***PRELIMINARY GEOTECHNICAL INVESTIGATION FOR THE PROPOSED WINCHESTER COMMERCIAL CENTER NORTHEAST CORNER OF WINCHESTER ROAD AND WILLOW SPRINGS AVENUE IN THE CITY OF TEMECULA RIVERSIDE COUNTY CALIFORNIA***

***Project No. I05759-10***

***Dated: April 7, 2005***

***Prepared For:***

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April 7, 2005

Project No. I05759-10

Mr. Jeff Adams
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***Subject:*** ***Preliminary Geotechnical Investigation for the Proposed Winchester Commercial Center, Northeast Corner of Winchester Road and Willow Springs Avenue in the City of Temecula, Riverside County, California***

LGC Inland, Inc. (LGC) is pleased to submit herewith our geotechnical investigation report for the proposed Winchester Commercial Development, located at the northeast corner of Winchester Road and Willow Springs Avenue in the Murrieta Hot Springs area of Riverside County, California. This work was performed in accordance with the scope of work outlined in our proposal, dated January 21, 2005. This report presents the results of our field investigation, laboratory testing and our engineering judgment, opinions, conclusions and recommendations pertaining to the geotechnical design aspects of the proposed development.

It has been a pleasure to be of service to you on this project. Should you have any questions regarding the content of this report or should you require additional information, please do not hesitate to contact this office at your earliest convenience.

Respectfully submitted,

***LGC INLAND, INC.***

Mark Bergmann
President

CW/AS/SMP/ts

Distribution: (6) Addressee

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*Attachments:*

## *1.0 INTRODUCTION*

LGC Inland, Inc. (LGC) is pleased to present this geotechnical investigation report for the subject property. The purposes of this investigation were to determine the nature of surface and subsurface soil conditions, evaluate their in-place characteristics, and then provide preliminary grading and foundation design recommendations based on the accompanying site map provided by you. The general location of the property is indicated on the Site Location Map (Figure 1). The Topographic Plot Plan Map you provided was used as the base map to show geologic conditions within the subject site (see Geotechnical Map, Plate 1).

### *1.1 Purpose and Scope of Services*

The purposes of this investigation were to obtain information on the surface/subsurface soil and geologic conditions within the subject site, evaluate the data, and then provide preliminary grading and foundation design recommendations. The scope of our investigation included the following:

- Review of readily available published and unpublished literature and geologic maps pertaining to active and potentially active faults that lie in close proximity to the site which may have an impact on the proposed development (see Appendix A, References).
- Field reconnaissance to observe existing site conditions and coordinate with Underground Service Alert to locate any known underground utilities.
- Geologic mapping of the site.
- Excavating, logging, and selective sampling of five (5) hollow-stem-auger borings to depths between 21 to 5¼ feet. Exploration locations are shown on the enclosed Geotechnical Map (Plate 1) and descriptive logs are presented in Appendix B.
- Laboratory testing and analysis of representative samples of soil materials (bulk and undisturbed) obtained during exploration to determine their engineering properties (Appendix C).
- Engineering and geologic analysis of the data with respect to the proposed development.
- An evaluation of faulting and seismicity of the region as it pertains to the site (Appendix D).
- A Liquefaction Analysis to address the potential for liquefaction at the site (Appendix E).
- Asphaltic concrete pavement analysis (Appendix F).
- Preparation of General Earthwork and Grading Specifications (Appendix G).
- Preparation of this report presenting our findings, conclusions and preliminary geotechnical recommendations for the proposed development.

### *1.2 Location and Site Description*

The subject site is located at the northeast corner of Winchester Road and Willow Springs Avenue, south of Murrieta Hot Springs Road in the Murrieta Hot Springs area of Riverside County, California. The general location and configuration of the site is shown on the Site Location Map (Figure 1).

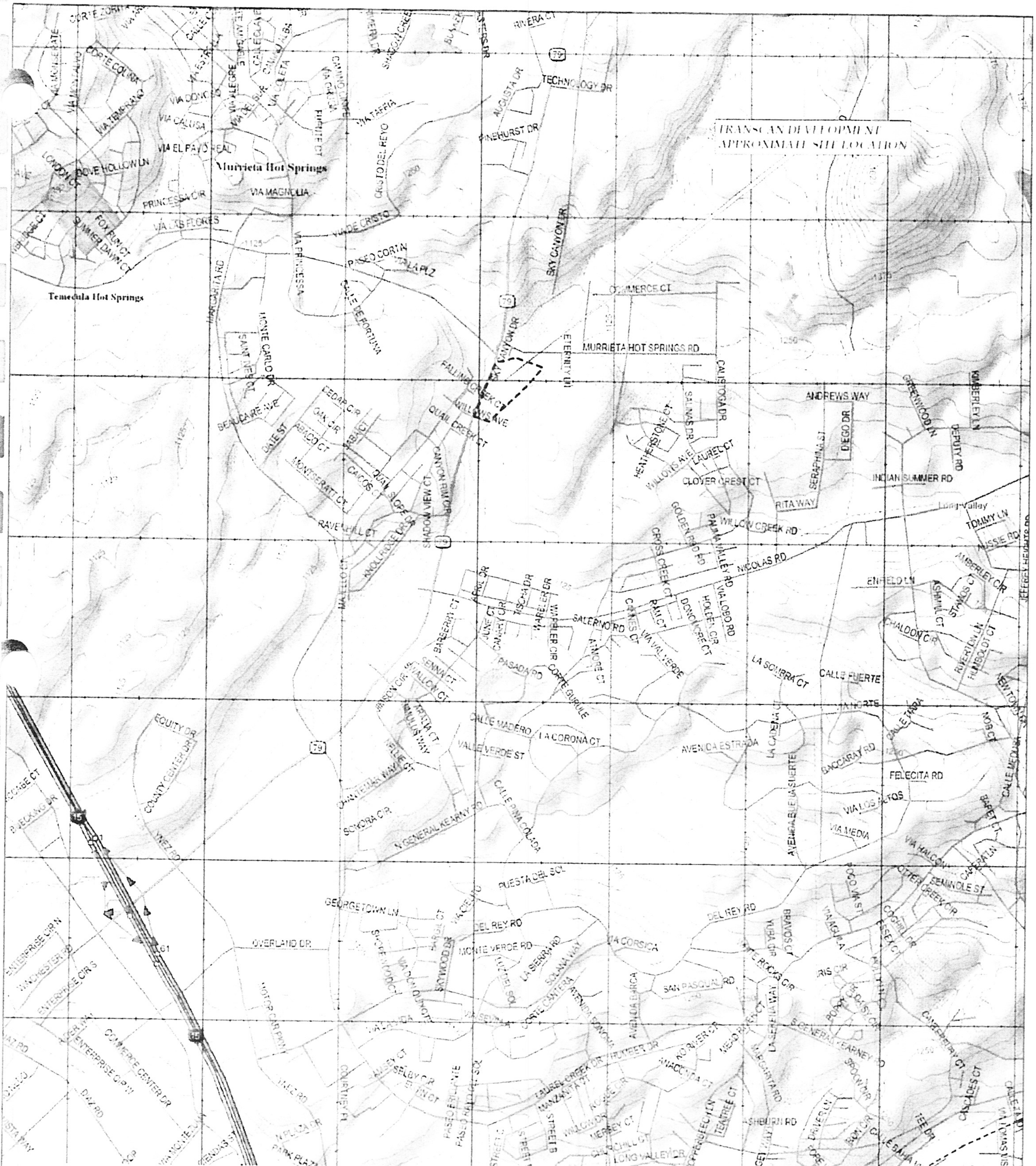


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FIGURE 1
SITE LOCATION MAP

| | |
|---|---|
| Project Name | TRANSCAN DEVELOPMENT |
| Project No. | I05759-10 |
| Geol./ Eng. | MB/ SMP |
| Scale | NOT TO SCALE |
| Date | March 2005 |

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The topography of the site is relatively flat. The general elevation of the property is 1102 feet above mean sea level (msl) with differences of less than 12± feet across the entire site. Local drainage is generally directed to the south.

No underground structures are known to exist at the site. The property has been used to stockpile imported fill dirt. The exploratory test borings revealed at least 13 feet of undocumented fill suggesting a blanket of undocumented fill across most of the subject site. At the time of this report undocumented fill is still being placed at the subject site.

Vegetation consists of a moderate cover of annual weeds/grasses. The site is bounded by Winchester Road (State Route 79), Sky Canyon Drive (proposed alignment), and Willows Avenue to the west, east, and south respectively. An existing commercial and retail development is located north of the subject property. Additionally, the Tucaloca Creek descends to the south and is located roughly parallel with the easterly boundary of the site on the east side of Sky Canyon Drive. The creek currently has running water.

### *1.3 Proposed Development and Grading*

The proposed commercial development is expected to be concrete tilt-up one- and two-story structures utilizing slab on ground construction with associated streets, landscape areas, and utilities. The proposed development includes eight (8) buildings (Buildings A through H) generally located along the perimeter of the site. No grading plan was available at the time this report was prepared.

The Topographic Plot Plan Map, provided by you, was utilized in our investigation and forms the base for our Geotechnical Map (Plate 1). Since the site has never been rough graded and due to the elevations of the existing development to the north, LGC assumes that existing grade elevations will remain essentially unchanged. Cuts and fills should be less than 20 feet in height.

## *2.0 INVESTIGATION AND LABORATORY TESTING*

### *2.1 Field Investigation*

Subsurface exploration within the subject site was performed on February 16, 2005 for the exploratory borings. A hollow-stem-auger drill rig was utilized to drill five (5) borings throughout the site to depths ranging from 21 to 51¼ feet. Prior to the subsurface work, an underground utilities clearance was obtained from Underground Service Alert of Southern California.

Earth materials encountered during exploration were classified and logged in general accordance with the visual-manual procedures of ASTM D 2488. The approximate exploration locations are shown on Plate 1 and descriptive logs are presented in Appendix B.

Associated with the subsurface exploration was the collection of bulk (disturbed) samples and relatively undisturbed samples of soil materials for laboratory testing. The relatively undisturbed samples were obtained with a 3-inch outside diameter modified California split-spoon sampler lined with 1-inch high brass rings. In addition, samples were obtained using a Standard Penetration Test (SPT) sampler. The soil samples obtained with the hollow stem auger drill rig were driven mechanically with successive 30-inch drops of an automatic 140-pound, sampling hammer. The blow count for each six inch increment was recorded in the boring logs. The central portions of the driven-core samples were placed in sealed containers and transported to our laboratory for testing.

### *2.2* *Laboratory Testing*

Maximum dry density/optimum moisture content, expansion potential, grain size distribution, sulfate, R-value, and in-situ density/moisture content were determined for selected undisturbed and bulk samples of soil materials, considered representative of those encountered. A brief description of laboratory test criteria and summaries of test data are presented in Appendix C. An evaluation of the test data is reflected throughout the Conclusions and Recommendations section of this report.

## *3.0* *FINDINGS*

### *3.1* *Regional Geologic Setting*

Regionally, the site is located in the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are characterized by steep, elongated valleys that trend west to northwest. The northwest-trending topography is controlled by the Elsinore fault zone, which extends from the San Gabriel River Valley southeasterly to the United States/Mexico border. The Santa Ana Mountains lie along the western side of the Elsinore fault zone, while the Perris Block is located along the eastern side of the fault zone. The mountainous regions are underlain by Pre-Cretaceous, metasedimentary and metavolcanic rocks and Cretaceous plutonic rocks of the Southern California Batholith. Tertiary and Quaternary rocks are generally comprised of non-marine sediments consisting of sandstone, mudstones, conglomerates, and occasional volcanic units. A map of the regional geology is presented on the Regional Geologic Map, Figure 2.

### *3.2* *Local Geology and Soil Conditions*

The earth materials on the site are primarily comprised of artificial fill and Quaternary aged young axial channel deposits. A general description of the soil materials observed on the site is provided in the following paragraphs:

- Artificial Fill, Undocumented (map symbol Afu): Undocumented artificial fill materials were encountered throughout the site in the upper 1 to 13 feet within the borings. These materials have been imported onto the site and consist generally of olive brown sandy clay, silty sand and clayey sand. These materials are generally inconsistent, poorly consolidated fills.

- Quaternary Young Axial Channel Deposits (map symbol Qya): Quaternary aged young axial channel deposits were encountered below the existing fill to the maximum depth explored of 51¼feet. This alluvial unit consists predominately of interbedded olive grey and brown to yellow brown, fine to coarse grained silty sand, fine to coarse grained sand, sandy clay and silt. This unit is generally slightly moist to wet and loose to medium dense (medium stiff) in condition.

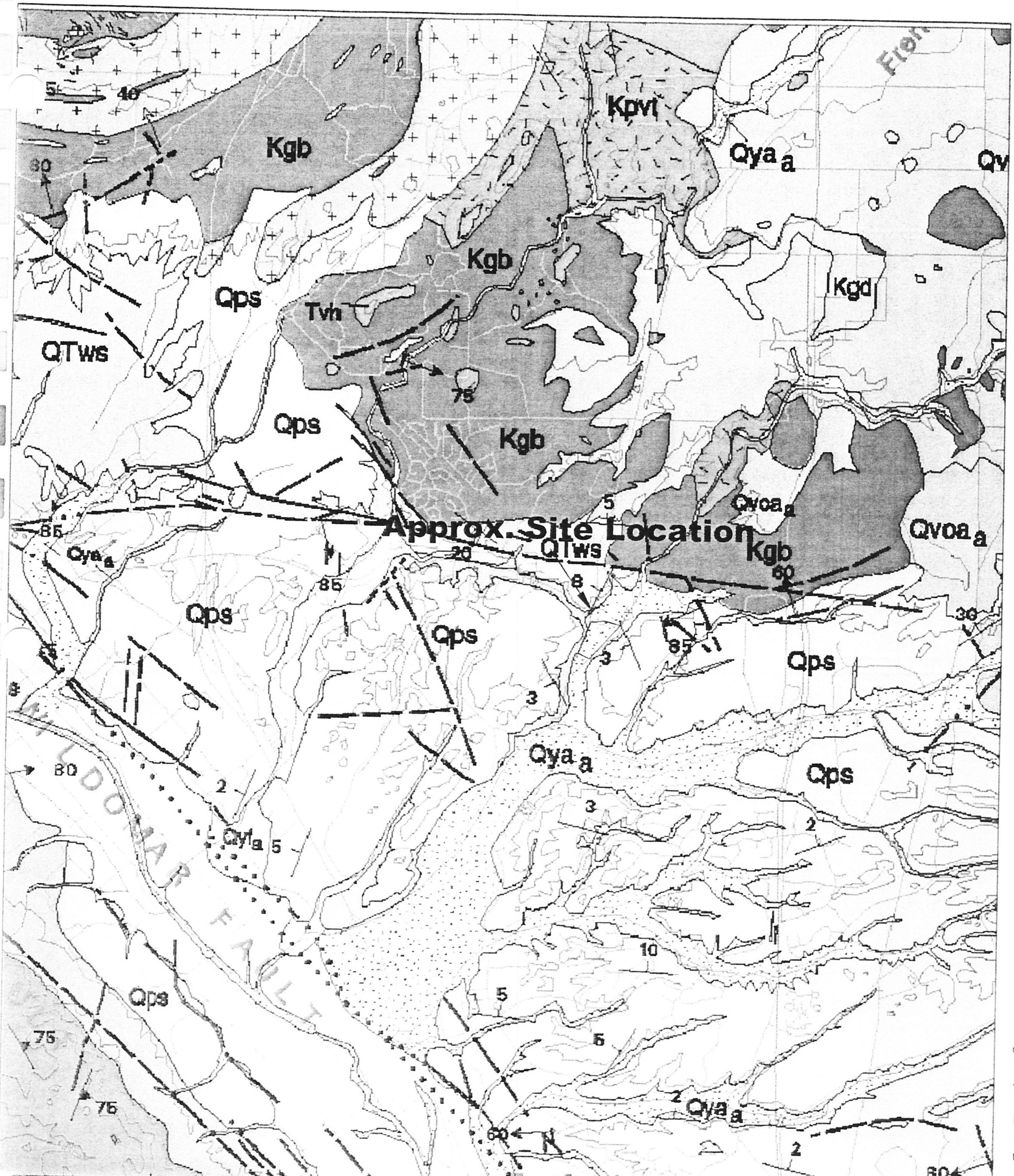


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FIGURE 2
REGIONAL GEOLOGIC MAP

| | |
|---|---|
| Project Name | TRANSCAN DEVELOPMENT |
| Project No. | I05759-10 |
| Geol./ Eng. | MB/ SMP |
| Scale | NOT TO SCALE |
| Date | March 2005 |

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### *3.3 Groundwater*

Groundwater was encountered in Boring 1, Boring 3, and Boring 5 (B-1, B-3, & B-5) at approximately 43, 34, 24 feet, respectively.

### *3.4 Faulting*

The geologic structure of the entire Southern California area is dominated by northwest-trending faults associated with the San Andreas Fault system. Faults, such as the Newport-Inglewood, Whittier-Elsinore, San Jacinto and San Andreas are major faults in this system and all are known to be active. In addition, the San Andreas, Elsinore, and San Jacinto faults are known to have ruptured the ground surface in historic times.

Based on our review of published and unpublished geologic maps and literature pertaining to the site and regional geology, the closest active fault producing the highest anticipated peak ground acceleration at site is the Elsinore - Temecula Fault located approximately 22.5 kilometers to the west. This fault is capable of producing a moderate magnitude earthquake. No active or potentially active faults are known to project through the site and the site does not lie within an Alquist-Priolo Earthquake Fault Zone (previously called an Alquist-Priolo Special Studies Zone).

### *3.5 Landslides*

No landslide debris was noted during our subsurface exploration and no ancient landslides are known to exist on the site.

## *4.0 CONCLUSIONS AND RECOMMENDATIONS*

### *4.1 General*

From a soils engineering and engineering geologic point of view, the subject property is considered suitable for the proposed development, provided the following conclusions and recommendations are incorporated into the design criteria and project specifications.

### *4.2 Earthwork*

#### *4.2.1 General Earthwork and Grading Specifications*

All earthwork and grading should be performed in accordance with all applicable requirements of the Grading and Excavation Code and the Grading Manual of the County of Riverside, in addition to the provisions of the 1997 Uniform Building Code (UBC), including Appendix Chapter 33. Grading should also be performed in accordance with applicable provisions of the General Earthwork and Grading Specifications (Appendix G), prepared by LGC, unless specifically revised or amended herein.

### 4.2.2 *Clearing and Grubbing*

All weeds, grasses, brush, shrubs, debris and trash in the areas to be graded should be stripped and hauled offsite. During site grading, laborers should clear from fills any roots, branches, and other deleterious materials missed during clearing and grubbing operations.

The project geotechnical engineer or his qualified representative should be notified at appropriate times to provide observation and testing services during clearing operations and to verify compliance with the above recommendations. In addition, any buried structures or unusual or adverse soil conditions encountered that are not described or anticipated herein should be brought to the immediate attention of the geotechnical consultant.

### 4.2.3 *Excavation Characteristics*

Based on the results of our exploration, the near surface soil materials, will be readily excavated with conventional earth moving equipment.

### 4.2.4 *Groundwater*

Groundwater was encountered during our subsurface exploration, and is reported to be at a depth of approximately 24 to 43 feet below the existing ground surface. Therefore, groundwater may be a factor during grading or construction.

### 4.2.5 *Ground Preparation – Fill Areas*

All existing low density and potentially collapsible soil materials, such as topsoil, alluvium, and loose manmade fill, should be removed to underlying competent alluvium, from each area to receive compacted fill. Dense native soils are subject to verification by the project engineer, geologist or their representative. Prior to placing structural fills, the exposed bottom surfaces in each removal area should first be scarified to a depth of 6 inches or more, watered or air-dried as necessary to achieve near-optimum moisture conditions and then re-compacted in-place to a minimum relative compaction of 90 percent.

Based on LGC's exploration, anticipated depths of removal are shown on the enclosed Geotechnical Map (Plate 1). In general, the anticipated removal depths should vary from 15 to 21 feet. However, actual depths and horizontal limits of any removals will have to be determined during grading on the basis of in-grading observations and testing performed by the geotechnical consultant and/or engineering geologist.

### 4.2.6 *Disposal of Oversize Rock*

Oversize rock/concrete may be encountered during grading. The disposal of oversize rock is discussed in General Earthwork and Grading Specifications, Appendix G.

#### 4.2.7 *Fill Placement*

Any fill should be placed in 6- to 8-inch maximum (uncompacted) lifts, watered or air-dried as necessary to achieve uniform near optimum moisture content (preferred at or slightly above optimum moisture content) and then compacted in-place to a minimum of 90 percent relative compaction. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D1557-00.

#### 4.2.8 *Import Soils for Grading*

In the event import soils are needed to achieve final design grades, all potential import materials should be free of deleterious/oversize materials, non-expansive, and approved by the project geotechnical consultant prior to commencement of delivery onsite.

#### 4.2.9 *Cut/Fill Transition Lots*

To mitigate distress to structures related to the potential adverse affects of excessive differential settlement, cut/fill transitions should be eliminated from all building areas where the depth of fill placed within the "fill" portion exceeds proposed footing depths. The entire structure should be founded on a uniform bearing material. This should be accomplished by overexcavating the "cut" portion and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation are provided in the following table:

| *DEPTH OF FILL ("fill" portion)* | *DEPTH OF OVEREXCAVATION ("cut" portion)* |
|---|---|
| Up to 5 feet | Equal Depth |
| 5 to 10 feet | 5 feet |
| Greater than 10 feet | One-half the thickness of fill placed on the "fill" portion (10 feet maximum) |

Overexcavation of the "cut" portion should extend beyond the perimeter building lines a horizontal distance equal to the depth of overexcavation or to a minimum distance of 5 feet, whichever is greater.

#### 4.2.10 *Shrinkage, Bulking and Subsidence*

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of shrinkage and bulking factors for the various geologic units found onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

| *GEOLOGIC UNIT* | *SHRINKAGE PERCENT* |
|---|---|
| Artificial Fill | 20 to 25 |
| Channel Deposits | 10 to 15 |

Subsidence from scarification and recompaction of exposed bottom surfaces in removal areas to receive fill is expected to be negligible.

The above estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. **However, these estimates should be used with some caution since they are not absolute values.** Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading.

#### *4.2.11 Geotechnical Observations*

An observation of clearing operations, removal of unsuitable materials, and general grading procedures should be performed by the project geotechnical consultant or his representative. Fills should not be placed without prior approval from the geotechnical consultant.

The project geotechnical consultant or his representative should also be present onsite during all grading operations to verify proper placement and adequate compaction of all fill materials, as well as to verify compliance with the other recommendations presented herein.

### *4.3 Post Grading Considerations*

#### *4.3.1 Slope Landscaping and Maintenance*

Adequate slope and pad drainage facilities are essential in the design of the finish grading for the subject site. An anticipated rainfall equivalency of 60 to 100 inches per year at the site can result due to irrigation. The overall stability of graded slopes should not be adversely affected provided all drainage provisions are properly constructed and maintained thereafter and provided all engineered slopes are landscaped with a deep rooted, drought tolerant and maintenance free plant species, as recommended by the project landscape architect. Additional comments and recommendations are presented below with respect to slope drainage, landscaping and irrigation. A discussion of drainage is given in the following section.

#### *4.3.2 Site Drainage*

Positive-drainage devices, such as sloping sidewalks, graded swales and/or area drains, should be provided around buildings to collect and direct all water away from the structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Roof gutters and downspouts may be required on the sides of buildings where yard drainage devices cannot be provided and/or where roof drainage is directed onto adjacent slopes. All drainage should be directed to adjacent driveways, adjacent streets or storm drain facilities.

#### *4.3.3 Utility Trenches*

All utility trench backfill within the street right-of-ways, utility easements, under sidewalks, driveways and building floor slabs, as well as within or in proximity to slopes should be compacted to a minimum relative compaction of 90 percent. Where onsite soils are utilized as backfill, mechanical compaction will be required. Density testing, along with probing, should be performed by the project geotechnical engineer or their representative to verify proper compaction.

For deep trenches with vertical walls, backfill should be placed in approximately 1- to 2-foot maximum lifts and then mechanically compacted with a hydro-hammer, pneumatic tampers or similar equipment. For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch maximum lifts and then compacted by rolling with a sheepsfoot tamper or similar equipment.

To avoid point loads and subsequent distress to vitrified clay, concrete or plastic pipe, imported sand bedding should be placed at least 1-foot above the pipe in areas where excavated trench materials contain significant cobbles. Sand-bedding materials should be thoroughly jetted prior to placing the backfill.

Where utility trenches are proposed parallel to any building footing (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 horizontal to vertical (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

## *5.0 SEISMIC DESIGN CONSIDERATIONS*

### *5.1 Ground Motions*

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in the 1997 UBC Sections 1626 through 1633. The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and building height.

For structural design in accordance with the 1997 UBC, a computer program developed by Thomas F. Blake (UBCSEIS, 1998) was used that compiles fault information for a particular site using a modified version of a data file of approximately 183 California faults that were digitized by the California Division of Mines and Geology and the U.S. Geological Survey. This program computes various information for a particular site, including; the distance of the site from each of the faults in the data file, the estimated slip rate for each fault and the "maximum moment magnitude" of each fault. The program then selects the closest Type A, Type B and Type C faults from the site and computes the seismic design coefficients for each of the fault types. The program then selects the largest of the computed seismic design coefficients and designates these as the design coefficients for the subject site.

The probabilistic seismic hazard analysis for the site was completed for three (3) different attenuation relationships (Campbell & Bozorgnia, 1997, Sadigh et al., 1997, and Abrahamson & Silva, 1997). The peak ground acceleration value of 0.59 g is the mean of the three (3) values obtained. The probability of exceedance versus acceleration waves for the different attenuation relationships are presented in Appendix D.

Probability curves were calculated using the computer program FRISKSP Version 4.0 (Blake, 2000).

Based on our evaluation, the Elsinore - Temecula Fault zone would probably generate the most severe site ground motions with an anticipated maximum moment magnitude of 6.8 and anticipated slip rate of 5.0 mm/yr. The following 1997 UBC seismic design coefficients should be used for the proposed structures. These criteria are based on the soil profile type as determined by subsurface geologic conditions, on the proximity of the Elsinore - Temecula Fault and on the maximum moment magnitude and slip rate.

| UBC 1997 TABLE | FACTOR |
|---|---|
| Figure 16-2 Seismic Zone | 4 |
| Table 16-I Seismic Zone Factor Z | 0.4 |
| Table 16-U Seismic Source Type | B |
| Table 16-J Seismic Profile Type | $S_D$ |
| Table 16-S Near-Source Factor, $N_a$ | 1.0 |
| Table 16-T Near-Source Factor, $N_v$ | 1.2 |
| Table 16-Q Seismic Coefficient, $C_a$ | 0.44 |
| Table 16-R Seismic Coefficient, $C_v$ | 0.75 |

### *5.2 Secondary Seismic Hazards*

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure as well as induced flooding. Various general types of ground failures, which might occur as a consequence of severe ground shaking of the site, include land sliding, ground lurching, shallow ground rupture, and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsurface soils, groundwater conditions, and other factors. Based on our subsurface exploration, all of the above secondary effects of seismic activity are considered unlikely.

Seismically induced flooding normally includes flooding due to a tsunami (seismic sea wave), a seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. Since the site is located more than 30 miles inland from the nearest coastline of the Pacific Ocean at an elevation in excess of 1,100 feet above mean sea level, the potential for seismically induced flooding due to a tsunamis run-up is considered nonexistent. Since no enclosed bodies of water lie adjacent to the site, the potential for induced flooding at the site due to a seiche is also considered nonexistent.

### *5.3 Liquefaction*

Liquefaction involves the substantial loss of shear strength in saturated soil, usually taking place within a soil medium exhibiting a uniform, fine grained characteristic, loose consistency and low confining pressure when subjected to impact by seismic or dynamic loading. Factors influencing a site's potential for liquefaction include area seismicity, onsite soil type and consistency and groundwater level. The project site will be underlain by compacted fill and competent alluvium with groundwater at a depth of approximately 24 to 43 feet

Liquefaction analyses were performed for the existing (un-graded) site conditions. The soil and groundwater conditions encountered in Boring Nos. 1 were utilized in our analyses. Our field investigation indicated groundwater to be at a depth of 43 feet below the existing surface in Boring Number 1. A conservative level of 5 feet was used for the liquefaction analyses to represent the historic high groundwater level. Our analyses indicated potentially liquefiable soils in Boring No. 1 at depths of 13 to 28 and 43 to 48 feet below the existing ground surface. However, the effects of liquefaction should not be a factor due to the depth of the liquefiable soils along with the volume of overburden materials recommended to be removed and/or reconditioned down to approximately 15 to 23 feet. Therefore, liquefaction should not manifest itself at the surface. The results of the liquefaction analyses are presented in Appendix E.

The potential for earthquake induced liquefaction within the site is considered *low to remote* due to the recommended engineered fill, relatively *low* groundwater, and the dense nature of the deeper onsite soils.

## 6.0 TENTATIVE FOUNDATION DESIGN RECOMMENDATIONS

### 6.1 General

Provided site grading is performed in accordance with the recommendations of this report, conventional shallow foundations are considered feasible for support of the proposed structures. Tentative foundation recommendations are provided herein. However, these recommendations may require modification depending on as-graded conditions existing within the building site upon completion of grading.

### 6.2 Allowable Bearing Values

An allowable bearing value of 2,000 pounds per square foot (psf) is recommended for design of 24-inch square pad footings and 12-inch wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional 1-foot of width and/or depth to a maximum value of 3,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third when designing for short duration wind and seismic forces.

### 6.3 Settlement

Based on the general settlement characteristics of the soil types that underlie the building sites and the anticipated loading, it has been estimated that the maximum total settlement of conventional footings will be less than approximately ¾-inch. Differential settlement is expected to be about ½-inch over a horizontal distance of approximately 20 feet, for an angular distortion ratio of 1:480. It is anticipated that the majority of the settlement will occur during construction or shortly thereafter as loads are applied.

The above settlement estimates are based on the assumption that the grading will be performed in accordance with the grading recommendations presented in this report and that the project geotechnical consultant will observe or test the soil conditions in the footing excavations.

### *6.4 Lateral Resistance*

A passive earth pressure of 250 psf per foot of depth to a maximum value of 2,500 psf may be used to determine lateral bearing resistance for footings. Where structures are planned in or near descending slopes, the passive earth pressure should be reduced to 150 psf per foot of depth to a maximum value of 1,500 psf. In addition, a coefficient of friction of 0.40 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for short duration wind or seismic forces.

The above values are based on footings for an entire structure being placed directly against compacted fill. In the case where footing sides are formed, all backfill placed against the footings should be compacted to a minimum of 90 percent of maximum dry density.

### *6.5 Footing Observations*

All foundation excavations should be observed by the project geotechnical engineer to verify that they have been excavated into competent bearing materials. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture softened soil should be removed prior to concrete placement.

Excavated materials from footing excavations should not be placed in slab-on-grade areas unless the soils are compacted to a minimum 90 percent of maximum dry density.

### *6.6 Expansive Soil Considerations*

Results of preliminary laboratory tests indicate onsite earth materials exhibit expansion potentials ranging from **VERY LOW** to **LOW** as classified in accordance with 1997 UBC Table 18-I-B. Accordingly, expansive soil conditions should be evaluated for individual lots during and at the completion of rough grading. The design and construction details herein are intended to provide recommendations for the various levels of expansion potential, which may be evident at the completion of rough grading.

#### *6.6.1 Very Low Expansion Potential (Expansion Index of 20 or Less)*

Results of our laboratory tests indicate onsite soils exhibit a **VERY LOW** expansion potential as classified in accordance with Table 18-I-B of the 1997 Uniform Building Code (UBC). Since the onsite soils exhibit expansion indices of less than 20, the design of slab-on-ground foundations is exempt from the procedures outlined in Section 1815. Based on the above soil conditions, it is recommended that footings and floors be constructed and reinforced in accordance with the following minimum criteria. However, additional slab thickness, footing sizes and/or reinforcement should be provided as required by the project architect or structural engineer.

#### *6.6.1.1 Footings*

- Exterior continuous footings may be founded at the minimum depths indicated in UBC Table 18-I-C (i.e. 12-inch minimum depth for one-story and 18-inch minimum depth for two-story construction). Interior continuous footings for both one- and two-story construction may be founded at a minimum depth of 12 inches below the lowest adjacent grade. All continuous footings should have a minimum width of 12 and 15 inches, for one-story and two-story buildings, respectively, and should be reinforced with two (2) No. 4 bars, one (1) top and one (1) bottom.

#### *6.6.1.2 Building Floor Slabs*

- Concrete floor slabs should be 4 inches thick and reinforced with No. 3 bars spaced a maximum of 24 inches on center, both ways. All slab reinforcement should be supported on concrete chairs or bricks to ensure the desired placement near mid-depth.

- Concrete floor slabs should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 6 mil visqueen, or equivalent. All laps within the membrane should be sealed, and at least 2 inches of clean sand be placed over the membrane to promote uniform curing of the concrete.

- Prior to placing concrete, the subgrade soils below all living-area and garage area floor slabs should be pre-watered to promote uniform curing of the concrete and minimize the development of shrinkage cracks.

### *6.6.2 Low Expansion Potential (Expansion Index of 21 to 50)*

Onsite soils may exhibit a **LOW** expansion potential as classified in accordance with Table 18-I-B of the 1997 Uniform Building Code (UBC). The 1997 UBC specifies that slab on ground foundations (floor slabs) resting on soils with expansion indices greater than 20, require special design considerations in accordance with 1997 UBC Section 1815. The design procedures outlined in 1997 UBC Section 1815 are based on the thickness and plasticity index of each different soil type existing within the upper 15 feet of the building site. For final design purposes, we have assumed an effective plasticity index of 15 for in accordance with 1997 UBC Section 1815.4.2.

#### *6.6.2.1 Footings*

- Exterior continuous footings may be founded at the minimum depths indicated in UBC Table 18-I-C (i.e. 12-inch minimum depth for one-story and 18-inch minimum depth for two-story construction). Interior continuous footings for both one- and two-story construction may be founded at a minimum depth of 12 inches below the lowest adjacent grade. All continuous footings should have a minimum width of 12 and 15 inches, for one-story and two-story buildings, respectively, and should be reinforced with a minimum of two (2) No. 4 bars, one (1) top and one (1) bottom.

#### *6.6.2.2* ***Building Floor Slabs***

- The project architect or structural engineer should evaluate minimum floor slab thickness and reinforcement in accordance with 1997 UBC Section 1815 based on an effective plasticity index of 15. Unless a more stringent design is recommended by the architect or the structural engineer, we recommend a minimum slab thickness of 4 inches for floor slabs, and be reinforced with No. 3 bars spaced a maximum of 18 inches on center, both ways. All slab reinforcement should be supported on concrete chairs or bricks to ensure the desired placement near mid-depth.

- Concrete floor slabs should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 6 mil visqueen, or equivalent. All laps within the membrane should be sealed, and at least 2 inches of clean sand be placed over the membrane to promote uniform curing of the concrete.

- Prior to placing concrete, the subgrade soils below all floor slabs should be pre-watered to achieve a moisture content that is at least equal or slightly greater than optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches into the subgrade soils.

### *6.7* ***Post Tensioned Slab/Foundation Design Recommendations***

In lieu of the proceeding recommendations for conventional footing and floor slabs, post tensioned slabs may be utilized for the support of the proposed structures. We recommend that the foundation engineer design the foundation system using the geotechnical parameters provided below in Table 1. These parameters have been determined in general accordance with Chapter 18 Section 1816 of the Uniform Building Code (UBC), 1997 edition. Alternate designs are allowed per 1997 UBC Section 1806.2 that addresses the effects of expansive soils when present. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural engineer/architect.

Please note that the post tensioned design methodology reflected in UBC Chapter 18 is in part based on the assumption that soil moisture changes around and beneath the post-tensioned slabs are influenced only by climatological conditions. Soil moisture change below slabs is the major factor in foundation damages relating to expansive soil. The UBC design methodology has no consideration for presaturation, owner irrigation, or other nonclimate related influences on the moisture content of subgrade soils. In recognition of these factors, we have modified the geotechnical parameters obtained from this methodology to account for reasonable irrigation practices and proper homeowner maintenance. In addition, we recommend that prior to foundation construction, slab subgrades be presoaked to 12 inches prior to trenching and maintained at above optimum moisture up to concrete construction. We further recommend that the moisture content of the soil around the immediate perimeter of the slab be maintained near optimum moisture content (or above) during construction and up to occupancy.

The following geotechnical parameters provided in Table 1 assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage so ponding, which causes significant moisture change below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Sunken planters placed adjacent to the foundation, should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters. Based on the design parameters we have provided, and our experience with monitoring similar sites on these types of soils, we anticipate that if the soils become saturated below the perimeter of the foundations due to incorrect landscaping irrigation or maintenance, then up to approximately ¾-inch of uplift could occur at the perimeter of the foundation relative to the central portion of the slab.

Future owners should be informed and educated regarding the importance of maintaining a consistent level of soil moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to become too dry. The soil will undergo shrinkage as it dries up, followed by swelling during the rainy winter season, or when irrigation is resumed. This will result in distress to site improvements and structures.

## *TABLE 1:*

### *Preliminary Geotechnical Parameters for Post Tensioned Foundation Slab Design*

| ***PARAMETER*** | | ***VALUE*** | |
|---|---|---|---|
| Expansion Index | | Very Low | Low |
| Percent that is Finer than 0.002 mm in the Fraction Passing the No. 200 Sieve. | | < 20 percent (assumed) | < 20 percent (assumed) |
| Clay Mineral Type | | Montmorillonite (assumed) | Montmorillonite (assumed) |
| Thornthwaite Moisture Index | | -20 | -20 |
| Depth to Constant Soil Suction (estimated as the depth to constant moisture content over time, but within UBC limits) | | 7 feet | 7 feet |
| Constant Soil Suction | | P.F. 3.6 | P.F. 3.6 |
| Moisture Velocity | | 0.7 inches/month | 0.7 inches/month |
| Center Lift | Edge moisture variation distance, $e_m$<br>Center lift, $y_m$ | 5.5 feet<br>1.5 inches | 5.5 feet<br>2.0 inches |
| Edge Lift | Edge moisture variation distance, $e_m$<br>Edge lift, $y_m$ | 2.5 feet<br>0.4 inches | 3.0 feet<br>0.8 inches |
| Soluble Sulfate Content for Design of Concrete Mixtures in Contact with Site Soils in Accordance with 1997 UBC Table 19-A-4 | | Negligible | Negligible |
| Modulus of Subgrade Reaction, k (assuming presaturation as indicated below) | | 200 lbs/in$^3$ | 200 lbs/in$^3$ |
| Minimum Perimeter Foundation Embedment | | 12 | 18 |
| Rebar in Exterior Footing | | - | - |
| Sand and Visqueen | | Type 1 | Type 1 |

Additional Recommendations:

1. Presoak to 12 inches prior to trenching, maintain at above optimum up to concrete construction

**Sand & Visqueen**

**Type 1**

Install a 10-mil Visqueen (or equivalent) moisture barrier covered by a minimum of 1-inch layer of sand. Note: The builder must ensure that the Visqueen has been lapped and sealed and not punctured as a result of being placed in direct contact with the native soils or by other construction methods.

**Type 2**

Install a 6-milVisqueen (or equivalent) moisture barrier covered by a minimum of 1-inch layer of sand and 2 inches below. **Or** , install a 10-mil Visqueen ( or equivalent) moisture barrier in contact with the native soils an dcovered by a minimum of at least 2 inches of sand. Note: For both options, the builder must ensure that the Visqueen has been lapped and sealed and not punctured as a result of being placed in direct contact with the native soils or by other construction methods.

* The above sand and Visqueen recommendations are traditionally included with geotechnical foundation recommendations although they are generally not a major factor influencing the geotechnical performance of the foundation. The sand and Visqueen requirements are the purview of the foundation engineer/corrosion engineer and the builder to ensure that the concrete cures correctly is protected from corrosive environments and moisture penetration of the floor is acceptable to the future owners. Therefore, the above recommendations may be superceded by the requirements of the previously mentioned parties.

### *6.8 Corrosivity to Concrete and Metal*

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment." From a geotechnical viewpoint, the "environment" is the prevailing foundation soils and the "substances" are the reinforced concrete foundations.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Table 19-A-4 of the U.B.C., 1997, provides specific guidelines for the concrete mix design when the soluble sulfate content of the soils exceeds 0.1 percent by weight.

Based on testing performed within the project area, the onsite soils are classified as having a negligible sulfate exposure condition in accordance with Table 19-A-4, of U.B.C., 1997. Therefore, in accordance with Table 19-A-4 structural concrete in contact with earth materials should have cement of *Type I or II.*

This recommendation is based on limited samples of the subsurface soils. The initiation of grading at the site could blend various soil types and import soils may be used locally. These changes made to the foundation soils could alter sulfate content levels. Accordingly, it is recommended that additional testing be performed at the completion of grading to verify sulfate contents and other chemical properties.

Despite the minimum recommendation above, LGC is not a corrosion engineer, therefore, we recommend that you consult with a competent corrosion engineer and conduct additional testing (if required) to evaluate the actual corrosion potential of the site and provide recommendations to mitigate the corrosion potential with respect to the proposed improvements. The recommendations of the corrosion engineer may supercede the above requirements.

### *6.9 Structural Setbacks*

Structural setbacks, in addition to those required per the UBC, are not required due to geologic or geotechnical conditions within the site. Building setbacks from slopes, property lines, etc. should conform to 1997 UBC requirements.

## *7.0 RETAINING WALLS*

### *7.1 Active and At-Rest Earth Pressures*

An active earth pressure represented by an equivalent fluid having a density of 40 pounds per cubic foot (pcf) should tentatively be used for design of cantilevered walls up to 10 feet high retaining a drained level backfill. Where the wall backfill slopes upward at 2:1 (h:v), the above value should be increased to 60 pcf. All retaining walls should be designed to resist any surcharge loads imposed by other nearby walls or structures in addition to the above active earth pressures.

For design of retaining walls that are restrained at the top, an at-rest earth pressure equivalent to a fluid having a density of 63 pcf should tentatively be used for walls up to 10 feet high supporting a level backfill. This value should be increased to 95 pcf for ascending 2:1 (h:v) backfill. All retaining walls should be designed to resist any surcharge loads imposed by other nearby walls or structures in addition to the above at-rest earth pressures.

### *7.2 Drainage*

Weep holes or open vertical masonry joints should be provided in retaining walls to prevent entrapment of water in the backfill. Weep holes, if used, should be 3 inches in minimum diameter and provided at minimum intervals of 6 feet along the wall. Open vertical masonry joints, if used, should be provided at 32-inch minimum intervals. A continuous gravel fill, 12 inches by 12 inches, should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric to prevent infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

In lieu of weep holes or open joints, a perforated pipe and gravel subdrain may be used. Perforated pipe should consist of 4-inch minimum diameter PVC Schedule 40 or ABS SDR-35, with the perforations laid down. The pipe should be embedded in 1½ cubic feet per foot of ¾- or 1½-inch open graded gravel wrapped in filter fabric. Filter fabric may consist of Mirafi 140N or equivalent.

The backfilled side of the retaining wall supporting backfill should be coated with an approved waterproofing compound to inhibit infiltration of moisture through the walls.

### *7.3* *Temporary Excavations*

All excavations should be made in accordance with OSHA requirements. LGC is not responsible for job site safety.

### *7.4* *Wall Backfill*

Retaining-wall backfill materials should be approved by the soils engineer prior to placement. All retaining-wall backfill should be placed in 6- to 8-inch maximum lifts, watered or air dried as necessary to achieve near optimum moisture conditions and compacted in place to a minimum relative compaction of 90 percent.

## *8.0* *CONCRETE FLATWORK*

### *8.1* *Thickness and Joint Spacing*

To reduce the potential of unsightly cracking, concrete sidewalks and patio type slabs should be at least 3½ inches thick and provided with construction or expansion joints every 6 feet or less. Any concrete driveway slabs should be at least 5 inches thick and provided with construction or expansion joints every 10 feet or less.

### *8.2* *Subgrade Preparation*

As a further measure to minimize cracking of concrete flatwork, the subgrade soils underlying concrete flatwork should first be compacted to a minimum relative compaction of 90 percent and then thoroughly wetted to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content. This moisture should extend to a depth of 12 inches below subgrade and be maintained in the soils during the placement of concrete. Pre-watering of the soils will promote uniform curing of the concrete and minimize the development of shrinkage cracks. A representative of the project geotechnical engineer should observe and verify the density and moisture content of the soils and the depth of moisture penetration prior to placing concrete.

## 9.0 PRELIMINARY ASPHALTIC CONCRETE PAVEMENT DESIGN

A representative sample of soil was tested. The laboratory test results indicated an R-value of 30. Assumed Traffic Indicies are presented in the table below. This table shows our minimum recommended street sections. Further evaluation should be carried out once grading is complete, and R-values have been confirmed. The following asphaltic concrete pavement sections have been computed in accordance with the State of California design procedures. These and alternative asphaltic concrete pavement calculations are attached in Appendix F.

| Preliminary Asphalt Concrete Pavement Design | | |
|---|---|---|
| | ***Auto Parking Area*** | ***Entrance and Heavy Truck Areas*** |
| Assumed Traffic Index | 5.0 | 7.0 |
| Design R-value | 30 | 30 |
| AC Thickness | 0.25 feet | 0.35 feet |
| AB Thickness | 0.50 feet | 0.80 feet |

Notes: AC – Asphaltic Concrete (feet)
AB – Aggregate Base (feet)

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of Class 2 aggregate base conforming to Section 26-1.02B of the State of California Standard Specifications or crushed aggregate base conforming to Section 200-2 of the Standard Specifications for Public Works Construction (Greenbook). Base materials should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557. The base materials should be at or slightly below optimum moisture content when compacted. Asphaltic concrete materials and construction should conform to Section 203 of the Greenbook.

## 10.0 GRADING PLAN REVIEW AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of **TRANSCAN DEVELOPMENT, LLC**. to assist the project engineer and architect in the design of the proposed development. It is recommended that **LGC** be engaged to review the final design drawings and specifications prior to construction. This is to verify that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If **LGC** is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that **LGC** be retained to provide geotechnical engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design, specifications or recommendations and to allow design changes in the event that the subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly (e.g., building loads or type of structures), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during the construction operations that appear to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

## 11.0 *INVESTIGATION LIMITATIONS*

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical investigation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants and incorporated into the plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office at your earliest convenience.

Respectfully submitted,

***LGC INLAND, INC.***

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