

LKQ Adelanto Expansion (PROJ-2020-00030)

Gyant Compliance



29982 Ivy Glenn Drive Ste 201

Laguna Niguel Ca 92677

Date: 5/26/2018

County of San Bernardino
Planning and Building Department
Attn: County Engineer

RE: LKQ Adelanto Geology, Soils and Geotechnical Reports
Plan Check No.: COMM-1-18-43607
Project Location: Phase I & II – 11200 Alden Road, Adelanto, Ca 92301 (2015 to 2017 Year)
Phase III – 11010 Calleja Road, Adelanto, Ca 92301 (2016 to 2018 Year)

To the County Engineer

Geology, Soils and Geotechnical Reports covered Phase I Building Structure. Phase II general site grading for salvage vehicle storage yard and Phase III general site grading for salvage vehicle storage yard with no structure buildings or pavements. Phase III is a smaller part of this project which has been going since April 2015 to present. With the Land Annexation between the County and City of Adelanto through LAFCO there will be another 2 to 3 phases still to complete.

Attached is a geology, soil and geotechnical report for all phases of the project.

Let me know if you need any further information. You can contact me on my cell phone at 949 677-7862 or by email at jgypsyn@gyantgroup.com.

Sincerely

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Cc: LKQ Corporation



CHJ Consultants

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January 23, 2016

Job No. 14852-3

Engineering and Consulting Alliance
1740 Dell Range Boulevard, Suite 454H
Cheyenne, Wyoming 82009
Attention: Mr. Phillip Regeski

Dear Mr. Regeski:

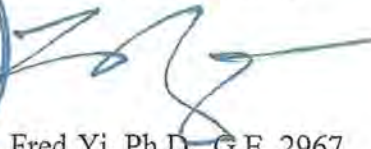
Attached herewith is the Geotechnical Investigation report prepared for the proposed auto dismantling facility improvements and land expansion (44-acres site), to be located at 11200 Alden Road, Adelanto, California.

This report was based upon a scope of services generally outlined in our proposal to perform geotechnical and percolation investigations, dated November 19, 2015, and other written and verbal communication.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.



Respectfully submitted,
CHJ CONSULTANTS


Fred Yi, Ph.D., G.E. 2967
Chief Engineer

FY:1b

**GEOTECHNICAL INVESTIGATION
PROPOSED IMPROVEMENTS
AUTO DISMANTLING FACILITY
ADELANTO, CALIFORNIA
PREPARED FOR
ENGINEERING AND CONSULTING ALLIANCE
JOB NO. 14852-3**

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GEOTECHNICAL INVESTIGATION
PROPOSED IMPROVEMENTS
AUTO DISMANTLING FACILITY
ADELANTO, CALIFORNIA
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INTRODUCTION

During January of 2016, a geotechnical investigation for the proposed improvements and land expansion (44-acre Calleja Road site) at the auto dismantling facility, located at 11200 Alden Road, Adelanto, California, was performed by this firm. The purposes of this investigation were to explore and evaluate the geotechnical engineering conditions at the subject site and to provide geotechnical engineering recommendations for design and construction of the proposed improvements.

To orient our investigation at the site, an electronic copy of an 80-scale boundary and topographic survey plan, dated November 2014, was furnished for our use. The Google Earth aerial photograph for the area was also utilized to orient our investigation. The approximate location of the site is shown on the attached Index Map (Enclosure "A-1").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this updated report included the following:

- Review of published and unpublished literature and maps
- Geologic field reconnaissance of the site and surrounding area
- Marking of exploration locations in the field and notification of Underground Service Alert
- Placement of three exploratory borings on the site

- Logging and sampling of exploratory borings for testing and evaluation
- Laboratory testing on selected samples
- Evaluation of the geotechnical data to develop site-specific recommendations for site grading, foundation design, preliminary asphalt concrete (AC) and Portland cement concrete (PCC) pavement structural section design, and mitigation of potential geotechnical concerns and hazards such as liquefaction and seismic settlement.

PROJECT CONSIDERATIONS

It is our understanding that the approximately 30-acre existing site is in use as an auto dismantling facility. Approximately 51,000 square feet of steel-framed metal buildings are proposed to be constructed on the southwest portion of the site. Included is another 44-acre for expansion directly north located at Calleja Road for temporary vehicle storage only and no building structures. In addition, a leach field septic system is proposed adjacent to the buildings. Associated infrastructure includes loading ramps, driveways and parking spaces.

A percolation investigation was also performed for the leach field septic system and is reported under separate cover.

Project grading plans were not available at the time of our investigation. However, observation of site topography and of adjacent developments indicates that development of this site will entail minor cuts and fills. The final project grading plan should be reviewed by the geotechnical engineer.

SITE DESCRIPTION

The 30-acre site is located approximately 1/4 mile east of Highway 395 at 11200 Alden Road in the city of Adelanto, California and the other 44-acre land expansion for temporary vehicle storage located at Calleja Road between City and County Land. At the time of our investigation, the southwest corner of the site was occupied by an approximately 5,000-square-foot structure and PCC and AC pavement. The remaining acreage of the site was unpaved. Rows of stacked automotive wheels were noted throughout the site. No significant vegetation was noted. Overhead electrical lines were located

along Alden Road, ending at the southwest corner of the site. The site is relatively flat and planar and slopes downward to the northwest at an overall gradient of less than 2 percent.

The site is generally surrounded by vacant desert land with natural vegetation. A residential property that includes three structures is located south of Alden Road.

Aerial imagery dated 1994 to 2013 indicates that the facility has occupied the site since 1994. The southwest corner of the site has been occupied by a structure throughout this time period. No features suggesting geologic hazard, including faulting, landslide or flooding, were noted in the imagery examined.

FIELD INVESTIGATION

The soil conditions underlying the subject site were explored by means of three exploratory borings drilled to a maximum depth of 51-1/2 feet below the existing ground surface (bgs) with a truck-mounted CME 75 drill rig equipped for soil sampling. The approximate locations of our exploratory borings are indicated on the attached Site Plan (Appendix "A").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Both a standard penetration test (SPT) sampler (2-inch outer diameter and 1-3/8-inch inner diameter) and a modified California ring sampler (3.25-inch outer diameter and 2.42-inch inner diameter) were utilized in our investigation. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The sampler was driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, the sampler was advanced up to 18 inches, providing up to three sets of blowcounts at each sampling interval. The recorded blows are raw numbers *without* any corrections for hammer type (automatic vs. manual cathead) or sampler size (ring sampler vs. SPT sampler). Both relatively

undisturbed and bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our uncorrected blowcount data and in-place density data, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

At the completion of drilling, all borings were backfilled to the initial grade of the boring with soil boring cuttings and tamped using the drilling augers. It is possible that some settlement of the backfilled material may occur. Our firm will not monitor boring locations for any settlement. This is deemed to be and is accepted to be the responsibility of our client.

LABORATORY INVESTIGATION

Included in our laboratory testing program were moisture content tests on all samples returned to the laboratory and dry density tests on all relatively undisturbed samples. The results are included on the exploratory boring logs. An optimum moisture content - maximum dry density relationship was established for typical soil types in order that the relative compaction of the subsoils might be evaluated. Direct shear testing was performed on a selected relatively undisturbed sample in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. A consolidation test was performed on a selected sample in order to provide parameters for settlement and hydroconsolidation evaluations. Sieve analyses were performed on selected samples of soil for classification purposes. Sand equivalent and R-value tests were performed on a probable subgrade soil to develop criteria for on-site parking and drive area pavement design recommendations. A selected sample of material was delivered to HDR for preliminary corrosivity analysis.

Laboratory test results appear in Appendix "C". Soil classifications provided in our geotechnical investigation are as per the Unified Soil Classification System.

SUBSURFACE SOIL CONDITIONS

Dibblee (1960) indicates the site as being underlain by native alluvial soils. As encountered in our exploratory borings, the site is mantled by fill materials to depths of 2 to 3 feet bgs. The fill consists primarily of silty sand. The fill is underlain by alluvium to the maximum 51-1/2-foot depth attained in the borings. The alluvium generally consists of silty sands and sandy silts with local sand lenses.

Groundwater was not encountered within the recent exploratory borings to the maximum depth attained of approximately 51-1/2 feet bgs.

The soils encountered were generally granular and considered to be non-critically expansive.

Bedrock was not encountered within any of the exploratory borings.

Refusal was not experienced in any of the exploratory borings utilized for the investigation.

More detailed descriptions of the subsurface soil conditions encountered are included within our exploratory boring logs (Appendix "B").

SEISMIC DESIGN PARAMETERS

Based on the geologic setting and anticipated earthwork for construction of the proposed project, the soils underlying the site are classified as Site Class "D, stiff soil", according to the 2013 California Building Code (CBC). The seismic design parameters according to CBC are summarized in the following table.

2013 CBC Seismic Design Parameters	
Mapped Spectral Acceleration Parameters	$S_s = 1.27$ and $S_1 = 0.52$
Site Coefficients	$F_a = 1.0$ and $F_v = 1.5$
Adjusted Maximum Considered Earthquake Spectral Response Parameters	$S_{MS} = 1.27$ and $S_{M1} = 0.77$
Design Spectral Acceleration Parameters	$S_{DS} = 0.85$ and $S_{D1} = 0.52$
Design Peak Ground Acceleration	0.48

GROUNDWATER AND LIQUEFACTION

Available groundwater data were reviewed in order to provide an estimate of the historical groundwater conditions for the site.

Groundwater data from State of California Department of Water Resources (2015) is summarized in the following table.

1/2-mile WNW

Well No.	Date of Measurement	Depth to Water (feet)	Location
06N05W05P001S	3-8-1994	75	1 mile NW
06N05W08F005S	5-17-1989	76	1 mile NW
06N05W08F004S	1-17-1958	77	1/2 mile WNW
	3-29-2007	70	

Groundwater was not encountered within the maximum 51-1/2-foot depth attained in our exploratory borings at the site. Based on the data examined, the historic high depth to water beneath the site is in excess of 70 feet bgs.

Based on the depth of groundwater and density of soils encountered, liquefaction and associated ground deformation is not considered a hazard to the site, and further evaluation is not warranted.

SEISMIC SETTLEMENT

Calculations were performed to evaluate the potential seismic settlement of the soils underlying the site by the method proposed by Tokimatsu and Seed (1987) and Pradel (1998). A peak horizontal ground acceleration of 0.48g and a deaggregated earthquake magnitude of 7.79 (U.S.G.S., 2013) were utilized as input into the geotechnical analysis program GeoSuite® Version 2.2 (Yi, 2014). The results of the dry sand settlement are included in Enclosure "D-1". The results indicate the potential seismic-induced settlement is less than 1/10 inch. It is our opinion that both total and differential seismic settlements are insignificant for this site.

CONCLUSIONS

On the basis of our field and laboratory investigations, it is the opinion of this firm that the proposed development is feasible from a geotechnical engineering standpoint, provided the recommendations contained in this report are implemented during grading and construction.

Moderate to severe seismic shaking can be expected at the site.

Groundwater was not encountered within the maximum 51-1/2-foot depth attained in our exploratory borings. Historic high groundwater is estimated to be greater than 70 feet bgs at the site. Liquefaction is not considered to be a potential hazard to the site.

Refusal to further advancement of the drilling augers was not experienced.

Although caving was not experienced within the exploratory borings utilized for this investigation, trenches, larger diameter borings or excavations that remain open for longer periods of time may be subject to significant caving.

Bedrock was not encountered in any of our exploratory borings.

Fill was encountered to depths between 2 and 3 feet below existing grade.

Based on the classification and density of soils and the lack of significant soil cementation exhibited by the soils encountered in our exploratory borings, site grading and utility trenching are expected to be feasible with conventional heavy grading and trenching equipment, respectively.

Conditions conducive to land sliding or lateral spreading (lurching) are not present at the site. No significant slopes are proposed.

The on-site soils encountered during this investigation are generally granular and considered non-critically expansive.

Settlement resulting from seismic shaking of dry to partially saturated sands may result in total settlement of less than 1/10 inch. The maximum seismic differential settlement and hydroconsolidation settlement are considered to be negligible.

Based upon our field investigation and test data, it is our opinion that the upper native soils and/or undocumented fills will not, in their present condition, provide uniform or adequate support for the proposed structure. Our density testing indicated variable in situ conditions of the native soils, ranging from medium dense to dense states.

Because of site conditions and in order to assist in the identification of undocumented fill, it will be necessary to remove a minimum of the upper 2 feet of existing soil in all areas to be graded. All existing undocumented fill should then be completely removed. Further subexcavation may be necessary depending on the density of the underlying soils. To provide adequate support for the proposed structures, it is our recommendation that the building area be further subexcavated as necessary and recompact to provide a compacted fill mat beneath footings and slabs. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with a compacted fill mat.

RECOMMENDATIONS

DESIGN ACCELERATION PARAMETERS:

Based on the geologic setting and anticipated earthwork for construction of the proposed project, the soils underlying the site are classified as Site Class "D", according to the 2013 CBC and ASCE 7-10. The design acceleration parameters are summarized in the following table.

2013 CBC Seismic Design Parameters	
Mapped Spectral Acceleration Parameters	$S_s = 1.27$ and $S_1 = 0.52$
Site Coefficients	$F_a = 1.0$ and $F_v = 1.5$
Adjusted Maximum Considered Earthquake Spectral Response Parameters	$S_{MS} = 1.27$ and $S_{M1} = 0.77$
Design Spectral Acceleration Parameters	$S_{DS} = 0.85$ and $S_{D1} = 0.52$
Design Peak Ground Acceleration	0.48

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site pre-job meeting with the property owner, the contractor and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the 2013 CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped of man-made obstructions and other deleterious materials. These materials should be removed from the site for disposal. Any existing utility lines should be traced, removed and rerouted from the building area.

In order to assist the identification of undocumented fill, it is recommended that the upper 2 feet of native soils be removed in all areas to be graded. All existing undocumented fill should then be completely removed. Further subexcavation may be necessary depending on the density of the underlying soils. The actual depth of removal should be determined at the time of grading by the project geotechnical engineer/geologist. The determination will be based on soil conditions exposed in the excavation. The bottoms of the excavations should be observed and approved by the project geologist. The on-site soils may be reused as compacted fill.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment and backfilled as recommended for site fill.

PREPARATION OF FILL AREAS:

Prior to placing fill, and after the mandatory sub excavation operation, the surfaces of all areas to receive fill should be scarified to a depth of 6 inches or more. The scarified soils should be brought to near optimum moisture content and recompact to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

PREPARATION OF FOOTING AREAS:

All footings should rest upon at least 24 inches of properly compacted fill material. In areas where the required thickness of compacted fill is not accomplished by site rough grading and the mandatory subexcavation operation, the footing areas should be subexcavated to a depth of 24 inches or more below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines, where possible. The bottom of this excavation should then be scarified to a depth of at least 6 inches, brought to near optimum moisture content and recompact to a minimum of 90 percent relative compaction in accordance with ASTM D1557 prior to refilling the excavation to grade as properly compacted fill.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials.

Import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 6 inches in maximum dimension. The contractor shall notify the geotechnical engineer of import sources sufficiently ahead of their use so that the sources can be observed and approved as to the physical characteristic of the import material. For all import material, the contractor shall also submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current (ACI) criteria and is not corrosive to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the job.

Fill should be spread in near-horizontal layers, approximately 8 inches thick. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to near-optimum moisture content and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

SHRINKAGE AND SUBSIDENCE:

Based upon the relative compaction of the native soils tested during this investigation and the relative compaction anticipated for compacted fill soils, we estimate compaction shrinkage of approximately 5 to 10 percent. Therefore, 1.05 cubic yards to 1.10 cubic yards of in-place soil material would be necessary to yield 1 cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 feet. These values are exclusive of losses due to stripping, tree removal or the removal of other subsurface obstructions, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made to accommodate possible variations in actual quantities during site grading.

FOUNDATION DESIGN:

If the site is prepared as recommended, the proposed warehouse may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 24 inches of compacted fill. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 12 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil bearing pressure of 1,600 pounds per square foot (psf) for dead plus live loads. This allowable bearing pressure may be increased by 400 psf for each additional foot of width and by 1,100 psf for

each additional foot of depth to a maximum safe soil bearing pressure of 4,500 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, static settlement analysis indicates a potential total settlement on the order of 1 inch, depending on loading. Differential static and seismic settlement of no more than 1/2 inch in 50 feet should be anticipated.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 400 psf per foot of depth. Base friction may be computed at 0.39 times the normal load. Base friction and passive earth pressure may be combined without reduction.

Other than conservative soil modeling, the lateral passive earth pressure and base friction values recommended do not include factors of safety. If the design is to be based on allowable lateral resistance values, we recommend that minimum factors of safety of 1.5 and 2.0 be applied to the friction coefficient and passive lateral earth pressure, respectively. The resulting allowable lateral resistance values are:

	Ultimate	Allowable	Factor of Safety
Passive Lateral Earth Pressure (psf/ft)	400	200	2.0
Base Friction Coefficient	0.39	0.26	1.5

For preliminary retaining wall design, a lateral active earth pressure developed at a rate of 40 psf per foot of depth should be utilized for unrestrained conditions.

For restrained conditions, an at-rest earth pressure of 60 psf per foot of depth should be utilized. The "at-rest" condition applies toward braced walls that are not free to tilt. The "active" condition applies toward unrestrained cantilevered walls where wall movement is anticipated. The structural designer should use judgment in determining the wall fixity and may utilize values interpolated between the "at-rest" and "active" conditions where appropriate.

For walls 10 feet high or less, a uniform construction surcharge load of 72 psf or an alternative traffic surcharge load of 100 psf should be applied in addition to active earth pressure. If the wall is higher than 10 feet, a uniform construction surcharge load of 72 psf or an alternative traffic surcharge load of 100 psf should be applied only up to 10 feet. The resulting additional surcharge pressure should be applied to the wall as a rectangular distribution, from top to bottom, or 10 feet, whichever is smaller.

These values are based on backfills with on-site materials compacted to 90 percent of relative compaction and should be verified prior to construction. These values are applicable only to level, properly drained backfill with no additional surcharge loadings and do not include a factor of safety other than conservative modeling of the soil strength parameters. *If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters.* If import material is to be utilized for backfill, an engineer from this firm should verify the backfill has equivalent or superior strength values.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SEISMIC LATERAL EARTH PRESSURE:

The seismic earth pressure acting on a cantilevered retaining wall was calculated by the Mononobe-Okabe ("M-O") method (Okabe, 1926; Mononobe and Matsuo, 1929). According to AASHTO (LRFD Bridge Design Specifications, Sixth Edition, 2012, Section C11.8.6.2 and A11.3.2), the resulting pseudostatic horizontal seismic coefficient, k_h , could be reduced by 50 percent when 1.0 to 2.0 inches of permanent ground deformation is permitted during the design seismic event, i.e., the pseudostatic horizontal seismic coefficient (k_h) be taken equal to one-half of the peak ground acceleration, which equates to 0.24g. The pseudostatic vertical seismic coefficient (k_v) is usually taken as 0.0g in accordance with AASHTO (2012). For retaining walls with on-site soils as backfill materials, a unit weight of 125.5 pounds per cubic foot (pcf) and a friction angle of 32 degrees were utilized in the calculation. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to level, properly drained backfill with no additional surcharge loadings.

The total lateral active seismic earth pressure (including static active earth pressure) developed at a rate of 60 psf per foot of depth should be utilized for unrestrained conditions. A triangular distribution of total seismic earth pressure should be used in the design (Atik & Sitar, 2010).

The above lateral earth pressures are for level backfill. If inclined backfills are proposed, this firm should be contacted.

SLABS-ON-GRADE:

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 24 inches of compacted soil. Concrete slabs-on-grade should be a minimum of 4 inches in thickness. The soil should be compacted to 95 percent relative compaction. The final pad surfaces should be rolled to provide smooth, dense surfaces.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute 302.1R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder/barrier should comply with ASTM E1745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage. Per the Portland Cement Association (www.cement.org/tech/cct_con_vapor_retarders.asp), for slabs with vapor-sensitive coverings, a layer of dry, granular material (sand) should be placed under the vapor retarder/barrier. For slabs in humidity-controlled areas, a layer of dry, granular material (sand) should be placed above the vapor retarder/barrier.

A modulus of vertical subgrade reaction of 115 ksf/ft can be utilized in the design of slabs-on-grade for the proposed structure.

EXPANSIVE SOILS:

As all soils materials encountered during this investigation were sufficiently granular to be non-critically expansive, the need for specialized construction procedures to specifically resist expansive soil forces is not anticipated at this time. Requirements for reinforcing steel to satisfy structural criteria are not affected by this recommendation. Additional evaluation of soils for expansion potential should be conducted by the soils engineer during the grading operation.

SOIL CORROSION:

A selected sample of material was delivered to our subconsultant, HDR, for soil corrosivity testing. Laboratory testing consisted of pH, resistivity and major soluble salts commonly found in soils. The results of the laboratory tests appear in Enclosure "C-9". These tests have been performed in order to screen the site for potentially corrosive soils.

Resistivity values from the soil tested are considered "mildly corrosive" to "moderately corrosive" to ferrous metals at the site at as-received and saturated conditions, respectively. Specific corrosion

control measures, such as coating of pipe with non-corrosive material or alternative non-metallic pipe material, are considered to be needed if there is potential of saturation.

Results of the soluble sulfate testing indicate a "not applicable" (Class S0) anticipated exposure to sulfate attack, as indicated on the enclosed test results. Based on the criteria from Table 4.2.1 of the American Concrete Institute Manual of Concrete Practice (2011), no special measures, such as specific cement types or water-cement ratios, will be needed for this "not applicable" exposure to sulfate attack.

Soluble chloride content of soil was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

Testing indicated that both ammonium and nitrate contents are not considered corrosive to copper.

CHJ Consultants does not practice corrosion engineering. If further information concerning the corrosion characteristics or if interpretation of the results submitted herein is required, then a competent corrosion engineer should be consulted.

PRELIMINARY FLEXIBLE PAVEMENT DESIGN:

The following recommended structural sections were calculated based on traffic indices provided in the Caltrans Highway Design Manual for Safety Roadside Rest Areas (Caltrans, 2012). Based upon our preliminary sampling and testing, the structural sections tabulated below should provide satisfactory asphalt concrete pavement.

Usage	Traffic Index	R-Value	Recommended Structural Section
Auto Parking Areas	5.0	20	0.25' HMA/0.60' Class 2 AB
Auto Roads	5.5	20	0.25' HMA/0.75' Class 2 AB
Truck Parking Areas	6.0	20	0.30' HMA/0.80' Class 2 AB
Truck Ramps and Roads	8.0	20	0.40' HMA/1.15' Class 2 AB

HMA = hot mix asphalt

AB = aggregate base

The above structural sections are predicated upon proper compaction of the utility trench backfills and the subgrade soils, with the upper 6 inches of subgrade soils and all aggregate base material brought to a minimum relative compaction of 95 percent in accordance with ASTM D1557 prior to paving. The aggregate base should meet Caltrans requirements for Class 2 base.

It should be noted that the above pavement designs were based upon the results of preliminary sampling and testing and should be verified by additional sampling and testing during construction when the actual subgrade soils are exposed.

CHJ Consultants does not practice traffic engineering. The traffic indices used to develop the recommended pavement sections are typical for projects of this type. We recommend that the project civil engineer or traffic engineer review the traffic indices to verify that they are appropriate for this project.

PRELIMINARY RIGID PAVEMENT DESIGN:

Based on an R-value of 20, which correlates to a modulus of subgrade reaction of approximately 107 pounds per square inch per inch (k), we recommend the following PCC pavement designs. This

design is based on the American Concrete Institute Guide for the Design and Construction of Concrete Parking Lots (ACI 330R-08).

Design Area	Recommended Section
Car Parking and Access Lanes ADTT = 1 (Category A)	4.5" PCC*/Compacted Soil
Truck Parking Areas Multiple Units ADTT = 25 (Category B)	6.0" PCC*/Compacted Soil

ADTT = Average Daily Truck Traffic

* Rough-textured, angular-shaped aggregates

The above recommended concrete sections are based on a design life of 20 years, with integral curbs or thickened edges. In addition, the above structural sections are predicated upon proper compaction of the utility trench backfills and the subgrade soils, with the upper 12 inches of subgrade soils brought to a uniform relative compaction of 95 percent (ASTM 1557).

Slab edges that will be subject to vehicle loading should be thickened at least 2 inches at the outside edge and tapered to 36 inches back from the edge. Typical details are given in the ACI Guide for Design and Construction of Concrete Parking Lots (ACI 330R-08). Alternatively, slab edges subject to vehicle loading should be designed with dowels or other load transfer mechanism. Thickened edges or dowels are not necessary where new pavement will abut areas of curb and gutter, buildings or other structures preventing through-vehicle traffic and associated traffic loads.

The concrete sections may be placed directly over a compacted subgrade prepared as described above. The concrete to be utilized for the concrete pavement should have a minimum modulus of rupture of 550 pounds per square inch. This approximates a 28-day compressive strength of 3,650 pounds per square inch. However, the design strength should be based upon the modulus of rupture and not the compressive strength. Contraction joints should be sawcut in the pavement at

maximum spacing of 30 times the thickness of the slab, up to a maximum of 15 feet. Saw cutting in the pavement should be performed within 12 hours of concrete placement, or preferably sooner.

Sawcut depths should be equal to approximately one-quarter of the slab thickness for conventional saws or 1 inch when early-entry saws are utilized on slabs 9 inches thick or less. The use of plastic strips for formation of jointing is not recommended. The use of expansion joints is not recommended, except where the pavement will adjoin structures. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other or the joints are properly doweled with smooth dowels. It should be noted that distributed steel reinforcement (welded wire fabric) is not necessary, nor will any decrease in section thickness result from its inclusion.

The above pavement designs were based upon the results of preliminary sampling and testing and should be verified by additional sampling and testing during construction when the actual subgrade soils are exposed. CHJ Consultants does not practice traffic engineering. The ADTT figures used to develop the recommended PCC pavement sections are typical for projects of this type. We recommend that the ADTT figures used be reviewed by the project civil engineer or traffic engineer to verify that they are appropriate for this project.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The geotechnical engineer's field representative will be present to provide observation and field testing and will not supervise or direct any of the actual work of the contractor, his employees or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

CHJ Consultants has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of CHJ Consultants. This report is therefore, subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling were performed. However, conditions between these locations may vary significantly. Should conditions that appear different from those described herein be encountered in the field by the client or any firm performing services for the client or the client's assign, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

Respectfully submitted,
CHJ CONSULTANTS

Fred Yi. Ph.D., G.E. 2967
Chief Engineer

FY/RJJ:lb

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AERIAL IMAGERY EXAMINED

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APPENDIX "A"
GEOTECHNICAL MAPS