PRELIMINARY GEOTECHNICAL
ENGINEERING REPORT
Lancaster Highlands Master-Planned Community
Avenue L and 80th Street West
Lancaster, Los Angeles County, California
PL-07018-01

Prepared For
LANCASTER HIGHLANDS, LLC

November 16, 2007

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Attention: Mr. Jack Hartung

Subject: Preliminary Geotechnical Engineering Report
Lancaster Highlands Master-Planned Community
Avenue L and 80th Street West
Lancaster, Los Angeles County, California

Presented herewith is Earth Systems Southern California’s (ESSC’s) Preliminary Geotechnical Engineering Report prepared, as authorized, for the site of a proposed master-planned development in Lancaster, Los Angeles County, California. The approximate 1,900-acre project site is located adjacent to the southwest corner of Avenue L and 80th Street West, in the City of Lancaster, Los Angeles County, California. The conclusions and recommendations contained in this preliminary report are based upon ESSC’s understanding of the proposed development and on analyses of the data obtained from the field and laboratory testing programs. The preliminary recommendations provided in this report generally relate to criteria for site grading and foundation design. ESSC strives to provide its analyses and recommendations in accordance with the applicable standards of care for the geotechnical engineering profession at the time this study was conducted.

This preliminary report completes ESSC’s scope of geotechnical engineering services authorized on March 5, 2007, which were performed in accordance with ESSC’s proposal dated January 23, 2007. Other services that may be required, such as additional reports, grading plan review, grading observation and construction testing, are additional services and will be billed according to the Fee Schedule in effect at the time such services are provided. Budgets for these services, which are dependent upon design and construction schedules, can be provided when requested.

Earth Systems Southern California appreciates this opportunity to provide professional geotechnical engineering services for this project. If you need clarification of the information contained in this report, or if we can be of additional service, please contact the undersigned.

Respectfully submitted,

Earth Systems
Southern California

Bruce A. Hick
Project Geotechnical Engineer
Distribution: 4 – Lancaster Highlands, LLC
4 – Pacific Stantec
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EARTH SYSTEMS SOUTHERN CALIFORNIA
PRELIMINARY GEOTECHNICAL ENGINEERING REPORT
LANCASTER HIGHLANDS MASTER-PLANNED COMMUNITY
AVENUE L AND 80TH STREET WEST
LANCASTER, LOS ANGELES COUNTY, CALIFORNIA

INTRODUCTION

This Preliminary Geotechnical Engineering Report has been prepared for a proposed master-planned development in Lancaster, Los Angeles County, California. The approximate 1,900-acre project site is located adjacent to the southwest corner of Avenue L and 80th Street West, in the City of Lancaster, Los Angeles County, California. The purpose of Earth Systems Southern California’s (ESSC’s) services was to provide preliminary geotechnical engineering characteristics of the on-site subsurface soils relative to the anticipated site development.

This report includes:

1. Descriptions of the field exploration and laboratory tests performed.

2. Conclusions and preliminary recommendations relating to construction of the proposed development based upon analyses of data obtained from the exploration and testing programs and on knowledge of the general and site specific characteristics of the subsurface soils.

PROJECT DESCRIPTION

Based upon review of the preliminary conceptual plan provided by Pacific Stantec, ESSC understands that the approximate 1,900-acre master-planned community will include residential developments, commercial/retail developments, parks, schools, and open space, along with typical City of Lancaster infrastructure improvements. ESSC has not received site, building or foundation plans for the proposed structures as of this writing. However, based upon past experience, it is anticipated the proposed structures will be single or two-story of wood frame or masonry construction with slab-on-grade ground floors. It is possible that an access bridge may be constructed across the California Aqueduct to allow access to the southern portion of the site. However, deep foundation analysis was not performed to address this condition in this report. Estimated maximum structural loads are 2,500 plf for continuous foundations and 80 kips for isolated column loads.

Due to the existing site topography, ESSC has assumed that a moderate amount of site grading will be required to construct conventional building pads. Maximum cut and fill slope heights of approximately 30 feet anticipated. The proposed mass grading is anticipated to include cut, fill, and transition (cut/fill) lots. All graded slopes are proposed to be finished at gradients of two horizontal to one vertical (2:1), or flatter, and will consist of alluvial cuts, bedrock cut/fill conditions, and fill slopes. Sewage disposal will be provided by a public sewer system. ESSC understands that drainage (both storm water and nuisance water) will be directed into proposed drainage easements and culverts. No significant development/grading is planned in the hillside areas in the southwestern portion of the project. These assumptions were used as the basis for the exploration, testing, and analyses programs, and for the recommendations contained in this report.

EARTH SYSTEMS SOUTHERN CALIFORNIA
PURPOSE AND SCOPE OF SERVICES

The purpose of ESSC’s services was to evaluate the project site soil conditions and to provide preliminary geotechnical engineering conclusions and recommendations relative to the project site and the proposed development. ESSC’s scope of services included the following:

A. A general geotechnical engineering reconnaissance of the site.

B. A geomorphic and stratigraphic assessment and geologic mapping of Quaternary depositional units.

C. Shallow subsurface exploration of the project site by drilling forty (40) test borings and excavation of thirty (30) test pits.

D. Geotechnical laboratory testing of selected soil and bedrock samples obtained from the exploratory soil borings and trenches excavated for this project.

E. Engineering analyses of the data obtained from the exploration and testing programs.

F. A summary of findings and recommendations in this written report.

Contained in this report are:

1. Discussions on local and site specific soil stratigraphic and bedrock conditions.

2. Results of laboratory tests and field data.

3. Preliminary recommendations relating to the proposed site development, including allowable foundation bearing capacity, recommendations for foundation design, estimated total and differential settlements, site grading criteria, lateral earth pressures, soil expansion characteristics, soil corrosion potential, site liquefaction potential, and preliminary pavement sections.

SITE DESCRIPTION

The approximate 1,900-acre project site is located adjacent to the southwest corner of Avenue L and 80th Street West, in the City of Lancaster, Los Angeles County, California. The project site is irregular shaped and extends approximately 3 miles west and approximately 1 mile south from the afore mentioned intersection. The site is located at approximately 34.652° latitude and approximately 118.299° longitude. Access to the site is available from Avenue L, Avenue M and 80th Street West; poorly maintained dirt roads are located adjacent to the northern, southern and eastern boundaries of the site, respectively. Numerous dirt roads and trails traverse the site (see Geologic Map in Appendix A).

Topographically, the site lies at the northern base of the Sierra Pelona and is characterized by rolling hills in the southwestern portions of the site, and a nested set of at least three broad, gently sloping
alluvial fan surfaces within the northern portions of the site. Ground elevations vary from approximately 2,520 feet above mean sea level (msl) in the northeast portion of the site to approximately 3,950 feet msl in the southwestern portion of the site. Each of the defined alluvial fans has a distinct surface gradient, soil profile and geomorphic features. The fan surfaces have intervening moderately dissected drainage courses that reflect modern and ancient drainage patterns. Predominant drainage is to the north and northeast within well-defined drainage courses that exhibit distinct evidence of relatively recent erosion. The California Aqueduct traverses the approximate center portion the site from the northwest corner to the southeast corner of the site. Other improvements include cut and fill slopes associated with the construction of the California Aqueduct. High-tension power lines are located along the northeastern edge of the site.

Vegetation in the project area is generally sparse in the northeastern portion with increased vegetation in the hillier southwest portion of the site. The vegetation observed consisted of native junipers, yucca trees, brush and grasses. A few old foundations were observed in the valleys, south of the California aqueduct. Most of the site is vacant and currently utilized as rangeland.

FIELD EXPLORATION

Field exploration for this study was performed from April 2007 through July 2007 and consisted of geologic mapping, exploratory soils borings and backhoe test pits to review the structural conditions of the bedrock and of the alluvial soils. Select technical literature pertaining to the site geology was also reviewed.

Forty (40) exploratory soil borings were drilled to depths ranging from approximately 16 to 51 feet below the existing ground surface. The borings were drilled with a Mobil B-51 truck-mounted drilling rig using eight-inch diameter continuous flight hollow stem auger in accordance with generally accepted geotechnical exploration procedures (ASTM D 1452). The approximate location of the exploratory borings, as indicated on the attached Geologic Map in Appendix A, were determined by sighting and tape measuring from existing site improvements. The exploration locations should be considered accurate only to the degree implied by the measurement method used.

Thirty (30) shallow exploratory test pits were excavated by backhoe. The walls of the backhoe test pits were observed by ESSC’s engineering geologists to document the structural geologic features observed. The approximate backhoe test pit locations excavated for this report, as indicated on the attached Geologic Map in Appendix A, were determined by pacing and sighting from existing roads and topographic features. The exploration locations should be considered accurate only to the degree implied by the measurement method used.

Bulk disturbed samples of the subsurface soils were obtained from tailings developed during excavation of the test borings and test pits. These samples were secured for classification and testing purposes and represent a mixture of soils within the noted depths.

Soil samples ("ring samples") were secured from within the soil borings using a three-inch O. D. ring sampler (ASTM D 3550). The sampler shoe is similar to the type specified in ASTM D 1586. A 140-pound hammer falling approximately 30 inches (ASTM D 1586) drove the sampler. The number of blows required to drive the sampler one-foot was recorded in six-inch increments.
Recovered soil samples were sealed in plastic containers and brought to ESSC’s laboratory for further classification and testing.

The Boring Logs and Test Pit Logs for this report, included in Appendix A, represent ESSC’s interpretation of the field logs prepared for each boring and test pit by our staff, along with their interpretation of soil conditions between samples and results of laboratory tests. While the noted stratification lines represent approximate boundaries between soil types, the actual transitions may be gradual.

LABORATORY TESTING

After visual and tactile classification in the field, the soil samples were brought to ESSC’s laboratory. The soil classifications were checked in accordance with the Unified Soil Classification System and a testing program was established as follows:

A. Soil samples and field logs were reviewed to assess which samples would be analyzed further.

B. In-situ moisture content and dry unit weight for soil core samples were developed in accordance with ASTM D 2937.

C. The relative strength characteristics of the subsurface soils were estimated from the results of direct shear tests (ASTM D 3080) conducted on select ring samples. The samples were placed in contact with water for at least 24 hours before testing and then sheared under normal loads ranging from 0.5 to 2.3 KSF.

D. The relative strength characteristics of remolded (compacted) samples of the near-surface soils were estimated from the results of direct shear tests (ASTM D 3080) conducted on samples remolded to approximately 90% of maximum dry density as determined by ASTM D 1557 test procedures. The remolded samples were submerged in water for at least 24 hours before testing and then sheared under normal loads ranging from approximately 0.5 to 2.3 KSF.

E. Consolidation tests (ASTM D 2435) were conducted on select ring samples. The maximum stress during testing was 4.6 KSF. The samples were saturated at 2.3 KSF to check the hydrocompression potential. The samples were unloaded to 1.2 KSF to check rebound.

F. M. J. Schiff & Associates of Claremont, California performed soil chemistry tests on a sample of the site soil provided by ESSC. Tests consisted of sulfate, pH and Soil Resistivity, as well as several other chemical content tests.

G. Additional tests consisted of Maximum Density-Optimum Moisture (ASTM D 1557), Expansion Index (ASTM D 4829), and “R”-Value (California Test Method 301).
Refer to Appendix B for the laboratory test results. Presentation of the test results provides only that information considered pertinent. References to ASTM and other test standards refer to the standard currently in effect.

**SUBSURFACE SOIL CONDITIONS**

The soils encountered in the preliminary exploratory borings and trenches are predominately alluvial deposits, consisting of interbedded layers of silty sands and clayey sands (SM and SC soil type based upon the Unified Soil Classification System). Shallow bedrock and several bedrock outcrops were observed in the southwestern portion of the site. Some of the upper seven to nine feet of the site soils were found to be relatively loose/soft, non-uniform, and of low relative compaction. The granular soils (SM and SC soil types) encountered below a depth of approximately nine feet were found to be medium dense to very dense. The Boring Logs and Test Pit Logs in Appendix A contain more detailed descriptions of the soil encountered in the exploratory test boring. Per 2001 California Building Code (CBC) Table 16-J, the site subgrade classification is a “stiff soil” profile ($S_p$) for the $Q_{1a}$, $Q_{1b}$, $Q_2$, and $Q_3$ geologic units and a “very dense soil and soft rock” profile ($S_c$) for the $pos$ and $gr$ geologic units (see Geologic Map in Appendix A).

**Descriptive Geology**

The site is located in an area of low foothills and alluvial fans at the northern base of the Sierra Pelona located at southwestern edge of the Antelope Valley. On-site lithographic units consist of Mesozoic Pelona schist ($pos$), Quartz monzonite ($gr$), Quaternary older alluvial fan sediments ($Q_2$ and $Q_3$), Quaternary old alluvial fan sediments($Q_{1a}$ and $Q_{1b}$), and artificial fill ($af$). The northeastern portion of the project site includes gently sloping younger alluvial fan deposits. The southwestern portion of the site has a ridge and valley topography where Pelona schist outcrops are common. Many naturally occurring drainage channels extend in a north/northeast direction within this portion of the project site. The limited preliminary distribution of geologic units is shown on the Geologic Map in Appendix A. Descriptions of the units encountered on the site are as follows:

**Artificial Fill ($af$)**: Various artificial fill deposits are present related mostly to the construction of the existing California Aqueduct. The material consists predominately of locally derived silty sand with gravel (SM soil type).

**Quaternary Recent Alluvial Sediments ($Q_{1a}$, $Q_{1b}$)**: Alluvial and colluvial sediments are located within the broad drainage courses of the site and upon the lower reaches of the project hillsides. This material consists primarily of loose to medium dense moderate brown silty sand with gravel (SM soil type). These deposits typically exhibit moderate to high hydroconsolidation potentials.

**Quaternary Older Alluvial Sediments ($Q_2$)**: The mid-level nested alluvial fan sediments are found adjacent to modern drainages. They are similar in composition to sediments being deposited in active drainages today. The $Q_2$ sediments comprise an elevated surface on upper slopes that is stranded from the influence of modern drainages. On the lower portion of the site, $Q_2$ soils are buried beneath the modern sediments. The upper portion of $Q_2$ alluvial fan deposit is made up largely of gravelly, interbedded channel deposits that grade downfan into overbank deposits of fine sand and silt. The $Q_2$ soils have a thin, weak argillic horizon that grades coarser with depth. This material
consists primarily of loose to medium dense moderate brown silty sand with gravel (SM soil type). Some of these deposits exhibit moderate to high hydroconsolidation potentials.

**Quaternary Older Alluvial Sediments (Q2):** The native soils encountered on the high-level alluvial fan deposit comprise a relict paleosol. They were found to consist predominantly of dark reddish-brown dense silty sand and gravel, stiff silts, clayey silts and sandy clays (SM, SP-SM, ML and CL soil types). These sediments are capped with a well-developed argillie textural B-horizon and typically grade coarser with depth. They have textural features that are characteristic of very old soils. The Logs of the Test Borings and Test Pits in Appendix A, contain more detailed descriptions of the soils encountered. Some of these deposits exhibit low to moderate hydroconsolidation potentials.

**Mesozoic Pelona Schist (pns):** Bedrock exposed at the site includes Pelona schist, a mica schist with quartz veins and quartzite. Local areas of meta-volcanic rock may also be present. The schist varies in color from gray to black, is well foliated, and highly folded. Foliation attitudes vary over short distances with the fold axes generally having east-west trends. The near surface schist is severely weathered and soft. Moderately hard rock occurs at depth, generally at depths greater than 10-20 feet. The schist, where exposed in cut faces, has high potentials for raveling and instability where foliation or joint attitudes are adverse.

**Mesozoic Quartz Monzonite (gr):** The on-site Quartz monzonite is a felsic igneous rock that has an approximately equal proportion of orthoclase and plagioclase feldspars. The plagioclase is typically intermediate to sodic in composition, andesine to oligoclase. Quartz is present in significant amounts. Biotite and/or hornblende constitute the dark minerals. The near-surface rock is severely weathered and soft. Moderately hard rock occurs at depth, generally at depths greater than 5-10 feet.

**Stratigraphic Soil Sequence**

The sequence of soils in the area of investigation is comprised of at least three different-age soils that are developed on different-age deposits. These physical features and the stratigraphic nomenclature used in this study to define these deposits are described below from youngest to oldest:

A. **Quaternary Recent Alluvium** (Map Symbol- Q1a and Q1b): The youngest Quaternary geological units in the area consist of modern wash/stream channel deposits (Qsc, Qsvc of Ponti, et al., 1981). Wash deposits consist of unconsolidated, coarse to very coarse-grained materials that occupy the modern stream channels. These units are equivalent to Qal and Qsc of Barrows and others (1985) and to Qg of Dibblee (2001). Within the vicinity of the subject property, stream deposits consist of medium to fine-grained sand and occurs near each of five drainages that cross the subject parcel from southwest to northeast.

B. **Quaternary Old Alluvium** (Map Symbol- Q2): Late Pleistocene alluvial fan deposits (Q6m, Q6c; Ponti et al, 1981) is exposed throughout the study area. It consists of predominantly unconsolidated, sandy and silty sediments with moderately developed paleosols that have incipient argillie horizons.

EARTH SYSTEMS SOUTHERN CALIFORNIA
C. Quaternary Older Alluvial Fan (Map Symbol: Q₃): Ancient, pedologically well-developed elevated alluvial fan and stream bed deposits. Late Pleistocene alluvial and fluvial deposits (Map symbol: Q₃) (Q₄c of Ponti, et al., 1981) occur in the southern and western portion of the study area. These deposits also correspond in part to deposits mapped as older alluvium (Qoa, Qos) by Dibblee (2001) and a variety of units mapped by Barrows and others (1985) south of the San Andreas Fault in the southern portion of the quadrangle. Ponti et al, (1981) describe their stratigraphic unit Q₄c as unconsolidated, uplifted, and slightly dissected coarse-grained deposits that have moderately developed soils and argillic clay horizon accumulation.

All of these deposits appear to have been comprised primarily of silty sand and gravel prior to the soil forming process. These deposits and the soils formed on them are considered to be comparable members of a soil chronosequence in which the most distinguishing characteristics are related to the duration of soil development and geomorphic position. Accordingly, the mapped geology should be considered morpho-stratigraphic units. Ages are estimated for each major Quaternary morpho-stratigraphic unit based mainly on soil stratigraphic and geomorphic dating techniques (Table I). Geomorphic criteria described below were emphasized in the age estimations of soil deposits and landforms.

<table>
<thead>
<tr>
<th>STRATIGRAPHIC UNIT</th>
<th>ESTIMATED MINIMUM AGE (X 10³ years)</th>
<th>MARINE OXYGEN ISOTOPE STAGE</th>
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<tr>
<td>A) Flash Flood Deposits₂(Q₁a)</td>
<td>0.1+</td>
<td>Modern Sediment Stage 1</td>
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<td>B) Channel Deposit₂(Q₁b)</td>
<td>2-11</td>
<td>Stage 1</td>
</tr>
<tr>
<td>B) “Basal Gravel”₂(Q₁b)</td>
<td>11-18</td>
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<td>C) Greenfield Terrace₁(Q₂)</td>
<td>58-72</td>
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<tr>
<td>(Paleosol)₄</td>
<td>80-105</td>
<td>Stage 5b</td>
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<tr>
<td>(Sediments)₃</td>
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<tr>
<td>D) Hanford Terrace₁(Q₃)</td>
<td>105-120</td>
<td>Stage 5c</td>
</tr>
<tr>
<td>(Paleosol)₄</td>
<td>120-135</td>
<td>Stage 6</td>
</tr>
<tr>
<td>(Sediments)₃</td>
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NOTES:
1) Stratigraphic nomenclature of the U.S.D.A. (1970) loosely correlates with that adapted for this study (in parentheses)
2) Relative age estimates based on Shlemon & LaChapelle (1993b)
4) Age estimates based on Stratigraphic position & Marine Oxygen Isotope Stage Chronology of Shackleton & Opdyke (1973)
Geomorphic Assessment

The mapping of morpho-stratigraphic units was done for the purpose of distinguishing areas of differing engineering characteristics. The method makes use of the stratigraphic sequence described in the preceding section and considers depositional aspects of the ancient landscape that generated the sequence. It also provides the basis for ESSC's conclusion that the last displacement of the Hitchbrook fault took place prior to the late Pleistocene deposition of depositional unit Q_2 and should not be considered to be active under State of California criteria.

The upper portion of the large apron of coalescing alluvial fans that underlies the site is dissected by streams draining the northeastern flank of the Sierra Pelona. The radial profile of these coalescing fans comprises a set of segmented geomorphic surfaces; three distinct segments are recognized onsite. Each has segment a unique gradient that results from the associated stream profile that existed at the time of deposition.

The oldest segment if the alluvial fan apron is adjacent to the mountain front. This segment comprises the upper portion of the large apron of coalescing alluvial fans that underlies the site is dissected by streams draining the northeastern flank of the Sierra Pelona. The next two younger segments (described above) were observed to underlie the site at progressively shallower surface gradient and higher stream sinuosity. According to Bull (1964, 1977), this geomorphic configuration is an indication of relative landscape stability. A segmented alluvial fan with progressively younger soils on the fan segments in the downfan direction indicates that the mountain front is stable over the period of time represented by the overlying members of the chronostratigraphic sequence.

The first, lowest (stratigraphically) alluvial deposit (map symbol- Q_1) is comprised of subrounded, schist-bearing, well-graded cobble conglomerate that was originally deposited near the base of the north-trending streams at a relatively higher elevation than the modern stream channel adjacent to the mountain front. The base of this ancient channel deposit was not observed except where exposed at locations where the basal contact of the deposit daylights. It may be expected to thin in these areas due to the obsequent topography (i.e., "reversed" with the oldest stratigraphic unit at the highest elevation) upstream from the "hinge point" of the fan. North (i.e., downfan) from the "hinge point" the normal (i.e., progressively older with depth) stratigraphic sequence occurs. The maximum thickness of the deposit was not determined although it may be expected to thicken toward the axis of the ancient channel.

Preliminary exploratory trenching was performed which together with the mapped distribution specifically depict the relationship of units in the stratigraphic sequence. Some soils (pedogenic profiles) that were used to date site geomorphic surfaces and their underlying sediments are described in an agricultural survey of the area (U.S. Department of Agriculture, Soil Conservation Service, 1970). These soils usually have a well-developed subsoil horizon (argillic B) that has at least 20-percent clay accumulation. Additionally, these soil profiles are usually colored reddish-brown and have strong structural features suggesting a strong pedologic development over an extensive period of time.

In contrast, entisols (map symbols Q_1a & Q_1b) typify actively eroding surfaces as well as aggrading valley floors. Soils there are generally pedologically undeveloped; that is, they are characterized
only by a thin organic (topsoil) horizon forming an essentially unweathered parent material (alluvium and colluvium).

The general origin of the site soils, their age and applicability to the paleoseismic assessment of the study area is summarized on Table 1 (page 7). The soils and landform criteria relevant to this stratigraphic assessment are described below:

A. Quaternary Recent Alluvium (Map Symbol - Q₁a and Q₁b)
Undissected alluvial deposits have accumulated across the broad geomorphic surfaces that comprise the floors of each of the five drainages that make up the study area. These deposits are relatively the youngest units observed in the study area. Although no radiometrically dateable material was found during the field exploration phase of this study, these soils are almost certainly young. They are minimally developed entisols that lack diagnostic soil horizonation and appears to comprise a geomorphic surface of recent aggradation with a typically unconformable, abrupt, wavy basal contact with underlying older sediments. Where observed, Q₁a and Q₁b sediments are less than 10-feet thick bearing at most a weakly developed (A-C) soil profile.

B. Quaternary Old Alluvium (Map Symbol - Q₂)
The older alluvium that underlies the recent alluvium is similar in overall composition and texture to the younger alluvium but is quite distinct in terms of pedological development. The basal horizon boundary is an abrupt wavy contact with underlying bedrock and locally, older alluvial fan deposits. The deposit has a distinct stone line at the basal contact and thickens to at least 30-feet near the northeast corner of the parcel (based on unpublished consultant reports on adjacent property to the east).

C. Quaternary Older Alluvial Fan (Map Symbol - Q₃)
These deposits have comparatively well-developed depositional features. They are comprised of interbedded, laterally continuous, well-stratified interfingerling silt and sand lenses. The capping soil has a well-developed B-horizon with clay fraction that generally exceeds that of the parent material by more than 20-percent. The clay appears to be primarily pedogenic in origin. The gravels at the base of this unit are in gradual, broken contact with the underlying older bedrock: granitic rocks north of the Hitchbrook fault and Pelona Schist south of the Hitchbrook fault.

Relative Ages of Stratigraphic Units

The sequence of Quaternary deposition occurred in two stages. First, interfingerling, relatively thin sheets of coarse gravel (fanglematate) were deposited across the site. The source of these schist-bearing gravels was the Sierra Pelona near Portal Ridge south of the Hitchbrook fault. These gravels then served as a source for all subsequent deposition.

Subsequently, cyclic patterns of sedimentation created a set of nested alluvial fans. Each successive fan overlies older units in the northern part of the study area. This area comprises the proximal and medial portion of north-trending piedmont alluvial fans with the apices of the fans eroded or no longer present. Successively younger fans spread northward, burying the distal portion of previous

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fans on the Antelope Valley floor. Each successive fan then has a shallower mean gradient than the preceding one. Individual depositional units comprise a sequence of sediments that generally coarsens with depth and thickens to the north.

In the north-trending drainages, these deposits consist of at least three distinct generations of alluvial deposits (map symbols: Q₁₅, Q₁₆, Q₂, & Q₃). The pedologic development of the two older alluvial deposits has resulted in distinctive features that are useful for assigning a depositional age, for the assessment of the recency of displacement on the Hitchbrooke fault and assessment of areas where overexcavation of compressible sediments will be necessary.

The oldest alluvial fan deposits generally consist of reddish or orange-brown, sandy clay; clayey sand and silty sand with discontinuous sandy gravel lenses and poorly graded gravel conglomerate are near the base. These deposits are generally dense to very dense, dry to slightly moist and range in thickness from a few feet to over 20-feet thick in the study area. Extensive interconnecting fracture networks with carbonate or silcrete-coated fracture surfaces were observed below argillic horizons (i.e., in parent material) in the ancient alluvial fan deposits.

As observed in natural exposures and exploratory pits, alluvium shows a crude graded layering. Individual layers range from a few inches to several feet thick. Many layers have coarse sand and pebbles near the base of the section overlain by successively finer sand grading finer upward to a capping clayey argillic horizon. The contact between individual layers appears to be erosional, the upper (finer) layers thereby missing. This, combined with the observed buried soil horizons (paleosols) all suggest that fan deposition was episodic in nature as might occur during periodic, intense storms. Such processes are the present form of deposition common to arid environments.

The upper sections of some layers also show signs of soil development in the form of pedologic profiles (soil horizons) suggesting that they lay exposed in a stable environment for an extended period. Evidence for this includes blocky to columnar soil structure with carbonate veining in soil fractures, clay fractures, clay film coatings on pore faces, ped faces and bridging individual mineral grains.

Based on relative soil profile development associated with marine oxygen isotope stage chronology the minimum age of these deposits is inferred to be about 80,000 to 102,000 ybp± (years before present) for the middle level fan. The paleosol on this fan must therefore have formed during the subsequent interglacial period from 28,000 to 58,000 ybp± (based on geomorphic criteria and pedological development).

In the northern portion of the site, the youngest alluvium (map symbol- Q₁₆ and Q₁₇) conformably overlies each of the older alluvial deposits in the study area. The broad, northeast-trending alleviated drainages comprise depressions that have received the deepest sections of compressible young alluvium in the study area. The surficial soils in this area have a texture and morphology similar to alluvium that is being actively deposited. The base of the young surficial alluvium is in laterally continuous, smooth, sharp contact with the underlying paleosols. A strong stone line directly overlies this contact with the underlying paleosol and extends across the basal contact of the recent alluvium. This stone line represents a period of intensified erosion followed by regional deposition. It may be correlated with a change in the known climatic record about 12,000 to 18,000 ybp±. This
unconformity then represents a period of non-deposition that may be bracketed by the age of the relict paleosol (>28,000ybp±).

The relict paleosol consists of clayey sand that is very dense and slightly porous. This soil has textural features of very old soil (columnar structure with clay films coating ped faces and pores). The sediments underlying the paleosol are less clay-rich, lighter color fine clayey sand. The relict paleosol represents a period of relative landscape stability that was preceded by a period of unstable aggradation represented by the underlying C-horizon parent material. Past sedimentation and development of drainage patterns occurred under the influence of a considerable larger volume of runoff that occurs in the modern environment. The evidence for this includes the size, configuration and bedload characteristics of these ancient drainages. It is doubtful for example that even the relatively rare intense storms that characterize the modern Antelope Valley environment could possibly transport the large (up to 18-inches diameter) clasts observed near the base of the ancient channel deposits on the relatively gentle (approximate gradient=38:1) floor of the existing stream bed. The existing streams then are “underfit” not in the classic geomorphic sense, but in the sense that they are far too small to have created the drainage that they presently occupy.

**Engineering Characteristics**

Based upon the limited consolidation test results, some of the younger alluvium soils (Q1a, Q1b) and some of the older alluvium soils (Q2) encountered within the top seven to nine feet and some of the older alluvium soils (Q3) encountered within the top three to four feet demonstrated a moderate to high tendency to hydrocompress (i.e., experience a loss in volume upon wetting, with or without additional loading; commonly referred to as “collapsing soil”). The compressible soil consists of a medium to coarse silty sand in a dry, loose condition at depths ranging to 9-feet. The younger alluvium soils (Q1a, Q1b) and some of the older alluvium soils (Q2) tested below a depth of approximately nine feet and the older alluvium soils (Q3) tested below a depth of approximately three feet, through the depths tested, were found to demonstrate a negligible to slight tendency to hydrocompress. Additional geotechnical studies will be required at the grading plan stage to determine the actual depth based upon the specific grading plans.

Based upon the limited Expansion Index Test (ASTM D 4829) results, the upper site soils are considered to have a "very low" (0-20) expansion potential. However some of the bedrock of the Pelona Schist formation are anticipated to have expansion potential in the "low" to "medium" range. Additional tests should be performed on this material if construction is to be performed in areas of this formation. Refer to Section L of the Recommendations section for explanations and recommendations for dealing with expansive soils.

**GROUNDWATER**

No free groundwater was encountered in the limited borings conducted for this preliminary report. It is anticipated that seasonal shallow groundwater conditions may occur within the well-defined drainages that cross the project site, especially where bedrock is shallow. Evidence of shallow groundwater further up the canyons in the southwestern portion of the project site is apparent by the presence of bushes and other vegetation. This groundwater probably is the result of perched groundwater at or near the alluvial/bedrock contact.

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Localized seepage in bedrock areas is anticipated during and after seasonal precipitation, and due to landscape irrigation. Seasonal perched groundwater is also anticipated to occur near buried contacts between alluvium and bedrock. Fluctuations in groundwater levels will occur due to variations in rainfall, regional climate, and other factors.

REGIONAL GEOLOGY

The site is located in an area of low foothills and alluvial fans at the northern base of the Sierra Pelona ridgeline located along the southwestern edge of the Antelope Valley.

The San Andreas rift zone, which is several miles wide, dominates the geology of the southern Antelope Valley. The rift zone is an extensive zone of active and potentially active faults that extends from the Gulf of California to Cape Mendocino in northern California. The Hitchbrook Fault is anticipated to project through the project site (see Geologic Map). However, preliminary field exploration performed for this report indicated that this fault is in-active and not a potential problem for this site. The San Andreas fault, and associated subsidiary faults, is the closest active fault to the site. The San Andreas fault, at its nearest point, is approximately 0.75 miles southwest of the site.

GEOLOGIC HAZARDS

Based on the preliminary site reconnaissance and a review of selected geologic references, the geologic hazards that could affect the proposed development generally include seismically related hazards. These hazards are discussed below.

Fault Rupture

The site is not located within a currently delineated State of California Alquist-Priolo Earthquake Fault Zone (Hart, 2000); however, the Hitchbrook fault is anticipated to extend through the project site (see Geologic Map in Appendix A for the approximate location of this fault). Based upon the limited field exploration performed for this preliminary report, the Hitchbrook Fault is not considered to be an active fault and the potential for future surface fault rupture at the site is considered to be low. While fault rupture would most likely occur along previously established fault traces, future fault rupture may occur at other locations.

Liquefaction

Liquefaction is defined as a loss of strength of saturated cohesionless soil generally due to seismic shaking. Soil types most susceptible to liquefaction are loose, saturated silty to clean fine sands. Based upon the limited site exploration, the shallow alluvial soils encountered below this site consist of sands that are generally in a medium dense to dense state. Static groundwater depths from the borings performed for this report are greater than 50 feet, however there may be a potential along the
southwestern portion of the site (pos and gr geologic units) for a perched water table to be present along the soil/bedrock transition. Where groundwater levels are greater than 50 feet deep, it is generally thought that surface damage from deeper liquefaction will not occur. Therefore, because the preliminary static groundwater levels determined in the preparation of this report were determined to be greater than 50 feet deep and because the foundation soils are relatively dense in nature, it is ESSC’s opinion that the potential for liquefaction related hazards on this site is low.

Seismic Hazards

The site is located in Southern California, which is an active seismic area. The site is within Seismic Zone 4 as designated by the 2001 edition of the California Building Code. Major historic earthquakes felt in the vicinity of Lancaster have usually originated from faults located outside the area. These include the 1857 Fort Tejon, 1872 Owens Valley, 1952 Arvin-Tehachapi, 1971 San Fernando, 1987 Whittier, 1992 Landers and Big Bear events, 1994 Northridge and the 1999 Hector Mine earthquakes. Table II (below) lists significant recorded earthquakes felt in the Lancaster area and the estimated intensity of ground shaking at the site based on the Modified Mercalli Scale. A description of the Modified Mercalli Scale is included as Table III (page 15) of this report.

**TABLE II**

<table>
<thead>
<tr>
<th>Earthquake (Fault)</th>
<th>Approx. Distance to Epicenter (miles)</th>
<th>Earthquake Magnitude*</th>
<th>Estimated Intensity at the Site **</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fort Tejon (San Andreas)</td>
<td>96</td>
<td>8.0</td>
<td>VIII</td>
<td>1857</td>
</tr>
<tr>
<td>Owens Valley (Sierra Nevada)</td>
<td>142</td>
<td>7.6</td>
<td>VI</td>
<td>1872</td>
</tr>
<tr>
<td>Arvin-Tehachapi (White Wolf)</td>
<td>47</td>
<td>7.5</td>
<td>VII</td>
<td>1952</td>
</tr>
<tr>
<td>San Fernando (San Fernando)</td>
<td>18</td>
<td>6.6</td>
<td>VI</td>
<td>1971</td>
</tr>
<tr>
<td>Whittier</td>
<td>43</td>
<td>5.9</td>
<td>IV</td>
<td>1987</td>
</tr>
<tr>
<td>Landers</td>
<td>111</td>
<td>7.3</td>
<td>V</td>
<td>1992</td>
</tr>
<tr>
<td>Northridge</td>
<td>33</td>
<td>6.7</td>
<td>V</td>
<td>1994</td>
</tr>
<tr>
<td>Hector Mine</td>
<td>115</td>
<td>7.1</td>
<td>V</td>
<td>1999</td>
</tr>
</tbody>
</table>

* Moment Magnitude
** Modified Mercalli Scale

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From Table II, it appears that the past maximum intensity of historic earthquakes felt in the Lancaster area due to regional faults has been on the order of VIII on the Modified Mercalli Scale. Estimated maximum Mercalli intensities at the site for a 7.9+ moment magnitude earthquake occurring on the local San Andreas Fault are approximately VIII. Intense ground shaking lasting at least 60 seconds is anticipated. Aftershocks with magnitudes up to 7 are expected.

Landslides and Debris Flows

The alluvial portions of the site have fairly moderate topographic grades with minimal potential for landslides. As is, over-steepened slopes adjacent to current drainage courses may be unstable and subject to failure. However, anticipated site grading should mitigate this potential hazard.

The Pelona schist within the hillside portions of the site is prone to instability, either surficially within the regolith (shallow weathered zone) or along the many shear zones common within the formation. Cut slopes in schist bedrock typically ravel along joint and fracture planes, resulting in debris accumulation at the toe of slopes or shallow wedge failures.

Debris flow hazards typically exist within hillside areas where slopewash or colluvium has accumulated on the slope faces or within defined drainage courses. Potentials for debris flow hazards within or adjacent to the site are considered moderate to high as several defined drainage courses exist within and above the hillside portions of the project site. It is estimated that colluvial thickness in susceptible areas will range from 1-3 feet thick. Debris flow hazards should be further evaluated once development plans are better known.

Slope Stability

Based upon the limited information available to ESSC regarding future slopes within the project site, formal slope stability analysis was not performed for this investigation. Once development plans have been better known, additional field investigation and laboratory testing will be required in specific slope areas, and formal slope stability analysis performed at that time. For preliminary design, cut slopes in alluvial soil may be constructed at 3:1, or flatter gradients. If steeper cut slopes in alluvial soil are required, the slopes may be constructed as recompacted fill slopes and trimmed back to a gradient of 2:1, or flatter. All bedrock and fill slopes can be tentatively designed at 2:1, or flatter gradients. Actual planned slope design will need to be reviewed and analyzed using specific field and laboratory testing.
Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering.

*Masonry A:* Good workmanship, mortar, and design; reinforced, especially laterally and bound together by using steel, concrete, etc.; designed to resist lateral forces.

*Masonry B:* Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

*Masonry C:* Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

*Masonry D:* Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

<table>
<thead>
<tr>
<th>I.</th>
<th>Not felt. Marginal and long-period effects of large earthquakes.</th>
</tr>
</thead>
<tbody>
<tr>
<td>II.</td>
<td>Felt by persons at rest, on upper floors, or favorably placed.</td>
</tr>
<tr>
<td>IV.</td>
<td>Hanging objects swing. Vibrations like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frame creak.</td>
</tr>
<tr>
<td>VII.</td>
<td>Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.</td>
</tr>
<tr>
<td>VIII.</td>
<td>Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.</td>
</tr>
<tr>
<td>IX.</td>
<td>General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.</td>
</tr>
<tr>
<td>X.</td>
<td>Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.</td>
</tr>
<tr>
<td>XI.</td>
<td>Rails bent greatly. Underground pipelines completely out of service.</td>
</tr>
<tr>
<td>XII.</td>
<td>Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.</td>
</tr>
</tbody>
</table>

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Ground Fissuring

Areal subsidence could also occur at the site, but would probably occur on a regional basis. Ground fissuring is a recently observed phenomenon in the northwest Lancaster area and at Edwards Air Force Base. It is thought to be related to extensive groundwater withdrawal and tensional stresses. Documented hazards from ground surface fissuring observed in other areas of California have included foundation distress and adverse settlement, and cracking of pavement and utilities.

At this time, the areas of predominant fissuring in the Antelope Valley are located north of Avenue I in Lancaster. As of this date, ESSC is not aware of documented evidence of structural damage to buildings in the immediate area of the project site attributed to the ground fissuring phenomena. ESSC's personnel observed no obvious evidence of fissuring on this site at the time of the field exploration.

The location of ground fissuring in the Lancaster area appears to be related to specific soil types and the relative location within the area of areal subsidence. Prediction of future areas of fissuring is beyond the current state of practice for this profession, particularly because changes in groundwater pumping and location of well fields could alter the location and magnitude of areal subsidence and associated tensional stresses.

Other Secondary Seismic Hazards

Other seismic hazards related to ground shaking include ground lurching, landslides, tsunamis, seiches, and seismic-induced settlements. Ground lurching is generally associated with fault rupture and liquefaction. As these two hazards are considered unlikely, it is ESSC's opinion that the potential for ground lurching is also low. Because of the relatively flat site topography, hazards from landslides are considered negligible. Because of the inland location of the site, hazards from tsunamis are considered nonexistent.

The California Aqueduct extends from the northwest corner of the site through the southeastern portion of the site. ESSC understands that the Aqueduct is designed to release limited amounts of water if failure of aqueduct embankments occurs. If the aqueduct were to release water, flooding of the site is possible. It is anticipated that existing or planned storm water control facilities would minimize the impact of flooding on the project site.

Seismically induced settlement may occur within the on-site younger alluvial soils. However, the near surface soils will be densified by remedial grading to mitigate most settlement potentials. Additional settlement may occur because of seismic shaking, but will most likely occur on an areal basis.

Erosion

The project site is in an area where minor sheet flooding and erosion may occur within the northeastern portion of the site, while moderate to severe erosion may exist along the southwestern portion of the site. Appropriate site analyses, project design, project construction, and site maintenance should be designed to minimize the erosion potential.
DISCUSSION AND CONCLUSIONS

Based upon the preliminary field exploration, limited laboratory testing, ESSC's understanding of the proposed site development, and past experience, it is ESSC's opinion that the site, when modified as recommended in this report, is suitable for the intended construction. Additional field exploration, testing and analysis will be required once the specific site development plans are better known. The following discussions and conclusions are provided for preliminary site development.

Site Grading

As mentioned in the Subsurface Soil Conditions Section, the upper seven to nine feet of the younger alluvium soils and some of the older alluvium soils (Q1b, Q1b and Q2 Geologic Units) and the upper three to four feet of the older alluvium soils (Q3 Geologic Unit) encountered in this preliminary investigation were found to be relatively loose, non-uniform, of low relative compaction, and anticipated to be subject to significant hydrocompression. Based upon this condition, it is ESSC's opinion that the upper alluvium soils will not provide uniform support for the proposed structures without remedial grading. It is further anticipated that given the random foliation and expansive nature of the Pelona Schist bedrock that construction on this geologic unit will not provide uniform support for the proposed structures without remedial grading. To provide a more uniform bearing for the proposed structures, it is recommended that a recompacted soil mat be constructed beneath all structural foundations and slab-on-grade construction. Refer to Section A of the Recommendations of this report for more detailed discussions and recommendations regarding site preparation.

Due to the high moisture content and expansive nature of the Pelona Schist bedrock material, moisture conditioning and compaction of this material may be difficult. Close monitoring of the soil moisture content and proper choice of compaction and mixing equipment will be required to place these soils as engineered compacted fill.

Foundation Design and Settlements

If the recommendations for site preparation and grading are followed, conventional shallow (continuous and isolated pad) foundations may be used to support the proposed structures. If the recommendations for foundation design and construction are followed, preliminary estimates of total settlement of the proposed structures should be approximately three-quarters of an inch. Differential settlement across a 30-foot span may be as high as 50 percent of the total settlement. Refer to Section I and J of the Recommendations section of this report for more detailed discussions and recommendations regarding foundation design.

Site Geology Considerations

1. The Hitchbrook Fault is the only fault known to extend through the project site; however, based upon preliminary investigation performed for this study, it not considered to be an active fault and the potential for future surface fault rupture within the site is considered to be low.
2. With the last great earthquake occurring along the local portion of the San Andreas fault in 1857, and with an estimated recurrence interval of ±132 years, it is ESSC's opinion that a significant seismic event similar to the 1857 earthquake may occur within the lifetime of the proposed development. Past great earthquakes have had estimated Modified Mercalli intensities in Palmdale of approximately VIII. Peak ground accelerations may exceed 1.0g with design based ground motions on the order of 0.8 to 0.9 g (horizontal).

3. Pelona schist is prone to shallow instability and raveling within cut faces. The grading of stabilization fills for all schist bedrock cut slopes is recommended. Anticipated cut slopes within the parcel area may exhibit potential instability, especially under seismic conditions. Adverse jointing, shear zones, or faults within the internally deformed schist may contribute planar discontinuities or planes of weakness.

4. Potential hazards from debris flows are in the “low” to “high” ranges. The project site is susceptible to flooding and erosion due to seasonal heavy rainfall and runoff. Appropriate project design, site grading, construction, and maintenance should mitigate debris flow, flood and erosion hazards.

5. The on site soils were found to consist of cohesive soils that were relatively dense in nature, groundwater was not observed in any of the borings performed in this preliminary report, based upon this, it is ESSC’s opinion that the potential for on-site liquefaction is low.

Soil Engineering Considerations

1. Proposed cut and fill slopes are anticipated to be stable provided they are designed and constructed in accordance with the preliminary recommendations provided in this report. Bedrock cuts will require slope design that accommodates the local foliation, joints, or shear zones at the location of proposed cuts.

2. As mentioned in the Subsurface Soil Conditions Section, portions of the younger and older alluvial soil demonstrate a negligible to high tendency to hydroconsolidate. Hydroconsolidation is a loss in soil volume upon wetting, with or without additional loading. Before hydroconsolidation, the soil has a relatively open inter-granular structure held in-place by a clay-like or water-soluble binder located between the individual soil grains. The addition of water decreases the binding effect allowing the sand grains to slide together. This causes an overall decrease in soil volume resulting in consolidation. Remedial grading (removal and recompaction) of portions of the site alluvial soils will be required to mitigate the hydroconsolidation potential and low relative density of some of the native soils along with existing artificial fill deposits.

3. If the preliminary recommendations for site preparation and grading presented in this report are followed, conventional shallow continuous foundations may be used to support the proposed foundations or structures anticipated to be constructed for this project.

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4. Provided the recommendations contained in this report are incorporated into project design and construction, the estimated maximum total settlement of structures supported by compacted soils should be within acceptable limits. Settlement of the underlying street alignment soils because of construction of proposed fill slope embankments is estimated to be approximately one (1) inch. Settlement of the proposed fill is estimated to be on the order of 0.2% to 0.4% of the fill height. Total and differential settlement of structures supported by competent bedrock should be negligible.

5. Pavement support characteristics of the site alluvial soils, as determined from the "R"-Value testing results, ranges from good to moderate, depending upon the amount of "fines" (material passing the #200 sieve) within the soil. In general, the coarser-grained material has relatively better pavement support characteristics. Based upon previous experience, fill derived from the schist bedrock is considered to have poor to moderate pavement support characteristics.

RECOMMENDATIONS

Based upon the limited field exploration, laboratory testing, ESSC's interpretation of data from the exploration and testing programs, and past experience, it is ESSC's opinion that the following preliminary recommendations should be incorporated into site preparation, design, and construction of the proposed site.

A. General Site Preparation and Grading

1. Any existing pavements, foundations, vegetation, trash piles, abandoned underground utilities, and other debris should be removed from the proposed construction areas. It is possible that underground facilities (seepage pits, septic tanks, cisterns, foundations, etc.) may be present on the site. All such facilities should be removed in their entirety or properly abandoned. All stripplings and debris should be removed from the site in order to preclude their incorporation in site fill or remedial excavation backfill.

2. Exploration test pits and trench backfills are uncompacted, and are therefore unsuitable for support of structures or pavements. If any structure or other improvements, including paved roads, are located over, or immediately adjacent to, the exploration test pits, it is recommended that the test pit backfill be excavated and replaced with compacted engineered fill, or the structure must be designed to span the trench or pit. Approximate locations of exploratory test pits are shown on the Geologic Map in Appendix A. Locations of the test pits need to be determined in the field at time of grading.

3. Depressions resulting from removals under Items 1 and 2 above should have debris and loose soils removed and filled with suitable soils placed as recommended below.
4. Fill soils used to backfill the exploration pits should be moisture conditioned to near optimum moisture content (±2%) and be uniformly compacted to at least 90% of maximum dry density per ASTM D1557 using mechanical compaction equipment. To aid in the compaction operation, fill should be placed in maximum six inch compacted lifts. Compaction should be verified by testing.

5. In order to minimize potential settlement problems associated with a structure supported on a nonuniform thickness of compacted fill, the geotechnical engineers should be consulted for site grading recommendations relative to backfilling large and/or deep depressions resulting from removals under Item 1 and 2.

6. The excavated on-site soils and bedrock may be used for structural fill on the project when placed as specified in this report. Generation of oversize material (defined as rock clasts greater than six inches in nominal dimension) is anticipated, especially from the schist and monzonite bedrock. Recommendations for the placement of oversize rocks are as follows:

a. Rocks up to 6 inches in maximum dimension may be incorporated into engineered fill provided they are uniformly distributed in the fill.

b. Rocks in the range of 6 to 30 inches in maximum dimension may be incorporated in deeper fill areas. This material should be placed at least 10 feet below finished grade elevation, be no closer than 15 feet from finished slope faces, and should be below or outside any areas anticipated for underground infrastructure installation. Continuous observation by the geotechnical engineer's representative should be provided during placement to confirm adequate placement of fines around the oversize material. Construction problems, including excessive caving and harder digging, will occur in utility trench excavations where oversize rock is incorporated in the fill.

c. Oversize material between 30 and 60 inches in diameter should not be placed closer than 20 feet from the projected downward plane of the road, from any proposed structure, from any property line, or from finished slope faces. Large oversize material should not be placed where excavation for deep utilities will be performed after the fill is placed.

Such oversize material should not be "nested", but individually placed so as to provide a minimum of two feet of select fill between individual rocks and rock layers. Select fill is classified as non-expansive silty sands (SM type soil) with no rock clasts greater than 6 inches in nominal size. Fill placement thickness and compaction requirements should be as specified in this report, depending upon the fill thickness.

d. Rocks greater than 30 inches in maximum dimension should be broken down, disposed of offsite, or utilized in an acceptable manner such as decorative surface landscaping, erosion control or water velocity reduction devices, or as riprap in flood channels.

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7. Soils used in fills less than 30 feet in thickness (measured vertically) should be moisture conditioned to near optimum moisture content (±2%) and be uniformly compacted to at least 90% of maximum dry density per ASTM D 1557 using mechanical compaction equipment. **Compaction should be verified by testing.**

8. That portion of any fill below 30 feet in fills greater than 30 feet in thickness (measured vertically) should be moisture conditioned to optimum moisture content and be uniformly compacted to at least 95% of maximum dry density per ASTM D1557 using mechanical compaction equipment. Blending of the finer grained soils with selected granular soils for this fill zone may be required. Coarser grained soils should be more readily compactable at the higher (over optimum) moisture content. **Compaction should be verified by testing.**

9. Final site grades should be designed and constructed so that all surface water is diverted offsite and not allowed to pond on or near pavement areas. A minimum gradient of 2% is required in landscape areas. Drainage devices should be constructed to divert tributary drainage. Culverts should be constructed with cutoff walls and anti-seep collars to minimize the potential for piping and erosion around the exterior of the culvert.

10. It is recommended that Earth Systems Southern California be retained to provide geotechnical engineering services during the grading and site development phases of the project. This continuity of services will allow for the geotechnical review of the design concepts and specifications relative to the recommendations of this report and will more readily allow for design changes in the event that subsurface conditions differ from those currently anticipated.

B. **Site Specific Grading**

Alluvial soils within the project, within varying depths, demonstrate a significant (greater than 2%) potential to hydrocompress and will require remedial grading procedures to minimize settlement potentials. The following preliminary remedial grading recommendations are based upon geologic units as depicted on the attached Geologic Map. Soils, following scarification and moisture conditions, and fill soils used to backfill remedial excavations, should be moisture conditioned to near optimum moisture content and uniformly compacted to at least 90% relative compaction per ASTM D 1557 using mechanical compaction equipment. To aid in the compaction operations, fill should be placed in maximum six inch compacted lifts. **Compaction should be verified by testing.** The entire area of the level portion of each lot should have the remedial grading described below performed.
Younger Alluvial ($Q_{1a}$ and $Q_{1b}$) and Older Alluvium ($Q_{2}$) Soil Areas

Within younger alluvial ($Q_{1a}$ and $Q_{1b}$) and older alluvium ($Q_{2}$) areas, it is recommended that prior to placing fill, the existing younger soils beneath the areas to be filled or constructed upon should be excavated such that all loose alluvium is removed and replaced with compacted engineered fill. **Estimated** depths of remedial excavations are seven to nine feet. Depths of excavation should be verified in the field such that soils with in-place densities less than 110 pcf should be removed and replaced as engineered fill. The resulting surface should then be scarified to a depth of approximately 6 inches (see Plate C-I). **The base of the remedial excavation across an individual building pad should be a level elevation.**

The soils, following scarification, and fill soils used to backfill any remedial excavations, should be moisture conditioned to near optimum moisture content ($\pm 2\%$) and be uniformly compacted to at least 90% of maximum dry density per ASTM D1557 using mechanical compaction equipment. To aid in the compaction operation, fill should be placed in maximum 6-inch compacted lifts. **Compaction should be verified by testing.** All fills should be keyed and benched into firm soil or bedrock where natural grades to receive fill are 5:1 or steeper (see Plate C-II).

Older Alluvial ($Q_{3}$) Soil Areas

**Estimated** depths of remedial excavations within the older alluvial ($Q_{3}$) areas are a minimum of three (3) to four feet (4) below existing or finished grade, whichever is lower. The exposed surface should have a minimum in-place dry density of 110 pcf. The resulting surface should then be scarified to a depth of approximately 6 inches (see Plate C-I). **The base of the remedial excavation across an individual building pad should be a level elevation.**

The soils following scarification, and fill soils used to backfill any remedial excavations, should be moisture conditioned to near optimum moisture content ($\pm 2\%$) and be uniformly compacted to at least 90% of maximum dry density per ASTM D1557 using mechanical compaction equipment. To aid in the compaction operation, fill should be placed in maximum 6 inch compacted lifts. **Compaction should be verified by testing.** All embankment fills should be keyed and benched into firm soil or bedrock where natural grades to receive fill are 5:1 or steeper (see Plate C-II).

Bedrock (pos and gr) Areas

Building pad and swimming pool areas with bedrock at or near finished grade should be over-excavated to a minimum depth of two feet below finished grade to expedite foundation excavations. An ESSC geologist is required to inspect all foundations within bedrock areas and tests be performed to determine expansion potential of the bedrock material. Deeper removals will be required if expansive clay zones are encountered within the bedrock. In this case, the pad areas should be over-excavated at least five feet, the underlying exposed rock and clay saturated, and the pad reconstructed with the uniformly mixed clay/rock derived fill compacted to a minimum of 90% relative compaction per ASTM D 1557.
In street areas, the bedrock should be over-excavated to provide a minimum of two feet of engineered fill beneath the proposed pavement sections.

Transition Conditions

Areas of cut and thickening fill within building pad areas should be over-excavated to provide a uniform thickness of engineered fill under the level pad area of each lot. A minimum of five (5) feet of fill within the pad area is recommended (see Plate C-I). Steep transition areas (over five foot differentials) should be over-excavated such that the differential thickness of engineered fill under the level portion of each lot does not exceed a 3:1 ratio.

Where bedrock is encountered within the transition condition, the bedrock portion of the level portion of the pad should be over-excavated to provide a uniform thickness of engineered fill under that portion of the lot, with a minimum thickness of engineered fill of five feet.

C. Shrinkage and Subsidence

The following is a listing of the preliminary shrinkage due to excavation, scarification, and compaction of the soils within the designated material types:

Younger Alluvium (Q_{18} and Q_{1b})

The upper about 18 to 24 inches of the younger alluvial soils is estimated to exhibit shrinkage of about 18% when excavated and used as compacted fill. The soils below about 24 inches from the existing ground surface to about eight feet in depth are estimated to exhibit shrinkage of about 15% when excavated and replaced as compacted fill. Soils excavated below about eight-foot from existing grades in younger alluvial areas can be estimated to exhibit shrinkage of about 10% when excavated and used as compacted fill. The estimated shrinkage is based upon compactive effort needed to produce an average degree of compaction of about 92% and may vary depending on contractor methods. Compaction to higher relative compaction values will increase the shrinkage accordingly.

Subsidence of approximately 0.1 to 0.2 feet is estimated over the entire graded area in the younger alluvial areas. Losses due to the stripping, clearing, and grubbing operations may also affect quantity calculations and should also be taken into account. The grading contractor should verify shrinkage and grading yardage estimates.

Older Alluvium (Q_{2})

The upper approximately 18 to 24 inches of the older alluvial (Q_{2}) soils is estimated to exhibit shrinkage of about 14% when excavated and used as compacted fill. The soils below about 24 inches from the existing ground surface to approximately eight feet in depth are estimated to exhibit shrinkage of about 12% when excavated and replaced as compacted fill. Soils excavated below about eight-foot from existing grades in older (Q_{2}) alluvial areas can be estimated to exhibit shrinkage of about 10% when excavated and used as compacted fill. The estimated shrinkage is based upon
compactive effort needed to produce an average degree of compaction of approximately 92% and may vary depending on contractor methods. Compaction to higher relative compaction values will increase the shrinkage accordingly.

Subsidence of approximately 0.1 to 0.2 feet is estimated over the entire graded area in the younger alluvial areas. Losses due to the stripping, clearing, and grubbing operations may also affect quantity calculations and should also be taken into account. The grading contractor should verify shrinkage and grading yardage estimates.

Older Alluvium ($Q_2$)

The upper one to five feet of the older alluvial ($Q_2$) soils below the existing ground surface is estimated to exhibit shrinkage of about 15% when excavated and used as compacted fill. Soils excavated below about five-foot from existing grades in older ($Q_2$) alluvial areas can be estimated to exhibit shrinkage of about 5% when excavated and used as compacted fill. The estimated shrinkage values are based upon compactive effort needed to produce an average degree of compaction of approximately 92% and may vary depending on contractor methods. Compaction to higher relative compaction values will increase the shrinkage accordingly.

Subsidence of approximately 0.05 to 0.1 feet is estimated over the entire graded area in the older alluvial areas. Losses due to the stripping, clearing, and grubbing operations may also affect quantity calculations and should also be taken into account. The grading contractor should verify shrinkage and grading yardage estimates.

Bedrock areas (pos and gr)

The Pelona Schist and Granitic bedrock materials will most likely exhibit bulking characteristics when excavated material is placed as compacted fill. Bulking up to three (3) percent is expected. As previously discussed, grading of bedrock areas is anticipated to generate oversize material (defined as rock clasts greater than six inches in nominal dimension). The removal of significant oversize material (greater than 30 inches in size) from the excavated bedrock material may result in excessive material volume losses ("shrinkage"). Such losses should be taken into account in earthwork calculations.

D. Slope Construction

Fill Slopes

1. Based upon ESSC's preliminary investigation, fill slopes should be constructed at a maximum slope of 2:1 (horizontal to vertical). All fill slopes should be constructed with suitable structural fill that has been properly moisture conditioned and compacted as recommended below.

2. Fill slopes should be compacted in-place to at least 90% of maximum dry density as determined by ASTM D1557 test procedures, with a running average of 92% relative
compaction for the last five tests performed. Fill slopes should be overfilled and trimmed back to compacted material. The final surface of the slopes should be track-walked or grid-rolled to improve the slope resistance to erosion. Compaction should be verified by testing.

3. Where fill slopes are to be constructed on natural slopes steeper than 5:1 (horizontal to vertical), the fill should be keyed and benched into firm soil or bedrock (see Plate C-II, Appendix C). All proposed keyways should be accurately delineated on the grading plans.

4. Keys for all slope construction greater than five feet (5') in height should be cut a minimum of 2 feet into firm soil or bedrock (see Plate C-V, Appendix C). The minimum key dimensions are ten feet (10') horizontal and two feet (2') vertical from the lowest adjacent soil grade. Before fill is placed, the geotechnical engineers, or their representatives should be notified to verify compliance with the above recommendations.

5. Where fill is to be placed over bedrock, a subdrain should be located at the back of the keyway excavation approximately as shown on Plate C-III. It is recommended that all subdrains be located on the grading plan with correct flowline elevations and daylight locations clearly identified. Subdrain outlets should be spaced at maximum 100-foot horizontal intervals and maximum 15-foot vertical intervals. Subdrains should be sloped at a minimum 2% gradient towards the front of the slope (see Plate C-IV). Keyway subdrains are typically located below pad grade elevations. Appropriate outlet pipe elevations and locations should be delineated on the grading plans by the design civil engineer.

6. A protective berm should be constructed and maintained at the top of all fill slopes to divert any runoff away from the slope face.

7. It is recommended that the geotechnical engineers, or their representatives, be present during the fill construction to observe compliance with the above recommendations.

Cut Slopes

1. Based upon ESSC's preliminary investigation, understanding of the current site geology conditions and assumed site grading, cut slopes in on-site alluvial soils should be constructed at a maximum slope gradient of 3:1 (horizontal to vertical) or flatter.

If the proposed cut slope will expose both alluvium and bedrock, it is recommended that the entire portion of the cut should be reconstructed, and a subdrain provided per the recommendations of Section F of this report.
If the proposed cut slope will expose younger alluvium over older alluvium, it is recommended that the entire cut should be reconstructed, and a subdrain provided per the recommendations of Section F of this report.

2. Cut slopes in bedrock should be finished at a slope equal to or flatter than the local foliation or bedding at that location. In general, 2:1 slopes or flatter are recommended where foliation or bedding is not adverse. Local foliation or bedding may require slopes finished at a shallower gradient, or reconstruction as buttressed slopes. The engineering geologists, or their representatives, should monitor bedrock cut slopes during construction, to check for adverse bedding, joint patterns or other geologic features exposed within the cut face.

3. All 2:1 schist bedrock cut slopes greater than five feet in height should be reconstructed with a stabilization fill to minimize hazards from raveling and small surficial “slip-outs” or “pop-outs” (see Plate C–VII).

4. Temporary backcuts for keyways and benches may be excavated vertical up to five feet in height. Backcut failures in bedrock areas may occur during keyway and benching operations. Careful grading operations should be implemented when working adjacent to vertical backcuts. Where unstable conditions may exist, backcut slopes should be laid back to a 1:1.5 or flatter configuration as recommended by the project geologist or geotechnical engineer.

5. As an alternative to the construction of stabilization fills for bedrock cut slopes, it may be possible to construct debris walls at the base of the bedrock slopes and cover the slope faces with chain link fences or other devices. Debris fences or walls should be provided with means to allow for intermittent removal of accumulated debris.

6. Positive drainage should be provided at the tops of all cut slopes to divert runoff away from the cut face. Swales constructed in alluvial soils should be lined with gunite, concrete, or other suitable non-erodible material. Erosion protection should be provided, especially where concentrated runoff is anticipated.

7. Velocity reducers should be provided at the discharge points of the swales or down drains as deemed necessary by the design engineer.

E. Slope Protection and Maintenance

Proper slope protection and maintenance should help minimize erosion and improve the stability of the project slopes. All project slopes are prone to erosion and will require protection and maintenance.

1. A qualified landscape architect should provide recommendations for slope planting. It is strongly recommended that erosion control measures, such as planting, erosion control blankets or fabrics, sprayed tackifiers, or some combination of these, be utilized on the fill slopes.
2. It is critical to provide periodic maintenance and repair of all slopes and drainage systems. Drainage system inlets, outlets, and spillways should be periodically inspected and cleaned of soil and debris.

3. Slope plantings and irrigation systems should be maintained. It is recommended that all project landscaping be provided with automatic sprinkler shutoffs in order to help prevent over-saturation of slope faces and help mitigate surficial slope instability problems. Leaks in the irrigation system should be fixed without delay.

8. All slopes should be periodically inspected for evidence of cracking, erosion, and rodent infestation. Any problems should be repaired immediately.

F. Seepage Control

Based upon ESSC's interpretation of the proposed development plans, certain project areas will require specific grading and/or the installation of subdrains to mitigate potential seepage or mounding resulting from the construction of fill over bedrock or alluvium over bedrock cuts. Landscape irrigation and seasonal precipitation may result in mounded groundwater at alluvial/bedrock contacts and fill/bedrock contacts.

The initial location and type of subdrainage systems should be determined following preparation of project grading plans. Final locations and outlet of subdrainage systems should be determined in the field at the time of grading. The project civil engineer should survey all final outlet locations.

Depending upon the extent of yard landscaping, the use of appropriate "French drains", yard drains, swales, or other drainage devices may be required, especially on those lots where bedrock is relatively shallow.

G. Excavations

1. Special construction techniques or equipment may be required to excavate the bedrock encountered at portions of the site. All excavations should be made in accordance with applicable regulations (including CAL/OSHA). Project safety is the responsibility of the contractor and the owner. ESSC will not be responsible for project safety.

2. Open excavations may be cut vertically to a maximum depth of no more than four feet. Excavations extending between four and ten feet deep should be shored or sloped back from the base of the excavation to at least a 1.5:1 (horizontal to vertical) slope or flatter. If excavations dry out, sloughing may occur.

3. During the time excavations are open, no heavy grading equipment or other surcharge loads (i.e. excavation spoils) should be allowed within a horizontal distance from the
top of any slope equal to the depth of the excavation (both distances measured from the top of the excavation slope).

4. Pre-ripping or overexcavation of utility trench alignments should be considered for bedrock areas, if encountered, to reduce construction problems.

5. It should be realized that relatively impermeable schist or granite bedrock may be below the surface of some of the project lots following site grading. Bedrock may be encountered during excavations for landscaping, foundations for out-structures, retaining walls, swimming pools, spas, etc. on the lots. Excavations that extend into the bedrock may require extra effort depending upon the relative hardness of the schist bedrock. The effort required to excavate the bedrock is difficult to predict as the bedrock varies from relatively soft and easily excavated rock to very hard metavolcanic rock. **Purchasers of the lots on this project should be notified of possible construction difficulties associated with excavations in bedrock.**

6. Adequate measures should be taken to protect any structural foundations, pavements, or utilities adjacent to any excavations.

H. Utility Trenches

Special construction techniques may be required for site utility trench excavations, because of the presence of shallow bedrock within portions of the development. The surface of utility trench backfill frequently settles even when backfill is placed under optimum conditions. Structural units or pavement placed over such backfill should be designed to accommodate such movements. Jetting of utility trench backfill is not recommended.

1. Excavations for utility trenches typically will be in soil, but some rock areas may be encountered, therefore, trenching may require ripping and/or special excavation techniques to penetrate any bedrock or cemented soil layers. Because of the nature of the subsurface soils and presence of shallow bedrock, jetting of utility trench backfill is not recommended. Overbreak of utility trench excavations should be anticipated in areas of bedrock. Possible raveling or shallow “pop-outs” in schist or granite bedrock slopes is also possible during site excavations.

2. Backfill of utility trench excavations within rights-of-way should be placed in strict conformance with the requirements of the governing agency. However, as a minimum it is recommended that utility trench excavation backfill should be moisture conditioned and be uniformly compacted to at least 90% of maximum dry density using mechanical compaction equipment. To aid in the compaction operation, utility trench excavation backfill should be placed in maximum six-inch compacted lifts. **Compaction should be verified by testing.**

3. The provisions of this report relative to minimum compaction standards should govern utility trench excavation backfill within the project boundary. In general, service trench line excavations extending inside the site should be backfilled with
native soils that have been moisture conditioned and uniformly compacted to at least 90% of maximum dry density using mechanical compaction equipment. To aid in the compaction operation, utility trench backfill should be placed in maximum six-inch compacted lifts. **Compaction should be verified by testing.**

4. Pipe bedding material and compaction requirements should be in accordance with the pipe manufacturer’s requirements or the City of Lancaster’s standards. However, as a minimum, it is recommended that any pipe bedding material should be compacted to at least 90% of maximum dry density using mechanical compaction equipment. To aid in the compaction operation, utility trench excavation backfill should be placed in maximum six-inch compacted lifts. **Compaction should be verified by testing.**

5. Backfill operations should be reviewed and tested by the geotechnical engineer’s representative to verify conformance with these recommendations.

I. **Foundations**

1. It is recommended that any building or structure constructed on this site be designed to at least the minimum standards for Seismic Zone 4 as designated by the latest edition of the governing Building Code. The following Table is a summary of the estimated seismic parameters typically required for structural design per the 2001 California Building Code (CBC).

| TABLE IV  
**Summary of Seismic Parameters** |
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<tbody>
<tr>
<td>Seismic Zone</td>
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<tr>
<td>Seismic Source Type (2001 CBC Table 16A-U)</td>
</tr>
<tr>
<td>*San Andreas Fault (&lt;2 km)</td>
</tr>
<tr>
<td>Seismic Zone Factor “Z” (2001 CBC Table 16A-I)</td>
</tr>
<tr>
<td>Near Source Factor - N_s (2001 CBC Table 16A-S)</td>
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<tr>
<td>Near Source Factor - N_v (2001 CBC Table 16A-T)</td>
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</tbody>
</table>

**Alluvial Material (Q_{H1}, Q_{H2}, Q_{2} & Q_{3})**
- Subgrade Classification (2001 CBC Table 16A-J) \( S_D \)
- Seismic Coefficient - C_a (2001 CBC Table 16A-Q) \( 0.44N_s \)
- Seismic Coefficient - C_v (2001 CBC Table 16A-R) \( 0.64N_v \)

**Bedrock Material (pos & gr)**
- Subgrade Classification (2001 CBC Table 16A-J) \( S_C \)
- Seismic Coefficient - C_a (2001 CBC Table 16A-Q) \( 0.40N_s \)
- Seismic Coefficient - C_v (2001 CBC Table 16A-R) \( 0.56N_v \)

2. Foundations for the proposed structures should be supported by compacted soils prepared as recommended in Section A. of this report.
3. Excavations for foundations should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the floor slab unless properly moisture conditioned and compacted.

4. Friction acting along the foundation base may provide resistance to lateral loading. For preliminary design the coefficient of friction may be estimated to be 0.36 for site soils recompacted to approximately 90% of maximum dry density as determined by ASTM D 1557 test methods, and may be used with dead loads. This value includes a reduction factor of 1/3.

5. Additional resistance to lateral loading may be provided by passive earth pressure acting against the sides of foundations or grade beams. For preliminary design this pressure was estimated to be 350 Z PSF, where \( Z = \) Depth (in feet) below the finished ground elevation. In passive pressure calculations, the upper one-foot of soil should be subtracted from the depth, \( Z \), unless confined by pavement or slab. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended). Passive and frictional resistance can be combined without reduction.

1. Slab-on-Grade Construction

1. Interior building concrete slab-on-grade construction should be supported by a uniform thickness of compacted soils prepared as recommended in Sections A and B of this report. Prior to placement of any slab reinforcement, moisture retarder, or sand material, all slab-on-grade subgrades (both interior and exterior) should be reviewed and tested for the required compaction and uniformity of conditions. **Compaction should be verified by testing.**

2. Exterior concrete slab-on-grade construction should be supported by at least 12 inches of compacted soils, uniform in thickness, prepared as recommended in this report (6 inches of excavated and recompacted soils and 6 inches of scarified and recompacted soils). Where slabs will extend over utility trench excavations, observation and testing of the trench backfill should be performed to confirm the compaction and uniformity of conditions of the utility trench excavation backfill. **Compaction should be verified by testing.**

3. In areas of moisture sensitive floor coverings, an appropriate vapor retarder should be installed in order to minimize vapor transmission from the subgrade soil to the slab. The vapor retarder should be evaluated for holes and/or punctures, and the edges overlapped and taped or sealed per the manufacture’s recommendations, prior to placement of concrete. Any holes or punctures observed should be properly repaired. The retarder should be covered with two inches of sand to help protect it during construction. The sand should be lightly moistened just prior to placing the concrete.
4. Reinforcement of slab-on-grade construction is contingent upon the structural engineer's recommendations and the Expansion Index of the supporting soils. Since the mixing of fill soils with native soils could change the Expansion Index, additional tests should be conducted during rough grading to determine the expansion characteristics of the new subgrade soils. It is recommended that all interior and exterior concrete slab-on-grade be reinforced with at least #3 bars on 18-inch centers. **Reinforcement should be placed at mid-depth of the slab.** Additional reinforcement may be required once the final expansion potential of the subgrade soils is known. The structural engineer may also require additional slab-on-grade reinforcement.

5. It should be realized that as a field manufactured project, concrete will crack even under ideal conditions. It is ESSC's experience that concrete shrinkage is more pronounced in the Antelope Valley area due to environmental conditions (high winds, low humidity, and large daily temperature differentials). The use of high slump concrete for foundations and slabs on this project will increase the occurrence and magnitude of shrinkage cracks. It is recommended that the project developers consult with the structural engineer, project concrete contractors and concrete suppliers to formulate appropriate mix designs, placement procedures, and concrete curing procedures in an attempt to reduce the occurrence and magnitude of concrete shrinkage cracking.

6. Cracks that develop in concrete slab-on-grade should be filled and sealed prior to placing floor coverings. Frequent control joints should be incorporated into the slab construction, particularly in the areas of re-entrant corners, to help control cracking.

7. Relatively impervious floor coverings (i.e. vinyl, linoleum, etc.) that cover concrete slab-on-grade may block the passage of moisture vapor through the slab, which may result in damage to the floor covering. It is suggested that after the concrete has sufficiently cured, the slab surface be sealed with a commercial sealant prior to placing the floor covering. The compatibility and recommendations for placing of the concrete sealer, mastic, and floor covering should be verified by the floor covering manufacturer prior to sealing the concrete or placing the floor covering.

8. It is recommended that the proposed exterior perimeter slabs (sidewalks, patios, walkways, etc.) be designed to be relatively independent of foundation stems (free-floating) to help mitigate cracking due to foundation settlement and/or expansion. Frequent joint spacing should be incorporated into concrete slab-on-grade construction, particularly in the areas of re-entrant corners, to help control cracking.

9. Subgrade soils for all concrete slab-on-grade construction should be moisture conditioned to at least optimum moisture content to a depth of at least six inches below the lowest adjacent soil grade within 24-hours prior to placement of concrete. Measures should be taken to maintain optimum moisture until concrete is placed. Actual depths of pre-moistening will be dependent upon the actual Expansion Index of the subgrade soils.