Project: Proposed Industrial/ Commercial Project:
700 Crocker Drive
Vacaville, CA

TEST PIT No.	DEPTH (feet)	USCS	DESCRIPTION
TP-1	0' – 1.5'	GC	Brown Sandy Clayey GRAVELS w/ Concrete Pieces; dry, loose. (FILL)
	1.5' – 3.5'	CL	Brown Sandy CLAY w/ Some Gravels; moist, very stiff. (FILL)
	3.5' – 5'	CH	Dark Brown Silty CLAY; moist, stiff. (NATIVE)
TP-2	0' – 2.5'	CL	Brown Sandy CLAY w/ Some Gravels; loose, dry to moist, plastic @ base. (FILL)
	2.5' – 3.5'	CL/CH	Dark Brown Silty CLAY; moist, firm to stiff. (NATIVE)

TP-3	0’ – 5’	CL/CH	Brown & Gray Silty CLAY w/ Some Gravels; dry, loose & soft in top 1.5’, then firm to 5’. (FILL)
	5’ – 7’	ML	Brown SILT; moist, firm. (NATIVE)
TP-4	0’ – 1.5’	CL/CH	Dark Brown Silty CLAY; dry, top 1’ discked then firm to stiff. (NATIVE)
TP-5	0’ – 7’	CL/CH	Brown Sandy CLAY w/ Some Gravels; dry to moist, soft to firm. (FILL)
	7’ – 8’	CL/CH	Dark Brown Silty CLAY; moist, firm. (NATIVE)
TP-6	0’ – 4’	CL/CH	Brown Sandy CLAY; dry to moist, top 12” loose, then stiff. (NATIVE)
	4’		Hit Water Main.
TP-7	0’ – 1.5’	CL	Dark Brown Sandy CLAY w/ Gravels & Minor Debris; dry, soft. (FILL)
	1.5’ – 3.5’	ML	Brown SILT; moist, firm w/ some voids. (NATIVE)
	3.5’ – 5’	CH	Brown Sandy CLAY; moist, stiff.
TP-8	0’ – 1.5’	CL	Brown Silty CLAY w/ Gravels; dry, stiff. (FILL)
	1.5’ – 3’	CL	Reddish Brown Sandy CLAY; moist, very stiff. (NATIVE)
	3’ – 5’	Rx	Reddish Brown Clayey SANDSTONE; dry, highly weathered, moderately strong.

TP-9	0' – 1.5' 1.5' – 2.5' 2.5' – 5'	CL/CH CL CL	Brown Silty CLAY; dry, stiff. (FILL?) Light Brown Silty CLAY; dry, stiff. (NATIVE) Reddish Brown Silty Sandy CLAY; dry to moist, very stiff to hard.
TP-10	0' – 5' 5' – 6'	CL/CH CH	Reddish Brown Silty Sandy CLAY w/ Rocks up to 2-3 inches; dry to moist, firm to stiff. (FILL) Dark Brown Silty CLAY; moist, stiff to very stiff. (NATIVE)
TP-11	0' – 4.5' 4.5' – 5.5'	CL/CH CH	Reddish Brown Silty Sandy CLAY w/ Rocks up to 2-3 inches; moist, firm. (FILL) Dark Brown Silty CLAY; moist, firm. (NATIVE)
TP-12	0' – 3' 3' – 4'	CL/CH CL	Brown Silty CLAY; dry to moist, firm to stiff. (NATIVE) Yellowish Brown Silty Sandy CLAY; dry to moist, very stiff to hard.
TP-13	0' – 2' 2' – 4'	CL CL	Brown Silty CLAY w/ Gravels; dry to moist, firm to stiff. (FILL) Yellowish Brown Silty Sandy CLAY; moist, very stiff to hard. (NATIVE)
TP-14	0' – 4' 4' – 5'	CL/CH ML	Brown CLAY; moist, stiff. (NATIVE) Pale Brown Clayey SILT w/ Trace Sand; moist, stiff.

Note: No Groundwater Encountered During the Field Exploration.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is retained on the No. 200 Sieve	GRAVEL More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (<5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines (Cu>4 & 1<Cc<3)
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines (Cu < 4 and/or 1>Cc>3)
		Gravel with fines (>12% fines)	GM	Silty gravels and gravel-sand-silt mixtures (PI<4 or below "A" line)
			GC	Clayey gravels and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
	SAND Half or more of the coarse fraction is smaller than No. 4 sieve	Clean sands (<5% fines)	SW	Well graded sands, gravelly sands, little or no fines (Cu>6 & 1<Cc<3)
			SP	Poorly graded sands, gravelly sands, little or no fines (Cu<6 and/or 1>Cc>3)
		Sand with fines (>12% fines)	SM	Silty sands and gravel-sand-silt mixtures (PI<4 or below "A" line)
			SC	Clayey sands and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
FINE GRAINED SOILS Half or more of the material passes the No. 200 Sieve	SILTS AND CLAYS Liquid Limit is less than 50%		ML	Inorganic silts with gravel and sand having slight plasticity (PI<4 or below "A" line)
			CL	Inorganic clays of low to med. plasticity with gravel and sand (PI>7 & on or above "A" line)
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS Liquid Limit is 50% or more		MH	Inorganic elastic silts (PI below "A" line)
			CH	Inorganic clays of high plasticity, fat clays (PI on or above "A" line)
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

MTI-KC ENGINEERING COMPANY
865 Cotting Lane, Ste A, Vacaville, CA 95688
8798 Airport Road, Redding, CA 96002

SAMPLER AND LAB TESTING LEGEND

- Auger
- Bulk Sample, taken from auger cuttings
- California Sampler
- Bulk/Grab Sample
- Pitcher
- Standard Penetration Test
- Shelby Tube
- No Recovery

LL=Liquid Limit (%)
PI=Plasticity Index
¦ =Friction Angle
C=Cohesion
UCC=Unconfined Compression
R value=Resistance Value
Consol=Consolidation Test

SOIL GRAIN SIZE

U.S. STANDARD SIEVE OPENINGS

	#200	#40	#10	#4	¾"	3"	12"	
CLAY	SILT	SAND			GRAVEL		COBBLES	BOULDERS
		FINE	MEDIUM	COARSE	FINE	COARSE		
0.002	0.075	0.425	2.00	4.75	19.0	75	300	

SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY (Coarse-grained soils)

SANDS & GRAVELS	BLOWS/FOOT[1]
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	> 50

CONSISTENCY (Fine-grained soils)

SILTS & CLAYS	STRENGTH[2]	BLOWS/FOOT[1]
Very Soft	< 500	0 – 2
Soft	500 – 1,000	2 – 4
Firm	1,000 – 2,000	4 – 8
Stiff	2,000 – 4,000	8 – 15
Very Stiff	4,000 – 8,000	15 – 30
Hard	> 8,000	>30

1 – Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. split spoon sampler (ASTM D1586)
2 – Unconfined compressive strength in lb/ft^2 as determined by lab testing or approximated by the standard penetration test (ASTM D1586) or pocket penetrometer.

WEATHERING (Bedrock)

Fresh	No visible sign of decomposition or discoloration; rings under hammer impact
Slightly weathered	Slight discoloration inwards from open fractures; little or no effect on normal cementation; otherwise similar to Fresh
Moderately weathered	Discoloration throughout; weaker minerals decomposed; strength somewhat less than fresh rock but cores can not be broken by hand or scraped with knife; texture preserved; cementation little to not affected; fractures may contain filling
Highly weathered	Most minerals somewhat decomposed; specimens can be broken by hand with effort or shaved with knife; texture becoming indistinct but fabric preserved; faint fractures
Completely weathered	Minerals decomposed to soil but fabric and structure preserved; specimens can be easily crumbled or penetrated

STRENGTH (Bedrock)

Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen will crumble under light hammer blows
Moderately strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments
Very strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

BEDDING (Bedrock)	SPACING (inches)
Very thickly bedded	> 48
Thickly bedded	24 to 48
Thin bedded	2.5 to 24
Very thin bedded	5/8 to 2.5
Laminated	1/8 to 5/8
Thinly laminated	<1/8

FRACTURING (Bedrock)	SPACING (inches)
Very little fractured	> 48
Occasionally fractured	12 to 48
Moderately fractured	6 to 12
Closely fractured	1 to 6
Intensely fractured	5/8 to 1
Crushed	<5/8

M T I

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client: LDK Ventures, LLC
3140 Peacemaker Way
McClellan, CA 95652

Client No.: VV4376-001
Report No.: 0300-001
Date: 11/02/17

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Submitted by: KC Engineering

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
1-1 @ 2.0'	Grayish Brown Sandy Silt (visual)	103.9	6.5	---	---	---
1-2 @ 7.0'	Yellowish Brown Silty Sand (visual)	93.7	18.4	---	---	---
1-4 @ 19.0'	Mottled Olive Yellow & Orange Clayey Sand (visual)	103.9	23.0	---	---	---
2-1 @ 4.0'	Brown Clayey Silt with Sand (visual)	115.4	10.6	---	---	---
2-2 @ 9.0'	Yellow Brown Sandy Clay (visual)	103.3	23.5	---	---	---
2-3 @ 14.0'	Mottled Brown, Orange & Yellow Clayey Sand (visual)	113.1	16.0	---	---	---
3-1 @ 2.0'	Brown Sandy Silt (visual)	106.9	6.2	---	---	---
3-1B @ 2.0'	Brown Sandy Clay (visual)	---	---	60	19	41
3-2 @ 5.0'	Brown Sandy Silt (visual)	119.3	12.7	---	---	---
3-3 @ 10.0'	Yellowish Brown Sandy Clay (visual)	102.6	24.3	---	---	---
4-1 @ 3.0'	Dark Brown Sandy Clay with Gravel (visual)	113.1	15.7	30	14	16
4-3 @ 14.5'	Mottled Yellow, Gray & Brown Sandy Silt (visual)	82.5	16.6	---	---	---
4-4 @ 24.5'	Mottled Gray & Yellow Sandy Clay (visual)	107.4	21.1	---	---	---
5-1 @ 2.5'	Dark Brown Sandy Clay (visual)	101.5	16.4	38	15	23
5-2 @ 7.5'	Brown Sandy Clay (visual)	114.3	16.2	---	---	---

M T I

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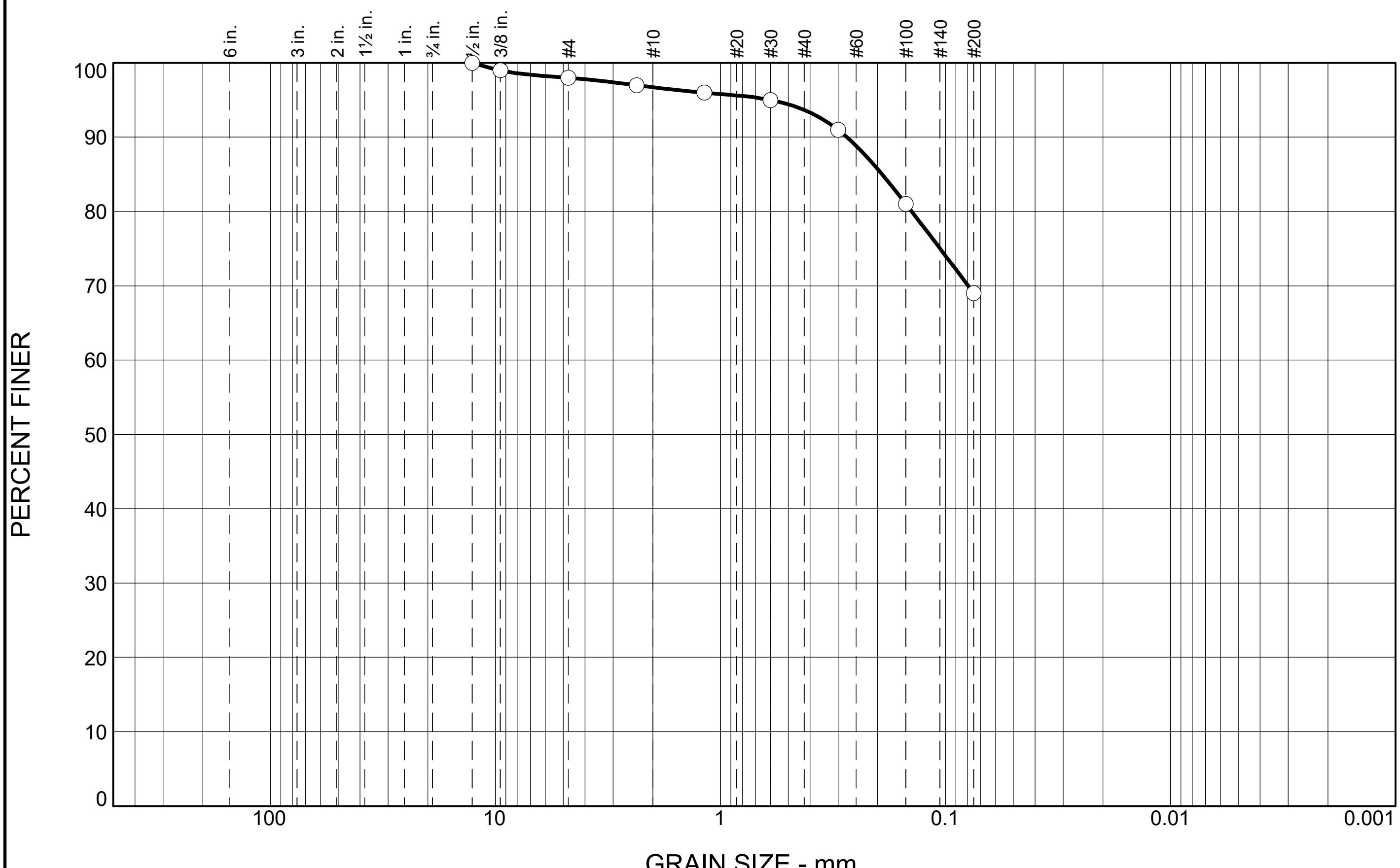
Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
5-3 @ 13.0’	Mottled Gray, Brown & Yellow Sandy Clay (visual)	104.1	22.9	---	---	---
6-1 @ 4.0’	Brown Sandy Clay (visual)	111.9	16.4	---	---	---
6-2 @ 9.0’	Brown Sandy Clay with Gravel (visual)	118.1	11.4	---	---	---
7-1 @ 2.0’	Brown Clayey Sand (visual)	117.7	12.1	---	---	---
7-2 @ 6.0’	Light Brown Clayey Sand (visual)	106.4	15.6	---	---	---
7-3 @ 11.0’	Brown Sandy Clay (visual)	121.4	15.0	---	---	---
7-4 @ 16.0’	Mottled Yellow, Brown & Gray Sandy Clay (visual)	106.4	22.7	---	---	---
8-1 @ 3.5’	Brown Sandy Silt with Gravel (visual)	100.5	9.7	---	---	---
8-2 @ 6.0’	Brown Sandy Clay (visual)	112.1	17.9	---	---	---
8-3 @ 11.0’	Brown Clayey Sand (visual)	107.0	16.2	---	---	---
Pad/Subgrade 0-4.0’ Bulk	Brown Sandy Clay (visual)	---	---	36	15	21

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	2	1	3	25	69	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	99		
#4	98		
#8	97		
#16	96		
#30	95		
#50	91		
#100	81		
#200	69		

* (no specification provided)

Material Description

Grayish Brown Sandy Silt (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2740 D_{85}= 0.1915 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

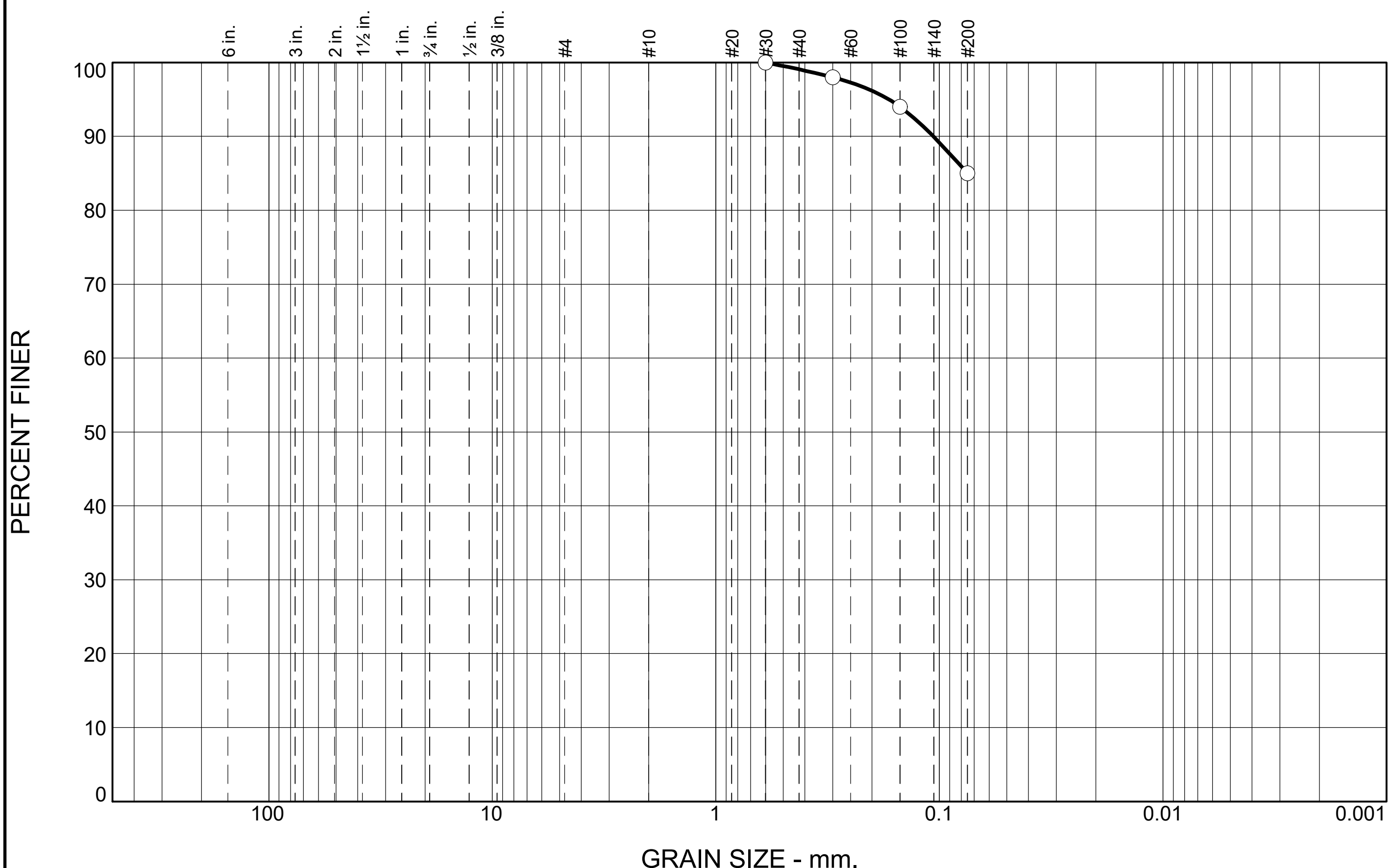
Material tested in accordance with ASTM D6913.

Location: 1-1
Sample Number: 4 **Depth:** 2.0' **Date:** 11/02/17

Materials Testing, Inc.

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-001 **Figure** 0300-002

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	1	14	85	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#30	100		
#50	98		
#100	94		
#200	85		

* (no specification provided)

Material Description

Yellowish Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1065 D_{85}= 0.0750 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

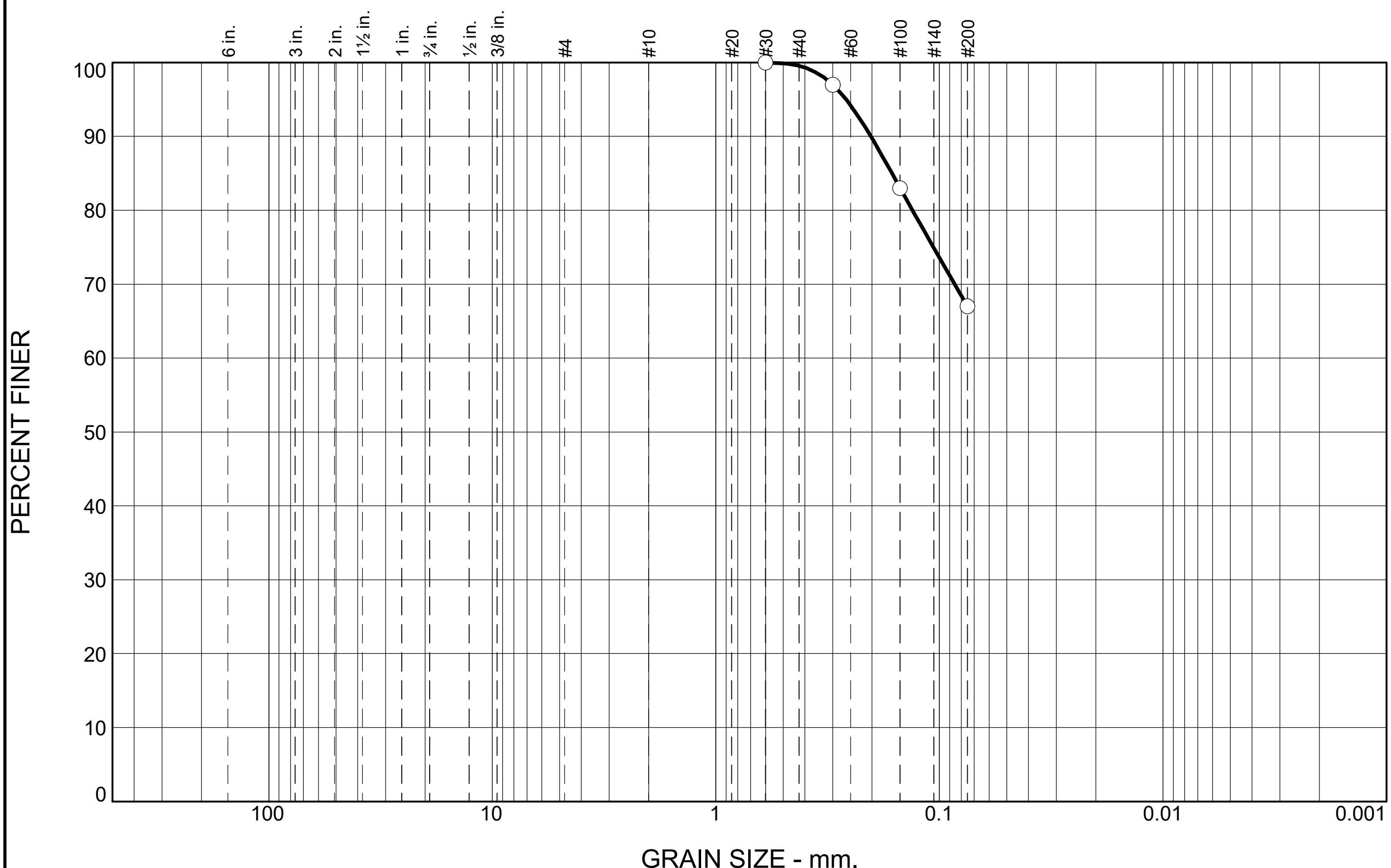
Material tested in accordance with ASTM D6913.

Location: 3-3
Sample Number: 11 **Depth:** 10.0' **Date:** 11/02/17

Materials Testing, Inc.

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-001 **Figure** 0300-003

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	33	67	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#30	100		
#50	97		
#100	83		
#200	67		

* (no specification provided)

Material Description

Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2028 D_{85}= 0.1632 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

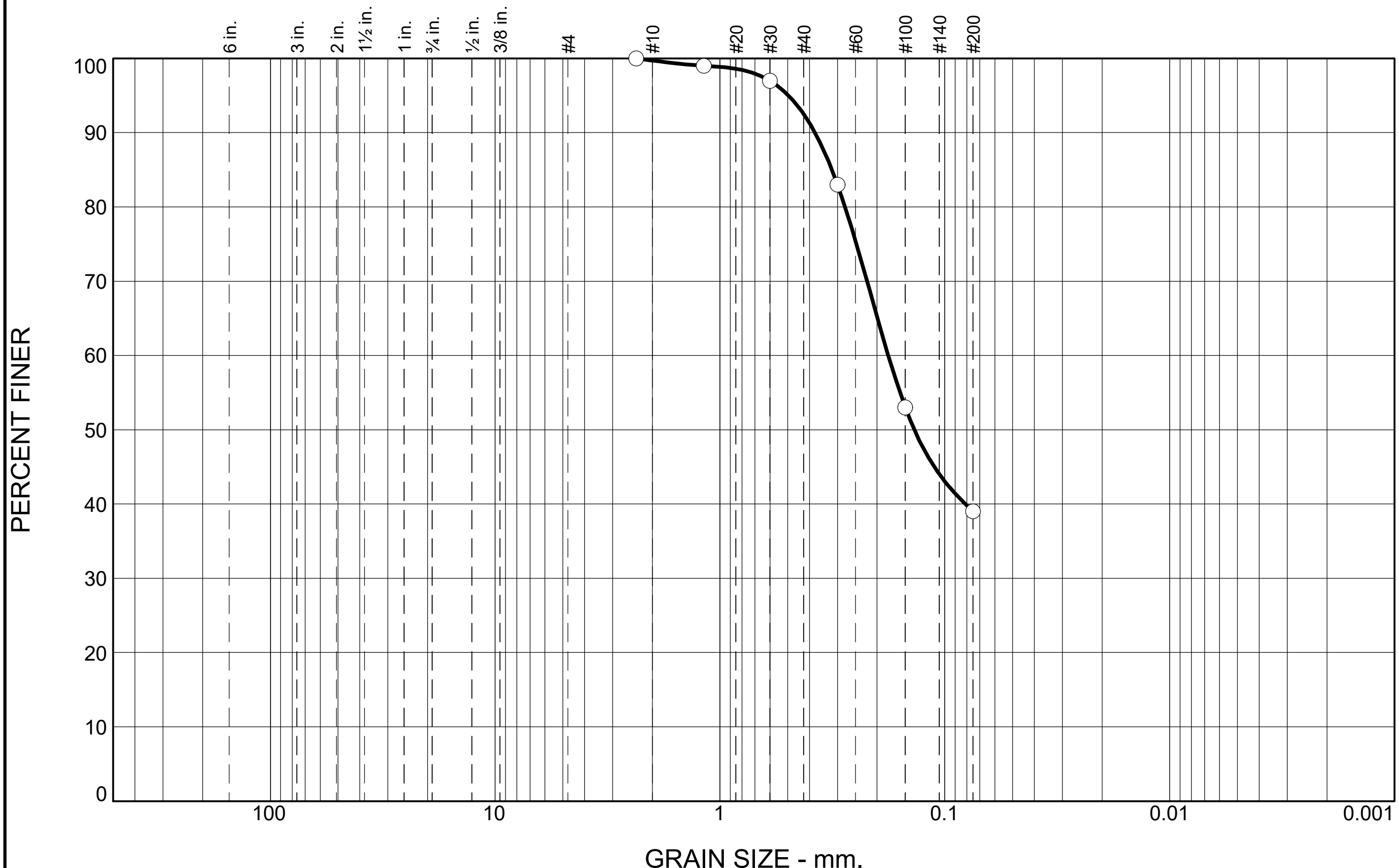
Material tested in accordance with ASTM D6913.

Location: 6-1
Sample Number: 21 **Depth:** 4.0' **Date:** 11/02/17

Materials Testing, Inc.

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-001 **Figure** 0300-004

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	7	54	39	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#8	100		
#16	99		
#30	97		
#50	83		
#100	53		
#200	39		

* (no specification provided)

Material Description

Brown Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3779 D_{85}= 0.3176 D_{60}= 0.1787
D_{50}= 0.1367 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

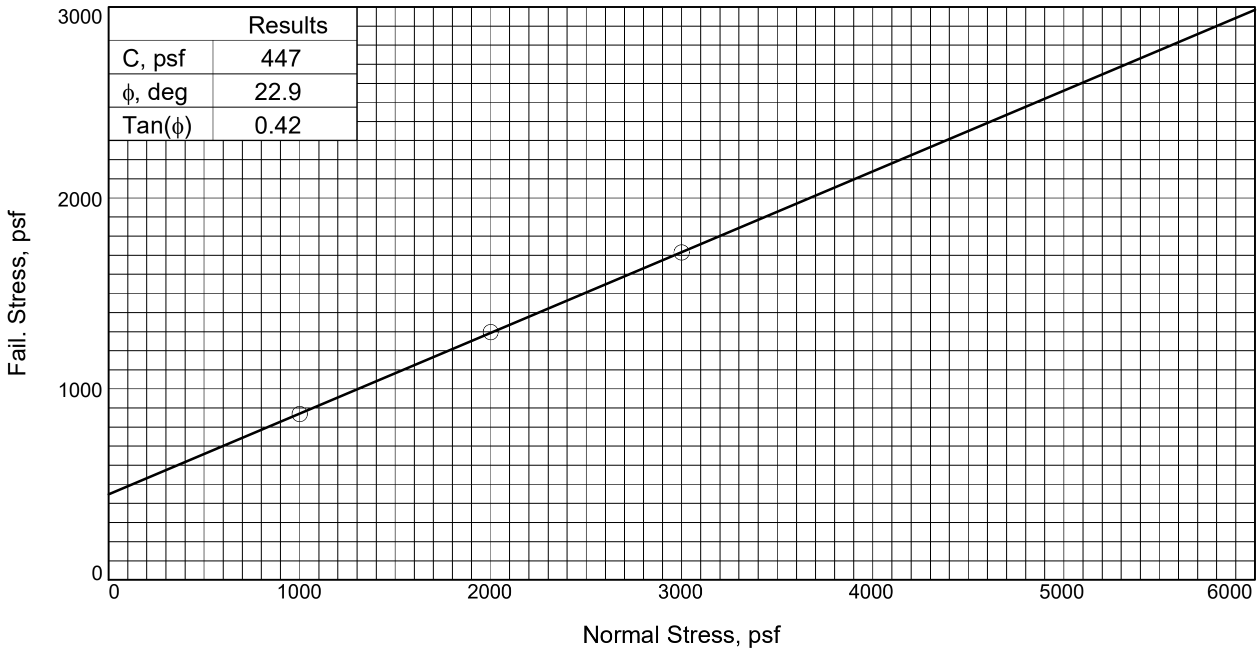
Remarks

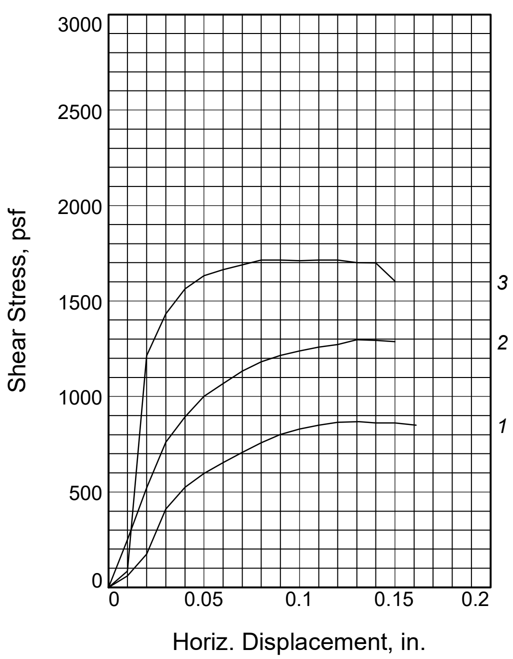
Material tested in accordance with ASTM D6913.

Location: 7-1
Sample Number: 24 **Depth:** 2.0' **Date:** 11/02/17

Materials Testing, Inc.

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-001 **Figure** 0300-005





	Sample No.	1	2	3
Initial	Water Content, %	10.6	10.6	10.6
	Dry Density, pcf	94.2	116.1	136.0
	Saturation, %	39.4	73.6	162.1
	Void Ratio	0.6836	0.3657	0.1662
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	16.8	21.8	22.2
	Dry Density, pcf	87.5	109.0	127.3
	Saturation, %	52.5	122.0	229.4
	Void Ratio	0.8127	0.4544	0.2459
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.08	1.06	1.07
	Normal Stress, psf	1000	2000	3000
	Fail. Stress, psf	868	1297	1714
	Displacement, in.	0.13	0.13	0.08
	Ult. Stress, psf			
	Displacement, in.			
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Tube

Description: Brown Clayey Silt with Sand (visual)

Specific Gravity= 2.54

Remarks: DIRECT SHEAR TEST REPORT
Material tested in accordance with ASTM D3080.

Figure 0300-006

Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

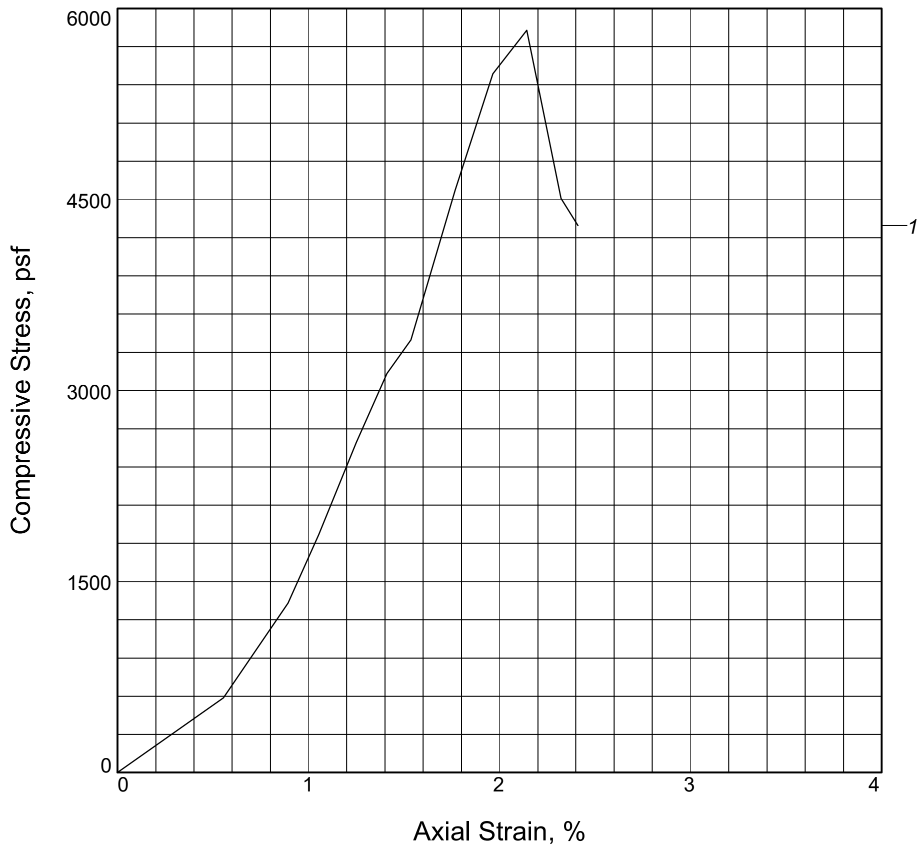
Location: 2-1

Sample Number: 5 **Depth:** 4.0'

Proj. No.: VV4376-001 **Date Sampled:** 10/12/17

Materials Testing, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	5829			
Undrained shear strength, psf	2915			
Failure strain, %	2.1			
Strain rate, in./min.	N/A			
Water content, %	6.2			
Wet density, pcf	113.5			
Dry density, pcf	106.9			
Saturation, %	29.6			
Void ratio	0.5596			
Specimen diameter, in.	2.41			
Specimen height, in.	5.60			
Height/diameter ratio	2.32			

Description: Brown Sandy Silt (visual)

LL =	**PL =**	**PI =**	**GS=** 2.67	**Type:** Tube

Project No.: VV4376-001
Date Sampled: 10/12/17

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Cone & Split

Figure 0300-007

Client: LDK Ventures, LLC

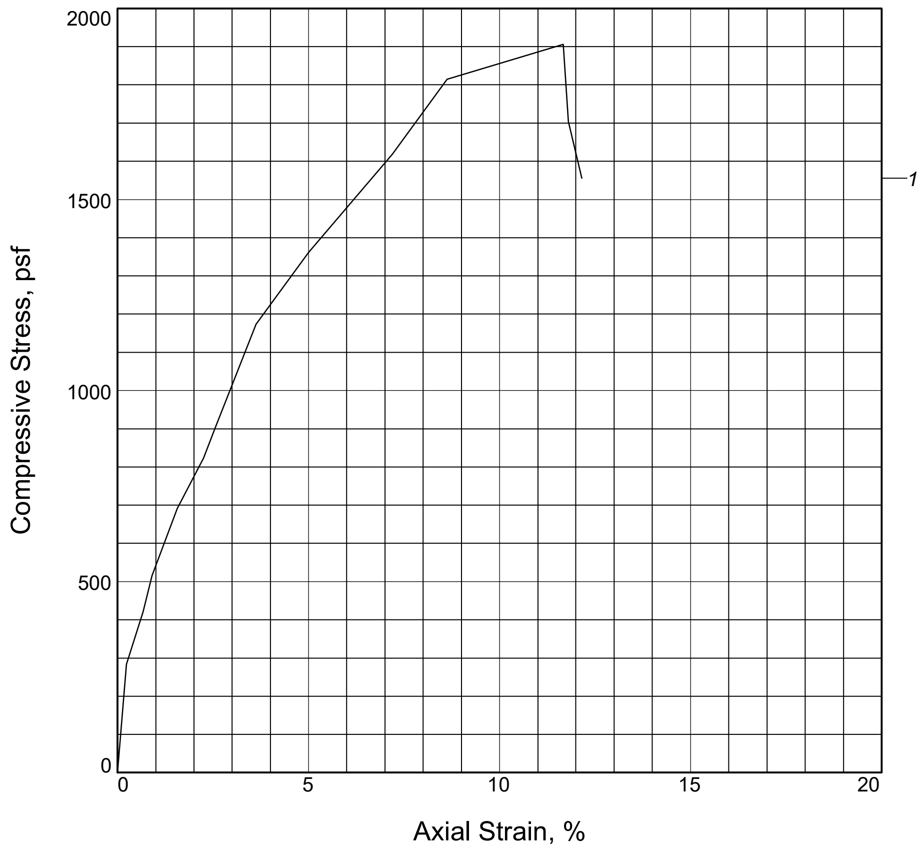
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Location: 3-1

Sample Number: 8 **Depth:** 2.0'

Materials Testing, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1906			
Undrained shear strength, psf	953			
Failure strain, %	11.7			
Strain rate, in./min.	N/A			
Water content, %	15.7			
Wet density, pcf	130.9			
Dry density, pcf	113.1			
Saturation, %	93.0			
Void ratio	0.4403			
Specimen diameter, in.	2.41			
Specimen height, in.	5.10			
Height/diameter ratio	2.12			

Description: Dark Brown Sandy Clay with Gravel (visual)

LL = 30	**PL =** 16	**PI =** 14	**GS=** 2.61	**Type:** Tube

Project No.: VV4376-001
Date Sampled: 10/12/17

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Bulge & Columnar.

Figure 0300-008

Client: LDK Ventures, LLC

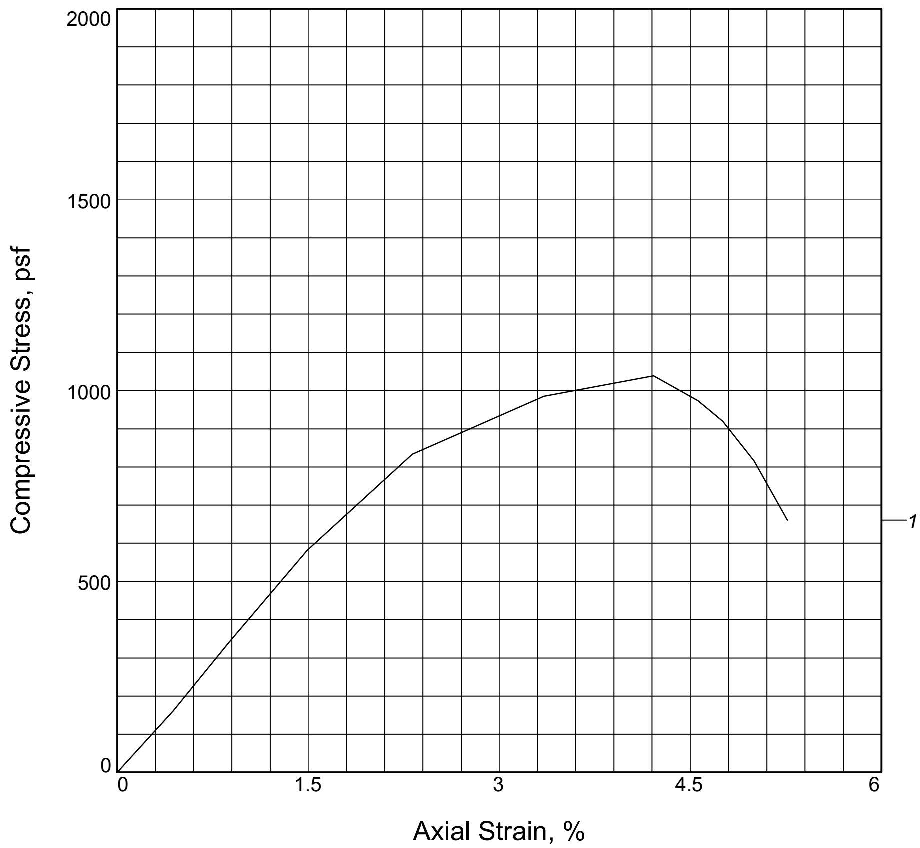
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Location: 4-1

Sample Number: 13 **Depth:** 3.0'

Materials Testing, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1038			
Undrained shear strength, psf	519			
Failure strain, %	4.2			
Strain rate, in./min.	N/A			
Water content, %	15.6			
Wet density, pcf	122.9			
Dry density, pcf	106.4			
Saturation, %	77.6			
Void ratio	0.5202			
Specimen diameter, in.	2.41			
Specimen height, in.	5.70			
Height/diameter ratio	2.37			

Description: Light Brown Clayey Sand (visual)

LL =	PL =	PI =	GS= 2.59	Type: Tube

Project No.: VV4376-001
Date Sampled: 10/12/17

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Columnar

Figure 0300-009

Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Location: 7-2
Sample Number: 25 **Depth:** 6.0'

Materials Testing, Inc.

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8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client	LDK Ventures, LLC 3140 Peacemaker Way McClellan, CA 95652	**Client No.** **Report No.** **Date:**	VV4376-001 0300-010 11/02/17
Project:	Proposed Industrial Building 700 Crocker Drive, Vacaville, California	**Submitted By:** **Submitted Date:**	KC Engineering 10/16/17

EXPANSION INDEX
(ASTM D4829)

Sample #:	Pad/Subgrade, 0-4.0' Bulk
Soil Description:	Brown Sandy Clay (visual)
Initial Moisture Content (%):	10.5
Moisture Content after Test (%):	21.4
Initial Dry Density (lb/ft^3):	108.6
After Test Wet Density (lb/ft^3):	131.9
Degree of Saturation (%):	51.9
Expansion Index:	42

Table 1 Classification of Potential Expansion of Soils Using EI (ASTM D4829-11)

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21 – 50	Low
51 – 90	Medium
91 – 130	High
>130	Very High

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Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client:	LDK Ventures, LLC 3140 Peacemaker Way McClellan, CA 95652	**Client No:** **Report No:** **Date:**	VV4376-001 0300-011 11/02/17
Subject:	Proposed Industrial Building 700 Crocker Drive, Vacaville, California	**Submitted By:**	KC Engineering

---

"R" VALUE TEST REPORT
(CTM 301)

Sample:	35
Description:	Brown Sandy Clay (visual)
Location:	Pad/Subgrade 0-4.0' Bulk

SIEVE ANALYSIS

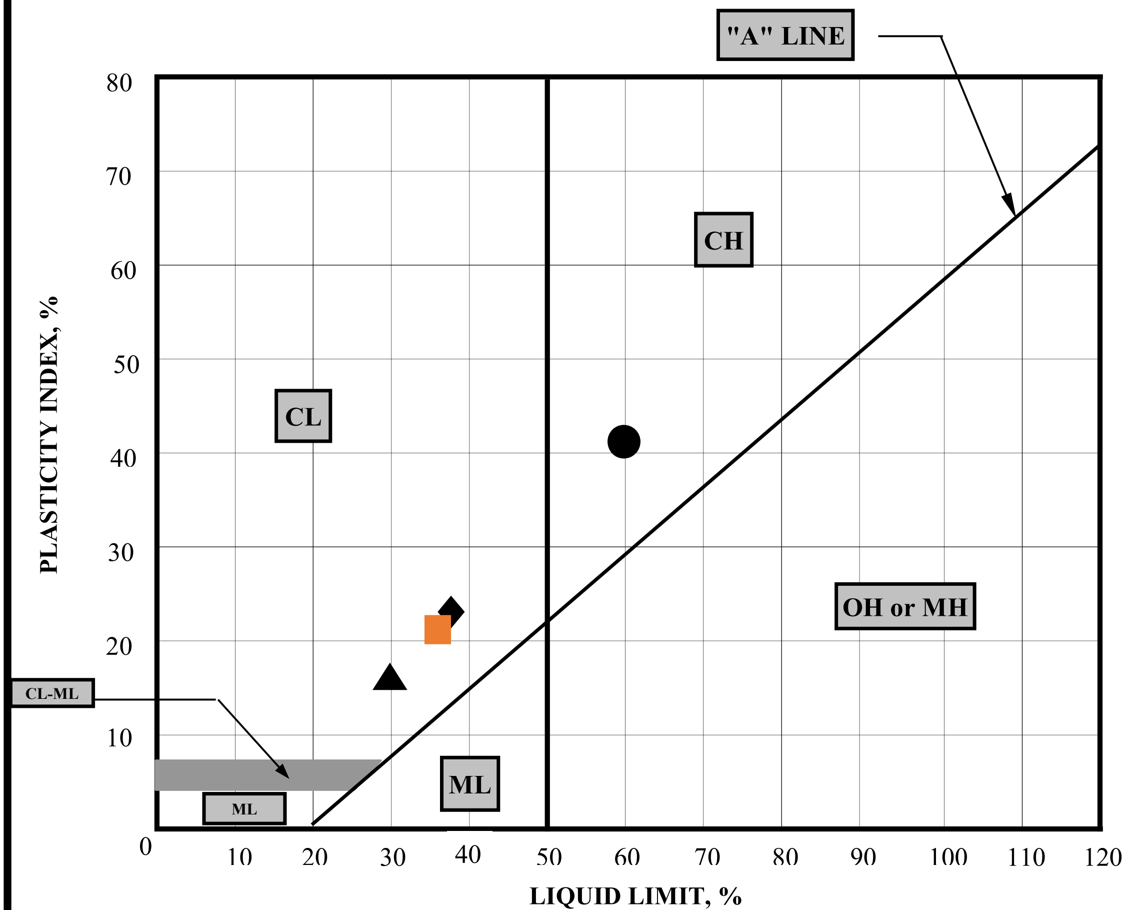
Sieve Size	1"	3/4"	1/2"	3/8"	#4
"As Received" (Percent Pass)	---	---	---	---	100
"As Used" (Percent Pass)	---	---	---	---	100

RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	115.1	14.7	443	16	69	20
2	111.3	17.2	377	11	48	10
3	105.2	19.1	258	5	22	8

R-Value @ 300 PSI Exudation Pressure = 8

Notes:



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	3-1B	2.0'	N/A	60	19	41	N/A	CH
▲	4-1	3.0'	N/A	30	14	16	N/A	CL
◆	5-1	2.5'	N/A	38	15	23	N/A	CL
■	35	Subgrade	N/A	36	15	21	N/A	CL

Note: Atterberg Limits tested in accordance with ASTM D4318.

Materials Testing, Inc.

PLASTICITY CHART AND DATA

Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Client No:	Date:	Report No:
VV4376-001	11/2/2017	0300-012

M T I

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(707) 447-4025, fax 447-4143

Client:	LDK Ventures, LLC 3140 Peacekeeper Way McClellan, CA 95652	**Page No.:**	1 of 2
		Client No.:	VV4376-002
		Report No.:	0300-001
		Date:	10/18/19
Project:	Proposed Industrial Building 700 Crocker Drive, Vacaville, California	**Submitted by:**	KC Engineering

---

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
9-1 @ 3.0’	Mottled Brown Silty Clay (visual)	81.5	30.1	---	---	---
9-2 @ 8.0’	Light Brown Silty Sand (visual)	88.6	30.8	---	---	---
9-3a @ 12.5’	Tan Silty Sand (visual)	93.4	7.9	---	---	---
9-4 @ 19.0’	Tan Silty Sand (visual)	89.4	12.1	---	---	---
11-1 @ 2.5’	Brown Sandy Clay (visual)	107.0	15.9	---	---	---
11-2 @ 7.5’	Brown Sandy Silt (visual)	83.0	22.0	---	---	---
11-3 @ 12.5’	Brown Silt with Sand (visual)	83.1	22.5	---	---	---
11-4a @ 19.0’	Brown Clayey Sand (visual)	96.5	10.1	---	---	---
11-4b @ 19.5’	Brown Clayey Sand (visual)	102.6	18.5	---	---	---
12-1 @ 8.0’	Tan Sandy Silty (visual)	88.3	13.7	---	---	---
12-2 @ 13.0’	Light Brown Silty Sand (visual)	100.1	6.0	---	---	---
12-3 @ 17.0’	Brown Sandy Clay (visual)	87.2	12.8	---	---	---
13-1 @ 2.5’	Brown Sandy Clay (visual)	104.9	21.5	59	13	46
13-2 @ 8.0’	Brown Sandy Clay with Gravel (visual)	113.4	17.0	---	---	---
13-3 @ 20.5’	Light Brown Clay (visual)	108.4	22.0	---	---	---
14-1 @ 3.0’	Brown Sandy Mudstone with Gravel (visual)	125.2	9.1	---	---	---
14-2 @ 7.5’	Brown Sandy Clay (visual)	95.5	15.2	---	---	---
15-1 @ 2.5’	Brown Claystone (visual)	117.4	18.2	67	21	46
15-2 @ 8.0’	Brown Sandy Clay (visual)	104.1	17.9	---	---	---

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.

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(707) 447-4025, fax 447-4143

Client: LDK Ventures, LLC
3140 Peacekeeper Way
McClellan, CA 95652

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

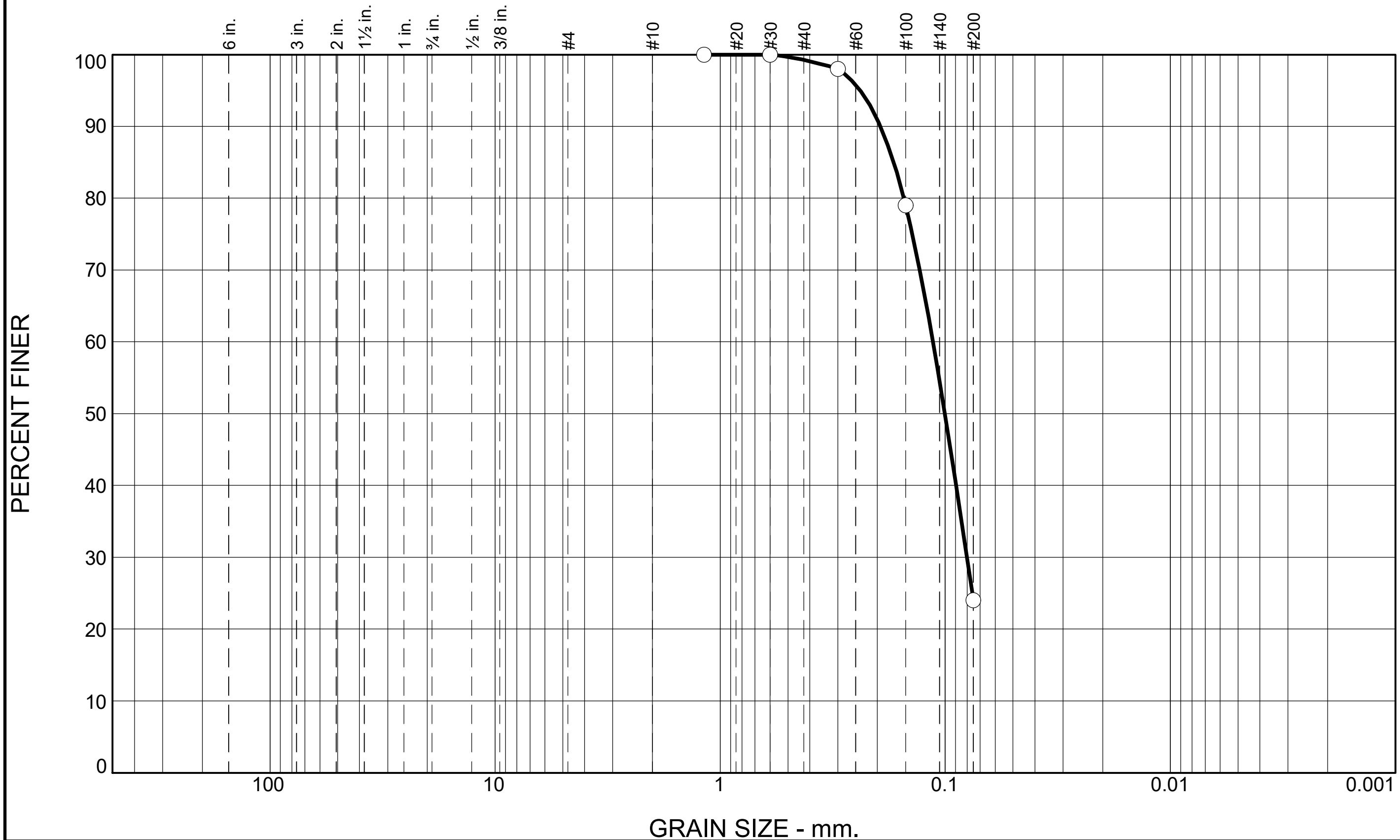
Client No.: VV4376-002
Report No.: 0300-001
Date: 10/18/19
Submitted by: KC Engineering

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
16-1 @ 7.5’	Light Brown Clay (visual)	100.1	21.7	---	---	---
17-1 @ 3.0’	Brown Clay with some coarse Sand (visual)	97.7	25.1	---	---	---
17-2 @ 8.0’	Brown Clay (visual)	94.8	25.4	---	---	---
17-3 @ 13.0’	Brown Sandy Clay (visual)	93.2	25.5	---	---	---
17-4 @ 19.0’	Tan Sandy Clay (visual)	108.1	18.4	---	---	---
18-1 @ 3.0’	Brown Sandy Clay (visual)	109.8	18.4	45	16	29
18-2 @ 8.0’	Light Brown Sandy Clay (visual)	102.6	23.7	---	---	---
18-3 @ 13.0’	Brown Sandy Clay (visual)	105.2	17.9	---	---	---
18-4 @ 19.5’	Light Brown Clay (visual)	97.3	25.2	---	---	---

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	1	75	24	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100		
#30	100		
#50	98		
#100	79		
#200	24		

* (no specification provided)

Material Description

Tan Silty Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1943 D_{85}= 0.1697 D_{60}= 0.1133
D_{50}= 0.1003 D_{30}= 0.0801 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

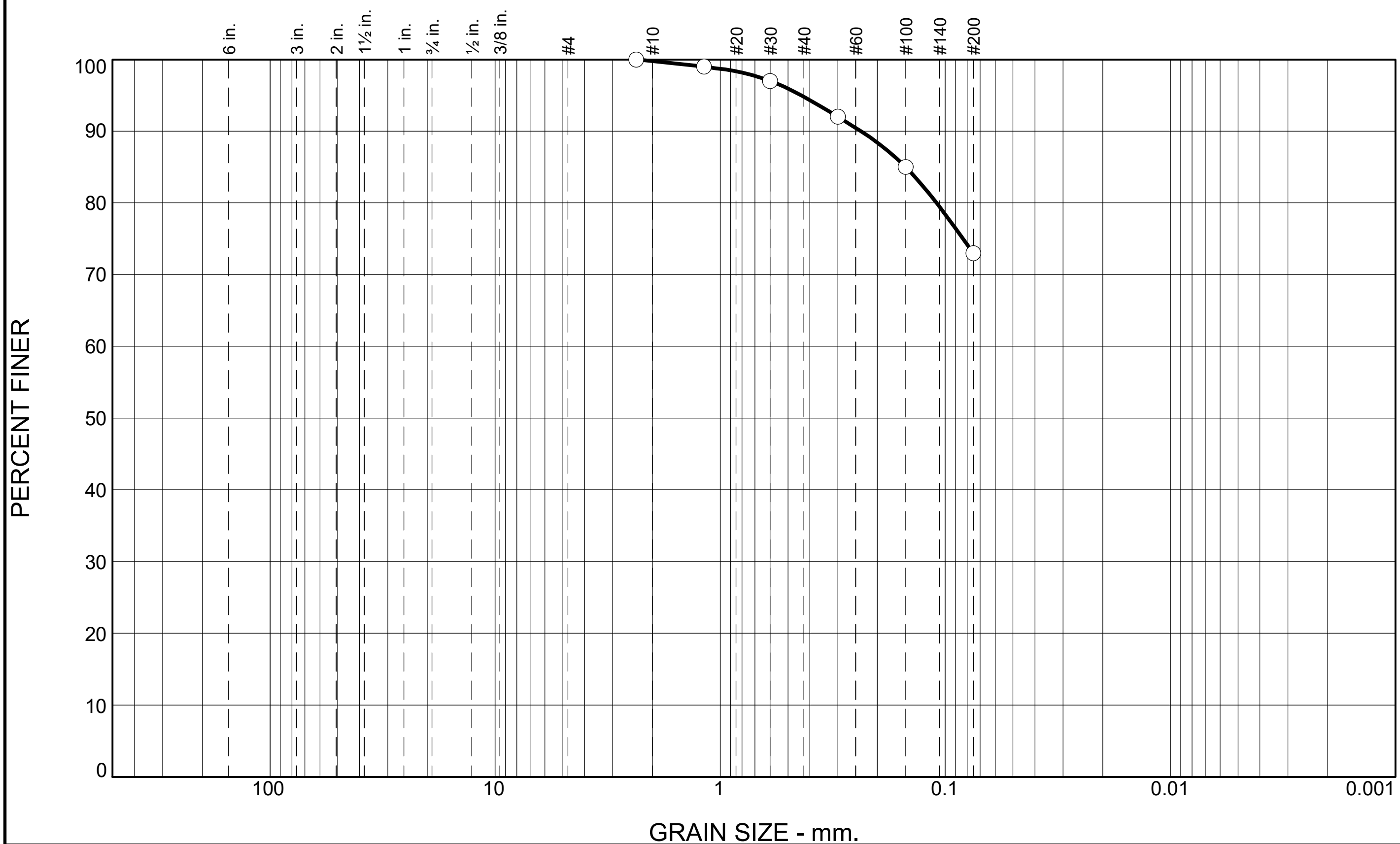
Material tested in accordance with ASTM D6913

Location: 10-2
Sample Number: 8 **Depth:** 8.0' **Date:** 10/18/19

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-002 **Figure** 0300-002

Tested By: Allante Blocker

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	5	22	73	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#8	100		
#16	99		
#30	97		
#50	92		
#100	85		
#200	73		

* (no specification provided)

Material Description

Tan Silt with Sand (visaul)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2371 D_{85}= 0.1500 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

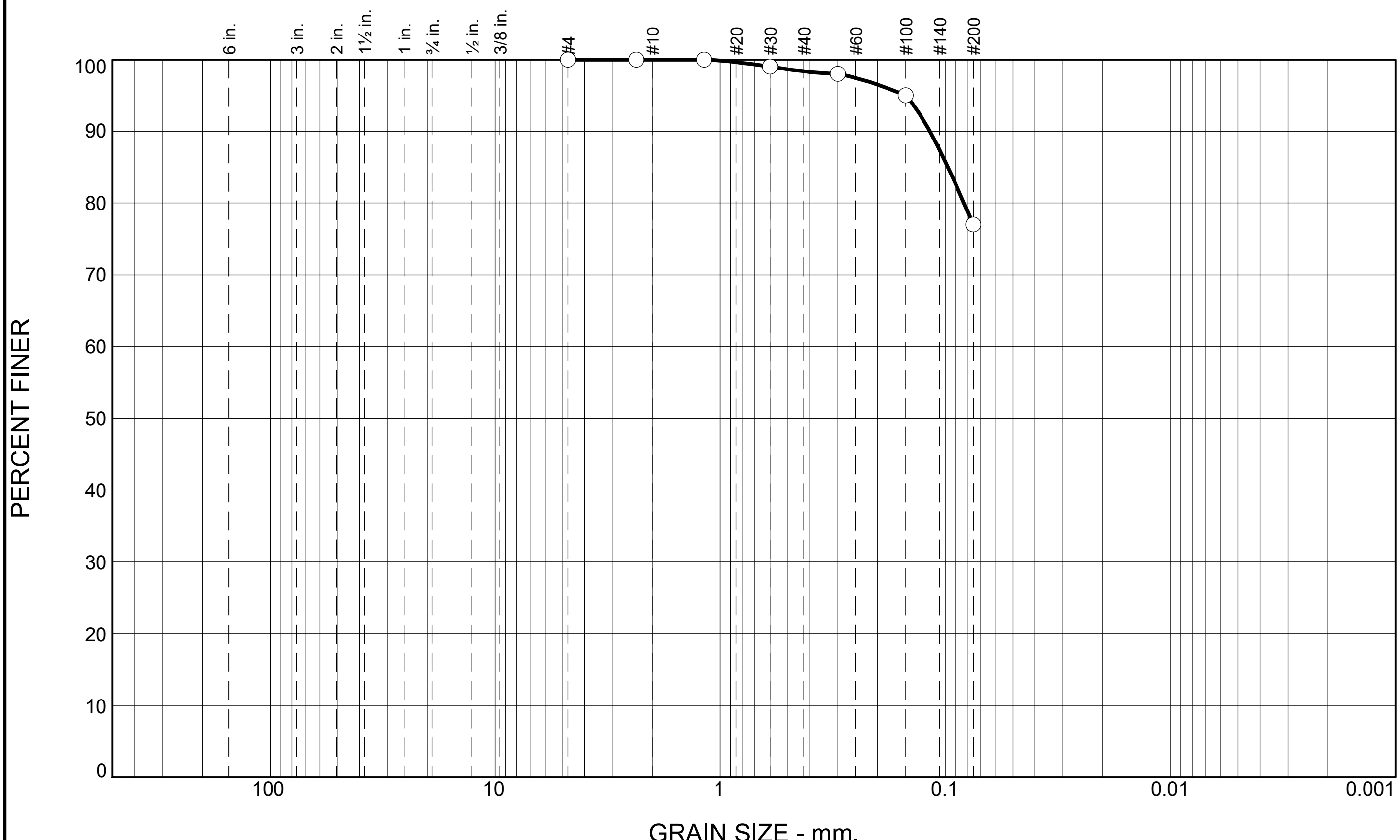
Material tested in accordance with ASTM D6913

Location: 10-3
Sample Number: 9 **Depth:** 12.5' **Date:** 10/18/19

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-002 **Figure** 0300-003

Tested By: Allante Blocker

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	2	21	77	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#8	100		
#16	100		
#30	99		
#50	98		
#100	95		
#200	77		

* (no specification provided)

Material Description

Brown Silt with Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1170 D_{85}= 0.0973 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Material tested in accordance with ASTM D6913

Location: 11-3
Sample Number: 13 **Depth:** 12.5' **Date:** 10/18/19

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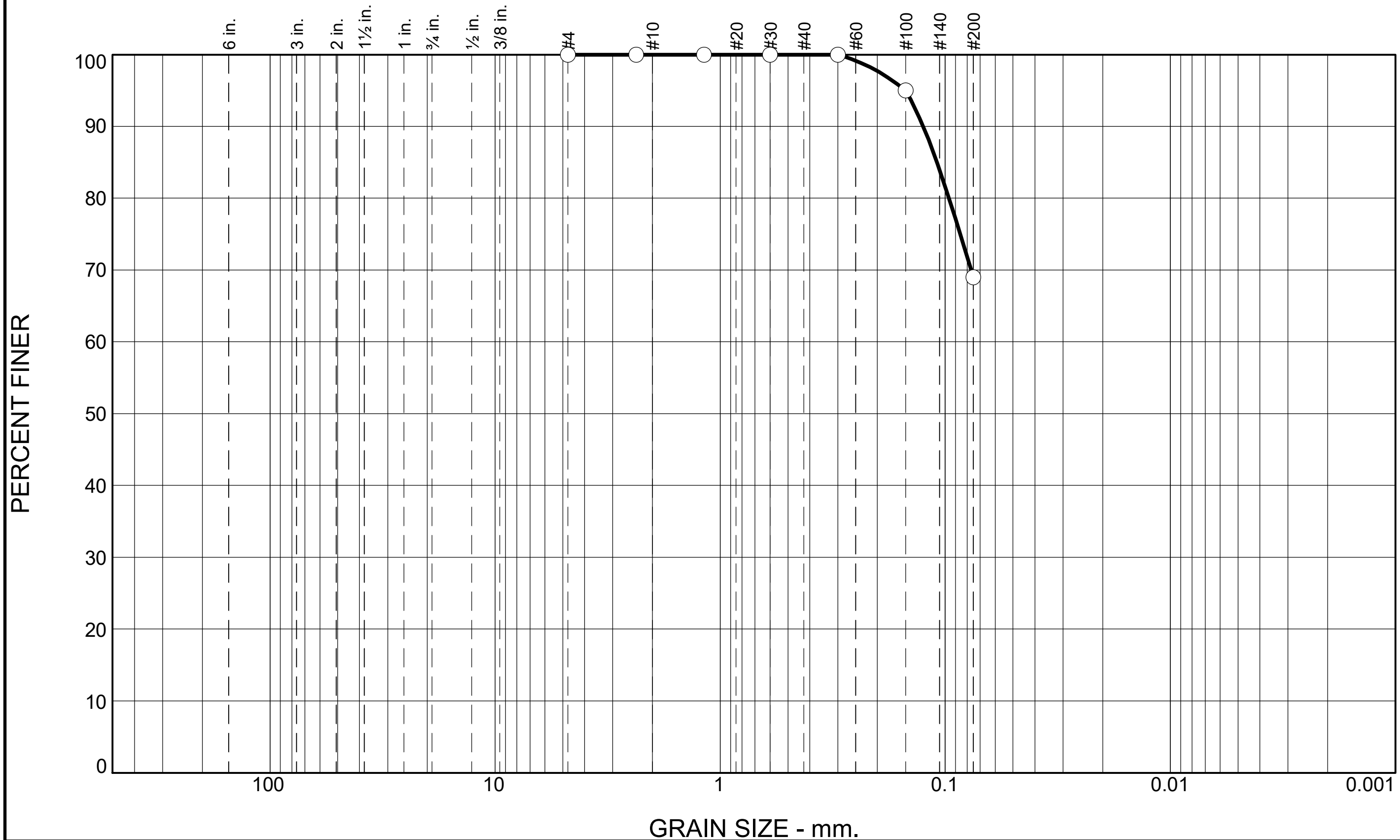
Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-002 **Figure** 0300-004

Tested By: Jack Bianchin

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	31	69	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#8	100		
#16	100		
#30	100		
#50	100		
#100	95		
#200	69		

* (no specification provided)

Material Description

Tan Sandy Silt (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1252 D_{85}= 0.1089 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

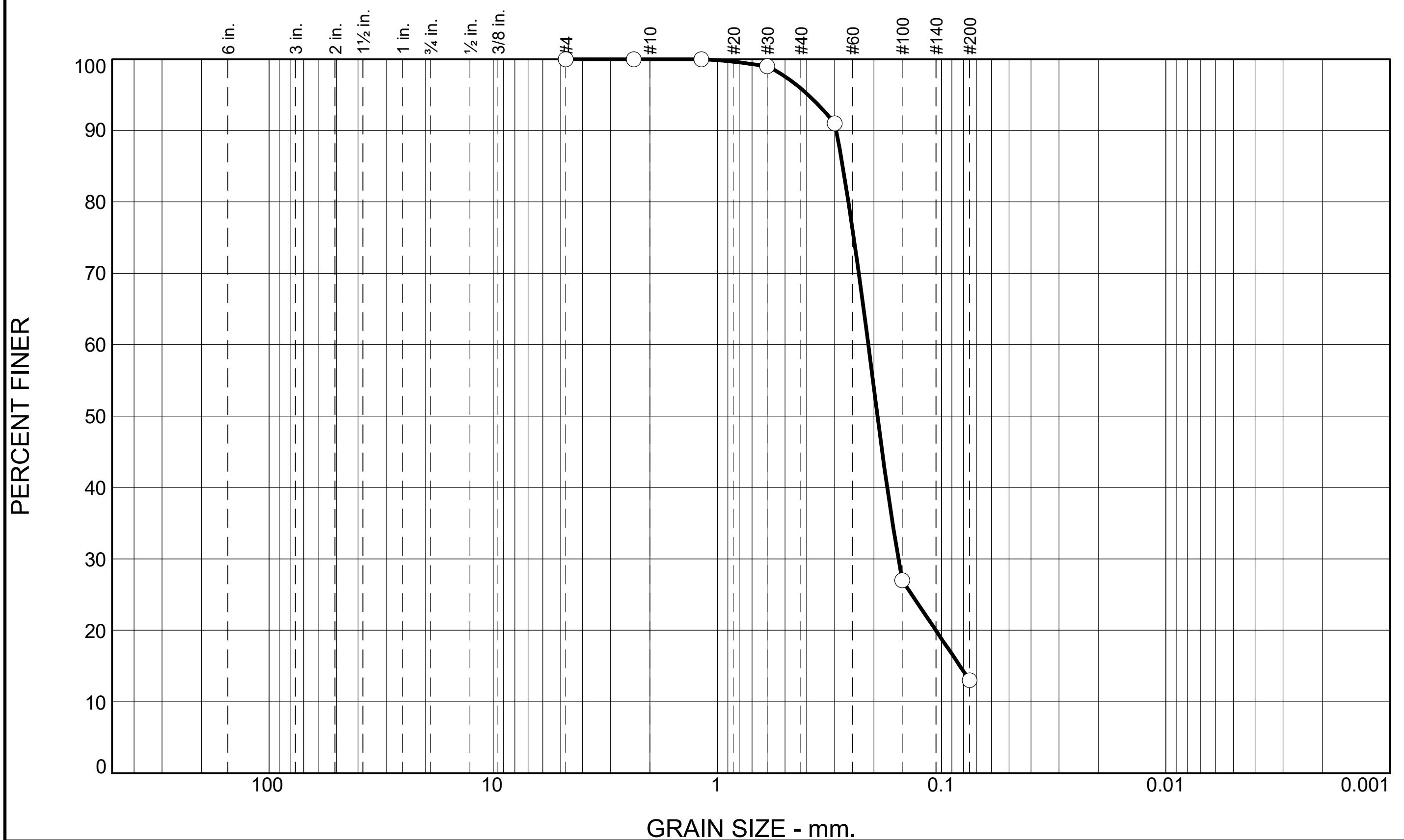
Material tested in accordance with ASTM D6913

Location: 12-1
Sample Number: 16 **Depth:** 8.0' **Date:** 10/18/19

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-002 **Figure** 0300-005

Tested By: Jack Bianchin

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	4	83	13	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#8	100		
#16	100		
#30	99		
#50	91		
#100	27		
#200	13		

* (no specification provided)

Material Description

Light Brown Silty Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2953 D_{85}= 0.2758 D_{60}= 0.2127
D_{50}= 0.1934 D_{30}= 0.1562 D_{15}= 0.0828
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

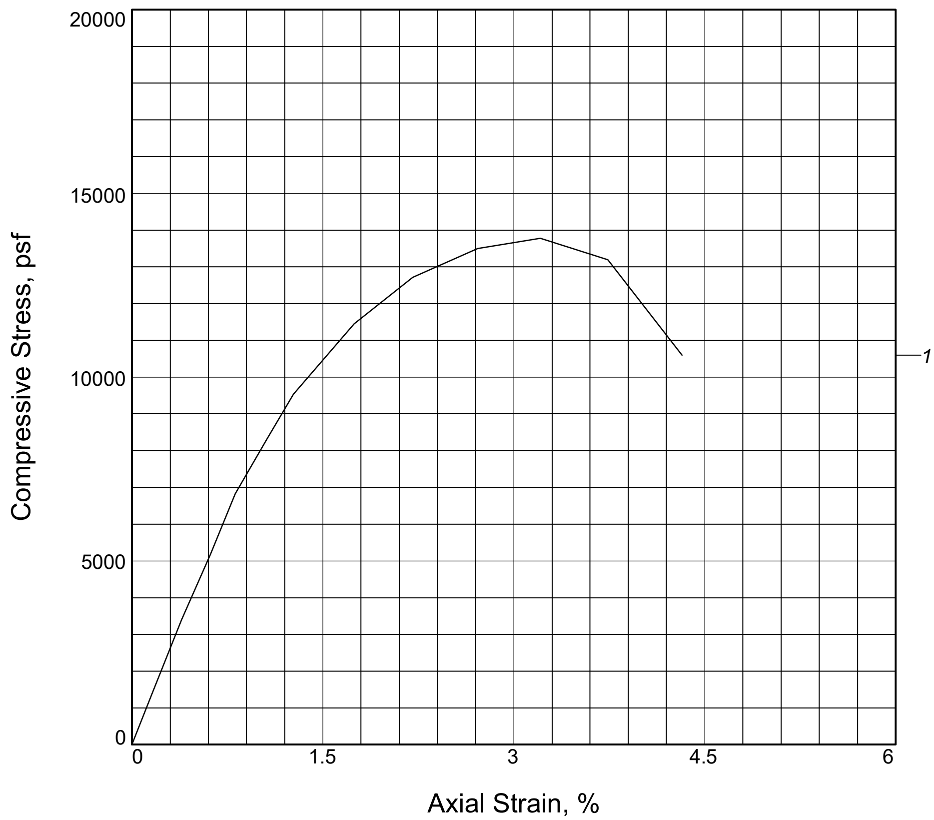
Material tested in accordance with ASTM D6913

Location: 12-2
Sample Number: 17 **Depth:** 13.0' **Date:** 10/18/19

Client: LDK Ventures, LLC
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Project No: VV4376-002 **Figure** 0300-006

Tested By: Jack Bianchin

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	13780			
Undrained shear strength, psf	6890			
Failure strain, %	3.2			
Strain rate, in./min.	0.030			
Water content, %	15.9			
Wet density, pcf	124.0			
Dry density, pcf	107.0			
Saturation, %	79.3			
Void ratio	0.5227			
Specimen diameter, in.	2.41			
Specimen height, in.	5.67			
Height/diameter ratio	2.35			

Description: Brown Sandy Clay (visual)

LL =	**PL =**	**PI =**	**GS=** 2.61	**Type:** Tube

Project No.: VV4376-002

Date Sampled: 10/18/19

Remarks:
Material tested in accordance with ASTM D2166
Type of Failure - Columnar

Figure 0300-007

Client: LDK Ventures, LLC

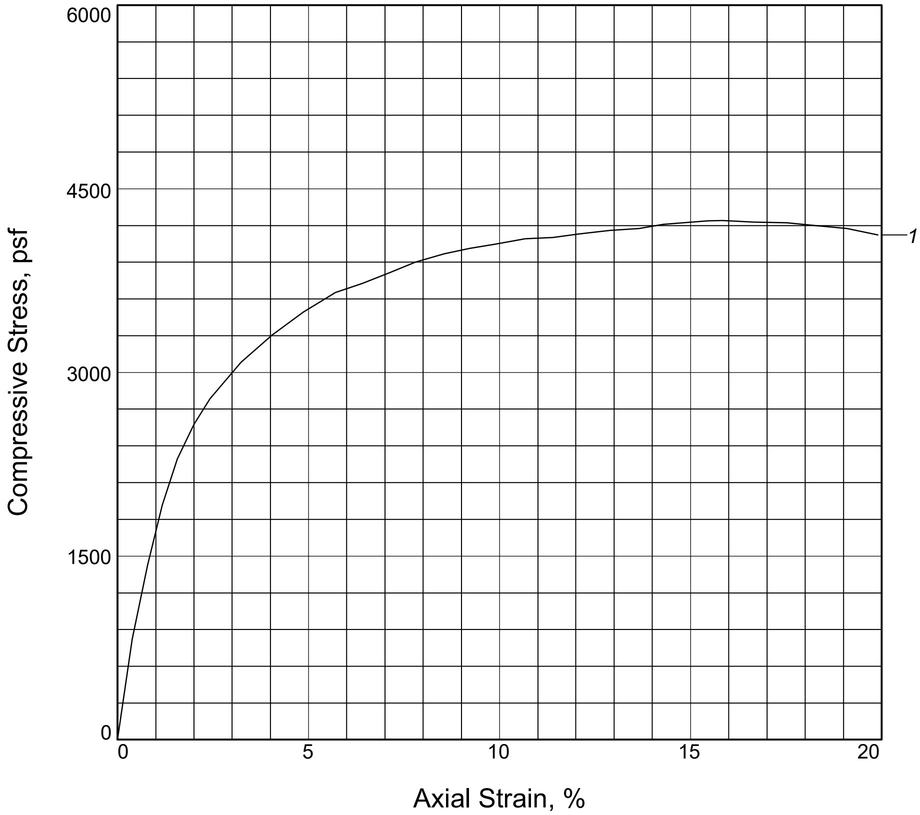
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Location: 11-1

Sample Number: 11 **Depth:** 2.5'

Tested By: Jack Bianchin

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	4240			
Undrained shear strength, psf	2120			
Failure strain, %	15.8			
Strain rate, in./min.	0.050			
Water content, %	21.5			
Wet density, pcf	127.5			
Dry density, pcf	104.9			
Saturation, %	95.9			
Void ratio	0.6061			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Brown Sandy Clay (Visual)

LL = 59	**PL =** 13	**PI =** 46	**GS=** 2.70	**Type:** Tube

Project No.: VV4376-002

Date Sampled: 10/18/19

Remarks:
Material tested in accordance with ASTM D2166
Type of Failure - Shear

Figure 0300-008

Client: LDK Ventures, LLC

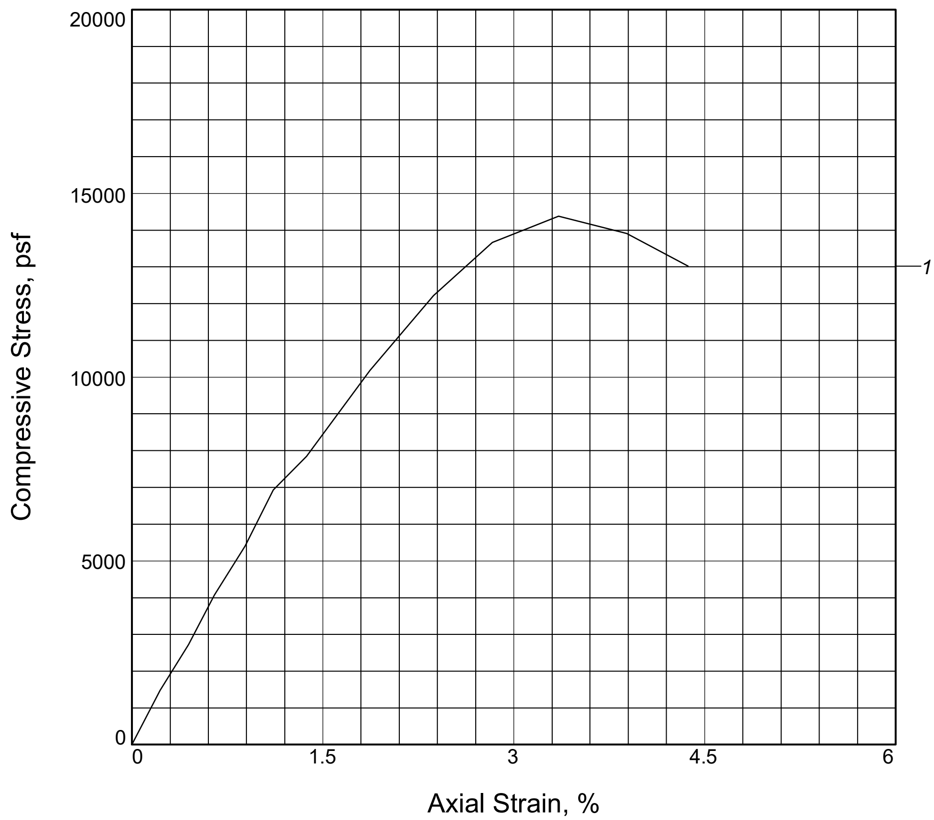
Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Location: 13-1

Sample Number: 19 **Depth:** 2.5'

Tested By: Cindy Gooden

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	14382			
Undrained shear strength, psf	7191			
Failure strain, %	3.4			
Strain rate, in./min.	0.030			
Water content, %	18.2			
Wet density, pcf	138.7			
Dry density, pcf	117.4			
Saturation, %	116.5			
Void ratio	0.4149			
Specimen diameter, in.	2.41			
Specimen height, in.	5.40			
Height/diameter ratio	2.24			

Description: Brown Claystone (visual)

LL = 67	**PL =** 21	**PI =** 46	**GS=** 2.66	**Type:** Tube

Project No.: VV4376-002
Date Sampled: 10/18/19

Remarks:
Material tested in accordance with ASTM D2166
Type of Failure - Columnar

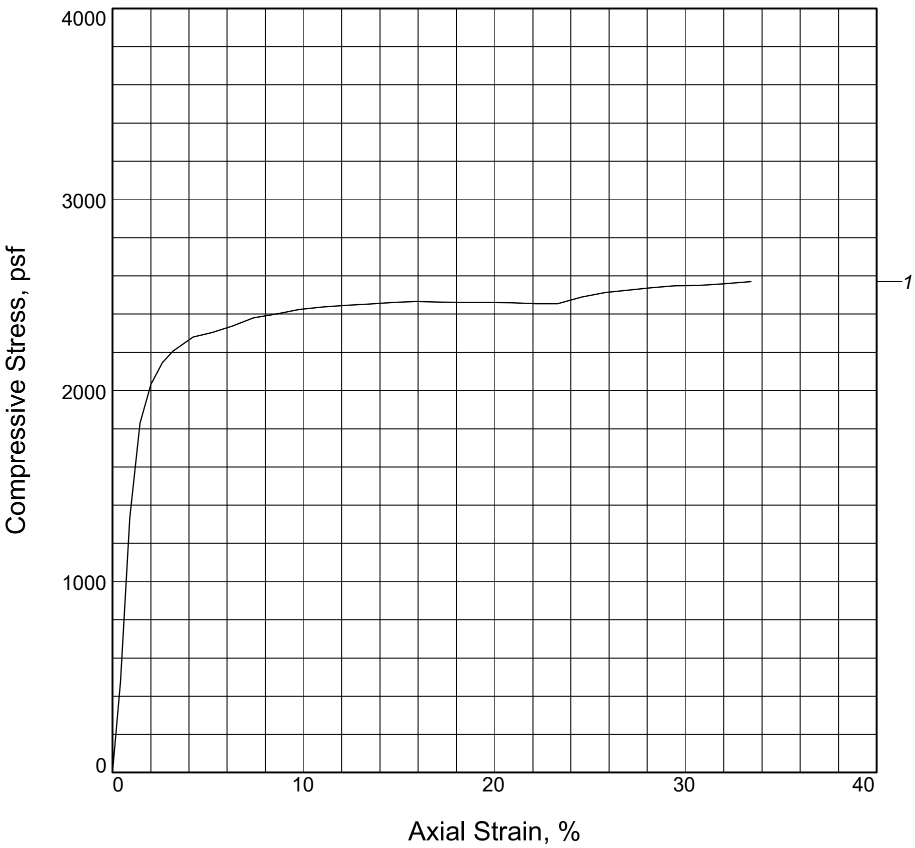
Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Location: 15-1
Sample Number: 25 **Depth:** 2.5'

Figure 0300-009

Tested By: Cindy Godden

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	2571			
Undrained shear strength, psf	1285			
Failure strain, %	33.4			
Strain rate, in./min.	0.075			
Water content, %	25.1			
Wet density, pcf	122.3			
Dry density, pcf	97.7			
Saturation, %	111.4			
Void ratio	0.5459			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Brown Clay with some coarse Sand (visual)

LL =	PL =	PI =	GS= 2.42	Type: Tube

Project No.: VV4376-002
Date Sampled: 10/18/19

Remarks:
Material tested in accordance with ASTM D2166
Type of Failure - Shear

Figure 0300-010

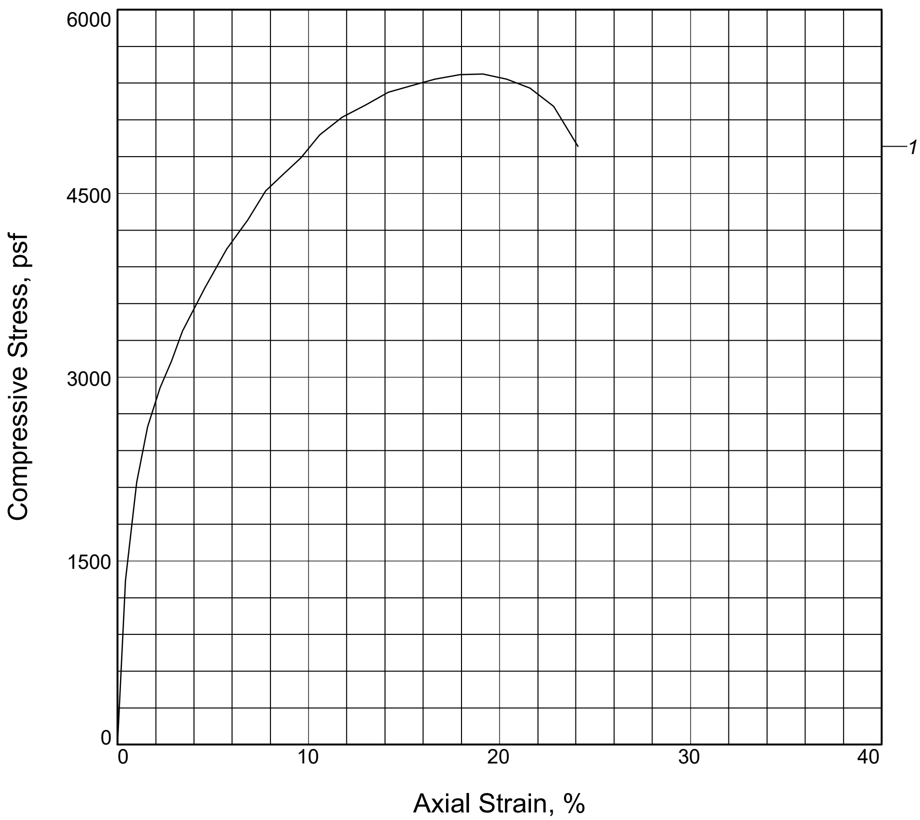
Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Location: 17-1

Sample Number: 32 **Depth:** 3.0'

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	5476			
Undrained shear strength, psf	2738			
Failure strain, %	19.1			
Strain rate, in./min.	0.075			
Water content, %	18.4			
Wet density, pcf	130.1			
Dry density, pcf	109.8			
Saturation, %	99.4			
Void ratio	0.4836			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Brown Sandy Clay (visual)

LL = 45	**PL =** 16	**PI =** 29	**GS=** 2.61	**Type:** Tube

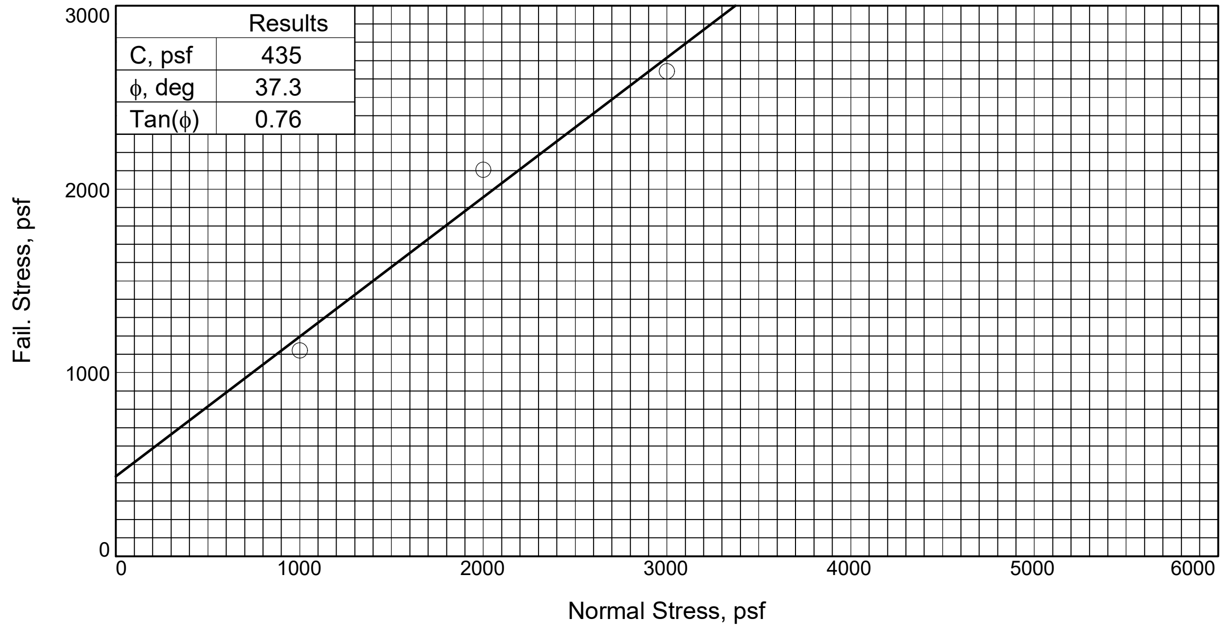
Project No.: VV4376-002
Date Sampled: 10/18/19
Remarks:
Material tested in accordance with ASTM D2166
Type of Failure - Bulge

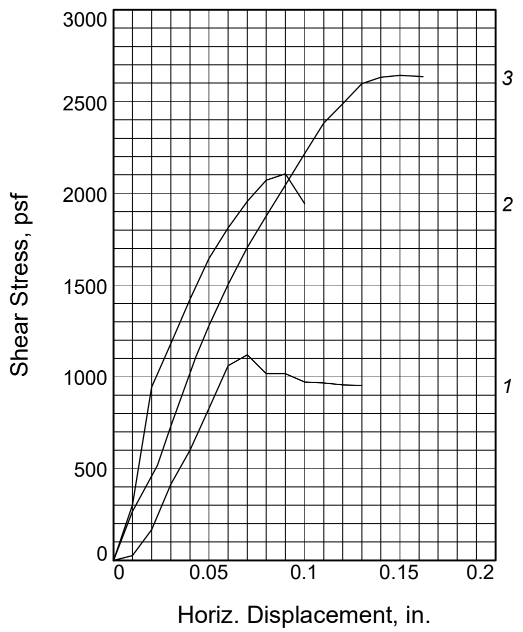
Figure 0300-011

Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Location: 18-1
Sample Number: 36 **Depth:** 3.0'

Tested By: Jack Bianchin





	Sample No.	1	2	3
Initial	Water Content, %	30.1	30.1	30.1
	Dry Density, pcf	87.1	84.1	73.4
	Saturation, %	81.7	76.5	59.9
	Void Ratio	1.0582	1.1294	1.4420
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	35.0	36.4	38.1
	Dry Density, pcf	86.1	78.9	71.7
	Saturation, %	92.8	82.1	73.0
	Void Ratio	1.0812	1.2712	1.4994
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.01	1.07	1.02
	Normal Stress, psf	1000	2000	3000
	Fail. Stress, psf	1121	2106	2642
	Displacement, in.	0.07	0.09	0.15
	Ult. Stress, psf			
	Displacement, in.			
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Tube
Description: Mottled Brown Silty Clay (visual)

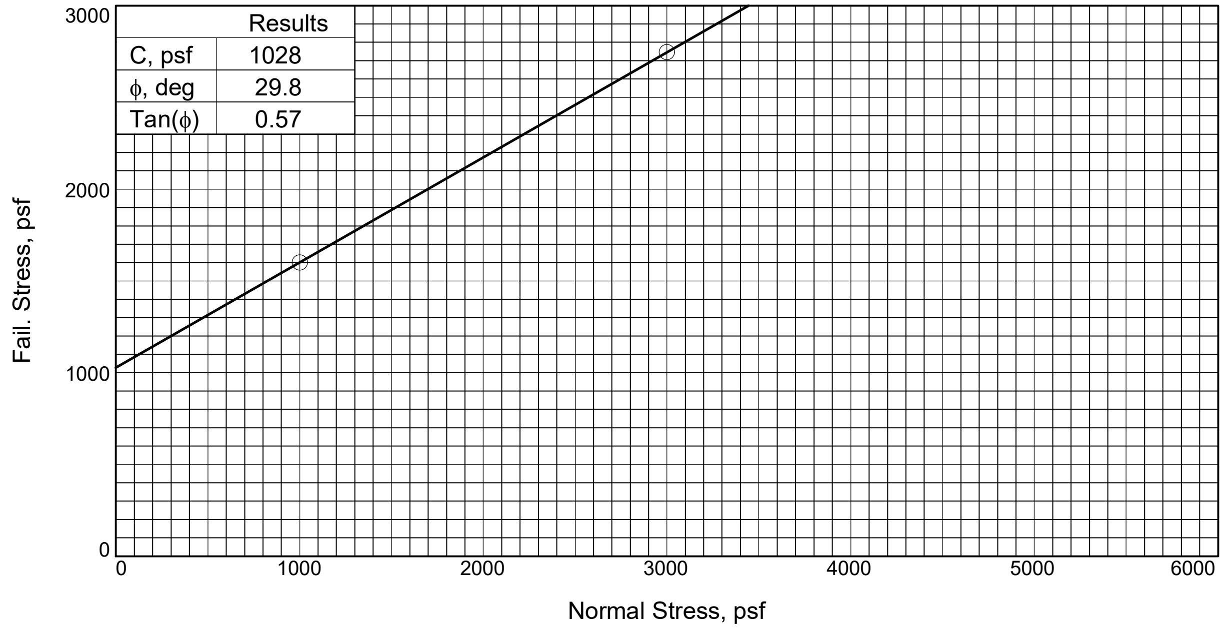
Specific Gravity= 2.87
Remarks: Material tested in accordance with ASTM D3080

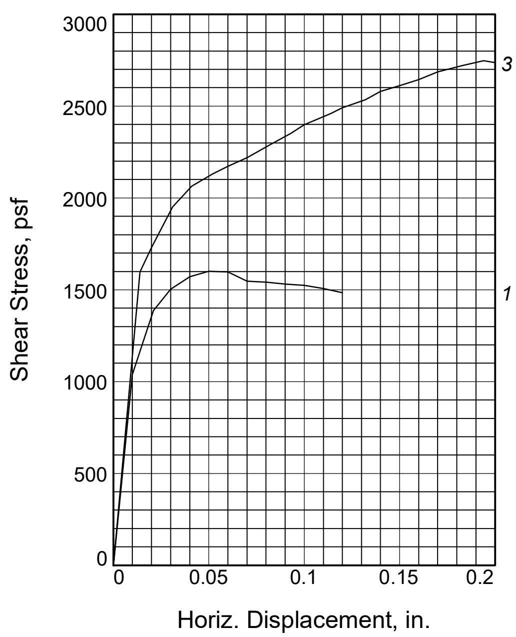
Figure 0300-012

Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Location: 9-1
Sample Number: 2 **Depth:** 3.0'
Proj. No.: VV4376-002 **Date Sampled:** 10/18/19

Tested By: Cindy Gooden





	Sample No.	1	3
Initial	Water Content, %	9.1	9.1
	Dry Density, pcf	121.9	128.5
	Saturation, %	68.8	85.9
	Void Ratio	0.3468	0.2776
	Diameter, in.	2.41	2.41
	Height, in.	1.00	1.00
At Test	Water Content, %	16.9	18.1
	Dry Density, pcf	121.9	123.3
	Saturation, %	128.0	143.2
	Void Ratio	0.3471	0.3316
	Diameter, in.	2.41	2.41
	Height, in.	1.00	1.04
	Normal Stress, psf	1000	3000
	Fail. Stress, psf	1600	2746
	Displacement, in.	0.05	0.19
	Ult. Stress, psf		
	Displacement, in.		
	Strain rate, in./min.	0.002	0.002

Sample Type: Tube
Description: Brown Sandy Mudstone with Gravel (visual)

Specific Gravity= 2.63
Remarks: Material tested in accordance with ASTM D3080

Figure 0300-013

Client: LDK Ventures, LLC

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California
Location: 14-1
Sample Number: 22 **Depth:** 3.0'
Proj. No.: VV4376-002 **Date Sampled:** 10/18/19

Tested By: Cindy Gooden

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client: LDK Ventures, LLC
3140 Peacekeeper Way
McClellan, CA 95652

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Client No: VV4376-002
Report No: 0300-014
Date: 10/18/19
Submitted by: KC Engineering
Date Submitted: 10/02/19

"R" VALUE TEST REPORT
(CTM 301)

Sample: 1
Description: Brown Sandy Clay (visual)
Location: B-13 Subgrade @ 0-3'

SIEVE ANALYSIS

Sieve Size	1-1/2"	1"	3/4"	1/2"	3/8"	#4
As Received (% Pass)	---	---	---	---	---	100
As Used (% Pass)	---	---	---	---	---	100

RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	114.6	14.1	675	0	0	38
2	112.6	15.2	465	0	0	11
3	112.0	16.4	277	0	0	6

R-Value @ 300 PSI Exudation Pressure = **6**

Notes:

Tested by Ricky Mathews

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client: LDK Ventures, LLC
3140 Peacekeeper Way
McClellan, CA 95652

Project: Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Client No: VV4376-002
Report No: 0300-015
Date: 10/18/19
Submitted by: KC Engineering
Date Submitted: 10/02/19

"R" VALUE TEST REPORT
(CTM 301)

Sample: 14
Description: Brown Sandy Clay (visual)
Location: B 18 Subgrade @ 0-3'

SIEVE ANALYSIS

Sieve Size	1-1/2"	1"	3/4"	1/2"	3/8"	#4
As Received (% Pass)	---	---	---	---	---	100
As Used (% Pass)	---	---	---	---	---	100

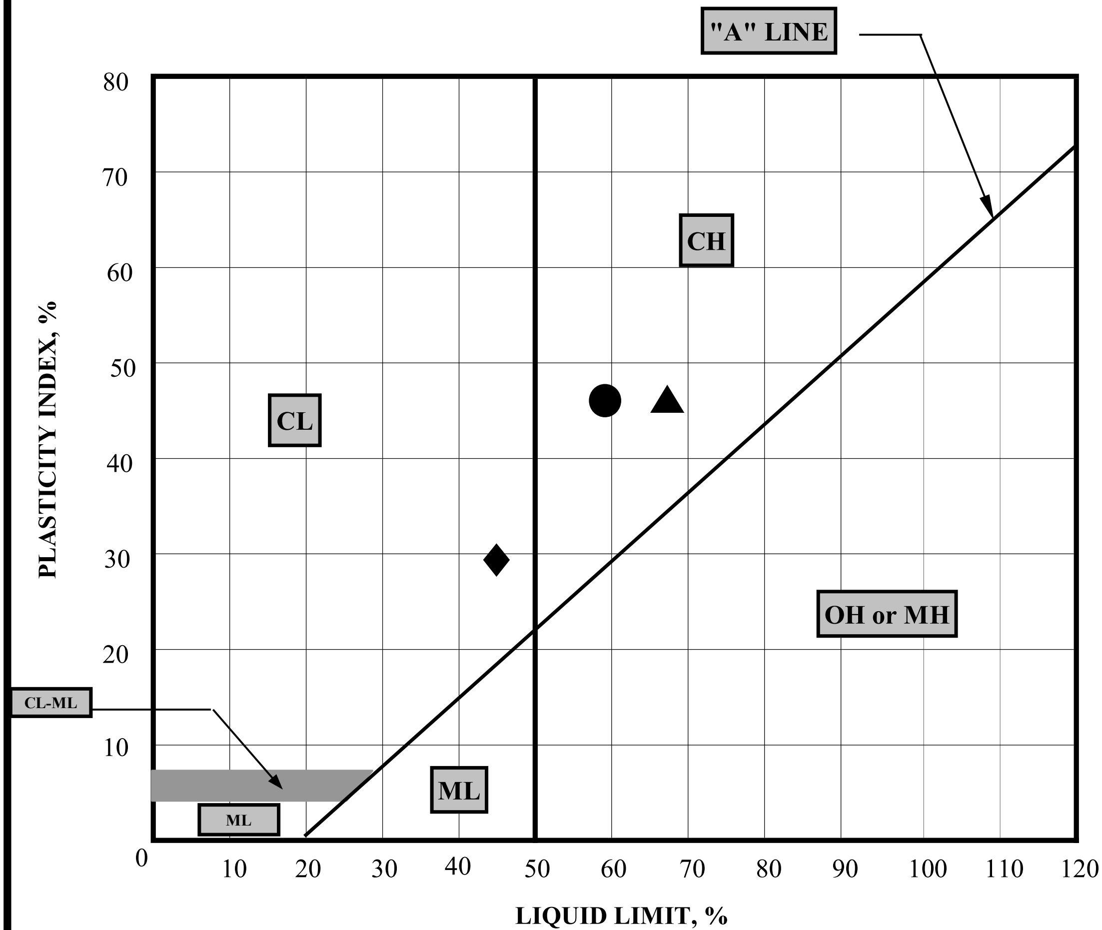
RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	117.5	14.2	654	0	0	49
2	114.1	15.3	430	0	0	31
3	111.5	16.4	239	0	0	14

R-Value @ 300 PSI Exudation Pressure = **20**

Notes:

Tested by Ricky Mathews
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested. Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	13-1	2.5'	N/A	59	13	46	N/A	CL
▲	15-1	2.5'	N/A	67	21	46	N/A	CH
◆	18-1	3.0'	N/A	45	16	29	N/A	CL

Note: Atterberg Limits tested in accordance with ASTM D4318.

MATERIALS TESTING INCORPORATED M T I

Materials Testing, Inc.

PLASTICITY CHART AND DATA

Proposed Industrial Building
700 Crocker Drive, Vacaville, California

Client No:	Date:	Report No:
VV4376-002	10/18/2019	0300-016

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/27/2017
Date Submitted 10/23/2017

To: Dave Cymanski
K.C. Engineering
8798 Airport Road
Redding, CA 96002

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : VV4376 Site ID : 34 @ 0-5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 75495-157583.

EVALUATION FOR SOIL CORROSION

Soil pH	6.57			
Minimum Resistivity	1.10	ohm-cm (x1000)		
Chloride	6.5	ppm	00.00065	%
Sulfate	15.3	ppm	00.00153	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2019
Date Submitted 10/03/2019

To: David Cymanski
K.C. Engineering
865 Cotting Lane Suite A
Vacaville, CA 95688

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : VV4376 Site ID : 18-1A.
Thank you for your business.

* For future reference to this analysis please use SUN # 80705-168647.

EVALUATION FOR SOIL CORROSION

Soil pH	5.82	
Minimum Resistivity	3.48 ohm-cm (x1000)	
Chloride	3.2 ppm	0.00032 %
Sulfate-SO4	11.6ppm	0.00116 %

METHODS

pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate-SO4 ASTM C1580, Chloride CA DOT Test #422m

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2019
Date Submitted 10/03/2019

To: David Cymanski
K.C. Engineering
865 Cotting Lane Suite A
Vacaville, CA 95688

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : VV4376 Site ID : 13-1A.
Thank you for your business.

* For future reference to this analysis please use SUN # 80705-168646.

EVALUATION FOR SOIL CORROSION

Soil pH	5.74		
Minimum Resistivity	0.88 ohm-cm (x1000)		
Chloride	16.1 ppm	0.00161	%
Sulfate-SO4	38.0ppm	0.00380	%

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate-SO4 ASTM C1580, Chloride CA DOT Test #422m

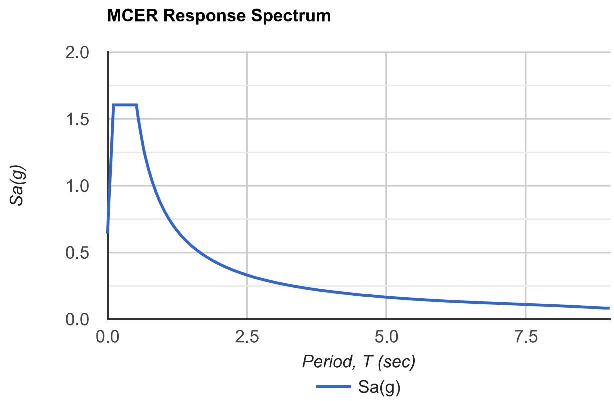
700 Crocker Dr., Vacaville

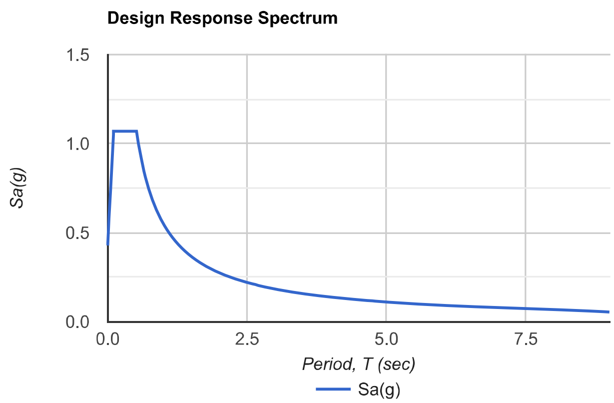
Latitude, Longitude: 38.4143, -121.9487

Date	11/5/2019, 9:52:53 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.605	MCE_R ground motion. (for 0.2 second period)
S_1	0.55	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.605	Site-modified spectral acceleration value
S_{M1}	0.825	Site-modified spectral acceleration value
S_{DS}	1.07	Numeric seismic design value at 0.2 second SA
S_{D1}	0.55	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.584	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.584	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.605	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.618	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.685	Factored deterministic acceleration value. (0.2 second)
S1RT	0.55	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.532	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.636	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.992	Mapped value of the risk coefficient at short periods
C_{R1}	1.035	Mapped value of the risk coefficient at a period of 1 s





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700 Crocker Dr., Vacaville

Latitude, Longitude: 38.4143, -121.9487

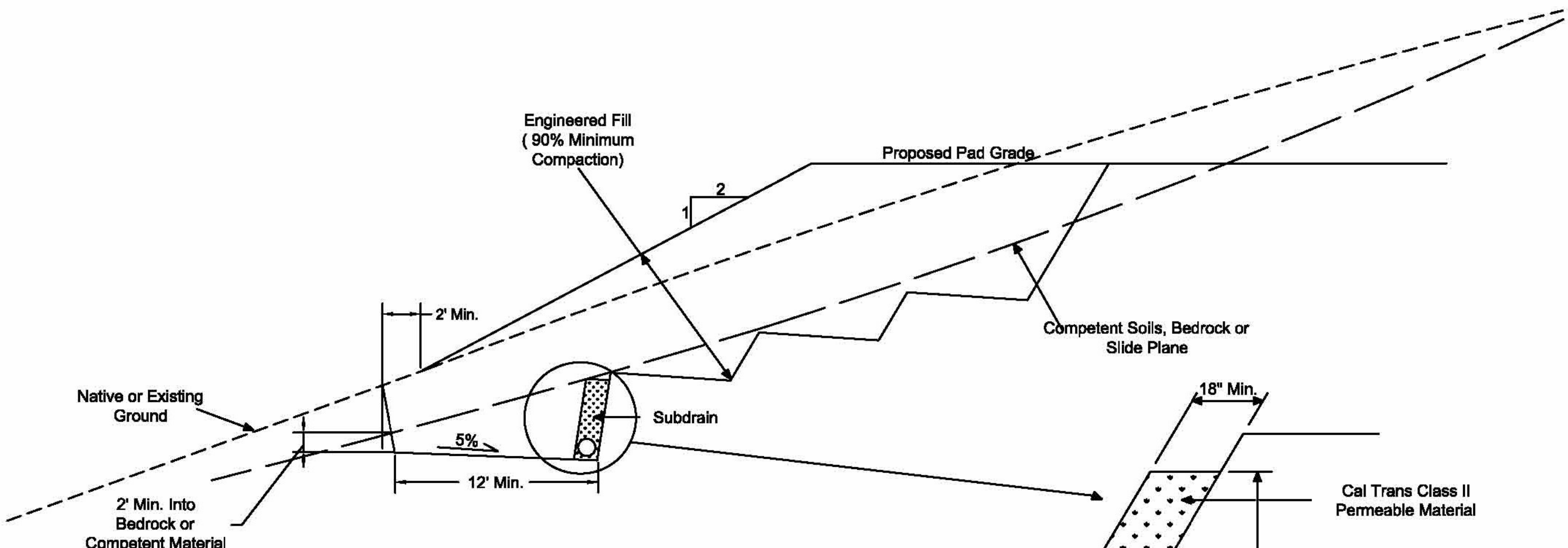
Date	11/5/2019, 9:55:29 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

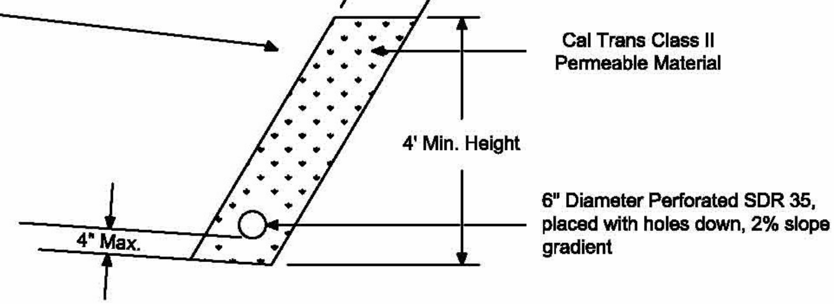
Type	Value	Description
S_S	1.248	MCE_R ground motion. (for 0.2 second period)
S_1	0.448	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.249	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.833	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.001	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.522	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.574	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.248	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.353	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.292	Factored deterministic acceleration value. (0.2 second)
S1RT	0.448	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.483	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.765	Factored deterministic acceleration value. (1.0 second)
PGAd	0.928	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s

TYPICAL FILL SLOPE & SUBDRAIN DETAILS

(Not to Scale)





SUBDRAIN DETAIL

NOTES:

1. This is a typical cross-section for fill slope construction. Significant variations may be encountered during grading.
2. Dimensions of keyways and benches are subject to approval in the field by the Soil Engineer and Geologist.
3. The number and location of subdrains will be determined by the Soil Engineer and Geologist in the field.

KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688

Proposed Hillside Fill Slope
TYPICAL FILL SLOPE, KEYWAY, BENCHING & SUBDRAIN DETAILS