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Wright and Company
Attention: Bill Wright
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Project: Proposed Residential and Commercial Development, Site No. 1
Near the Intersection of Garden and Yanonali Streets
Santa Barbara, California

As authorized, we have performed a geotechnical study for a residential and commercial development to be located near the intersection of the Garden Street and Yanonali Street in Santa Barbara, California. The accompanying Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, as well as our conclusions and recommendations pertaining to geotechnical aspects of project design.

We have appreciated the opportunity to be of service to you on this project. Please call if you have any questions, or when we can be of further service.

Respectfully submitted,

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INTRODUCTION

A. Project Description

This report presents findings and recommendations from a Geotechnical Engineering study performed for a proposed residential and commercial development to be located near the intersection of the Garden and Yanonali Streets in Santa Barbara, California. The project consists of two separate sites. Site No. 1 will consist of about 91 residential condominium/townhome units. The structures will be three-story units with attached garages at the lower level. Site No. 2 is not a part of the project. Site No. 3 is a part of the project and will include retail/commercial and self-storage units. The structures will be one- to three-story. This report presents our findings and recommendations for Site No. 1. The findings for Site No. 3 are included in a separate report. The site development will also include related utility, roadways, and parking areas. This firm has performed several geotechnical reports for similar buildings in the immediate vicinity of this site and some of those data were referenced in preparing this report.

The project site does not lie in a zone of required investigation for fault rupture on a State of California Special Studies Zone Map (CDMG, Official Map, 1978). The site however does lie in zone of required investigation for liquefaction (City of Santa Barbara Geological Map, Hoover, 1978). Therefore, liquefaction potential was evaluated.

Site grading is expected to consist of excavation and backfill for the structures, related new utilities, and the associated parking areas. The traffic on the site is expected to include light trucks and automobiles and occasional heavy trucks.

Earth Systems Southern California (ESSC) has not received building or foundation plans for the proposed structures. However, it is anticipated that buildings will be supported by post-tensioned and/or structural slabs. Structural loads are assumed to be 50 kips for the column footings and about 3 kips per linear foot at the perimeter walls. If these loads are not correct, ESSC should be notified so that the recommendations of this report can be reviewed and modified if necessary.

B. Purpose and Scope of Services

The purpose of the geotechnical study was to evaluate seismicity and soil conditions of the site with respect to the proposed construction. The soil conditions include surface and subsurface soil types, expansion potential, settlement potential, bearing capacity, and the presence or absence of subsurface water. In particular, the scope of work included:

1. Reconnaissance of the site and marking locations for borings and CPT soundings.
2. Drilling, sampling, and logging ten continuous flight auger borings to a maximum depth of 51-1/2 feet below existing ground surface to study soil and groundwater conditions.
3. Performing two cone penetrometer tests (CPT's).
4. Testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
5. Analyzing the geotechnical data obtained.
6. Preparing this report.

Contained in the report are:

1. Discussions on local soil and groundwater conditions.
2. Conclusions and recommendations pertaining to structural design.

C. Site Setting

The subject site is located at the southeast corner of the intersection of Garden Street and Yanonali Street in Santa Barbara, California (see Vicinity Map, Appendix A). The subject site is bounded by Yanonali Street to the west, Garden Street to the north and northeast, Santa Barbara Street to the south. The Union Pacific Railroad tracks run on the east side of the property. The site is relatively flat. The area surrounding the site is also relatively flat with slight surface drainage to the south. The elevation of the site is approximately 11 feet above sea level on its north side and approximately 14 feet above mean sea level on its south side (based on the City of Santa Barbara Topographic Maps, 1999). The site is currently occupied by trailers, storage containers, and piles of stone.

REGIONAL GEOLOGY

The parcel lies within the Santa Barbara coastal plain in the western portion of the Transverse Ranges geologic province. Numerous east-west trending folds and reverse faults indicative of active north-south transpressional tectonics characterize the region. The site does not lie within a State of California Fault Rupture Hazard Zone or a City of Santa Barbara fault hazard zone (Hoover, 1978). No faults or landslides are mapped on or trending into the parcel during site reconnaissance by a representative of this office or in attached review of regional geology maps prepared by others [Dibblee (1986), and Gurrola (2002)].

STRATIGRAPHY

The subsurface stratigraphy below the subject site is mapped by Dibblee, Gurrola, and Hoover as alluvium (see Regional Geology Maps, Appendix A). The CPT's and borings performed as part of this study encountered artificial fills and interbedded alluvium consisting of silty clays, clayey sandy silts, and silty sands.

GEOLOGIC HAZARDS

Geologic hazards that are common to the Southern California area include fault rupture, seismic shaking, landslides, rockfalls, tsunami, seiche, and flooding (seismic related and non-seismic related). Below, we will address each of these hazards as they relate to the subject site.

Fault Rupture

The parcel does not lie within a State of California designated fault hazard zone or City of Santa Barbara designated fault hazard zone. There are no other mapped faults trending towards the site on the referenced regional geology map. Therefore, the potential for fault rupture hazard on the subject site is considered low.

Landslides and Rockfall

There are no identified landslides or rockfalls either on or trending into the site, therefore, hazards associated with this phenomena should be considered low.

Seismically Induced Flooding (Tsunamis, Seiches, and Reservoir Failure)

The potential for earthquake induced tsunamis must be considered a hazard because of the site's relative elevation (El. 11 to 14 feet above mean sea level) and close positioning to the Pacific Ocean (approximately 2,000 feet). The County of Santa Barbara Seismic Safety and Safety Element (1980) has the subject site located in an area with a moderate potential for tsunami inundation (the County does not have any tsunami designation greater than moderate). The County indicates that sites below elevation of 40 feet could be susceptible to tsunami inundation. Project designers should consider the effects of tsunami inundation.

Because of the subject sites relative position away from any reservoirs, the potential for seiche and reservoir failure should be considered low.

SOIL CONDITIONS

The site is capped with about 2 to 4 inches of asphalt or gravel. Below that, uncertified fill material was encountered in all borings. The fill material consists of silty fine to coarse sand with gravel, pieces of brick, asphalt, glass, and debris. The depth of fill ranges from about 3-1/2 feet to 8 feet below existing ground surface. Below the fill material, the soils encountered were alluvium to the maximum depth explored. Generally, the alluvium consists of silty clays, clayey silts, and silty sands. Soils within the anticipated influence of the building foundations and the parking areas have less than 90% relative compaction and include significant quantities of uncertified fill. Consolidation testing indicated that the clays within the influence of anticipated foundations are generally compressible. The in-place densities in the upper 5 feet of the site soils range from about 72 pcf to 118.5 pcf. Moisture contents range from about 11% to 29%.

Groundwater was encountered at a depth of about 6 feet below the ground surface. Based on experience with similar coastal sites, it may be assumed that the

groundwater beneath the site is hydraulically connected to the ocean and the groundwater surface elevation is influenced by tide elevations. Soil moisture content above the groundwater, at the time the site was explored was above the optimum moisture content for compaction. Soil moisture may change with variations in weather patterns, the time of year, irrigation, and other factors.

Expansion determination indicates that bearing soils lie in the "Low" range (51-90) in accordance with Table 18-I-B of the CBC. Table 18-I-D, a locally adopted variation of CBC Table 18-I-B, provides recommended minimum foundation and slab requirements as a function of expansion index, and is included in Appendix B of this report. Because there are several buildings that will be graded, it is recommended that the expansion index of individual pads be determined after the completion of grading.

Corrosion characteristics of shallow soils were determined by testing for resistivity, pH, soluble sulfates, and soluble chlorides. The results are presented in Appendix B and should be brought to the attention of the project Structural or Mechanical Engineer or anyone designing facilities in contact with soil. Soluble sulfate content is in the "Negligible" range of Table 19-A-4 of the CBC. Therefore, it appears that special designs for concrete in contact with the ground will be necessary. Soil resistivity is in the "Moderately Corrosive" range for ferrous metal according to a table in the Los Angeles County Guidelines of Preparing Geotechnical Reports (2001). Designers of metals in contact with soil should be made aware of this potential. Please note that ESSC does not provide corrosion-engineering services.

SEISMICITY

The project site, like any other site in the region, is subject to relatively severe ground shaking in the event of a maximum earthquake on a nearby fault. It is the standard of practice, when evaluating the seismicity of a residential type development, to consider the design basis (10% probability of exceedance in 50 years) accelerations. The California Division of Mines and Geology, in concert with the U.S. Geological Survey and the scientific community, has presented results of a statewide probabilistic seismic hazard assessment (CDMG 1996). The

focus of the assessment was to generate a seismic hazard map showing zones of estimated peak ground accelerations at a hazard level of 10% probability of exceedance in 50 years. At this site, based on the soil type encountered, a uniform ground condition of "Alluvium" is used. More recently, the data from the 1996 evaluation has been refined and modified to include effects of local surface conditions (The Revised 2002 California Probabilistic seismic Hazard Zone Maps, June, 2003). The data from that effort is now available at the CGS internet web site. Searches can be done using the site geographic coordinates. This latest probabilistic analysis indicates the site area to have an estimated 0.50-g of predicted peak horizontal surface acceleration at the design level. The 1996 evaluation also included a statewide mapping of the magnitude of earthquake responsible for the design level acceleration. The subject site is located within a zone indicated as having predicted earthquake magnitudes of 6.5 to 7.0.

The 2001 CBC seismic design values are different from earlier versions by having more site-specific input. The following CBC geotechnical related values could be used in the buildings' earthquake design:

Seismic Zone - Figure 16-2	4
Seismic Zone Factor Z - Table 16A-I	0.40
Soil Profile Type - Table 16A-J	S_D
Seismic Coefficient C_a - Table 16A-Q	$0.44N_a$
Seismic Coefficient C_v - Table 16A-R	$0.64N_v$
Near Source Factor N_a -Table 16A-S	1.3
Near Source Factor N_v -Table 16A-T	1.6
Seismic Source Type - Table 16A-U	B
Distance to Seismic Source	≈2 km

LIQUEFACTION

Earthquake-induced vibrations can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction can result in a complete loss of strength and can cause structures to settle or even overturn if it occurs in the bearing zone. If liquefaction occurs beneath sloping ground, a phenomenon known as lateral spreading can occur. Liquefaction is typically limited to the upper 50 feet of the subsurface soils.

There are a number of conditions that need to be satisfied for liquefaction to be a potential hazard. Of primary importance is that groundwater, perched or otherwise, usually must be within the upper 50 feet of soils. Groundwater was encountered during our site exploration at a depth of about 6 feet. In addition, the site is in a required liquefaction hazard study zone (City of Santa Barbara Geological Map, Hoover).

There are several layers of sandy silts and silty sands that have less than 15% clay sized particles throughout the depth of the borings advanced. Consequently, a liquefaction analysis was performed using the CPT data at each location that the CPT was advanced, a magnitude 7.0 earthquake, and a 0.50-g peak ground acceleration. The analyses were done with a proprietary spreadsheet developed by this firm. This spreadsheet is based on the CPT data and methods developed by Robertson and Wride (1998). The analyses were done at every data point, which were taken at 0.05m intervals (about every 2 inches). No data averaging was performed. The analysis is presented in Appendix C. It indicates that at the location of the CPT Nos. 1 and 2, liquefaction is a potential in several soil layers and in thin soil lenses between the depths of about 5 to 50 feet. These layers have a cumulative total thickness of about 17.4 feet and 17.1 feet, respectively.

Liquefaction induced settlement has been estimated using the procedure developed by Tokimatsu and Seed (1987) to be on the average about 3 inches at this site. It can be assumed that this amount of strain would be experienced at the ground surface as areal settlement. Differential settlement is estimated to be half that amount or about 1-1/2- inches.

SEISMIC INDUCED SETTLEMENT OF DRY SANDS

Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. Procedures to evaluate this type of settlement were developed by Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

Because the soils above the groundwater contain more than 15% clay sized particles (Boring No. 7), and that the soils in the upper 5 feet of the site will be graded, the potential for seismic induced settlement of dry sands is considered to be low.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomenon in which naturally occurring soil deposits, or non-engineered fill soils, collapse when wetted. Natural soils that are susceptible to this phenomenon are typically aeolian, debris flow, alluvial, or colluvial deposits with high apparent strength when dry. The dry strength is attributed to salts, clays, silts, and in some cases capillary tension, "bonding" larger soil grains together. So long as these soils remain dry, their strength and resistance to compression are retained. However, when wetted, the salt, clay, or silt-bonding agent is weakened or dissolved, or capillary tension reduced, eventually leading to collapse. Soils susceptible to this phenomenon are found throughout the southwestern United States.

Methods to analyze hydroconsolidation include double oedometer tests proposed by Jennings and Knight (1956), and a modified version of the Jennings and Knight procedure presented by Houston, et al. (1988) involving a single oedometer test on a soil sample. We have performed hydroconsolidation testing according to the later procedure.

The consolidation tests on soils from above groundwater showed no tendency for the soils to hydroconsolidate. Soils below groundwater are not susceptible to hydroconsolidation.

CONCLUSIONS AND RECOMMENDATIONS

The site is suitable for the proposed development from a Geotechnical Engineering standpoint provided that the recommendations contained in this report are successfully implemented into the project.

The main geotechnical concerns at the site are the presence of uncertified fill material, shallow groundwater, potential large liquefaction induced settlement, and that the existing soils within the anticipated influence of the foundations are compressible and have less than 90% relative compaction and in their present condition are not suitable for the support of the buildings. In addition, some areas of the site appear to be underlain in the top 20 feet by some soft, normally consolidated clays that are compressible. Therefore, to help mitigate these conditions, the upper soils within the building areas require removal and recompaction. Additionally, because of liquefaction induced settlement, use of structural slabs and/or stiff post-tensioned slabs capable of tolerating the estimated differential settlement is recommended to support the structures. Auxiliary structures (signs, garden walls, retaining walls) can be supported on conventional footings with slab-on-grade floors.

Settlement from the maximum anticipated foundation loads (50 kips at column and 3 klf at continuous footings) is estimated to be about 1-3/4 inches without recompaction and about 3/4-inch when the recommended recompaction is completed. However, in the areas underlain by soft clays between the depths of 14 feet to 19 feet, total settlement could be about 2 inches before recompaction and about 1 inch after recompaction. The following sections will provide more detailed recommendations for grading and foundation design.

The site is in a seismically active area. Seismic induced settlement of dry soils and hydroconsolidation are not potentials at this site.

Calculations indicate that liquefaction is a potential in soil layers and in thin soil lenses from about 5 to 50 feet up to 5 feet in thickness. The result of liquefaction is

estimated to be approximately 3 inches of settlement, on an average, at the ground surface and differential settlement of about 1-1/2 inches.

Groundwater was encountered at a depth of about 6 to 10 feet during drilling. Therefore, dewatering of the excavation during overexcavation and recompaction may be needed, depending on its depth.

A. Grading

1. General Grading

a. Grading at a minimum should conform to Chapter 33 of the CBC.

b. The existing ground surface should be initially prepared for grading by removing all vegetation, noncomplying fill (from a few feet to about 8 feet thick at the boring locations), and all buried obstacles, buried irrigation lines, and debris. Voids created by removal of such material should be properly backfilled and compacted.

c. The bottom of all excavations should be observed by a representative of this firm prior to processing or placing fill.

d. Fill and backfill material should be placed at about optimum moisture content, in layers with loose thickness not greater than 8 inches and should be compacted to a minimum of 90% of the maximum dry density obtainable by the ASTM D 1557 test method, unless otherwise recommended or specified. Random compaction tests by ESSC can assist the Grading Contractor in evaluating whether the Grading Contractor is meeting compaction requirements. However, compaction tests pertain only to specific locations, and do not guaranty that all fill has been compacted to the prescribed percentage of maximum density. It is the ultimate responsibility of the Grading Contractor to achieve uniform compaction in accordance with the requirements of this report and the grading ordinance.

e. Shrinkage of soils affected by compaction is estimated to be about 15%. Shrinkage from clearing and grubbing the site is not included in this figure.

f. Import soils used to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the

Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

g. Roof draining systems and exterior decking should be designed so that water is not discharged into bearing soils or near the structures. Final site grade should be designed so that all water is diverted away from the structures, and is not allowed to pond. A minimum gradient of 2% is recommended for landscaped areas.

h. It is recommended that ESSC be retained to provide Geotechnical Engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

i. Plans and specifications should be provided to ESSC prior to grading. Plans should include the grading plans, foundation plans, and foundation details. ESSC will review these plans only for conformity with geotechnical parameters not including drainage. It is the responsibility of the Client and other Engineers to review and approve designs and plans for conformity with all engineering and design requirements necessary to the proper function and performance of the structures.

2. <u>Site Grading/Development - Main Buildings</u>

a. Recompaction in the building areas will be necessary to decrease the potential for differential settlement and provide more uniform bearing conditions. In the foundation areas, soils should be overexcavated to the deeper depth of 2 feet below the bottom of footings or 3 feet below existing grade and through any uncertified fill material. As mentioned earlier, the depths of fill at the location of the borings appear to be between about 3-1/2 to 8 feet below existing ground surface. Therefore, the depth of overexcavation should increase as necessary to remove the uncertified fill. The actual depth of fill should be determined in the field during construction by the Geotechnical Engineer. The overexcavation should extend laterally to 5 feet beyond the foundation edges. The resulting surface should then be scarified an additional 1 foot, moisture conditioned to about optimum moisture content and recompacted to at least 90% of

maximum dry density as determined by ASTM 1557 test method. The intent of these recommendations is to provide a minimum of 3 feet of compacted fill below bottom of footings and to remove all uncertified fill.

b. On-site soils may be used for fill once they are cleaned of organic material, rock, debris and irreducible material larger than 8 inches. Alternatively, a granular import soil may be used for fill.

c. As mentioned earlier, expansion determination indicates that bearing soils lie in the "Low" range (21 to 50) in accordance with Table 18-I-B of the CBC. However, because there are several pads that will be constructed, the expansion index of the soils should be determined after the completion of the grading.

d. Based on soil moisture content of the soils in the upper 5 feet during drilling, pumping soils or otherwise unstable soils will most likely be encountered at overexcavation bottoms. Alternate ways to stabilize excavation bottoms include working thin lifts of 1-1/2-inch (minimum size) float rock into the excavation bottoms until stabilization is achieved, or lime or cement treatment of the soils. Use of geotextiles in combination with crushed rock is another possibility. In this method, an extra 1 foot of soil should be removed and the bottom of the excavations covered with a woven geotextile fabric equivalent to Mirafi 500X, and then the fabric covered with a 12-inch thick layer of crushed rock. After the rock has been wheel rolled or track-walked, it should be smoothed, rolled, and then covered with a non-woven filter fabric equivalent to Mirafi 140N. Fill can then be placed in thin moisture conditioned compacted lifts until finished grade is reached.

3. <u>Grading Recommendations - Site Walls and Miscellaneous Foundations</u>

a. Foundation areas should be overexcavated to a depth of 12 inches below the bottom of footings and through any uncertified fill, and to a distance of 3 feet beyond the footing perimeters. The resulting surfaces should then be scarified an additional 1 foot, moisture conditioned, and recompacted to at least 90% of maximum density. On-site soils may be used for fill once they are cleaned of organic material, rock, debris, and irreducible material larger than 8 inches. Alternatively, a granular import soil may be used for fill.

4. Grading Recommendations - Exterior Slabs and Pavements
   a. Areas to receive exterior slabs-on-grade or pavements should be overexcavated to the deeper depth of 24 inches below existing grade or 24 inches below finishes subgrade, whichever is deeper and through any uncertified fill, and to a distance of 2 feet beyond the perimeter of the slab or pavement. The resulting surface should then be scarified an additional 1 foot, moisture conditioned, and recompacted to at least 90% of maximum density. On-site soils may be used for fill once they are cleaned of organic material, rock, debris, and irreducible material larger than 8 inches.

5. Utility Trenches
   a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90% of maximum density. Backfill of off site service lines will be subject to the specifications of the jurisdictional agency or this report whichever are greater.
   b. If water is present in trenches, backfill should be gravel to 6 inches above the water. If the elevation of a utility does not allow this, water should be pumped from its trench.
   c. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.
   d. Jetting should not be utilized for compaction in utility trenches.

B. Structural Design
   1. Post-Tension or Structural Slab Designs
      a. Post-tensioned or structural slab foundations are recommended for the proposed residential and commercial complex because of the potential for liquefaction at relatively shallow depths below the ground surface and estimated differential settlement. Either slab type should be designed to accommodate a differential ground movement of 2 inches in 15 feet in all directions.
      b. Post-tensioned or structural slabs and their footings should bear on firm recompacted soils as recommended elsewhere in this report. Foundation

excavations should be observed by a member of this firm after excavation and before reinforcing steel or concrete are placed.

c. Footing and thickened slab edges should have a minimum depth of 24 inches for the proposed three-story construction.

d. Slabs can be designed for a bearing capacity of 1,000 psf at the ground surface. At a depth of 24 inches, a bearing value of 1,500 psf can be used. These are net bearing values (weight of footing and soil surcharge can be ignored) and are applicable for dead plus live loads. These values can be increased by one-third when transient loads from wind or seismicity are included.

e. For slabs designed by the PTI method, edge lift and center lift conditions will not be factors because the soils are non-expansive. Rather differential settlement will be a factor and slabs should be designed for a differential settlement of 2 inches in 15 feet.

f. Slabs can be designed using a modulus of subgrade reaction of 60 pci.

g. Lateral loads can be resisted by friction on the base of the post-tensioned slab and foundations, and by passive resistance on the edge of the foundations. Lateral capacity is based on the assumption that backfill adjacent to foundations will be properly compacted.

h. Bearing soils under footings should be premoistened to 3% over the optimum moisture content to a depth of 18 inches below lowest adjacent grade. Premoistening should be confirmed by testing.

i. Slabs should be underlain by a minimum of 4 inches of clean sand (less than 5% fines). The sand found at the site will meet this criterion. A vapor retarder should be placed on the subgrade and covered with the 2 inches of clean sand. The sand above the vapor retarder should be lightly moistened just prior to placing concrete.

2. <u>Conventional Foundations - Miscellaneous Structures</u>

a. Conventional continuous footings and/or isolated pad footings may be used for support of miscellaneous structures (not buildings).

b. Footings with a minimum embedment depth of 15 inches should bear into firm recompacted soils. Foundation excavations should be observed by a representative of ESSC after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.

c. Conventional continuous and isolated pad footings may be designed based on an allowable bearing value of 1,500 psf. This value includes a factor of safety greater than three, and should not be increased because of settlement potential.
d. The allowable bearing value is net (weight of footing and soil surcharge may be neglected) and is applicable for dead plus live loads.
e. The bearing value may be increased by one-third when transient loads such as wind and/or seismicity are included.
f. Lateral loads may be resisted by soil friction on floor slabs and foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.
g. The data that follow regarding depths, widths, reinforcement and premoistening for footings are generally the same as those given in Table 18-I-D for the "Low" expansion range. It should be noted, however, that these values are minima and that other more stringent structural considerations may govern. Actual footing designs, depths, widths and reinforcement should be provided by the Structural Engineer but should not be less than values given herein.
h. Continuous footings bottomed in soils in the "Low" expansion range should, at a minimum, be reinforced with one No. 4 bar along the bottom and one No. 4 bar along the top.
i. Bearing soils under footings should be premoistened to 3% over the optimum moisture content to a depth of 18 inches below lowest adjacent grade. Premoistening should be confirmed by testing.
j. ESSC's representative should observe foundation excavations prior to placing reinforcing steel or concrete.

3. <u>Slabs-on-Grade (miscellaneous slabs)</u>
   a. Concrete slabs should be supported by compacted structural fill as recommended earlier in this report.
   b. It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e., free floating) so foundation adjustment will be less likely to cause cracking.

c. Slabs should be underlain with a minimum of 4 inches of clean sand. Areas where floor wetness would be undesirable should also be underlain with a vapor retarder to reduce moisture transmission from the subgrade soils to slabs. In these areas, the vapor retarder should be centered in the 4 inches of sand. The sand should be lightly moistened just prior to placing concrete.

d. Reinforcement and premoistening data given herein for slabs are the same as those given in Table 18-I-D for the "Low" expansion range. It should be noted, however, that these values are minima and that other more stringent structural consideration; such as large construction loads may govern. Actual reinforcement and slab thickness should be determined by the Structural Engineer, but should not be less than values given herein.

e. Slabs bottomed on soils in the "Low" expansion range should be reinforced with No. 3 bars placed 24 inches on center in each direction. Reinforcing should be located at mid-slab.

f. Slab subgrade soils should be premoistened to 3% above the optimum moisture content to a depth of 18 inches below lowest adjacent grade. Premoistening should be confirmed by testing.

4. Frictional and Lateral Coefficients

a. Resistance to lateral loading may be provided by friction acting on the base of foundations, grade-beams, and slabs-on-grade. A coefficient of friction of 0.30 may be applied to dead load forces. This value includes factor-of-safety of 1.5.

b. Passive resistance acting on the sides of foundation stems equal to 190 pcf of equivalent fluid weight may be included for resistance to lateral load. This value includes a factor-of-safety of 1.5. When passive resistance is used in conjunction with friction, the coefficient of friction should be reduced by one-third in determining the total lateral resistance.

5. Settlement Considerations

a. Maximum expected settlement of less than 1 inch is anticipated from static loads for foundations and slabs designed as recommended when recompaction is completed. Differential settlement between adjacent load

bearing members from static loads could be about one-half the total estimated settlement, or 1/2-inch.

b. The potential for seismic induced settlement of dry sands is estimated to be non-existence. The potential for hydroconsolidation settlement is estimated to be non-existence.

c. Liquefaction induced settlement is estimated to be about 3 inches. Differential settlement from this phenomenon could be about 1-1/2 inches.

d. Total potential settlement from all sources could be about 4 inches and total differential settlement from all sources could be about 2 inches.

6. Retaining Walls

a. Conventional cantilever retaining walls backfilled with compacted on-site soils may be designed for active pressures of 49 pcf of equivalent fluid weight for well-drained, level backfill.

b. The pressures listed above were based on the assumption that the on-site soils will be compacted to approximately 90% of maximum dry density as determined by ASTM D 1557 test procedures. The use of select granular fill may lower the recommended driving earth pressures.

c. The lateral earth pressure to be resisted by the retaining walls or similar structures should be increased to allow for surcharge loads. The surcharge considered should include the loads from any structures or temporary loads that would influence wall design.

d. A backdrain or an equivalent system of backfill drainage should be incorporated into retaining wall design. Backfill immediately behind retaining structures should be a free-draining granular material a minimum of 1 foot thick and extending to within 12 inches of the top of the backfill. The granular backfill should be capped with about 12 inches of on-site soils. Efflorescence on the face of walls can be mitigated by water proofing which, if done, should be according to the Architect or Civil Engineer's recommendations.

e. Compaction on the uphill side of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.

f. Water should not be allowed to pond near the top of walls. To accomplish this, the final backfill site grade should be such that all water is diverted away from retaining walls.

7. Paving Design

a. The R-Value measured on a sample of anticipated subgrade soil was 11 and this value is used in the designs that follow. Note that this value was measured on a sample from the adjacent site, but is similar to the values measured on a sample from this site (R=14).

b. If a Traffic Index of 4.5 is assumed (automobiles and infrequent light trucks), and using the R-Value of 11, paving sections should have a minimum gravel equivalent of 1.28 feet. This can be achieved by using 3 inches of asphalt concrete over 7 inches of aggregate base.

c. If a traffic Index of 5.0 (truck traffic area with two three- to five-axle trucks per day or nine three-axle trucks per day) is assumed, and using the measured R-Value of 11, paving sections should have a minimum gravel equivalent of 1.42 feet. This can be achieved by using 3 inches of asphalt concrete over 8.5 inches of aggregate base.

d. In the truck traffic areas described above, a PCC paving design would be 6 inches of PCC over 4 inches of aggregate base. For the lighter traffic areas the concrete thickness can be reduced to 5 inches. The concrete should have a minimum compressive strength of 3,700 psi. Cracks can be controlled by placing contraction joints spaced about 15 feet apart in each direction. Slabs should have aspect ratios of about 1:1. Reinforcing would provide additional crack control.

e. The above paving sections have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads should be taken into account.

f. Subgrade should be prepared in accordance with the recommendations in the grading section of this report. The subgrade should be firm and unyielding when proof rolled with a loaded water truck or similar load. Aggregate base should be compacted to a minimum of 95% maximum density and should be firm and unyielding prior to placing concrete.

g. Paving sections probably can be reduced significantly if the subgrade soils are lime-treated. This option can be explored during rough grading.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by ESSC during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

1. Review of the structural and grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of engineered fill, backfilling, and foundation construction.
3. Consultation as required during construction.
4. Special Inspection and material testing.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the borings drilled and CPT's performed on the site and other existing data. The nature and extent of variations between or beyond the borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed are strictly for the information of our Client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

In the event that any changes in the nature, design, or location of the structures, and other improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to insure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth System Southern California has strived to provide our services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of Client and their authorized agents.
It is recommended that Earth Systems Southern California be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If ESSC is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.