VINJE & MIDDLETON ENGINEERING, INC.

2450 Auto Park Way
Escondido, California 92029-1229

Phone (760) 743-1214
Fax (760) 739-0343

Job #13-116-P

April 3, 2013

Spieth & Wohlford, Inc.
c/o Mr. Jack Henthorn
Jack Henthorn & Associates, Inc.
P.O. Box 237
Carlsbad, California 92018

Geotechnical Investigation, Proposed Residential Subdivision
661 Bear Valley Parkway, Escondido, California (A.P.N. 237-131-01 & -02)

Pursuant to your request, Vinje & Middleton Engineering, Inc. has completed the attached Geotechnical Investigation Report for the planned residential subdivision at the above-referenced property.

The following report summarizes the results of our field investigation, including sampling, laboratory analyses and engineering analysis, and provides conclusions and recommendations for the proposed development as understood. From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the planned residential subdivision and associated improvements provided the recommendations presented in this report are incorporated into the design and construction of the project.

The conclusions and recommendations provided in this study are consistent with the site geotechnical conditions and are intended to aid in preparation of final development plans and allow more accurate estimates of development costs.

If you have any questions or need clarification, please do not hesitate to contact this office. Reference to our **Job #13-116-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

S. Mehdi S. Shariat
GE #2885

REGISTERED PROFESSIONAL ENGINEER
SEYED M. SAMSAM SHARIAT
No. 2885
Exp. 12/31/14
GEOTECHNICAL
STATE OF CALIFORNIA

Geotechnical Investigation

Proposed Residential Subdivision
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Prepared For:
Spieth & Wohlford, Inc.
c/o Mr. Jack Henthorn
Jack Henthorn & Associates, Inc.
P.O. Box 237
Carlsbad, California 92018

Prepared By:
VINJE & MIDDLETON ENGINEERING, INC.
2450 Auto Park Way
Escondido, California 92029

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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
661 BEAR VALLEY PARKWAY, ESCONDIDO, CALIFORNIA
(A.P.N. 237-131-01 & -02)

I. INTRODUCTION

The property investigated herein consists of approximately 40 acres of largely undeveloped terrain located on the east side of Bear Valley Parkway, south of Highway 78 within the limits of the City of Escondido. The location is shown on a Vicinity Map attached to this report as Plate 1. The approximate site coordinates are 33.10°N latitude and -117.06°W longitudes.

We understand that the project property is planned for the development of residential subdivision with the associated roadways and underground improvements. Consequently, the purpose of this investigation was to determine soil and geotechnical conditions at the property and to ascertain their influence upon the planned development, as currently understood. Test pit digging, seismic survey lines, soil sampling, and laboratory tests were among the activities conducted in conjunction with this effort which has resulted in the planning and geotechnical development recommendations presented herein.

II. SITE DESCRIPTION / HISTORY

A Geotechnical Map, reproduced from a topographic survey provided to us by Jack Henthorn & Associates, Inc. is included as Plate 2. As shown, the property consists of two parcels with a south trending, teardrop shape configuration and a combined area of approximately 42 acres.

Topographically, the site is characterized by a large hilltop and a north-south trending ridgeline that enters the northeast area of the property and transitions into descending hillside terrain in the southern reaches of the property. South and southwest hillside regions of the property are marked by local, less prominent ridgelines with intervening canyon flowlines or swale-like terrain. Northerly portions of the property ascend at gentle to modest gradients from Bear Valley Parkway eastward to the property line and beyond. Southeasterly hillside regions descend at modest gradients to a local flowline that marks the eastern property limits. Overall, lower site topography along the east and west site margins descend toward the south to prominent local flowlines.

Maximum vertical relief from the highest point at the property (hilltop at 678 feet MSL) to the lowest areas on the site (southerly flowline areas at 540 feet MSL) is on the order of 140 feet. Natural hillside gradients largely approach 3:1 (horizontal to vertical) maximum overall, however, more steeper terrain locally ranging to near vertical occurs in association with local canyon terrain and the prominent flowlines that cross the southeast and south portions of the property.

Surface areas of the property consist generally of natural, barren hillside terrain. Drainage terraces, consisting of shallow fills, were constructed within the last five years to help control site runoff from entering off-site locations. Existing graded surfaces at the property include small level graded pads in the upper northerly areas of the property that support an occupied dwelling and detached garage, and a small vacant cut-fill pad on the more gentle hillside west of the existing dwelling. A lower access roadway was constructed by filling across the southern drainage course flowline (see Plate 2). A water weir and culverts are present to allow for water to continuously flow under the access way, which no longer appears in use. Another flowline crossing, with a subsurface culvert, was constructed over the lower portions of the southwest canyon flowline. Additional fill soils, apparently associated with previous mining activities at the property, were used to fill vertical mine excavations or cover horizontal excavations.

Existing graded embankments at the property are limited to near vertical cut slopes, ranging to approximately 12 feet high, constructed along portions of the western site boundary adjacent to Bear Valley Parkway (see Plate 2). The exposed cuts were likely created in conjunction with the construction of Bear Valley Parkway. Brow ditches are not present and runoff is allowed to sheet flow over the slope faces which are subject to local surface erosion. However, gross instability is not in evidence.

Site drainage sheetflows over site sloping terrain to developed drainage terraces where the runoff is slowed and directed toward intervening canyon areas and lower flowlines. Local erosion of the drainage terraces has occurred. Shallow erosional ruts that include local piping through more consolidated upper soils occur in the southeast and southwest areas of the property near the prominent flowlines and lower canyon areas, as indicated on Plate 2. Some scouring and erosion are present along the flowline that crosses the southeast corner of the property. Portions of this flowline have little to no vegetation to impede largely uncontrolled runoff. This flowline was dry at the time of our field investigation. The other prominent flowline marking the southwest property boundary is heavily overgrown with mature trees. An earthen berm marks the top of the project side of the flowline to help control runoff from the project site. A continuous flow of water was noted within this flowline during our field investigation. Excessive scouring or erosion is not present along this drainage course.

More recently, the property was utilized for agricultural purposes and supported an avocado grove with associated improvements. Tree stumps, older irrigation pipeline, and valves remain as remnants of the grove (including a stockpile of metal irrigation pipe on the project hilltop).

Prior to the avocado grove, the site was subject to mining activities which were conducted approximately 50 to 70 years ago. Detailed documentation of the mining activities is not available. However, from a research of available mining documents for the local area and interviews, we understand there are existing mining shafts and horizontal adits at the property. Consequently, as part of the seismic investigation, three seismic tomography lines were conducted at the project property along suspected mine locations. The tomography surveys were conducted by SubSurface Surveys under the direction of this office. Seismic tomography uses mathematical modeling of P and S wave travel time to map subsurface structures, such as mine shafts or adits. The speeds of the generated shock waves relate to layers of differing density or rock type. In this case, a Bison 9024 signal enhancement seismograph and a standard spread using 24 geophones at 10-foot spacing was used. The results are presented with this report in an Attachment.

Our field investigation included test pits at suspected mine locations and anomalies shown on the seismic tomography lines. Subsequently, four existing mining excavations were identified at various test pit locations within the property as follows:

1. A nearly horizontal mine tunnel (adit) was exposed at the bottom of Test Pit 4, (TP-4) excavated through the underlying bedrock. The exposed excavation measured approximately 6 feet high by 7 feet wide trending in an N15°E direction. No supports were present, and the extent of the mine tunnel is unknown.

2. A nearly horizontal fille shaft was encountered at the Test Pit 7, (TP- 7) location, on the south side of the existing detached garage. The shaft excavation measured 9 feet square with the bottom of shaft extending beyond the 16-foot limit of our backhoe into crystalline bedrock. A secondary filled excavation appeared to the present on the westerly side of the shaft at 9 feet below the surface. This excavation appeared to trend slightly downward in an N75°W direction.

3. A well-developed horizontal mine tunnel (adit) was exposed 4 feet below the surface at the Test Pit 9, (TP- 9) location, near the upper beginning of the southwest canyon flowline. The hillside opening to this horizontal mine excavation measured 17 feet wide by 7 feet high and extended due west into the hillside for 17 feet. At this point, the mine tunnel narrows to approximately 3 feet wide and continues for an estimated 50-60 feet due west where it appears to trend northward. The entire mine tunnel was excavated into crystalline bedrock with no evidence of artificial support.

4. A filled mine excavation was identified at the Test Pit 15, (TP-15) location. This excavation originates on the south facing hillside with the mine opening measured 7 feet high by 4 feet wide and was exposed at 7 feet below the surface. The excavation appears to be nearly horizontal and trending N25°E into the hillside.

Other locations at the property are suspected of mine excavations due to periodic filling of sinkholes that generally appear after periods of heavy or prolonged rains. Overall extent of the vertical shafts or horizontal adits is unknown. Methods used to destroy or backfill the shafts, other than filling with soil, are also unknown. All known mine excavation locations, and those identified during site development will require special handling and mitigation as outlined in following sections.

III. PROPOSED DEVELOPMENT

Project grading and improvement plans are not yet developed and specific development details including final design elevations and height and gradients of planned graded embankments are not currently known. A tentative development map, provided by Hunsaker & Associates, is reproduced herein as Preliminary Development Concept, Plate 3.

The Preliminary Development Concept depicts the general proposed cut and fill areas (blue represents fill areas while red represents cut areas) as well as a subdivision and roadway layout for the planned development at the property. Specific design lines and grades are unknown, however, relatively significant cuts and fills are expected for developing level building pad surfaces and roadway alignments. Based on the Preliminary Development Concept (Plate 3), the higher areas of the property will be cut down and the generated material used to fill the lower perimeter terrain for achieving design elevations. Graded embankments are expected to be on the order of 20 feet high and are recommended at 2:1 gradients maximum. Required setbacks from the existing flowline tributaries will be maintained, as required.

Detailed building construction is not available. Future residential building construction is anticipated to consist of conventional wood-framed with exterior stucco structures supported on shallow foundations with stem-walls and slab-on-grade floors, or slab-on-ground with turned-down footings.

IV. SITE INVESTIGATION

Geotechnical and subsurface conditions at the property were chiefly determined by the excavation of 27 test pits dug with a tractor-mounted backhoe. Subsurface investigations were supplemented by six seismic refraction lines and three seismic tomography lines conducted along the project ridgelines, corestone outcrop areas, and suspected mining locations. All the test pits were logged by our project geologist who also retained representative soil and rock samples at selected locations and intervals for laboratory testing. Approximate locations of the test excavations and seismic refraction / tomography lines are shown on the Geotechnical Map, Plate 2. Logs of the test pits are attached as Plates 4 through 31. Compiled seismic data is detailed in the enclosed Attachment. Laboratory test results and engineering properties of elected samples are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

The property consists of ridgeline and hillside terrain underlain by crystalline bedrock mantled by surficial soils. Previous mining activities conducted at the site have resulted in vertical and nearly horizontal mine shafts and tunnels. Evidence of impending slope instability, faults, or significant shear zones are not indicated at the project property. The following geotechnical conditions are apparent and will influence site development:

A. Earth Materials

Bedrock - Bedrock units beneath the property are chiefly dark-colored gabbroic rocks intruded by felsic quartz-rich veins and lighter granitic rock types that are common in local areas. Project exposures are dominated by deeply weathered fine to coarse-grained rocks that grade harder with depth. The bedrock also locally includes spherical corestones. These are much harder rocks that consist of residual boulders in surface exposures and subsurface seismic surveys in the southeast area of the property.

Excavations into the project bedrock are expected to largely produce good quality gravelly sandy granular soils suitable for site new fills and backfills.

Colluvium - Lower perimeter terrain and local hillside areas are covered by a modest to thick mantle of colluvial soils. Site colluvium consists largely of fine to medium grained sandy deposits that locally include some clay-bearing soils. Project colluvium thickens toward the south margin of the property where developed exposures appear ancient and consolidated. Overall, shallow deposits of colluvium were found in damp and loose conditions, and thicker colluvium deposits become more uniformly medium dense to dense at depth.

Alluvium - Local alluvium deposits were exposed within local canyon flowlines in the south areas of the property. Upslope of the canyon flowline crossing in the southwest area of the site, the alluvium was measured to be 10 feet thick and was found in a very loose condition overall.

Topsoil - Shallow topsoil deposits occur along ridgelines and hilltops at the property. Site topsoil consists of silty sand that was found in damp and very loose conditions overall.

Fill - Existing fill deposits at the property include minor fills associated with existing pad areas and newer drainage terraces. More significant existing fill deposits are associated with the southwest canyon crossing and with covering or filling old mine excavations. The mine-related fills range from sandy to clayey deposits that were found in very loose conditions overall.

The approximate distribution of earth deposits at the site are shown on the enclosed Geotechnical Map, Plate 2. Details of site earth materials are given on the enclosed test pit logs, Plates 4-31. The subsurface relationship of site earth materials is depicted on Geologic Cross-Sections enclosed with this report as Plates 31 through 32.

B. Slope Stability

Project hillsides are underlain by competent crystalline bedrock units which typically perform well in natural and graded slope conditions. Slope instability is not indicated along the project natural hillside areas. Oversteepened graded cut slopes along Bear Valley Parkway are impacted by surface erosion and should be laid back to more gentle gradients as part of the project development / street improvements.

Future graded cut embankments exposing crystalline bedrock are expected to be grossly stable to anticipated design heights. All graded slopes should be constructed as recommended herein and provided with well-developed brow ditches. Runoff should not be allowed to occur in a concentrated flow conditions or flow over slope faces.

C. Surface Flow and Subsurface Groundwater

Subsurface water was not encountered in project test excavations to the depths explored. However, local weathered bedrock units can transmit near-surface water from up-slope terrain through joint and shear surfaces that could impact lower improvements. At the project site, moisture sensitive improvements may require the installation of a slope toe drains. All canyon flowlines within the planned development areas will also require the installation of canyon type subdrain, as specified in the following sections.

An active stream flow condition was noted in the southerly tributary flowline that marks the southwest margin of the site. The flowline that traverses the southeast portion of the property was dry at the time of our investigation. Both tributary flowlines will be subject to seasonal conditions and varying degrees of flow conditions. The southwest flowline is overgrown with trees and vegetation and has no evidence of excessive scouring or erosion. The southeast flowline, however, is largely barren of vegetation and is susceptible to scouring and erosion if not improved. Storm water control is critical to the stability of project lower terrain. Uncontrolled runoff should not be allowed to flow over project sloping terrain to the lower flowlines. Storm water runoff control facilities should be designed and installed as necessary and appropriate. Rock-lines sidewalls or armor protection for the southeasterly flowline may be necessary to limit erosion.

Like all graded hillside developments, the proper control of surface and storm water drainage is an important component in the continued stability of the site. Ponding should not be allowed and over watering of site vegetation should be avoided.

D. Rock Hardness

Local bedrock units are largely weathered gabbroic rocks that become more dense and hard at depth. Final grades are unknown, however, significant cutting of the hilltop and hillsides are expected to achieve finish pad grades and new street alignments. Consequently, added study of hard rock conditions at the project site were conducted as a part of this work. Six seismic refraction lines were conducted across ridgelines, hilltops, and suspected hard rock areas at the project site. The surveys were conducted by SubSurface Surveys under the direction of this office. Seismic refraction surveys measure the velocity of controlled compressional waves (P-waves) through the underlying rock medium. The speed of the generated shock waves relates to rock density or hardness and can also indicate layers of differing density. In this case a Bison 9024 signal enhancement seismograph and a standard spread using 24 geophones at 10-foot spacing were used. The results are presented with this report in Attachment A.

The data indicates the presence of weathered bedrock units beneath the project site. In general, project bedrock is in the rippable range for a Caterpiller D9 to depths ranging from 20 to 40 feet below the surface. Rocks below are harder, but still considered to be rippable (D9 or equivalent). Very hard rocks that will require blasting are currently not indicated. Isolated residual corestone rocks are present in the southeast area of the property, but are not indicated or expected elsewhere. Cross-sections of the seismic survey lines indicating subsurface velocities are included in Attachment A.

E. Faults/Seismicity

Faults or significant shear zones are not indicated on or near proximity to the project site.

As with most areas of California, the San Diego region lies within a seismically active zone; however, coastal areas of the county are characterized by low levels of seismic activity relative to inland areas to the east. During a 40-year period (1934-1974), 37 earthquakes were recorded in San Diego coastal areas by the California Institute of Technology. None of the recorded events exceeded a Richter magnitude of 3.7, nor did any of the earthquakes generate more than modest ground shaking or significant damages. Most of the recorded events occurred along various offshore faults which characteristically generate modest earthquakes.

Historically, the most significant earthquake events which affect local areas originate along well known, distant fault zones to the east and the Coronado Bank Fault to the west. Based upon available seismic data, compiled from California Earthquake Catalogs, the most significant historical event in the area of the study site occurred in 1800 at an estimated distance of 15.5 miles from the project area. This event, which is thought to have occurred along a segment of an offshore fault, reached an estimated magnitude of 6.5 with estimated bedrock acceleration values of 0.084g (site class B) and 0.145g (site class D) at the project site. The following list represents the most significant faults which commonly impact the region. Estimated ground acceleration data compiled from Digitized California Faults (Computer Program EQFAULT Version 3.00 updated) typically associated with the fault is also tabulated.

TABLE 1

Fault Zone	Distance from Site	Maximum Probable Acceleration (R.H.)	
Elsinore-Julian	16.9 miles	0.109g (B) 0.173g (D)	Site Class B Site Class D
Rose Canyon	16.2 miles	0.101g (B) 0.173g (D)	Site Class B Site Class D
Newport-Inglewood	21.7 miles	0.081g (B) 0.139g (D)	Site Class B Site Class D
Coronado Bank	30.8 miles	0.081g (B) 0.138g (D)	Site Class B Site Class D

The locations of significant faults and earthquake events relative to the study site are depicted on a Fault - Epicenter Map attached to this report as Plate 33.

More recently, the number of seismic events which affect the region appears to have heightened somewhat. Nearly 40 earthquakes of magnitude 3.5 or higher have been recorded in coastal regions between January 1984 and August 1986. Most of the earthquakes are thought to have been generated along offshore faults. For the most part, the recorded events remain moderate shocks which typically resulted in low levels of ground shaking to local areas. A notable exception to this pattern was recorded on July 13, 1986. An earthquake of magnitude 5.3 shook county coastal areas with moderate to locally heavy ground shaking resulting in $700,000 in damages, one death, and injuries to 30 people. The quake occurred along an offshore fault located nearly 30 miles southwest of Oceanside.

A series of notable events shook county areas with a (maximum) magnitude 7.4 shock in the early morning of June 28, 1992. These quakes originated along related segments of the San Andreas Fault approximately 90 miles to the north. Locally high levels of ground shaking over an extended period of time resulted; however, significant damages to local structures were not reported. The increase in earthquake frequency in the region remains a subject of speculation among geologists; however, based upon empirical information and the recorded seismic history of county areas, the 1986 and 1992 events are thought to represent the highest levels of ground shaking which can be expected at the study site as a result of seismic activity.

In recent years, the Rose Canyon Fault has received added attention from geologists. The fault is a significant structural feature in metropolitan San Diego which includes a series of parallel breaks trending southward from La Jolla Cove through San Diego Bay toward the Mexican border. Test trenching along the fault in Rose Canyon indicated that at that location the fault was last active 6,000 to 9,000 years ago. More recent work suggests that segments of the fault are younger having been last active 1000 - 2000 years ago. Consequently, the fault has been classified as active and included within an Alquist-Priolo Special Studies Zone established by the State of California.

Fault zones tabulated in the preceding table are considered most likely to impact the region of the study site during the lifetime of the project. The faults are periodically active and capable of generating moderate to locally high levels of ground shaking at the site. Ground separation as a result of seismic activity is not expected at the property.

F. Seismic Ground Motion Values

For design purposes, site specific seismic ground motion values were determined as part of this investigation in accordance with the California Building Code (CBC). The following parameters are consistent with the indicated project seismic environment and our experience with similar earth deposits in the vicinity of the project site, and may be utilized for project design work:

TABLE 2

(Bedrock or less than 10 feet of fills under building foundations)

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
B	1.079	0.395	1.0	1.0	1.079	0.395	0.719	0.263
According to Chapter 16, Section 1613 of the 2007 California Building Code.								

TABLE 3

(10 feet or more of fills under building foundations)

Site Class	S_s	S_1	F_a	F_v	S_{Ms}	S_{M1}	S_{DS}	S_{D1}
D	1.079	0.395	1.068	1.61	1.152	0.636	0.768	0.424
According to Chapter 16, Section 1613 of the 2007 California Building Code.								

Explanation:

Ss: Mapped MCE, 5% damped, spectral response acceleration parameter at short periods.
S1: Mapped MCE, 5% damped, spectral response acceleration parameter at a period of 1-second.
Fa: Site coefficient for mapped spectral response acceleration at short periods.
Fv: Site coefficient for mapped spectral response acceleration at 1-second periods.
SMS: The MCE, 5% damped, spectral response acceleration at short periods adjusted for site class effects (SMS=FaSs).
SM1: The MCE, 5% damped, spectral response acceleration at a period of 1-second adjusted for site class effects (SM1=FvS1).
SDS: Design, 5% damped, spectral response acceleration parameter at short periods (SDS=⅔SMS).
SD1: Design, 5% damped, spectral response acceleration parameter at a period of 1-second (SD1=⅔SM1).

Site peak ground accelerations (PGA) based on 2 percent probabilities of exceedance in 50 years defined as Maximum Considered Earthquake (MCE) with a statistical return period of 2,475 years is also evaluated herein in accordance with the requirements of CBC Section 1613 and ASCE Standard 7-05. Based on our analysis, the site PGA_{MCE} was estimated to be 0.45g using the web-based United States Geological Survey (USGS) ground motion calculator. The design PGA determined as two-thirds of the Maximum Considered Earthquake (MCE) was estimated to be 0.30g.

G. Geologic Hazards

Natural geologic hazards are not presently indicated at the project site. Hillside areas are underlain by competent crystalline bedrock and do not indicate gross geologic instability. The major geotechnical concerns at the property are those associated with existing known and unknown mining excavations which may result in future ground failures or sinkholes. Mitigation of the existing mine excavations and those identified during site development is given in following sections. Other less significant geologic hazards at the site are those associated with ground shaking in the event of a major seismic event. Liquefaction or related surface ruptures associated with faulting are not anticipated.

H. Field and Laboratory Tests and Test Results

Earth deposits encountered in our exploratory test excavations were closely examined and sampled for laboratory testing. Based upon our test pit and field exposures site soils have been grouped into the following soil types:

TABLE 4

Soil Type	Description
1	Brown-red brown silty fine to medium sand (Topsoil / Colluvium / Fill)
2	Red brown clayey sand (Topsoil / Colluvium)
3	Red brown-grey fine to coarse sand (Fill / Bedrock)
4	Red brown clayey sand - sandy clay (Fill / Topsoil)
5	Reddish tan fine grained granitic rock (Bedrock)

The following tests were conducted in support of this investigation:

1. **Grain Size Analysis:** Grain size analyses were performed on representative samples of Soil Types 1, 2, 3, and 4. The test results are presented in Table 5 and graphically presented on the enclosed Plate 34.

TABLE 5

Sieve Size		½"	#4	#10	#20	#40	#200
Location	Soil Type	Percent Passing					
TP-2 @ ½'	1	100	99	93	81	71	31
TP-1 @ 2'	2	100	100	98	93	86	53
TP-8 @ 4½'	3	100	89	60	38	25	7
TP-4 @ 3½'	4	100	100	96	85	73	47

2. **Maximum Dry Density and Optimum Moisture Content:** The maximum dry density and optimum moisture content of Soil Types 1, 2, 3, and 4 were determined in accordance with ASTM D-1557. The results are presented in Table 6.

TABLE 6

Location	Soil Type	Maximum Dry Density (Ym-pcf)	Optimum Moisture Content (ωopt-%)
TP-2 @ ½'	1	138.3	9
TP-1 @ 2'	2	131.8	12
TP-8 @ 4½'	3	131.0	10
TP-4 @ 3½'	4	125.8	13

3. **Moisture-Density Tests (Undisturbed Chunk Samples):** In-place dry density and moisture content of representative soil deposits beneath the site were determined from relatively undisturbed chunk samples using the water displacement test method. Results are presented in Table 7 and tabulated on the attached Test Pit Logs.

TABLE 7

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (Yd-pcf)	Max. Dry Density (Ym-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP- 1 @ 2'	2	14	104.0	131.8	79	56
TP- 1 @ 7'	3	12	-	131.0	Sample Disturbed	52
TP- 2 @ 3½'	1	7	105.8	138.3	77	29
TP- 2 @ 6½'	3	4	136.7	131.0	100+	38
TP- 3 @ 5½'	3	8	128.1	131.0	98	58
TP- 4 @ 6'	3	6	138.7	131.0	100+	61
TP- 5 @ 3'	2	13	112.7	131.8	86	64
TP- 5 @ 7'	1	15	116.7	138.3	84	82
TP- 5 @ 9½'	3	14	119.6	131.0	91	81
TP- 6 @ 7'	3	5	132.0	131.0	100+	41
TP- 10 @ 4'	1	11	120.4	138.3	87	65
TP- 10 @ 6'	1	10	118.0	138.3	85	56
TP- 10 @ 10'	1	11	120.3	138.3	87	65

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (Yd-pcf)	Max. Dry Density (Ym-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP- 12 @ 2'	1	9	123.5	138.3	89	58
TP- 12 @ 4'	1	13	117.2	138.3	85	71
TP- 12 @ 6'	1	12	-	138.3	Sample Disturbed	55
TP- 12 @ 10'	1	10	123.6	138.3	89	65
TP- 12 @ 14'	1	11	119.0	138.3	86	64
TP- 13 @ 5'	3	13	117.1	131.0	89	71
TP- 14 @ 4'	1	12	108.8	138.3	78	54
TP- 14 @ 8'	1	12	120.7	138.3	87	73
TP- 14 @ 10'	1	13	117.9	138.3	85	73
TP- 14 @ 12'	1	11	119.9	138.3	87	65
TP- 16 @ 4'	1	6	119.9	138.3	87	36
TP- 16 @ 6'	1	10	120.0	138.3	87	59
TP- 16 @ 9'	3	5	138.9	131.0	100+	51
TP- 17 @ 5'	1	9	120.7	138.3	87	55
TP- 20 @ 2'	1	9	123.2	138.3	89	58
TP- 20 @ 6'	1	13	123.3	138.3	89	84
TP- 21 @ 8'	3	4	129.3	131.0	99	30
TP- 22 @ 5'	1	10	117.1	138.3	85	55
TP- 22 @ 9'	3	7	121.4	131.0	93	43
TP- 23 @ 6½'	3	14	113.4	131.0	87	70
TP- 24 @ 6'	3	10	125.4	131.0	96	68

Note 1: Sample may be somewhat disturbed.
Assumptions and relationships:
In-place Relative Compaction = (Yd ÷ Ym) X100
Gs = 2.85
e = (Gs Yω ÷ Yd) - 1
S = (ω Gs) ÷ e

4. **Expansion Index Test:** Two expansion index (EI) tests were performed on representative samples of Soil Types 2 and 4 in accordance with the ASTM D-4829. The results are presented in Table 8.

TABLE 8

Sample Location	Soil Type	Molded ω (%)	Degree of Saturation (%)	Final ω (%)	Initial Dry Density (PCF)	Measured EI	EI 50% Saturation
TP-1 @ 2'	2	9.9	50.0	19.6	109.8	38	38
TP-4 @ 3½'	4	10.9	50.6	22.9	106.5	56	56

(ω) = moisture content in percent.
$EI_{50} = EI_{meas} - (50 - S_{meas}) ((65 + EI_{meas}) \div (220 - S_{meas}))$

Expansion Index (EI)	Expansion Potential
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
〉130	Very High

5. **Consolidation Test:** A consolidation test was performed on a representative remolded sample of on-site Soil Type 2. The test results are graphically presented in the enclosed Plate 35.

6. **Direct Shear Test:** Two direct shear tests were performed on representative samples of Soil Types 2 and 4. The prepared specimens were soaked overnight, loaded with normal loads of 1, 2, and 4 kips per square foot respectively, and sheared to failure in an undrained condition. The results are presented in Table 9.

TABLE 9

Sample Location	Soil Type	Sample Condition	Unit Weight (Yw-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
TP-1 @ 2'	2	remolded to 90% of Y_m @ % ωopt	132	26	250
TP-4 @ 3½'	4	remolded to 90% of Y_m @ % ωopt	129	29	200

7. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 4 was determined using "Method for Estimating the Service Life of Steel Culverts," in accordance with the California Test Method (CTM) 643. The result is tabulated in Table 10.

TABLE 10

Sample Location	Soil Type	Minimum Resistivity (OHM-CM)	pH
TP-4 @ 3½'	4	1400	7.5

8. **Sulfate Test:** A sulfate test was performed on a representative sample of Soil Type 4 in accordance with the California Test Method (CTM) 417. The result is presented in Table 11.

TABLE 11

Sample Location	Soil Type	Amount of Water Soluble Sulfate In Soil (% by Weight)
TP-4 @ 3½'	4	0.001

9. **Chloride Test:** A chloride test was performed on a representative sample of Soil Type 4 in accordance with the California Test Method (CTM) 422. The result is presented in Table 12.

TABLE 12

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
TP-4 @ 3½'	4	0.001

10. **R-Value Test:** One R-value test was performed on a representative sample of Soil Type 2 in accordance with the California Test Method 301. The result is presented in Table 13.

TABLE 13

Location	Soil Type	Description	R-Value
TP-1 @ 2'	2	Red brown sandy silt to sandy clay	14

VI. SITE CORROSION ASSESSMENT

A site is considered to be corrosive to foundation elements, walls and drainage structures if one or more of the following conditions exist:

* Sulfate concentration is greater than or equal to 2000 ppm (0.2% by weight).
* Chloride concentration is greater than or equal to 500 ppm (0.05 % by weight).
* pH is less than 5.5.

For structural elements, the minimum resistivity of soil (or water) indicates the relative quantity of soluble salts present in the soil (or water). In general, a minimum resistivity value for soil (or water) less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Appropriate corrosion mitigation measures for corrosive conditions should be selected depending on the service environment, amount of aggressive ion salts (chloride or sulfate), pH levels and the desired service life of the structure.

Results of limited laboratory tests performed on selected representative site samples indicate that the minimum resistivity is more than 1000 ohm-cm suggesting presence of low quantities of soluble salts. Test results further indicated pH levels are greater than 5.5, sulfate concentrations are less than 2000 ppm, and chloride concentration levels are less than 500 ppm. Based on the results of limited corrosion analyses, the project site is considered non-corrosive. The project site is not located within 1000 feet of salt or brackish water.

Results of the tested soil sample indicated that the amount of water soluble sulfate (SO_4) is 0.001 percent by weight which is considered negligible according to ACI 318, Table 4.3.1. Portland cement Type II may be used. Table 14 is appropriate based on the pH-Resistivity test result:

TABLE 14

Design Soil Type	Gage	18	16	14	12	10	8
4	Years to Perforation of Steel Culverts	28	36	44	61	78	95

VII. HYDRO MODIFICATIONS

Project stormwater management, if appropriate and as applicable, should be designed and constructed considering the site indicated geotechnical conditions. The implemented management practice(s) should also have no short and long term impacts on the building pad surfaces, graded slopes and natural embankments, fills and backfills, structures, and onsite and nearby off improvements.

Specific hydro modification designs, if required for the project development, are currently unavailable. Bio-retention and filtration systems consisting of vegetated buffers or strips and self-contained retention/detention areas with impermeable liners on sides and bottom, special engineered sand filter media and perforated pipe(s) which discharge into approved storm water facilities are typical methods for the stormwater Best Management Practices (BMP), if applicable. Additional and more specific recommendations should be provided by the project geotechnical consultant at the final plans review phase, as necessary.

VIII. CONCLUSIONS

Based upon the foregoing investigation, development of the project property for a residential subdivision and associated improvements substantially as proposed, is feasible from a geotechnical viewpoint. The site is underlain by crystalline bedrock units that are mantled by a thin to moderate soil cover. The most significant geotechnical factor expected to impact project development is the presence of known and unknown horizontal and vertical mine tunnels and excavations. The following geotechnical conditions are unique to the property and will most influence its development:

1. Natural geologic hazards including landsliding, faults, or significant shear zones are not indicated at the project property. The project is not located in or near Alquist - Priolo earthquake fault zone areas associated with nearby active faults.

2. On-site natural slopes and hillside terrain are geologically stable. Landslides or other forms of geologic instability which could preclude site development as planned are not indicated.

3. Mine related excavations are known to be present at the property which has resulted in vertical (shafts) and horizontal (adits) excavations. Limited site explorations completed as a part of this effort encountered evidence of mine shafts and adits as discussed herein and where indicated in the enclosed test pit logs. Presence of mine excavations is also suspected elsewhere at the property as evidenced by the reported periodic filling of sinkholes that generally develop after periods of heavy rain fall.

4. Overall extent of the vertical shafts or horizontal adits, and methods used to destroy or backfill the shafts are unknown. Some excavations and adit openings were apparently backfilled with loose soils. Additional tunnel conditions are suspected elsewhere. Actual locations and extend of all mine shafts and adits at the project property should be identifiable during site mass grading operations when surface soils have been stripped and underlying bedrock exposed. All mine shafts and adits, where currently backfilled with loose soils, should be re excavated and regraded during the project mass grading operations. Remaining shafts and adits should also be properly backfilled with compacted fills or cement slurry, or capped with concrete as appropriate and approved in accordance with the requirements of this report.

5. Surficial soils and upper highly weathered bedrock at the project site is unsuitable in their present condition for the support of new compact fills, embankments, structures and improvements. All existing fills, topsoil, alluvium, upper soft to loose colluviual deposits, and upper highly weathered bedrock in areas planned for new fills, structures, and improvements should be regraded as specified in following sections.

6. Below the upper soil cover and highly weathered bedrock, dense and competent natural colluvium and crystalline rock units occur, which will adequately support planned new fills, structures, and improvements.

7. Added removals of cut ground and reconstruction to design grades with compacted fill will also be necessary in the case of cut-fill pads so that uniform bearing soil conditions are constructed throughout the buildings and improvement surfaces. Undercutting the cut portions of the building pads will also help trenching for foundations and underground utilities into an otherwise very hard bedrock units.

8. Based upon our site observations, available subsurface exposures, and the results of a seismic refraction survey, weathered to moderately weathered bedrock units at the project property are expected to excavate to the anticipated design grades with moderate to heavy efforts using larger bulldozers (Caterpillar D-9 or equivalent). Very hard bedrock units requiring major specialized excavation techniques such as significant concentrated ripping and blasting efforts are not currently indicated. However, residual corestones or "floaters" are locally present at the property which are expected to create some handling and grading difficulties.

9. Earth deposits generated from the site bedrock excavations will predominantly consist of good quality gravelly sandy mixtures with possible small rock fragments. Harder bedrock units may generate some larger rock sizes. Selective rock sizes (less than 24 inches maximum) may be buried in deeper site fills and disposal of rocks up to 48 inches in diameter using "windrow" techniques may be allowed as specified in the following sections.

10. Potentially expansive clayey soils were also encountered at the property which are generally expected to occur in minor quantities overall. Potentially expansive clayey soils, where they are encountered, should be thoroughly mixed with an abundance of sandy granular soils available from the site bedrock excavations to manufacture a very low expansive mixture. Alternatively, expansive clayey soils may be selectively buried within deeper site fills and away from the finish fill slope faces, with the upper pad grades and embankment surfaces capped with good quality sandy soils as recommended in the following sections.

11. Project new fills and backfills should be clean deposits free of trash, debris, organic matter and deleterious materials consisting of minus 6-inch particles and maintain the specified fines to rock (minus 6 inches) ratio as outlined in the following sections. Added processing, moisture conditioning and mixing efforts should be anticipated in the case of overly wet or saturated soils generated from the flowline excavations as well as gravelly to rocky deposits anticipated from the site bedrock excavations, for manufacturing a uniform mixture suitable for reuse as site compacted fills.

12. Site new fills and backfills should be properly processed, throughly mixed, placed in thin lifts horizontal lifts and compacted as specified in the following sections. Higher levels of compaction, however, will be required for site fills and backfills subject to potential saturation or inundations, such as fill in the drainage course areas as well as foundation bearing and backfill soils for drainage structures in accordance with the requirements of the following sections.

13. Based on select grading recommendations specified herein, final bearing soils are expected to chiefly consist of gravelly silty sand (SW/SM) deposits with very low expansion potential (expansion index less than 20) based on based on ASTM D-4829 classification. Foundation bearing and subgrade soils at each individual lot should be additionally tested at the completion of rough grading to confirm expansion characteristics which will govern final foundations and slab design.

14. All project graded slopes should be programmed for 2:1 gradients maximum. Graded slopes greater than 30 feet in maximum vertical height (if any planned) should be provided with a minimum 6 feet drainage terrace as specified in the following sections.

15. The overall stability of graded fill slopes and building surfaces developed over sloping terrain is most dependent upon adequate keying and benching of fill into the undisturbed bedrock during the grading operations. At the project site, added care should be given to the proper construction of fill slope keyway and benching excavations.

16. An active stream flow condition was noted in the southwesterly flowline while dry conditions were noted in the flowline that traverses the southeast portion of the property. Surface flow within the site tributary flowlines are expected to varying in actual condition depending on the seasonal conditions. We anticipate that planned site development will maintain adequate setbacks from the site flowlines. However, erosional features are noted within the site flowlines which should be alleviated as a part of the project development and drainage improvements. Specific grading and ground stabilization recommendations near the project flowlines should be provided in plan review phase when final development and drainage improvement plans are available. Excavations, grading and earthworks operations during the dry seasons of the year are recommended.

17. Natural groundwater is not expected to be a major factor in the site development. However, hard bedrock surfaces at or near finish grade levels, may transmit irrigation or meteoric water creating excessive moisture conditions. Subsurface toe drains at the base of the project cut slopes may be appropriate in some cases for the protection of the nearby improvements.

18. Adequate site surface drainage and storm water control is considered a significant geotechnical factor in the future stability and performance of the developed hillside property. Storm water and drainage control facilities should be designed and installed for proper control and disposal of surface water as shown on the project approved grading or drainage improvements. Rip-rap or other channel protection armor may be necessary within the site flowlines to help alleviate continued erosion.

19. All trenching, excavations, and temporary slope constructions should be completed as specified in the following sections. Site excavations, trenching, grading and earthwork constructions will not impact the adjacent properties provided our recommendations are incorporated into the final designs and implemented during the construction phase. Added or modified field recommendations however, may also be necessary and should be given by the project geotechnical consultant for the protection of adjacent properties and should be anticipated.

20. Post construction settlements are not expected to be a major geotechnical concern in the development of project property provided our remedial grading and foundation recommendations are followed. Post construction settlements after remedial grading works as specified herein is not expected to exceed approximately 1-inch, and should occur below the heaviest loaded footing(s). The magnitude of post construction differential settlements as expressed in terms of angular distortion is not anticipated to exceed ½-inch between similar adjacent structural elements.

21. Liquefaction, seismically induced settlements, soil collapse and post construction settlements will not be a factor in the development of the project property provided our remedial grading and foundation recommendations are followed.

IX. RECOMMENDATIONS

The following recommendations are consistent with the indicated geotechnical conditions at the project site and should be reflected on the final plans and implemented during the construction phase. Added or modified recommendations should be given in an update report prepared by the project geotechnical consultant at the time of the plan review phase when site development designs are completed.

A. Grading and Earthworks

Cut-fill and remedial grading techniques may be used in order to achieve final design grades and construct stable surfaces for the support of the planned buildings, structures and improvements. All excavations, grading, earthworks, constructions and bearing soil preparations should be completed in accordance with Chapter 18 (Soils and Foundations) and Appendix "J" (Grading) of the 2010 California Building Code (CBC), the Standard Specifications for Public Works

Construction, City of Escondido Grading Ordinances, the requirements of the governing agencies and following sections, wherever appropriate and as applicable:

1. **Existing Underground Utilities and Structures:** All existing underground waterlines, sewer lines, utilities, pipes, storm drains, tanks, structures and improvements at or nearby the project construction site should be thoroughly potholed, identified and marked prior to the initiation of the actual grading and earthworks. Specific geotechnical engineering recommendations may be required based on the actual field locations and invert elevations, backfill conditions and proposed grades in the event of a grading conflict.

   Utility lines may need to be temporarily redirected, if necessary, prior to earthwork operations and reinstalled upon completion of earthwork operations. Alternatively, permanent relocations may be appropriate as shown on the approved plans.

   Abandoned irrigation lines, pipes and conduits should be properly removed, capped or sealed off to prevent any potential for future water infiltrations into the foundation bearing and subgrade soils. Voids created by the removals of the abandoned underground pipes, tanks and structures should be properly backfilled with compacted fills in accordance with the requirements of this report.

2. **Clearing and Grubbing:** Remove all existing vegetation, trees, stumps, roots, surface rocks and all other unsuitable materials and deleterious matter from all areas proposed for new fills, improvements, and structures plus a minimum of 10 horizontal feet outside the perimeter, where possible and as approved in the field.

   All unsuitable materials generated from clearing efforts should be properly removed and disposed of from the site. Trash and vegetation should not be allowed to occur or contaminate new site fills and backfills. The prepared grounds should be observed and approved by the project geotechnical consultant or his designated field representative prior to grading and earthworks.

3. **Stripping and Removals:** Site surficial soils in areas of planned new fills, embankments, structures and improvements plus 10 feet outside the perimeter where possible, and as approved in the field, should be removed to the underlying competent bedrock and placed back as properly compacted fills.

   Removal depths will vary throughout the site and should be established by the project geotechnical engineer in the field at the time of earthwork operations based on actual exposures. Approximate removal depths in the vicinity of exploratory test pits completed as part of this effort can be obtained from the attached Test Pit Logs, Plates 4 through 30, and are shown on the enclosed Geotechnical Map Plate 2. Locally deeper removals may be necessary based on the actual exposures and should be anticipated.

Existing fills, if encountered, should also be excavated and recompacted as a part of the project remedial grading operations. Exploratory test pits excavated in connection with our study at the indicated locations (see Plate 2) were backfilled with loose and uncompacted deposits. The loose/uncompacted exploratory trench backfill soils within the limits of project improvements and grading operations shall also be re excavated and placed back as properly compacted fills in accordance with the requirements of this report.

Bottom of all removals should be additionally prepared and recompacted in place to a minimum depth of 6 inches as directed in the field. Preparation of bottom of removals and over excavations shall construct neat surfaces which are adequately benched, keyed-in and heeled back into the hillside exposing competent bedrock as directed in the field. All ground steeper than 5:1 receiving fills or backfills should also be properly benched and keyed as directed in the field.

4. **Mine Excavation Mitigation:** Mining excavations consisting of vertical shafts or horizontal adits are present at the property. Mine excavations and other related features, where present at the project site, will require special consideration and mitigation. Limited site explorations and seismic surveys completed as a part of this effort encountered evidence of mine shafts and adits as indicated in the enclosed test pit logs and tomography images. However, site mass grading efforts should further locate and expose all mine features within the project development areas, as appropriate.

   Generally, all mine related excavations with less than 10 feet of competent bedrock overburden (measured from future finish grade) should be removed, exposed, and properly backfilled during the project mass grading operations in accordance with the requirements of this report.

   Horizontal excavations developed within competent bedrock with 10 feet or more of bedrock overburdens which are determined not to be susceptible to future collapse, as inspected and approved by the project geotechnical consultant may be sealed and capped. For this purpose, all access points to horizontal mine excavations should be cleaned and closed by filling with a minimum 5 feet thick concrete plug at the mine excavation opening. All access vertical shafts or sloping mine excavations should be excavated-out completely and backfilled with compacted fills during the mass grading operations. In the event vertical shaft and sloping excavation efforts are unable to expose the bottom, they should be excavated a minimum depth of 10 feet, provided at the base with a "mushroom" type concrete cap extending a minimum of 5 feet beyond the excavation perimeter, and then backfilled with a minimum 3-sack cement slurry mix or compacted fill. The "mushroom" type concrete cap should be a minimum of 9 inches thick and reinforced with minimum #4 mesh reinforcement at 12 inches on centers both ways, top and bottom. There should be a minimum 10 feet clear distance between top of mitigated mine excavation and rough finish design grades, unless otherwise specified or approved.

Elsewhere, horizontal excavations with 10 feet or more of the overburden determined by the project geotechnical consultant to be susceptible to future collapse potential, should be completely backfilled with 3-sack cement slurry mix using conventional and/or "injection grouting" or similar type techniques, as appropriate.

More specific recommendations should be given by the project geotechnical consultant in the forthcoming update report when final development plans are completed. Additional or modified recommendations may also be necessary at the time of actual field exposures and should be anticipated. Precise locations of mining excavations, where encountered, and specific mitigation procedures, should be accurately plotted and identified on the final As-Build plans.

5. **Canyon Subdrains:** Canyon subdrain system consisting of a 2-foot wide by 2-foot deep trench with a perforated pipe surrounded by ¾-crushed rocks wrapped in Mirafi 140N filter fabric, or Class 2 permeable aggregate, will be required at the bottom of the site canyon flowline cleanouts. A minimum 4-inch diameter, Schedule 40 (SDR 35) perforated pipe should be used, unless otherwise specified or directed in the field. Outlet and collector pipes should also be minimum 6-inch Schedule 40 (SDR 35) solid pipe.

   The subdrain system should be installed at suitable elevations to allow for adequate fall and outlet in an approved drainage facility. Filter fabric can be eliminated if Class 2 permeable material is used. Riser pipe cleanouts should also be provided at appropriate locations and intervals not exceeding 100 feet maximum along the alignment. The preliminary and approximate locations for recommended canyon subdrains within the flowlines are shown on the enclosed Geotechnical Map, Plate 2. The actual subdrain alignments should be established in the field and then accurately surveyed for precise depiction on the project final as-build grading plans. Typical canyon subdrain construction details are illustrated on the enclosed, Plate 36.

Fill soils may be needed in local areas at the bottom of the canyon removals in order to achieve design invert elevations for subdrain to gravity flow. In this case, the canyon removals should be backfilled with a minimum of 95% compacted sandy fills to the proposed subdrain inverts. Fills placed above the subdrain to achieve design grades should be placed and compacted as specified herein, depending on fill thickness.

6. **Cut-Fill Transitions and Undercuts:** Ground transition from excavated cut to compacted fills shall not be permitted underneath the proposed foundations, structures, and on-grade improvements. Buildings, wall foundations, floor and on-grade improvements should be supported entirely on compacted fills or founded entirely on competent undisturbed dense natural soils or bedrock units. Transition pads will require special treatment. The cut portions of the cut-fill pads plus 10 feet outside the perimeter, where possible and as directed in the field, should be undercut to a sufficient depth to provide for a minimum 3 feet of compacted fill mat below rough finish grades, or at least 12 inches of compacted fill beneath the deepest footing(s), whichever is more. In the roadways, driveway, parking and on-grade slabs/improvement transition areas there should be a minimum of 12 inches of compacted soils below rough finish subgrade.

   Undercutting the cut portion of the building pads and road improvements will also accommodate excavation of the foundation trenches and underground utilities in an otherwise harder bedrock. In the case of deeper utility trenches, undercutting to a minimum of 6 inches below the proposed inverts should be considered.

   Undercutting of cut-fill transition pads should be completed in substantial accordance with the enclosed Typical Undercutting detail, Plate 37.

7. **Excavation Characteristics:** Very hard rock conditions are not anticipated at the site to the anticipated cut depths. Site bedrock is expected to excavate to the design grades with moderate to heavy efforts using larger bulldozers (Caterpillar D-9 or equivalent). Major specialized excavation techniques such as significant concentrated ripping and blasting efforts are not currently indicated.

   Hard rock exposures which occur during cut excavations at or near design pad grades can cause difficulties for building foundations and common/utility trenching. Residual corestones or "floaters," are also locally present which are expected to create some handling and grading difficulties.

Good quality sandy to gravelly soils is expected from the site weathered bedrock excavations. However, excavations of harder bedrock units and corestone floaters will likely produce some larger rock debris. Disposal of selected rock sizes may be allowed within project deeper fills, as recommended below

8. **Excavation Setbacks, Trenching and Temporary Slopes:** Excavations and removals adjacent to the existing embankments, structures and improvements should be done under inspection of the project geotechnical engineer. Undermining existing embankments, improvements and structures by the removal and excavation operations shall not be allowed. Top of temporary construction slopes should be adequately set back from the existing embankment toes, structures and improvements as directed in the field.

   Trenching and construction slopes less than 5 feet high maximum exposing site topsoil, alluvial and colluvial deposits may be constructed at near vertical gradients, unless otherwise specified or directed in the field. Larger slopes within these deposits will require excavations support or trench shield, or laid back at 1:1 gradients, unless otherwise approved.

   Trenching, excavations and temporary slopes exposing competent undisturbed bedrock units may be constructed at near vertical gradients to a maximum height of 10 feet, unless otherwise directed. Larger temporary slopes developed within site bedrock units may be constructed at near vertical gradients within the lower 5-foot and laid back at ½:1 gradient maximum within the upper portions thereafter unless otherwise approved or directed in the field.

   The remaining wedge exposed at the laid back temporary slopes should then be properly benched out and new fills/backfills tightly keyed-in as the backfilling progresses. All temporary construction slopes require continuous geotechnical inspections during the excavation operations. Additional recommendations including revised slope gradients, set backs and the need for completing excavations in limited alternate sections and temporary shoring/trench shield support should be given at that time as necessary. The project contractor shall also obtain appropriate permits, as needed, and conform to Cal-OSHA and local governing agencies' requirements for trenching/open excavations and safety of the workmen during construction.

9. **Soil Properties and Select Grading:** Earth deposits generated from the site weathered bedrock excavations will largely produce sandy to gravelly materials which typically work well as site new compacted fills and backfills. However, excavations of the site harder bedrock units may generate rocky materials with some larger sizes creating handling and compaction difficulties. Larger rock sizes (plus 6 inches for fills and plus 3 inches for trench and wall backfills)

should be excluded from the new manufactured fill and backfill soil matrix. Excluding larger rocks from the fill matrix will also help trenching and excavations within these deposits and improve stability of the trench and excavation sidewalls.

Overly wet to saturated soils should be expected from the removals within the site flowline areas requiring added spreading, aerating and drying efforts. Potentially expansive silty to clayey surficial deposits are also locally present at the project property which are anticipated to minor in overall quantities. Site expansive plastic silty to clayey soils, where are encountered, should be thoroughly mixed with an abundance of sandy granular soils available from the site weathered bedrock excavations to manufacture a very low expansive mixture. Alternatively, site expansive soils may be selectively buried a minimum of 4 feet below rough pad grades (or 12 inches below the deepest footing, whichever is more) and a minimum of 15 feet away from the face of graded slopes within the fill mass. Good quality sandy granular soils available from the site weathered bedrock excavations should then be used within the upper pad grades and outer fill embankment slope surfaces. Improvement areas should be provided with a minimum 12 inches of good quality sandy soils. On-site plastic clayey soils are not suitable for use as wall or trench backfills. Plastic silty to clayey soils and rocky to gravelly materials typically requires added processing, mixing and moisture conditioning efforts in order to manufacture a uniform matrix suitable for reuse as site new compacted fills.

10. **Fill and Backfill Materials and Compaction:** Project fills shall be clean deposits free of trash, debris, organic matter and deleterious materials consisting of minus 6-inch particles and include at least 40% finer than #4 sieve materials by weight. Wall and trench backfills shall consist of minus 3 inches particles and maintain the minimum specified fines to rock ratios.

    Individual rocks up to 12 inches in maximum diameter may be allowed in compacted fills provided they are sparsely placed, surrounded with compacted fills and buried a minimum of 5 feet below the rough finish pad grades. The upper 5 feet in the building pad grades, and 10 feet in the areas of public right-of-way and easements should consist of minus 6-inch materials. Rocks up to 2 feet in maximum diameter may additionally be buried in deeper fills below 10 feet as directed in the field by the project geotechnical engineer. Rocks larger than 24 inches and less than 48 inches may also be buried in larger fills using the "windrow" techniques, if approved by the project geotechnical engineer. Rocks larger than 4 feet in maximum diameter should be properly disposed of from the site. All rock disposal areas should be shown on the final grading plans. Rock disposal should be completed in substantial accordance with the enclosed Rock Disposal Recommendations, Plates 38 and 39.

Uniform bearing soils conditions should be constructed at the site by grading operations. Site soils should be adequately processed, thoroughly mixed, moisture conditioned to slightly (2% to 3%) above optimum moisture levels as directed in the field, placed in thin (8 inches maximum) uniform horizontal lifts and mechanically compacted to a minimum 90% of the corresponding maximum dry density per ASTM D-1557, unless otherwise specified. Fills and backfills placed within the project flowline areas and where subject to potential saturations or flood inundations, should be mechanically compacted to a minimum 95% of the laboratory maximum dry density (ASTM D-1557). The upper 12 inches of subgrade soils under the asphalt pavement base layers should also be compacted to minimum 90% compaction levels.

11. **Shrinkage, Bulking and Import Soils:** Based upon our analyses and experience with similar earth materials, site existing upper surficial soils and alluvium deposits may be expected to shrink, on average, approximately 5%-15% and weathered bedrock units bulk nearly the same amount on volume basis, when compacted as specified herein.

    Import soils, if required, should be sandy granular non-corrosive deposits (SM/SW) with very low expansion potential (100% passing the 1-inch sieve, more than 50% passing the #4 sieve and less than 18% passing #200 sieve with expansion index less than 20). Import soils should be inspected, tested as necessary, and approved by the project geotechnical engineer prior to delivery to the site. Import soils should also meet or exceed engineering characteristic and soil design parameters as specified in the following sections.

12. **Permanent Graded Slopes:** Project permanent graded cut and fill slopes should be programmed for 2:1 gradients maximum, unless otherwise approved. Graded slopes constructed as recommended herein will be grossly stable with respect to deep seated and surficial failures for the anticipated design maximum vertical heights.

    All fill slopes should be provided with a lower toe keyway at the base. The toe keyway should maintain a minimum depth of 2 feet into the dense natural materials or competent bedrock with a minimum width of 15 feet, and as approved in the field. The keyway should expose firm natural undisturbed soils or bedrock throughout with the bottom heeled back a minimum of 2% into the natural hillside and inspected and approved by the project geotechnical engineer. Additional level benches should be constructed into the natural hillside as the fill slope construction progresses. Added excavation efforts should also be anticipated when developing lower fill slope keyways and subsequent level benches into the site harder bedrock units.

Fill slopes should be compacted to 90% minimum of the laboratory standard out to the slope face, unless otherwise specified. Over building and cutting back to the compacted core, or backrolling at a maximum of 4-foot vertical increments and "track-walking" at the completion of grading is recommended for site fill slope construction. Geotechnical engineering inspections and testing will be necessary to confirm adequate compaction levels within the fill slope face.

Cut slopes should also be inspected and approved by the project geotechnical consultant during the grading to confirm stability. Additional recommendations will be provided at that time in the event adverse geologic conditions such as unfavorable fracturing or jointing features are noted. Over excavations of the project cut slopes may also result in costly repairs and should be avoided.

Graded slopes 30 feet or more in maximum vertical height should be provided with a minimum of 6 feet wide terraces (per Appendix J, Section J109 of the CBC) to control surface drainage and debris. Wider terraces may also be appropriate for slopes larger than 60 feet in maximum. The specified terraces should be provided at 30 feet maximum vertical intervals except where only one terrace is required, it should be placed at mid-height.

All graded slopes should be constructed in substantial accordance with the enclosed Typical Keyway and Benching Details, Plates 40 and 41.

13. **Slope Toe Drainage Systems:** A subsurface toe drainage system may be appropriate and should be considered at the base of the project cut slopes transmitting up-slope water for the protection of the lower nearby improvements. The subsurface/toe drain should consist of a minimum 1½ feet wide by 2 feet deep trench with a 4-inch diameter Schedule 40 (SDR 35) perforated pipe surrounded in ¾-inch crushed rocks and wrapped in Mirafi 140-N filter fabric. Specific recommendations should be given in the field at the time of construction based on actual site conditions and cut slope exposures.

14. **Surface Flow, Subsurface Water and Dewatering:** Special ground stabilization and remedial grading techniques will be required for the grading and constructions improvements within or near the project flowlines. Specific recommendations should be provided by the project geotechnical consultant in the update report prepared at the time of plan review phase, when actual development and drainage improvement designs are completed. The following are appropriate:

   * Surface flow occur in the southwesterly flowline while dry conditions were noted in the southeastern flowline. Actual surface flow conditions within the site canyon flowline are expected to be seasonal. Surface water within the

site canyon flowlines, if occurs, may impact remedial grading and earthwork constructions within the impacted areas.

Temporary flow diversion efforts may be expected during seasonal surface streams. Any temporary diversion structures and methods such as diversion channels, sumps and pumps, sheet piles, earthen dicks and berms which could effectively redirect the flow from the project earthworks construction areas may be considered, provided it is reviewed and approved by the project geotechnical consultant.

Groundwater was not encountered in our exploratory test pit excavations at the time on our field investigations, however, dewatering efforts should also be anticipated for completing remedial grading and earthwork operations near the project flowline areas. Any dewatering technique suitable to the field conditions which can effectively remove the intruding water and allow soil removals and fill placement to proceed such as gravel-filled trench sumps with submersible pumps is acceptable unless otherwise considered inadequate or inefficient. Dewatering should continue until completion stabilization of the bottom of removals and over excavations, and initial backfill operations. Dewatering should only be discontinued upon approval of the project geotechnical engineer. Groundwater should be adequately lowered below the specified bottom of removals, over excavation, toe of temporary slope and trench excavations as approved in the field. A qualified contractor may be consulted in this regard. Performing grading and earthwork construction within the site canyon flowline during the dry months of the year should be considered. Stabilization of bottom of removals and over excavations with a rock mat placement may also be necessary and should be anticipated in the areas impacted by saturated yielding exposures.

15. **Surface Drainage and Erosion Control:** A critical element to the continued stability of the building pads and slopes is an adequate surface drainage system and protection of the slope face. Surface and storm water shall not be allowed to impact the developed construction and improvement sites. This can most effectively be achieved by appropriate vegetation cover and the installation of the following systems:

   * Concentrated surface run-off or overflow of water from the top of slope shall not be allowed. Drainage swales should be constructed at the top and toe of the slopes as shown on the approved grading or drainage improvement plans.

* Building pad surface run-off should be collected and directed away from the planned buildings and improvements to a selected location in a controlled manner. Area drains should be installed.

* The finished slope should be planted soon after completion of grading. Unprotected slope faces will be subject to severe erosion and should not be allowed. Over watering of the slope faces should also not be allowed. Only the amount of water to sustain vegetation should be provided. Planting large trees behind the site retaining walls should also be avoided.

* Temporary erosion control facilities and silt fences should be installed during the construction phase periods and until landscaping is fully established as indicated and specified on the approved project grading/erosion plans.

16. **Engineering Observations:** All grading and earthworks operations including excavations, trenching, stripping and removals, suitability of earth deposits used as compacted fills and backfills, and compaction procedures should be continuously observed and tested by the project geotechnical consultant and presented in the final as-graded compaction report. The nature of finished bearing and subgrade soils should be confirmed in the final compaction report at the completion of grading.

    Geotechnical engineering observations should include but are not limited to the following:

    * Initial observation - After the clearing limits have been staked but before grading/brushing starts.

    * Toe keyway, stripping and bottom excavation observation - After dense and competent bedrock is exposed and prepared to receive fill or backfill but before fill or backfill is placed.

    * Cut/excavation observation - After the excavation is started but before the vertical depth of excavation is more than 5 feet. Local and Cal-OSHA safety requirements for open excavations apply.

    * Fill/backfill observation - After the fill/backfill placement is started but before the vertical height of fill/backfill exceeds 2 feet. A minimum of one test shall be required for each 100-lineal foot maximum in every 2 feet vertical gain, with the exception of wall backfills where a minimum of one test shall be required for each 30-lineal foot maximum. Onsite plastic clayey soils are not suitable for wall backfills and good quality sandy granular soils should be used for this purpose. Wall backfills should consist of minus 3-inch sandy

granular materials and also be mechanically compacted to a minimum of 90% compaction levels unless otherwise specified or directed in the field. Finish rough and final pad grade tests shall be required regardless of fill thickness.

* Foundation trench and subgrade soils observation - After the foundation trench excavations and prior to the placement of steel reinforcing for proper moisture and specified compaction levels.

* Geotechnical foundation/slab steel observation - After the steel placement is completed but before the scheduled concrete pour.

* Underground utility, plumbing and storm drain trench observation - After the trench excavations but before placement of pipe bedding or installation of the underground facilities. Local and Cal-OSHA safety requirements for open excavations apply. Observation of pipe bedding may also be required by the project geotechnical engineer.

* Underground utility, plumbing and storm drain trench backfill observation - After the backfill placement is started above the pipe zone but before the vertical height of backfill exceeds 2 feet. Testing of the backfill within the pipe zone may also be required by the governing agencies. Pipe bedding and backfill materials shall conform to the governing agencies' requirements and project soils report if applicable. Onsite plastic clayey soils are not suitable for trench backfills and good quality sandy granular soils should be used for this purpose. All trench backfills shall consist of minus 3-inch sandy granular materials and mechanically compacted to a minimum of 90% compaction levels unless otherwise specified. Plumbing trenches more than 12 inches deep maximum under the floor slabs should also be mechanically compacted and tested for a minimum of 90% compaction levels. Flooding or jetting techniques as a means of compaction method should not be allowed.

* Pavement/improvements base and subgrade observation - Prior to the placement of concrete or asphalt for proper moisture and specified compaction levels.

B. Foundations and Slab-on-Grades

The following recommendations are consistent with very low expansive (expansion index less than 20) gravelly silty sand (SW/SM) foundation bearing soil and site specific geotechnical conditions. Additional recommendations may also be required and should be given in the update report and at the plan review phase. All design recommendations should be further confirmed and/or revised at the completion of rough grading based on the expansion characteristics of the foundation bearing soils and as-graded site geotechnical conditions, and presented in the final as-graded compaction report. Individual building sites may require different foundations/slab recommendations and should be anticipated.

1. Proposed buildings may be supported on shallow stiff concrete foundations. The shallow foundations should be uniformly supported on approved very low expansive compacted fills, or founded entirely on undisturbed competent bedrock. Acceptable building foundations may consist of a system of spread pad and strip footings with slab-on-grade floors.

2. Continuous strip stem wall foundations and turned-down footings should be sized at least 15 inches wide and 18 inches deep for single and two-story structures and 18 inches wide by 24 inches deep for three-story structures. Isolated pad footings should be at least 24 inches square and 12 inches deep. Footing depths are measured from the lowest adjacent ground surface, not including the sand / gravel beneath floor slabs. Exterior continuous footings should enclose the entire building perimeter.

   Continuous interior and exterior stem wall foundations should be reinforced with a minimum of four #4 reinforcing bars. Place 2-#4 bars 3 inches above the bottom of the footings and 2-#4 bars 3 inches below the top of the stem wall. Turned-down footings should be reinforced with a minimum of 2-#4 bars at the top and 2-#4 bars at the bottom. Reinforcement details for spread pad footings should be provided by the project architect/structural engineer.

3. All interior slabs should be a minimum of 4 inches in thickness, reinforced with #3 reinforcing bars spaced 18 inches on center each way, placed mid-height in the slab.

   Slabs should be underlain by 4 inches of clean sand (SE 30 or greater) which is provided with a well performing moisture barrier/vapor retardant (minimum 10-mil Stego) placed mid-height in the sand. Alternatively, a 4-inch thick base of compacted ½-inch clean aggregate provided with the vapor barrier (minimum 10-mil Stego) in direct contact with (beneath) the concrete may also be considered provided a concrete mix which can address bleeding, shrinkage and curling are used.

   Provide "softcut" contraction/control joints consisting of sawcuts spaced 10 feet on centers each way for all interior slabs. Cut as soon as the slab will support the weight of the saw and operate without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. The sawcuts should be a minimum of 1-inch in depth but should not exceed 1¼-inches deep maximum. Anti-ravel skid plates should be used and replaced with each blade to avoid spalling and ravelings. Avoid wheeled equipments across cuts for at least 24 hours.

Provide re-entrant corner reinforcement for all interior slabs. Re-entrant corners will depend on slab geometry and/or interior column locations. The enclosed Plate 42 may be used as a general guideline.

4. Adequate setbacks or deepened foundations shall be required for all foundations constructed on or near the top of descending slopes to maintain minimum horizontal distances to daylight or adjacent slope face. There should be a minimum of 7 feet horizontal setbacks from the bottom outside edge of the footing to daylight for foundations unless otherwise specified or approved. A minimum of 10 feet horizontal distances or setback shall be required for sensitive structures and improvements which cannot tolerate minor movements (including swimming pools and spas or portions thereof).

5. Foundation trenches and slab subgrade soils should be inspected and tested for proper moisture and specified compaction levels and approved by the project geotechnical consultant prior to the placement of steel reinforcement or concrete pour.

C. Soil Design Parameters

The following preliminary soil design parameters are based upon the tested representative samples of on-site earth deposits. All parameters should be re-evaluated when the characteristics of the final as-graded soils have been specifically determined:

1. Design soil unit weight = 129 pcf.
2. Design angle of internal friction of soil = 29 degrees.
3. Design active soil pressure for retaining structures = 45 pcf (EFP), level backfill, cantilever, unrestrained walls.
4. Design active soil pressure for retaining structures = 73 pcf (EFP), 2:1 sloping backfill, cantilever, unrestrained walls.
5. Design at-rest soil pressure for retaining structures = 67 pcf (EFP), non-yielding, restrained walls.
6. Design passive soil resistance for retaining structures = 369 pcf (EFP), level surface at the toe.
7. Design coefficient of friction for concrete on soils = 0.39.
8. Net allowable foundation pressure (minimum 15 inches wide by 18 inches deep footings) = 2000 psf.

9. Allowable lateral bearing pressure (all structures except retaining walls) = 200 psf/ft.

Notes:

* Use a minimum safety factor of 1.5 for wall overturning and sliding stability. However, because large movements must take place before maximum passive resistance can be developed, a minimum safety factor of 2 may be considered for sliding stability particularly where sensitive structures and improvements are planned near or on top of retaining walls.

* When combining passive pressure and frictional resistance, the passive component should be reduced by one-third.

* The indicated net allowable foundation pressure provided herein was determined based on minimum 15-inch wide by 18-inch deep footings and may be increased by 20% for each additional foot of depth and 20% for each additional foot of width to a maximum of 5500 psf. The allowable foundation pressures provided herein also apply to dead plus live loads and may be increased by one-third for wind and seismic loading.

* The lateral bearing earth pressures may be increased by the amount of designated value for each additional foot of depth to a maximum 1500 pounds per square foot.

D. Exterior Concrete Slabs / Flatworks

1. All exterior slabs (walkways, and patios) should be a minimum of 4 inches in thickness reinforced with 6x6/10x10 welded wire mesh carefully placed at mid-height in the slab.

2. Reinforcements lying on subgrade will be ineffective and shortly corrode due to lack of adequate concrete cover. Reinforcing bars should be correctly placed extending through the construction joints tying the slab panels. In construction practices where the reinforcements are discontinued or cut at the construction joints, slab panels should be tied together with minimum 18-inch long #3 dowels (dowel baskets) at 18 inches on centers placed mid-height in the slab (9 inches on either side of the joint).

3. Provide "tool joint" or "softcut" contraction/control joints spaced 10 feet on center (not to exceed 12 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as slab will support weight, and can be operated without disturbing the final finish which is

normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum of 1-inch but should not exceed 1¼-inches deep maximum. In case of softcut joints, anti-ravel skid plates should be used and replaced with each blade to avoid spalling and ravelings. Avoid wheeled equipments across cuts for at least 24 hours.

4. Joints shall intersect free-edges at a 90° angle and shall extend straight for a minimum of 1½ feet from the edge. The minimum angle between any two intersecting joints shall be 80°. Align joints of adjacent panels. Also, align joints in attached curbs with joints in slab panels. Provide adequate curing using approved methods (curing compound maximum coverage rate = 200 sq. ft./gal.).

5. Subgrade soils should be tested for proper moisture and specified compaction levels and approved by the project geotechnical consultant prior to the placement of concrete. All exterior slab designs should be confirmed in the final as-graded compaction report.

E. Asphalt and PCC Pavement Design

1. Asphalt Paving: Specific pavement designs can best be provided at the completion of rough grading based on R-value tests of the actual finish subgrade soils; however, the following structural sections are based on tested R-value of 14 and assumed traffic index (TI) of 5.0 and may be considered for initial planning phase cost estimating purposes only (not for construction):

   * A minimum section of 3 inches asphalt on 8 inches Caltrans Class 2 aggregate base or the minimum structural section required by City of Escondido, whichever is more, may be considered for the on-site asphalt paving surfaces outside the private and public right-of-way. In the areas where the longitudinal grades exceed 10%, ½-inch asphalt should be added to the design asphalt thickness for each 2% increase in grade or portions thereof. PCC paving should be considered for longitudinal grades greater than 15% maximum. Actual designs will depend on final subgrade R-value and design TI, and the approval of the City of Escondido.

   * Base materials should be compacted to a minimum 95% of the corresponding maximum dry density (ASTM D-1557). Subgrade soils beneath the asphalt paving surfaces should also be compacted to a minimum 95% of the corresponding maximum dry density within the upper 12 inches.

2. PCC Pavings: Residential PCC driveways and parking supported on very low expansive (expansion index less than 20) granular subgrade soils should be a minimum 5 inches in thickness, reinforced with #3 reinforcing bars at 18 inches on centers each way placed at mid-height in the slab. Subgrade soils beneath the PCC driveways and parking should also be compacted to a minimum 90% of the corresponding maximum dry density.

   In the areas where longitudinal grades exceed 15%, provide minimum 8 inches wide by 8 inches deep pavement anchors dug perpendicular to the pavement longitudinal profile into the approved subgrade at each 25-foot interval maximum. The pavement anchors should be poured monolithically with the concrete pavings.

   Provide "tool joint" or "softcut" contraction/control joints spaced 10 feet on center (not to exceed 15 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as the slab will support the weight and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum of 1-inch in depth but should not exceed 1¼-inches deep maximum. In case of softcut joints, anti-ravel skid plates should be used and replaced with each blade to avoid spalling and ravelings. Avoid wheeled equipments across cuts for at least 24 hours.

   Joints shall intersect free edges at a 90° angle and shall extend straight for a minimum of 1½ feet from the edge. The minimum angle between any two intersecting joints shall be 80°. Align joints of adjacent panels. Also, align joints in attached curbs with joints in slab panels. Provide adequate curing using approved method (curing compound maximum coverage rate = 200 sq. ft./gal.)

3. General Paving: Base section and subgrade preparations per structural section design, will be required for all surfaces subject to traffic including roadways, travelways, drive lanes, driveway approaches and ribbon (cross) gutters. Driveway approaches within the public right-of-way should have 12 inches subgrade compacted to a minimum of 95% compaction levels and provided with a 95% compacted Class 2 base section per structural section design.

   Base layer under curb and gutters should be compacted to a minimum 95%, while subgrade soils under curb and gutters, and base and subgrade under sidewalks should be compacted to a minimum 90% compaction levels. Base section may not be required under curb and gutters, and sidewalks, in the case of very low expansive subgrade soils (expansion index less than 20). Appropriate recommendations should be given in the final as-graded compaction report.

Base and subgrade soils should be tested for proper moisture and specified compaction levels, and approved by the project geotechnical consultant prior to the placement of the base or asphalt/PCC finish surface.

F. General Recommendations

1. The minimum foundation design and steel reinforcement provided herein are based on soil characteristics and are not intended to be in lieu of reinforcement necessary for structural considerations.

2. Adequate staking and grading control are a critical factor in properly completing the recommended remedial and site grading operations. Grading control and staking should be provided by the project grading contractor or surveyor/civil engineer, and is beyond the geotechnical engineering services. Staking should apply the required setbacks shown on the approved plans and conform to setback requirements established by the governing agencies and applicable codes for off-site private and public properties and property lines, utility easements, right-of-ways, nearby structures and improvements, leach fields and septic systems, and graded embankments. Inadequate staking and/or lack of grading control may result in illegal encroachments or unnecessary additional grading which will increase construction costs.

3. Open or backfilled trenches parallel with a footing shall not be below a projected plane having a downward slope of 1-unit vertical to 2 units horizontal (50%) from a line 9 inches above the bottom edge of the footing, and not closer than 18 inches from the face of such footing.

4. Where pipes cross under-footings, the footings shall be specially designed. Pipe sleeves shall be provided where pipes cross through footings or footing walls, and sleeve clearances shall provide for possible footing settlement, but not less than 1-inch all around the pipe.

5. Foundations where the surface of the ground slopes more than 1 unit vertical in 10 units horizontal (10% slope) shall be level or shall be stepped so that both top and bottom of such foundations are level. Individual steps in continuous footings shall not exceed 18 inches in height and the slope of a series of such steps shall not exceed 1 unit vertical to 2 units horizontal (50%) unless otherwise specified. The steps shall be detailed on the structural drawings. The local effects due to the discontinuity of the steps shall also be considered in the design of foundations as appropriate and applicable.

6. Expansive clayey soils should not be used for backfilling of any retaining structure. All retaining walls should be provided with a 1:1 wedge of granular, compacted backfill measured from the base of the wall footing to the finished surface, and a well constructed back drainage system as shown on the enclosed Plate 43. Planting large trees behind site retaining walls should be avoided.

7. All underground utility and plumbing trenches should be mechanically compacted to a minimum of 90% of the maximum dry density of the soil unless otherwise specified. Care should be taken not to crush the utilities or pipes during the compaction of the soil. Non-expansive, granular backfill soils should be used. Trench backfill materials and compaction beneath pavements within the public right-of-way shall conform to the requirements of governing agencies.

8. Site drainage over the finished pad surfaces should flow away from structures onto the street in a positive manner. Care should be taken during the construction, improvements, and fine grading phases not to disrupt the designed drainage patterns. Roof lines of the buildings should be provided with roof gutters. Roof water should be collected and directed away from the buildings and structures to a suitable location. Subterranean blanket drain and wall back drains should be provided as specified and their proper functioning periodically confirmed. Planter areas adjacent to basement walls and building foundations may be provided with impermeable liners and added subdrain, if appropriate.

9. Final plans should reflect preliminary recommendations given in this report. Final foundations and grading plans may also be reviewed by the project geotechnical consultant for conformance with the requirements of the geotechnical investigation report outlined herein. More specific recommendations may be necessary and should be given when final grading and architectural/structural drawings are available.

10. All foundation trenches should be inspected to ensure adequate footing embedment and confirm competent bearing soils. Foundation and slab reinforcements should also be inspected and approved by the project geotechnical consultant.

11. The amount of shrinkage and related cracks that occur in the concrete slab-on-grades, flatworks and driveways depend on many factors the most important of which is the amount of water in the concrete mix. The purpose of the slab reinforcement is to keep normal concrete shrinkage cracks closed tightly. The amount of concrete shrinkage can be minimized by reducing the amount of water in the mix. To keep shrinkage to a minimum, the following should be considered:

* Use the stiffest mix that can be handled and consolidated satisfactorily.

* Use the largest maximum size of aggregate that is practical. For example, concrete made with ⅜-inch maximum size aggregates usually require about 40-lbs. more (nearly 5-gal.) water per cubic yard than concrete with 1-inch aggregate.

* Cure the concrete as long as practical.

The amount of slab reinforcement provided for conventional slab-on-grade construction considers that good quality concrete materials, proportioning, craftsmanship, and control tests, where appropriate and applicable, are provided.

12. A preconstruction meeting between representatives of this office, the property owner or planner, city inspector as well as the grading contractor/builder is recommended in order to discuss grading and construction details associated with site development.

X. GEOTECHNICAL ENGINEER OF RECORD (GER)

Vinje & Middleton Engineering, Inc. is the geotechnical engineer of record (GER) for providing a specific scope of work or professional service under a contractual agreement unless it is terminated or canceled by either the client or our firm. In the event a new geotechnical consultant or soils engineering firm is hired to provide added engineering services, professional consultations, engineering observations and compaction testing, Vinje & Middleton Engineering, Inc. will no longer be the geotechnical engineer of the record. Project transfer should be completed in accordance with the California Geotechnical Engineering Association (CGEA) Recommended Practice for Transfer of Jobs Between Consultants.

The new geotechnical consultant or soils engineering firm should review all previous geotechnical documents, conduct an independent study, and provide appropriate confirmations, revisions or design modifications to his own satisfaction. The new geotechnical consultant or soils engineering firm should also notify in writing Vinje & Middleton Engineering, Inc. and submit proper notification to the City of Escondido for the assumption of responsibility in accordance with the applicable codes and standards (1997 UBC Section 3317.8).

XI. LIMITATIONS

The conclusions and recommendations provided herein have been based on available data obtained from the review of pertinent reports and plans, subsurface exploratory excavations as well as our experience with the soils and formational materials located in the general area. The materials encountered on the project site and utilized in our laboratory testing are believed representative of the total area; however, earth materials may vary in characteristics between excavations.

Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is necessary, therefore, that all observations, conclusions, and recommendations are verified during the grading operation. In the event discrepancies are noted, we should be contacted immediately so that an inspection can be made and additional recommendations issued if required.

The recommendations made in this report are applicable to the site at the time this report was prepared. It is the responsibility of the owner/developer to ensure that these recommendations are carried out in the field.

It is almost impossible to predict with certainty the future performance of a property. The future behavior of the site is also dependent on numerous unpredictable variables, such as earthquakes, rainfall, and on-site drainage patterns.

The firm of VINJE & MIDDLETON ENGINEERING, INC., shall not be held responsible for changes to the physical conditions of the property such as addition of fill soils, added cut slopes, or changing drainage patterns which occur without our inspection or control.

This report should be considered valid for a period of one year and is subject to review by our firm following that time. If significant modifications are made to your tentative reconstruction plan, especially with respect to the height and location of cut and fill slopes, this report must be presented to us for review and possible revision.

This report is issued with the understanding that the owner or his representative is responsible for ensuring that the information and recommendations are provided to the project architect/structural engineer so that they can be incorporated into the plans. Necessary steps shall be taken to ensure that the project general contractor and subcontractors carry out such recommendations during construction.

The project geotechnical engineer should be provided the opportunity for a general review of the project final design plans and specifications in order to ensure that the recommendations provided in this report are properly interpreted and implemented. If the project geotechnical engineer is not provided the opportunity of making these reviews, he can assume no responsibility for misinterpretation of his recommendations.

Vinje & Middleton Engineering, Inc., warrants that this report has been prepared within the limits prescribed by our client with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Once again, should any questions arise concerning this report, please do not hesitate to contact this office. Reference to our **Job #13-116-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
SEYED M. SAMSAM SHARIAT
No. 2885
Exp. 12/31/14
GEOTECHNICAL
STATE OF CALIFORNIA

S. Mehdi S. Shariat
GE #2885

PROFESSIONAL GEOLOGIST
STEVEN JAY MELZER
No. 2362
CERTIFIED ENGINEERING GEOLOGIST
EXP. 5/31/13
STATE OF CALIFORNIA

Steven J. Melzer
CEG #2362

Distribution: Addressee (5, e-mail)

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