FEASIBILITY LEVEL GEOLOGIC AND GEOTECHNICAL ENGINEERING STUDY

Tentative Tract Map 8143
2512 D Street, Alameda County, California

Prepared for:

The Grupe Company
3255 West March Lane, 4th Floor
Stockton, California

Prepared by:

GEOSPHERE CONSULTANTS, INC.
2001 Crow Canyon Road, Suite 210
San Ramon, California 94583
Geosphere Project No. 91-03626-A

March 2, 2016

The Grupe Company
3255 West March Lane, 4th Floor
Stockton, California

Attention: Mr. Kyle Master

Subject: **Feasibility Level Geologic and Geotechnical Engineering Study**
Tentative Tract Map 8143
2512 D Street, Alameda County, California
Geosphere Project No. 91-03626-A

Dear Mr. Master:

Geosphere Consultants, Inc. has completed feasibility level Engineering Geology and Geotechnical Engineering Study for the proposed development of 2512 D Street, Tentative Tract Map 8143, and Alameda County, California. This report has been prepared based on our discussion with you, a review of previous document for the project, and a review of select conceptual project plans. Transmitted herewith are the results of our findings, conclusions, and feasibility level recommendations for foundations, site preparation, and pavement design. In general, the proposed improvements at the site are considered to be geotechnically feasible provided the recommendations of this report are implemented in the design and construction of the project. Further design level investigations will be needed to develop the final design recommendations for the project. This report is intended for project planning purpose to better define the geologic hazards which affect the site and provide guidance for the development of grading plans and the quantities of remedial grading.

Should you or members of the design team have questions or need additional information, please contact the undersigned at (925) 314-7180, or by e-mail at ejs@geosphereinc.net. We greatly appreciate the opportunity to be of continuing service to the Grupe Company and to be involved in the design of this project.

Sincerely,
GEOSPHERE CONSULTANTS, INC.

Eric J. Swenson, GE, CEG
President/Principal Engineering Geologist

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Tentative Tract Map 8143

Client: **The Grupe Company**
Stockton, California

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this study was to evaluate the subsurface conditions at the site and prepare feasibility level engineering geology and geotechnical engineering recommendations for the proposed improvements. This study provides preliminary recommendations for foundations, site preparation, grading, drainage, and pavement design. This study was performed in accordance with the scope of work outlined in our proposal dated February 4, 2016.

The scope of this study included the review of available geotechnical and geologic literature for the site, the excavation of 12 test pits within the project site, laboratory testing of selected samples retrieved from the test pits, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The proposed redevelopment project is located at 2512 D Street in un-incorporated Alameda County east of Hayward. The site is a 3.14 acre parcel of land in the low rolling hills east of Hayward and south of Castro Valley. The parcel of land is roughly rectangular with approximate dimensions of 650 feet by 200 feet. The topography of the site includes two east-west trending swales with an intervening east-west ridge located in the center of the site. The elevation of the site ranges from a high of about 338 in the middle-east side to a low of about 287 in the lowest swales as they exit the property along the western property line. The natural slopes vary in inclination from about 3:1 to 4:1 (horizontal to vertical) with steeper 1:1 and 1-1/2:1 slopes in the swale located on the south central side of the site. A site access road has been graded through the center of the site from D-street to the south extending to the farthest extent to the north. Some minor excavations appear to have been made into the northeast corner of the site and also the eastern slopes adjacent to the southern swale. Two large fills were placed within the two swales to develop the access road through the site. The site drains into the two swales and then exists to the west. The aforementioned fills have impounded surface waters and created wetlands areas above the fills. The wetlands drain through the fills through culverts under the fill. In addition to the road, an existing improvement on the site is an abandoned residence in the center of the site on the top of the intervening ridgeline. This residence looks to have been built possibly in the 1960's and is a single story wood framed structure with stucco siding. Vegetation throughout the site consists primarily of grasses and small shrubs. In the low lying swale areas along the western property line are large eucalyptus, oak trees, and other tree species. In the wetland areas upstream of the artificial fills are tulles and other hydrophilic plant species.

1.3 Proposed Development

It is our understanding that the site is planned to be developed be subdividing the site into 12 planned lots for residential development. Vesting tentative Tract Map 8143 indicates that a central road with similar alignment will be developed through the north-south center of the site with lots 1 through 7 developed on the east side of the site and lots 8 through 12 on the west side the site. Grading will consist of filling the swale areas and cutting the upland area on the west side of the site. The grading is primarily for the development of the roadway and the residential grading appears to be predominately on-contour. In addition to the grading, site utilities, including new storm drains, culverts, water lines, sanitary lines, and joint trench will be built. Pavements will consist of an AC paved access road. The details regarding the residence are no known at this time, however we anticipate that they will be wood framed one to two story structures founded on either spread footings, PT Slabs, or drilled piers.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time period, Geosphere should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geosphere should be notified to determine if additional recommendations are required. Additionally, if Geosphere is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; Geosphere's geotechnical personnel should be retained to verify that the subsurface conditions anticipated when preparing this report are similar to the subsurface conditions revealed during construction. Geosphere's involvement should include foundation plan review, foundation excavation observation, and earthwork backfill and pavement and flatwork subgrade and baserock testing.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), water agencies, and other government agencies, as listed in the References section. In addition, the vesting tentative map 8143, dated 2/20/2014, prepared by Greenwood & Moore was reviewed.

2.2 Field Exploration

Field exploration consisted of geologic mapping of the site by a Registered Geologist in combination with excavation of test pits. On February 10, 2016, a Geosphere Consultants Registered Geologist observed the excavation of 12 test pits throughout the site. The test pits were excavated from depths ranging from about 5 feet to 12 feet deep and were 24-inches wide. The locations of the tests pits are shown on Figure 2, Site Plan/Geologic Map. The test pits were logged by our geologist and the logs are contained in Appendix A of this report. Bulk soil samples were obtained for additional laboratory testing and soil classification. Geologic structure, where encountered was also measured and logged. The test pits were loose backfilled following the excavation of the pits.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are either presented on the test pit logs, and/or are included in Appendix B. The following soil tests were performed for this study:

Atterberg Limits (ASTM D4318 and CT204) - Atterberg Limits test was performed on two samples of cohesive soils encountered at the site. Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, and help to evaluate the expansive characteristics of the soil and determine the USCS soil classification. Test results are presented in Appendix B, and on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis tests were conducted on three selected samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented in Appendix B.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Regional and Local Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

More specifically, the site is located in the hills that flank the eastern shoreline of the San Francisco Bay basin. These hills consist of sedimentary and metasedimentary rocks of the late Jurassic Knoxville and Late Cretaceous marine rocks. The underlying bedrock of the site has been mapped predominately as Cretaceous Great Valley Sequence.

3.2 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Hayward, and Calaveras. The project site is situated directly east of the active Hayward Fault, but no known active faults with Holocene movement (last 11,000 years) lie within the limits of the site. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

3.3 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located approximately five miles west of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Calaveras faults, as shown on Figure 5, *Regional Geology Map*. Active faults in general proximity to the project site the Hayward and Calaveras faults, located about 1.5 miles west and 7 miles east of the site.

In 2007, the Working Group on California Earthquake Probabilities (WGCEP 2007), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades. WGCEP 2007 estimated that there is a 93 percent probability that at least one magnitude 6.7 or greater earthquake will occur within the San Francisco Bay region over the next 30 years. This probability is an aggregate value that considered seven principal Bay Area fault systems and unknown faults (i.e., background values).

In 1868, the immediate vicinity of this site was heavily damaged by a magnitude 6.8 was one of the most significant earthquakes in Bay Area history with 30 people killed and most of down town Hayward destroyed.

4.0 SUBSURFACE CONDITIONS

4.1 Site Geologic Conditions

Utilizing the test pits, in combination with the site geologic mapping, we have developed a complete summary of the site geologic conditions including the site bedrock, soils, and ground water. The following is a summary of these conditions:

Bedrock Geology

The site is underlain by cretaceous Great Valley Sequence of sedimentary rocks. Structurally the rocks are striking at about N20W to N20 E across the site and dipping steeply from about 40 to 60 degrees to the west. These rocks consist of interbedded claystone, siltstone, and sandstone. The rocks are moderately strong with a relatively shallow weathering profile. Overlying the siltstone and claystone beds the residual soils are approximately 2 to 3 feet deep. Within the sandstone beds, the surface soils thin to about 1 foot or less. On steeper slopes the surface soil are 6-inches or less. The surface soils overlying the bedrock generally consist of moderate to highly plastic sandy to silty clay with a moderate to high potential for shrink/swell with moisture variation.

Alluvium

Within the two east-west trending swales, the geologic deposits consist of Quaternary age alluvium derived from the adjacent hillsides. These soils are approximately 5-feet deep and consist of loose to medium dense granular soil and medium stiff silt and clay. They have a high moisture content and moderate to highly compressible soils.

Artificial Fill

Two significant areas of artificial fill were mapped on the site. These areas are located in the east-west trending swales at the north and central parts of the site as shown on Figure 2, site Geology Map. The fill thickness in the test pits were identified up to about 10 feet deep, but based on site topography, the fills most likely range up to 15 feet deep in some areas. The fill was heterogeneous and contained deleterious materials including brick, wood, other debris.

Additional details of materials encountered in the exploratory borings, including laboratory test results, are included in the test pit logs in Appendix A, and laboratory test summaries are presented in Appendix B.

4.2 Groundwater Conditions

Groundwater was encountered not encountered in any of the test pits. However, we anticipate that seasonal perched groundwater will seep along the soil bedrock interface throughout the site. In addition, based on the observed surface ponded water, we anticipate that groundwater will be encounter in the deepest parts of the alluvial deposits in the center of the swales. The swale groundwater elevations will vary with seasonal changes, however groundwater most likely will be encountered within 5-feet in the center of the swales.

We note that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage. A detailed investigation of local groundwater conditions was not performed and is beyond the scope of this study.

4.3 Slope Stability

As shown on the Special Studies Zone Map, Figure 4, a portion of the site is located with the mapped seismically induced Landslide Hazard Zone by the California Geological Survey (CGS, 2006). The mapping of this hazard is based predominately on the steepness of hillsides, general regional geologic mapping, and proximity to strong ground motion from active faults. Sites within these zones require a special assessment of slope stability hazards

The portion of the site within this hazard zone include the steeper slopes of the swale at the southwest corner of the site. Test Pit 4 indicates that the natural slopes will have approximately 2 to 3 feet of stiff to very stiff sandy clay overlying moderately weathered and strong sandstone. Pocket Penetrometer values identify the unconfined compressive strength varying from 1.5 to 3.5 tones per square foot (tsf) in the soil. Based on the strength of the soil and strength and favorable structure of the bedrock, we judge that slope instability of the natural slopes in this area is not a concern.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

5.1 Conclusions

The site is considered geotechnically suitable for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that potentially impact this site are summarized below.

Artificial Fill – The two swales located within the swales are heterogeneous, compressible, and contain deleterious materials. The soils have been placed on moderately steep slopes with unknown site preparation and lack subdrains. These deposits will be susceptible to settlement and instability unless remediated in accordance with our recommendations which follow.

Expansive Soils – The native soils, as well as the non-engineered fill, at the site appear to have a moderate to high expansion potential. Structures supported on fill will be susceptible to expansive soil pressure as will flatwork and site pavements. Appropriate foundation solutions will be needed to resist these pressures.

Compressible Soils – The native alluvial soils within the center of the swales will be saturated and compressible. These soils are not capable of supporting the planned engineered fills above. As part of the remedial grading for the planned engineered fills, all alluvial soils will need to be removed and re-graded as engineered fills.

Site Drainage and Slope Stability – As with any hillside grading project, appropriate design of site drainage and grading details will affect long-term stability and performance of all site improvements and residential structures. All surface flows should be controlled on slopes through the use of lined swales and water should be controlled away from structures and away from slope faces. Additionally, the design and construction of subdrains around structures and below engineered fills is essential to control the naturally occurring seepage between surface soil and the underlying bedrock.

5.2 Site Grading

5.2.1 General Grading and Fill Material Requirements

Site grading is anticipated to be extensive with cuts on the order of 10 feet and fills up to 25 feet or more. The removal of the existing deep fills and grading of the access road will be the major grading in accordance with the preliminary grading scheme. We recommend that the cut slopes not exceed an inclination of 2:1 horizontal to vertical. Fill slopes up to a height of 25-feet should not exceed an inclination of 2:1 and fill slopes higher then 25-feet should be designed with a 2-1/2:1 inclination.

Existing onsite fill as well as all native soils can be reused as fill material provided the size requirements for import fill are met. Deleterious materials, particularly organic materials such as wood debris, should be removed as part of the regrading operations. Imported select fill should be non-expansive, having a Plasticity Index of 12 or less, an R-Value greater than 40, and enough fines so the soil can bind together. Imported soils should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. The Geotechnical Engineer should review and approve imported fill sources and materials prior to delivery onsite.

5.2.2 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geosphere prior to starting demolition operations at the site.

All non-engineered artificial fills on the site shall be completely removed in preparation for grading of the site. We anticipate that these fills will be up to 15-feet deep. Soft compressible soils underlying the fill are also unacceptable and will require removal and replacement.

Demolition of the existing building is to be performed, including removal of existing building foundations. Excavations resulting from the removal of foundations should be backfilled with engineered fill in accordance with this section

Existing underground utilities proposed to be abandoned, if any, should be properly grouted, closed, or removed as needed. If the utilities are removed, the excavations should be backfilled with properly compacted fill or other material approved by the Geotechnical Engineer. The extent of removal/abandonment depends on the diameter of the pipe, depth of the pipe, and proximity to building and pavement.

5.2.3 Hillside Grading Recommendations

Prior to grading, the native grasses should be disked to promote decay and drying of the grass. Provided the organic content of the grass does not exceed 3% by weight, no additional clearing of the grass is required and this soil can be incorporated into general engineered fill.

All fills placed on slopes exceeding a 5:1 inclination must be keyed into bedrock or competent materials has determined by the geotechnical engineer in the field. The keyway must be a minimum of 15-feet wide and 5-feet into stable material. A subdrain consisting of a 4-inch diameter SDR35 or better perforated pipe with 24-inchs of class 2 permeable rock shall be constructed along the entire length of the keyway. ¾-inch drain rock can be used in place of the permeable rock if it is surrounding by a high quality filter fabric such as Mirafi 140N. The subdrain shall drain by gravity flow into a drainage point below the fill. Fill slopes greater than 20-feet high should have additional horizontal subdrains every 10-feet in vertical height.

5.2.4 Project Compaction Recommendations

The following table provides the recommended compaction requirements for this project. Not all soils, aggregates and scenarios listed below may be applicable for this project. Specific grading recommendations are discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Min. Percent Relative Compaction (per ASTM D1557)	Percent Above/below Optimum Moisture Content
Fill Areas, Engineered Fill, Onsite Soil	90	+ 3
Fill Areas, Engineered Fill, Select Fill	90	+ 2
Concrete Flatwork, Subgrade Soil	90	+ 3
Concrete Flatwork, Baserock	95	± 2
Underground Utility Backfill – Onsite Soils	90	+ 3
Underground Utility Backfill – Import (Select) Soil	90	+ 2
Underground Utility Backfill - Below 3 feet in Pavement Areas	90	+ 3
Underground Utility Backfill - Upper 3 feet below Pavements	95	+ 3
AC Pavement – Onsite Subgrades – upper 6 inches	95	+ 3
Pavement – Class 2 Aggregate Base Section	95	± 2

5.2.5 Site Drainage Recommendations

We recommend that concrete lined brow ditches be constructed at the top of all cut-slopes. All slopes, both cut and fill, greater than 25-feet high should be designed with intermediate drainage benches every 20-feet with concrete lined v-ditches to control surface flows. Pads should be graded at a minimum of 2% away from the top of slope to limit the potential for surface flows over the face of slope. All surface drainage should be directed into appropriate storm drainage structures away from site improvements and structure foundations.

5.3 Preliminary Foundation Design

The final grading plan has not been design yet therefore we cannot provide final design recommendations. Based on the preliminary design concept, we can provide preliminary design recommendations for foundation types as well as typical foundation embedment or size. When additional planning is performed, design level recommendations can be provided

5.3.1 Flat lots and Bedrock Cut Lots – Lots 1-7

Based on the preliminary grading concept, we believe that a post-tension concrete slab would be the most economical foundations for these lots. The design will be driven by the final design grades including differential fill and potential cut into bedrock. Cut/Fill transition lots may require additional over-excavation to limit the potential for differential settlement. We anticipate that PT-stabs on the order of 10 to 12-inches thick would be required for these lots. Conventional spread footings may also be a possibility for these lots, however all expansive soils will need to be removed in the upper 12-inches under the slab and replaced with a non-expansive fill. Additionally, the footings would need to bear in a uniform support layer which might require additional deepening into bedrock with some of the lots.

5.3.1 Hillside lots – Lots 8-12

These lots are located on the west side of the site access road on a combination of cut, fill, and native soil slopes. Due to the variability of soil conditions as well as the expansive soil conditions, for preliminary design we recommend that these lots be designed with a drilled pier foundation. The drilled piers should be a minimum of 16-inches in diameter and bear uniformly into the underlying bedrock. Special design provisions for expansive soils will be required including uplift pressures on grade beams. In general we anticipate that the drilled piers will be on the order of 10 to 12 –feet deep however some of the deeper fill lot, such as #8, #9 and #12, may have deeper piers.

5.4 Pavement Design

Recommendations for the design of asphalt concrete pavement sections were developed in accordance with the procedures outlined in the latest edition of the Caltrans Highway Design Manual. The Caltrans design method uses

Traffic Indices (TI) to represent anticipated wheel loads and frequency of usage for a given design life. A design life of 20 years is typically used in California. Factors such as surface and subsurface drainage have an effect on the overall life of a pavement section.

Using established soil gradation and plasticity correlations, a preliminary R-value of 5 was selected for analysis and a Traffic Index of 4.5 was selected for passenger car and light truck parking lot and driveway traffic. As an option, a heavier pavement section assuming a TI of 6.0 was also evaluated should any exterior pavements be designed to receive loads from heavier truck traffic. The following are recommended structural asphalt concrete (AC) pavement sections. Aggregate base (AB) from recycled sources offered by AB suppliers as well as AC grindings mixed with existing baserock and meeting Class 2 specifications can be utilized in lieu of virgin Class 2 AB upon approval of the Geotechnical Engineer.

Table 7: Recommended Pavement Design Alternatives

Load Application (Traffic Index)	Asphalt Concrete (in.)	Class 2 AB (in.)	Total Section (in.)
Driveway and Parking Areas for Passenger Car and Light Truck Traffic (4.5)	3.0	8	11.5
Moderate Heavy Truck Traffic (6.0)	3.5	12	15.5

The traffic indices assumed for our design were established assuming typical automobile traffic for a TI = 4.5, and heavy truck access for a TI = 6.0 once construction has been completed. However, if the lighter pavements (TI = 4.5) are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, the heavier pavement section should be considered, or the asphalt concrete should be placed in phases (e.g., placement of final lift of AC after building construction is substantially completed) to minimize pavement damage caused by construction traffic.

Design based on the aforementioned traffic indices should provide the design pavement life with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on geotechnical engineering judgment, and not on an equivalent wheel load analysis either developed from a traffic study or furnished to us by the project civil engineer. Therefore, the traffic indices provided herein should be confirmed by the civil engineer as appropriate for the intended use. In addition, the R-value assumed for design should be confirmed by laboratory testing on a sample of actual pavement subgrade material as rough grading of pavement subgrades proceeds, and pavement designs adjusted if required.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

AB for use in flexible pavements should conform to Caltrans Standard Specification Sections 26-1.02A and 26-1.02B (2010) for Class 2 AB. AB used in pavement sections should be compacted to a minimum 95 percent relative compaction (ASTM D1557) and should be firm and unyielding at the time of asphalt concrete placement.

5.5 Design Level Investigation

The purpose of this investigation has been to provide a feasibility of construction and design as well as preliminary design guidance. As additional grading and drainage plans are developed, we recommend that a design level investigation be performed. This design level recommendation report would include:

- Fine grading recommendations
- Utility design recommendations
- Foundation design recommendations

Additional drilling or subsurface investigation may be needed, but that will not be known until additional planning is performed.

5.6 Observation and Testing During Construction

We recommend that Geosphere be retained to provide observation and testing services during site preparation, mass grading, underground utility construction, foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, Geosphere should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geosphere after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered Geosphere should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geosphere be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geosphere will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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FIGURES

APPENDIX A

Test Pit Logs

APPENDIX B

LABORATORY TEST RESULTS
Liquid and Plastic Limits Test Report
Particle Size Distribution Report