Geotechnical Investigation

Proposed Residential Subdivision
Ware Property, Attebury Drive
San Marcos, California

February 6, 2008

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GEOTECHNICAL INVESTIGATION, PROPOSED RESIDENTIAL SUBDIVISION, WARE PROPERTY, ATTEBURY DRIVE, SAN MARCOS, CALIFORNIA

Pursuant to your request, Vinje and Middleton Engineering, Inc. has completed the enclosed Geotechnical Investigation Report for the above-referenced project site.

The following report summarizes the results of our field investigation, including laboratory analyses and conclusions, and provides preliminary 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 #07-420-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

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GEOTECHNICAL INVESTIGATION
WARE PROPERTY, ATTEBURY DRIVE
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I. INTRODUCTION

The property investigated in this work is undeveloped hillside terrain south of San Elijo Road near the south limits of the City of San Marcos. The property location is shown on a Regional Index Map enclosed with this report as Plate 1. We understand that the property is planned for an 8-lot residential subdivision with the associated entrance roadway and underground improvements using cut-fill grading methods. Consequently, this investigation was initiated to determine soil and geotechnical conditions at the property and to ascertain their influence upon the proposed development. Test pit explorations, air-track drilling, soil sampling, laboratory testing and engineering analysis were among the activities conducted in conjunction with this effort which has resulted in the grading and development recommendations presented herein.

II. SITE CONDITIONS

Project topographic conditions are depicted on a Site Plan enclosed with this report as Plate 2. The property generally consists of west-facing hillside terrain with a small intervening canyon. Larger canyons define the west property boundary and support the unimproved Attebury Road. Slope gradients approach 2:1 (horizontal to vertical) at their steepest and rise a maximum of 150 feet from the lowest canyon areas along Attebury Drive to the highest areas along the east margin. Upper, more level site areas have been modified by previous grading and the creation of level surfaces likely used for previous agricultural support efforts. These areas are generally clear of vegetation. Remaining slope terrain at the site support a dense growth of native brush.

Site drainage generally sheetflows westward into natural canyon areas to the west. Some erosion is noted in association with site graded areas including an old graded entrance road. At the time of our field study, ponded and / or flowing surface water was noted in lower project terrain along the southwest margin and in association with an existing entrance roadway as shown on Plate 2.

III. PROPOSED DEVELOPMENT

Preliminary development concepts are shown on the enclosed Site Plan, Plate 2. As shown, cut-fill grading is planned to construct level building pads and improvement surfaces. Planned development will create 8 residential lots and an entrance roadway. Cutting of the highest site terrain to a maximum depth of 40 feet will generate soils for filling over canyon and lower perimeter areas. The largest graded cut slope is planned for 1½:1 gradients and will approach 50 feet in maximum vertical height. Large perimeter fill slopes will descend a maximum of 50 feet at 2:1 gradients into lower terrain.

Detailed residential construction plans are not yet developed. The use of conventional wood-frame and stucco building construction supported on shallow stiff foundations with stem-walls and slab-on-grade floors, or slab-on-ground with turned-down footings are assumed for the purpose of this investigation.

IV. SITE INVESTIGATION

Geotechnical conditions at the project site were determined from geologic mapping of available surface exposures and the following subsurface explorations:

* The excavation of 9 test pits dug with a tractor-mounted backhoe. The pits were logged by our project geologist who also retained representative soil/rock samples for laboratory testing. The test results are presented in a following section herein.

* The excavation of 8 air-track percussion borings drilled in proposed cut areas to beyond the depth of planned finish grade levels. The purpose of the borings was to determine hardness levels of bedrock units at the property planned for excavation.

Test pit and air-track boring locations at the site are shown on the enclosed Plate 2. Logs of the pits and air-track boring data are enclosed with this report as Plates 3-15.

V. GEOTECHNICAL CONDITIONS

The project site is undeveloped hillside terrain underlain by a series of shallow intrusive and metavolcanic rocks most commonly designated Santiago Peak Volcanics. In most areas, the rocks are mantled by a cover of natural topsoil or old fill soils associated with previous grading work at the property. The following geotechnical factors will influence site development:

A. Earth Materials

Bedrock units beneath the project site were noted in existing road cuts along the west boundary and in upper graded exposures. Additional exposures were developed and noted within test pit excavations. The rocks comprise a variety of volcanic to metavolcanic units that range in consistency from very hard and blocky to deeply weathered and relatively soft. Harder units often occur in tabular dike structures that occur throughout.

Bedrock units at the project site are mantled by undifferentiated surficial soils. These include a thin topsoil cover that thickens to alluvial soils in lower canyon terrain. Minor amounts of old fill are found in upper site areas associated with previous grading efforts. Site surficial soils consist chiefly of silty to clayey sands and occur in a loose condition. The distribution of site fills and thicker alluvial deposits at the property is mapped on Plate 2. Details of project earth materials are given on the enclosed logs Plates 3-7, and further defined in a following section herein. Indicated subsurface conditions and planned grades are depicted on Geologic Cross-Sections enclosed herein as Plate 16.

B. Surface And Subsurface Groundwater

Subsurface water seeps were encountered in test pit excavations dug in lower site terrain. The water typically originates from up-slope watershed areas and flows through fracture / joint surfaces in the near-surface bedrock. Significant amounts of water was not recorded in upper terrain and noted seeps typically diminish with depth.

Higher levels of subsurface water were found within alluvial soils that occupy lower site canyon terrain along the southwest margin. These areas also feature ponded or flowing surface water at the time of our field study and are expected to fluctuate in response to seasonal changes.

Earthworks and grading within or near the project canyon flowlines will likely be impacted by seasonal surface flow conditions. Temporary diversion of surface flow may be necessary in the event of seasonal surface streams until canyon subdrain as well as underground drainage improvements and storm drains are installed.

Dewatering efforts will also be necessary for removals of intruding water into the site excavations developed within the project canyon flowlines allowing fill placement and construction to proceed. A suitable dewatering program, depending on seasonal groundwater conditions at the time of grading, should be utilized. Added efforts for aerating and drying of wet soils removed from flowline areas prior to their reuse as new compacted fills should also be anticipated.

Local bedrock excavations often develop seeps along the base of graded cut embankments that can impact near-slope improvements. Subsurface toe drains may be appropriate as determined by the project geotechnical consultant when final plans are developed.

Like all graded hillside properties, well-developed site drainage is an important factor in overall site stability. Ponding of surface waters should not take place and overwatering of site vegetation should be avoided.

C. Slope Stability

Landslides or other forms of slope instability are not in evidence at the project site. The property is underlain by hard bedrock units that characteristically perform well in natural and graded slope conditions.

The rocks are impacted by fractures and joint surfaces. These are thin breaks in the otherwise massive rocks that can impact slope stability by translational failures. However, noted surfaces at the project site are typically discontinuous and steeply dipping features that are not expected to affect global slope performance.

D. Rock Hardness

Bedrock units beneath the project site are hard rocks whose excavations can be difficult and costly. Consequently, added attention was given in this study to the nature and consistency of project bedrock. Eight air-track percussion borings were excavated in proposed cut areas to below planned cut surface grades. Air-track drilling speed utilizing constant pressure represents an empirical measure of underlying rock hardness. At the project site, drilling was conducted with a 4½-inch diameter Ingersal-Rand 590 drill. With this equipment, drill rates up to 15 seconds per foot (spf) indicate hard rocks that will generally excavate with moderate to locally difficult ripping. Rates in excess of 20 spf are harder and more massive rocks that typically require blasting for excavation.

Resulting drill-rate data at the project site are included with this report as Plates 8-15. As shown, much of the project bedrock planned for excavation occurs in a weathered and fractured condition that will facilitate its excavation with moderate to difficult ripping procedures utilizing large dozers (Caterpillar D9H or equivalent). Slower drill rates (harder rock zones) recorded in Borings 2-4 indicate irregular zones of hard rock at depth within the softer surrounding rock. These are likely tabular dike structures that can be removed with intense and more concentrated ripping and rock breakers. However, the use of limited blasting will greatly increase ripping production levels and improve the quality of generated soils. Limited blasting will have a similar beneficial impact on remaining weathered rocks found in Borings 5-8.

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 10.9 miles from the project area. This event, which is thought to have occurred along an off-shore fault, reached an estimated magnitude of 7.6 with estimated bedrock acceleration values of 0.122g 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 UPDATE) typically associated with the fault is also tabulated:

TABLE 1

Fault Zone	Distance From Site	Maximum Probable Acceleration (R.H.)
Newport-Inglewood	15.7 miles	0.177g
Coronado Bank	26.1 miles	0.157g
Elsinore	20.1 miles	0.164g
Rose Canyon	11.2 miles	0.147g

The location of significant faults and earthquake events relative to the study site are depicted on a Fault - Epicenter Map enclosed with this report as Plate 17.

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 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.

A site specific probabilistic estimation of peak ground acceleration was also performed using the FRISKSP (T. Blake, 2000) computer program. Based upon Boore et. al. (1997) attenuation relationship, a 10 percent probability of exceedance in 50 years was estimated to produce a site specific peak ground acceleration of 0.19g (Design-Basis Earthquake, DBE) for site class B conditions, and 0.29g for site class D conditions. The results were obtained from the corresponding probability of exceedance versus acceleration curve.

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}
S_B	1.042	0.391	1.0	1.0	1.042	0.391	0.694	0.261
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}
S_B	1.042	0.391	1.083	1.617	1.128	0.633	0.752	0.422
According to Chapter 16, Section 1613 of the 2007 California Building Code.								

Explanation:

S_S: Mapped MCE, 5% damped, spectral response acceleration parameter at short periods.

S_1: Mapped MCE, 5% damped, spectral response acceleration parameter at a period of 1-second.

F_a: Site coefficient for mapped spectral response acceleration at short periods.

F_v: Site coefficient for mapped spectral response acceleration at 1-second period.

S_{MS}: The MCE, 5% damped, spectral response acceleration at short periods adjusted for site class effects ($S_{MS}=F_aS_s$).

S_{M1}: The MCE, 5% damped, spectral response acceleration at a period of 1-second adjusted for site class effects ($S_{M1}=F_vS_1$).

S_{DS}: Design, 5% damped, spectral response acceleration parameter at short periods ($S_{DS}=\frac{2}{3}S_{MS}$).

S_{D1}: Design, 5% damped, spectral response acceleration parameter at a period of 1-second ($S_{D1}=\frac{2}{3}S_{M1}$).

G. Geologic Hazards

Geologic hazards are not in evidence at the project site. Landslides or other forms of slope instability are not in evidence and proposed grading will expose hard bedrock units that are expected to perform well. The most significant geologic condition likely to impact site improvements will be ground shaking during periods of activity along distant active faults. Liquefaction or related ground failures are not expected at the property.

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 pits, air-track borings, and field exposures site soils have been grouped into the following soil types:

TABLE 4

Soil Type	Description
1	pale brown silty to clayey sand w/ gravel (Fill/Topsoil/Alluvium)
2	metavolcanic bedrock (Bedrock)
3	brown silty gravelly clay (Topsoil/Alluvium)

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 and 2. The test results are presented in Table 5.

TABLE 5

Sieve Size		3"	1½"	1"	¾"	½"	#4	#10	#20	#40	#200
Location	Soil Type	Percent Passing									
TP-1 @ 1½'	1	99	95	91	88	82	68	61	54	50	39
TP-3 @ 4'	2	94	82	70	60	47	25	14	10	8	6

2. **Maximum Dry Density and Optimum Moisture Content:** The maximum dry density and optimum moisture content of Soil Types 1 and 2 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-%)	Rock Corrected Maximum Dry Density (Ym-pcf)	Rock Corrected Optimum Moisture Content (ωopt-%)
TP-1 @ 1½'	1	124.6	11.5	128.2 (1)	10.4 (1)
TP-3 @ 4'	2	131.8	9.8	143.1 (2)	6.5 (2)

Assumption:
Gs = 2.60
(1) Corrected for 12% plus ¾-inch coarse fraction.
(2) Corrected for 42% plus ¾-inch coarse fraction.

3. **Expansion Index Test:** One expansion index (EI) test was performed on a representative sample of Soil Type 1 in accordance with the ASTM D-4829. The test results are presented in Table 7.

TABLE 7

Sample Location	Soil Type	Molded ω (%)	Degree of Saturation (%)	Final ω (%)	Initial Dry Density (PCF)	Measured EI	EI 50% Saturation
TP-1 @ 1½'	1	11.2	50.2	21.4	105.1	5	5

(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

4. **Direct Shear Test:** Two direct shear tests were performed on representative samples of Soil Types 1 and 2. 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 test results are presented in Table 8.

TABLE 8

Sample Location	Soil Type	Sample Condition	Wet Density (Yw-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
TP-1 @ 1½'	1	remolded to 90% of Ym @ % ωopt	125.2	31	373
TP-3 @ 4'	2	remolded to 90% of Ym @ % ωopt	129.6	33	70

5. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 1 was determined using " Method for Estimating the Service Life of Steel Culverts," in accordance with the California Test Method (CTM) 643. The test result is tabulated in Table 9.

TABLE 9

Sample Location	Soil Type	Minimum Resistivity (OHM-CM)	pH
TP-1 @ 1½'	1	1932	6.05

6. **Sulfate Test:** One sulfate test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 417. The test result is presented in Table 10.

TABLE 10

Sample Location	Soil Type	Amount of Water Soluble Sulfate In Soil (% by Weight)
TP-1 @ 1½'	1	0.010

7. **Chloride Test:** One chloride test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 422. The test result is presented in Table 11.

TABLE 11

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
TP-1 @ 1½'	1	0.006

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 exists:

* 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.

Laboratory test results performed on selected representative site samples indicated 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 the corrosion analyses, the project site is considered non-corrosive. Additional corrosion conformation testings are recommended and should be considered during the grading and earthwork operations at the planned underground storm drains, structures and each individual lot surfaces. The project site is not located within 1000 feet of salt or brackish water.

Based upon the result of the tested soil sample, the amount of water soluble sulfate (SO_4) was found to be 0.010 percent by weight which is considered negligible according to the ACI 318, Table 4.3.1. Portland cement Type II may be used. Table 12 is appropriate based on the pH-Resistivity test result:

TABLE 12

Design Soil Type	Gage	18	16	14	12	10	8
1	Years to Perforation of Steel Culverts	13	16	20	28	36	44

VII. CONCLUSIONS

Based upon the forgoing site study, development of the project site for residential purposes substantially as proposed is feasible from a geotechnical viewpoint. The property is underlain by hard and stable bedrock units that are mantled by a thin cover of surficial soil in a loose condition. The following factors are unique to the property and will most impact site development:

* Geologic hazards including faults or significant shear zones are not indicated at the project site. The most significant design factor will be ground motion during brief periods of seismic activity along distant active faults. Faulting, seismically induced ground failures or instability of natural terrain are not indicated at the property.

* The study area is underlain at relatively shallow to modest depths by competent metavolcanic bedrock units which will adequately support the planned new fills, structures and improvements, and perform well in graded slope conditions. Site upper surficial soils and alluvial deposits within the lower canyons consist of loose to soft deposits which should be regraded as recommended in the following sections, in order to construct safe and stable building surfaces. Higher levels of fill compaction are recommended for deeper site fills and within areas subject to inundations as specified below. Added removals of cut ground will also be necessary in the case of cut-fill pads which expose bedrock units so that uniform bearing soils conditions are constructed throughout the buildings and improvement surfaces.

* Proposed cutting of project hillside terrain will encounter hard bedrock units. Cuts as deep as 40 feet are planned into the hillside. Added cutting will provide undercuts for lot capping and facilitate utility line excavations. Proposed cutting in the south (lower) portion of the project (Lots 7 and 8) will encounter deeply weathered and fractured rocks that can likely be excavated with light to moderate ripping utilizing larger dozers (Caterpillar D9H or equivalent). Elsewhere on the property, similar weathered rocks are indicated, however hard dike structures are present that will require more intense and concentrated ripping efforts. In these upper areas, limited blasting techniques are recommended as a means of increasing ripping production to acceptable levels and improving the quality of generated soils that are more acceptable as project fill soils.

* Undercutting of building pads exposing hard bedrock units throughout and their reconstruction to grade with compacted fill is recommended to facilitate footing and utility-line excavations.

* Natural slope terrain at the project site are geologically stable. Graded slopes planned in conjunction with site development should be constructed as recommended in the following sections. Graded fill slopes should be provided with an adequate toe keyway and benched into the undisturbed bedrock during the grading operations. Over-blasting of graded cut slopes will require costly repairs and should be avoided. Larger graded slopes greater than 30 feet in maximum vertical height should be provided with adequate drainage terraces as specified below.

* Bedrock excavations will generate poor to marginal quality rock ladden fills that likely include inadequate fines and an abundance of larger debris creating disposal and compaction difficulties. Smaller rocks can be buried in selected site areas using appropriate rock burial techniques as specified in the following sections. Larger rock sizes should be selectively separated and excluded from the project fills.

* Site fills should maintain the specified fines to rock ratio. For this purpose, added crushing efforts of generated rocky materials or importing sandy soils for improving the quality of the fill matrix may be required and should be anticipated. Import soils should meet or exceed the minimum engineering properties as specified in the following sections.

* Based on our field observations, site topsoils and alluvial deposits locally include potentially expansive plastic clays which are thought be to minor in overall earthworks quantities. Site potentially expansive plastic clayey soils, where encountered, should be selectively buried in deeper fills or thoroughly mixed with an abundance of sandy to gravelly soils generated from site weathered bedrock excavations to manufacture a very low expansive mixture.

* Additional mixing and moisture conditioning efforts will be necessary when processing rocky and clayey deposits for manufacturing uniform materials suitable for reuse as site new fills. Excavations within the lower site canyons are also expected to encounter overly moist to saturated soils depending on seasonal conditions, requiring added spreading, aerating and drying efforts for achieving a suitable mixture at near optimum moisture levels.

* Based on select grading recommendations provided herein, expansive soils are not expected to be a major geotechnical factor in the site development according to the California Building Code Section 1802.3.2. Final bearing soil mixture may be anticipated to consist chiefly of gravelly silty sand to sandy silty gravel (GM/GP) with very low expansion potential (expansion index less than 21) based on ASTM D-4829 classification. Actual classification and expansion characteristic of the finished grade soil can only be provided in the final as-graded compaction report based on proper testing of foundation bearing and subgrade soils which may result in revised design recommendations.

* Natural groundwater and surface flow were noted within site canyon flowlines and are expected to impact project grading and construction works within the impacted areas. Locally, moderate to significant surface flow and subsurface water seeps should be anticipated at the bottom of flowlines and canyon cleanouts depending on the annual rainfall and seasonal storm conditions requiring surface flow

diversions and heavy dewatering efforts. Appropriate temporary surface flow diversion techniques and dewatering methods which can effectively remove the intruding water and allow for over-excavations and fill placement should be considered. Grading during the dry moths of the year is also recommended to help reduce the need for added dewatering efforts.

Elsewhere within the project higher terrains, natural groundwater is not expected to impact project grading or the long term stability of the individual developed lots.

* A subdrain system should also be provided at the bottom of the project canyon cleanouts as specified below. All fills and backfills placed within the potential flood innundation or saturation areas including the water quality basin embankment fill should be compacted to at least 95% compaction levels as specified in the following sections.

* Graded cut slopes at the project are expected to manifest seepage of up-slope watershed drainage that is transmitted through frature/joint surfaces in the rock. Moisture sensative improvements located near the toe of impacted cut slopes can be protected by subsurface drains constructed along the base of graded cut slopes. The need for slope toe drains can best be determined in the future when site improvements are known and actual graded conditions are apparent.

* The proper control of post development surface run-off drainage and storm waters are important factors in the continued stability of the graded property. Hard bedrock surfaces at, or near finish grade levels, may transmit irrigation or meteoric water creating excessive moisture conditions. Storm water and drainage control facilities should be also designed and installed for proper control and disposal of surface water as shown on the approved grading or drainage improvement plans.

* Post construction settlement of site fill deposits after completion of grading works as specified herein, is not expected to exceed approximately 1-inch and should occur below the heaviest loaded footings. The magnitude of post construction differential settlements of site fill deposits as expressed in terms of angular distortion, is not anticipated to exceed ½-inch between similar elements in a 20-foot span.

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

VIII. RECOMMENDATIONS

The following preliminary recommendations are consistent with site indicated geotechnical conditions and the general site development concept as understood. Added or modified recommendations may also be appropriate and should be provided at the final plan review phase when details of the project constructions and actual development schemes are known:

A. Remedial Grading And Earthworks

The property is mantled by shallow deposits of existing loose fills and natural topsoils over dense and competent bedrock in the high ground areas, while loose to soft saturated alluvial deposits occur within the intervening canyon flowlines. Removal and recompaction of upper soil cover and alluvium will be required as specified below. All grading and earthworks should be completed in accordance with the Appendix J of the California Building Code (CBC), City of San Marcos Ordinances, the Standard Specifications for Public Works Construction and the requirements of the following sections:

1. **Existing Underground Utilities And Structures:** All existing underground waterlines, sewer or leach pipes, storm drains, utilities, tanks, structures and improvements at the project 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 earthworks operations. Alternatively, permanent relocations may be appropriate as shown on the approved plans.

   Abandoned lines, irrigation pipes and conduits should be properly removed, capped or sealed-off to prevent any potential for future water infiltrations into the site fills and graded embankments. 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 surface and subsurface improvements, structures, vegetation, trees, roots, stumps, boulder rocks, construction debris, and all other unsuitable materials and deleterious matter

from all areas proposed for new fills, improvements, and structures plus a minimum of 10 feet outside the perimeter, where possible and as approved in the field.

All trash debris and unsuitable materials generated from site demolitions and vegetation from clearing efforts should be properly removed and disposed of from the site. Trash, vegetation and construction debris shall not be allowed to occur or contaminate new site fills and backfills.

The prepared grounds should be inspected and approved by the project geotechnical consultant or his designated field representative prior to grading and earthworks.

3. **Removals and Remedial Grading:** Site upper soil mantle and alluvial deposits in the areas to receive fill, structures and improvements, plus 10 feet outside the perimeter where possible and as directed in the field, should be removed to the underlying competent bedrock as determined in the field by the project geotechnical consultant and placed back as properly compacted fill.

   Typical removal depths in the vicinity of individual exploratory test pit sites are presented in Table 13. The tabulated values are subject to changes by the project geotechnical consultant based on actual field exposures at the time of grading. Locally deeper removals may be necessary based on the actual field exposures and should be anticipated.

TABLE 13

Trench Location	Total Depth of Trench (ft)	Estimated Removal Depth (ft)	Estimated Depth of Groundwater (ft)	Comments
TP-1	6'	5'	at surface	Road extension canyon fill areas. Trench caving in upper 5'. Divert surface flow / dewater as necessary. Install subdrain at bottom of canyon cleanouts.
TP-2	8½'	8'	1'	Lot 8 canyon fill areas. Trench caving in upper 8'. Divert surface flow / dewater as necessary. Install subdrain at bottom of canyon cleanouts.
TP-3	4½'	1½'	not encountered	Lot 7 cut slope areas. Depth of cut will govern.

TABLE 13 (continued)

TP-4	7'	2½'	not encountered	Lot 7 cut slope areas. Depth of cut will govern.
TP-5	6'	4½'	½'	Lot 6 canyon fill areas. Trench caving in upper 3'. Divert surface flow / dewater as necessary. Install subdrain at bottom of canyon cleanouts.
TP-6	5'	1½'	not encountered	Lot 2 fill slope areas. Depth of hillside benching may govern.
TP-7	5'	bedrock @ surface	not encountered	Lot 4 cut slope areas. Depth of cut will govern. Small Block failure in upper 2'. Will require geologic observations at the time of grading.
TP-8	6'	4'	not encountered	Lot 3 cut areas. Depth of pad cut - undercut will govern.
TP-9	5'	1'	not encountered	Lot 1 fill slope areas. Depth of toe keyway excavation / hillside benching may govern.

Notes:

1. All depths are measured from the existing ground levels. Flow diversions, dewatering and aerating of saturated soils within the site canyons should be anticipated as specified in the following sections.
2. Actual depths may vary at the time of construction based on seasonal conditions and field exposures.
3. Bottom of all removals should be additionally prepared and recompacted to a minimum depth of 6 inches as directed in the field.
4. Remove and recompact all existing site fills in accordance with the requirements of this report and as directed in the field.
5. Bottom of all removals should be additionally prepared, ripped and recompacted to a minimum depth of 6 inches as directed in the field.
6. Exploratory test pits excavated in connection with our study at the indicated locations were backfilled with loose and uncompacted deposits. The loose/uncompacted backfill soils within these trenches shall also be re-excavated and placed back as properly compacted fills as a part of the project grading operations.
7. All ground surfaces steeper than 5:1 receiving fill/backfill should be properly benched and keyed as directed in the field.

4. **Rock Hardness and Bedrock Excavation:** Much of the bedrock planned for excavation at the project site occurs in a weathered and fractured condition and can be excavated with moderate to locally heavy ripping with large dozers (Caterpillar D9H or equivalent). However, harder zones are present in upper site areas that will significantly slow production and require intense and

concentrated effort. Consequently, blasting of upper site areas is recommended in order to allow for economic production and also to improve the quality of generated fill by reducing the size and amount of rock debris.

Blasting at the site should be conducted by a qualified contractor with experience in similar projects. Care should be taken when blasting in proximity to proposed cut slopes. Over-blasting can result in unstable conditions and the need for costly slope reduction.

5. **Surface-Subsurface Water and Dewatering:** Surface flow and groundwater were encountered within the site canyon flowlines at the time of our field investigations. Surface water and high groundwater within the canyon alluvium will impact remedial grading and earthwork constructions depending on the seasonal conditions.

   Temporary flow diversion efforts should be considered during seasonal rainfall periods. Any temporary diversion structures and methods such as diversion channels, sumps and pumps, sheet piles, earthen dikes and berms, etc., which could effectively redirect the flow from the earthworks construction areas may be considered.

   Heavy dewatering efforts should also be anticipated in the southwestern canyons and impacted flowline areas. Any dewatering technique suitable to the field conditions which can also effectively remove the intruding water and allow soil removals and fill placement, is considered acceptable unless otherwise proved inadequate or inefficient. Dewatering should continue until completion of remedial grading operations and should be discontinued only upon approval of the project geotechnical engineer. Groundwater should be adequately lowered below the specified bottom of removals, over-excavations, toe of temporary slope and trenches as approved in the field.

   Performing grading and earthwork construction within the site flowlines during the dry months of the year should be considered.

6. **Canyon Subdrain:** Canyon subdrain system consisting of a 2-foot wide by 2-foot deep trench with a minimum 6-inch diameter, Schedule 40 (SDR 35) 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. 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 approximate locations for the recommended canyon subdrain within the main flowlines are shown on the enclosed Plate 2. Intervening and secondary canyons may also require a similar subdrain system as determined in the field by the project geotechnical consultant and should be anticipated. The actual drain alignments should be established in the field and then surveyed for precise depiction on the project final as-build grading plans. Typical canyon subdrain construction details is illustrated on the enclosed Plate 18.

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.

7. **Transition Pads and Undercuts:** Ground transition from excavated cut to placed fill should not be permitted underneath the proposed structures and improvements. Buildings, foundations and improvements should be uniformly supported on competent bedrock or entirely founded on compacted fills. Transition pads will require special treatment. The cut portion 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 whichever is more.

   Undercutting of cut pads exposing rock surfaces at finish grades to the specified depths and their reconstruction to design elevations with compacted sandy fills is also recommended. In the roadways, driveways, parking and on-grade slabs/improvement transition areas there should be a minimum 12 inches of compacted soils below rough finish subgrade.

   Undercutting cut pads and the cut portion of the transition pads will accommodate excavation of the foundation trenches, and underground storm drain and utilities in an otherwise very hard bedrock units. In the case of deeper utility / storm drain trenches, undercutting to a minimum 8 inches below the proposed inverts may be considered.

8. **Temporary Excavation Slopes:** Undermining existing nearby improvements, structures and adjacent properties by the excavations and removal operations should not be allowed. For this purpose, adequate excavation set-backs shall be maintained and excavation slopes laid back at safe gradients as specified herein and as directed in the field.

Temporary embankments less than 3 feet high maximum may be constructed at near vertical gradients, if approved in the field. Trench and excavation slopes greater than 3 feet maximum should be laid back at 1:1 gradient with the remaining wedge of soil properly benched and new fill/backfill tightly keyed-in as the fill placement progresses. Surface and subsurface water, if any encountered, should be effectively removed from the excavation areas as specified herein, and directed in the field. Some shoring or trench shield support may also be appropriate based on site conditions at the time of constructions and should be anticipated.

9. **Soil Properties, Fill and Backfill Materials:** Site bedrock excavations will predominantly generate poor to marginal quality fills with excessive rock debris that include larger rock sizes and inadequate fines. Generated rock laden materials may be considered for reuse as site new compacted fills provided they are properly processed and manufactured into an approved mixture as specified herein. Lager rock sizes should be excluded from the site fills and backfills as specified herein, and attempts should be made during bedrock excavations to increase fines production.

   Excavations of site topsoils and alluvial deposits will locally generate poor quality plastic clayey deposits. Plastic clayey soils can be detrimental to planned structures and improvements if they occur within upper finish grades. Plastic clayey soils are also not suitable for wall and trench backfills. Adverse effects of site expansive clayey soils should be mitigated by selective burial of these deposits, placed in deep fills a minimum of 4 feet below rough pad grades (or 2 feet below the deepest footing, whichever is more) and a minimum of 15 feet away from the face of graded slopes within the fill mass. Sandy to gravelly soils generated from the site weathered bedrock excavations should then be selectively used within the upper pad grades and outer fill slope surfaces. Improvement areas should be provided with a minimum 12 inches of good quality sandy soils. Alternatively, site clayey soils may be thoroughly mixed with an abundance of generated sandy to gravelly soils available from the site weathered bedrock excavations in order to manufacture a very low expansive mixture.

   Added efforts should be anticipated when processing rocky and clayey deposits for manufacturing a suitable and uniform fill mixture. Soils from the alluvial excavations within the site flowline canyons will also include wet to saturated deposits that will likely require added spreading, aerating and drying efforts in order to achieve near optimum moisture levels.

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. For this purpose, added efforts to increase fines production or importing sandy soils and thoroughly mixing with rocky fills may be necessary. Wall and trench backfills should consist of minus 3-inch particles and maintain the specified fines to rock ratio. Rocks up to 12 inches in maximum diameter may be allowed in compacted fills provided they are individually 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 also 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 as-build grading plans. Rock disposal should be completed in substantial accordance with the enclosed Rock Disposal Recommendations, Plates19 and 20.

10. **Shrinkage, Bulking, Import soils and Compaction:** Based on our analyses, site existing surficial soils and alluvial deposits may be expected to shrink, on average, approximately 10% to 20%, and soils generated from the excavations of onsite bedrock may be anticipated to bulk nearly the same amount on a volume basis when compacted as specified herein.

    Import soils, if needed to complete grading and backfilling or improve the quality of generated rocky fills and achieve the specified fines to rock ratios, should be thoroughly mixed with rock laden materials in order to manufacture a suitable uniform mixture which meets or exceeds the requirements of this report.

    Import soils should be clean sandy granular non-corrosive deposits (SM/SW) with very low expansion potential (100% passing #4 sieve and less than 18% passing #200 sieve with expansion index less than 21). Import soils should be inspected, tested as necessary, and approved by the project geotechnical consultant 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.

    Uniform bearing and subgrade soil conditions should be constructed at the site by the grading operations. Site fills should be adequately processed, thoroughly mixed, moisture conditioned to slightly above (2%-3%) the optimum moisture levels as directed in the field, placed in thin (8 inches maximum) uniform horizontal lifts and mechanically compacted with heavy construction equipments to at least the minimum compaction levels as specified herein based on ASTM D-1557.

Project fills 20 feet thick or shallower should be compacted to a minimum 90% compaction levels unless otherwise specified. Fills greater than 20 feet thick maximum should be compacted to a minimum 95% compaction levels below the upper 20 feet and to minimum 90% levels within the upper 20 feet unless otherwise specified. All fills and backfills placed within the potential flood areas or subject to saturations and inundations should also be compacted to minimum 95% compaction levels.

11. **Permanent Graded Slopes:** Project graded cut and fill slopes are programmed for 1½:1 and 2:1 gradients maximum, respectively. Graded slopes constructed as recommended herein will be grossly stable with respect to deep seated and surficial failures for the indicated design maximum heights and gradients.

    All fill slopes shall be provided with a lower toe keyway. The keyway should maintain a minimum depth of 2 feet into the competent bedrock with a minimum width of 15 feet unless otherwise specified or directed in the field. The keyway should expose competent and stable bedrock units throughout with the bottom heeled back a minimum of 2% into the natural hillside and observed and approved by the project geotechnical consultant. Additional level benches should be constructed into the natural hillside as the fill slope construction progresses. Added excavation efforts including the use of rock breakers 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 minimum 90% (or 95%) of the laboratory standard out to the slope face as specified. Over-building and cutting back to the compacted core, or backrolling at a maximum 4-foot vertical increments and "track-walking" at the completion of grading is recommended for site fill slope construction. Geotechnical engineering observations and testing will be necessary to confirm adequate compaction levels within the fill slope face.

    Cut slopes should 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.

Graded slopes more than 30 feet in maximum vertical height should be provided with a minimum 6 feet wide terraces for control of surface drainage and debris. Drainage terraces should be provided at 30 feet maximum vertical intervals except where only one terrace is required, it should be placed near mid-height. A minimum 12 feet wide drainage terraces should be provided near the mid-height of slopes greater than 60 feet unless otherwise approved.

All graded slopes should be constructed in general accordance with the enclosed Typical Key and Benching Details, Plates 21 and 22.

12. **Slope Toe Drainage Systems:** A subsurface toe drainage system may be considered at the base of project cut slopes likely to transmit up-slope water. The subsurface toe drain should consist a minimum of 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. Collected water should discharge into an approved outlet. Specific recommendations should be given by the project geotechnical engineer in the field at the time of construction based on actual subsurface exposures and exposed slope conditions. Cut slope toe drains should be shown on the final as-build grading plans.

13. **Surface Drainage and Erosion Control:** A critical element to the continued stability of the graded building pads, slopes and embankments is an adequate storm water and surface drainage control, and protection of the slope faces. This can most effectively be achieved by appropriate storm water control and drainage structures, vegetation cover and the installation of the following systems:

    * Soil erosion, scouring and sediment transport should not be allowed at the site. Erosion and scour control structures as well as energy dissipaters should be installed as shown on the approved civil drawings.

    * Drainage swales should be provided at the top of the slopes per the project civil engineer design. Slopes 30 feet or more in vertical height should be provided with appropriate drainage terrace.

    * 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.

    * Hard rock slope surface will create planning difficulties. However, finish slope faces should be considered for suitable planting program as determined by the project landscape consultant. Unprotected slope faces

should be avoided, and over-watering should not be allowed. Only the amount of water to sustain vegetation should be provided.

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

14. **Engineering Observations:** All grading and earthworks operations including 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 not limited to the following:

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

* Bottom of toe keyway/over-excavation observation - After competent bedrock or firm native ground 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 3 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 feet 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 feet maximum. Wall backfills should consist of minus 3-inch materials and also mechanically compacted to a minimum of 90% (or 95%) 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 observation - After the foundation trench excavations but before steel placement.

* Foundation bearing/slab subgrade soils observation - Prior to the placement of concrete 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 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 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. All trench backfills shall consist of minus 3-inch particles and mechanically compacted to a minimum of 90% (or 95%) 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% (or 95%) 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 Floor Slabs

The following preliminary recommendations are consistent with the anticipated very low expansive (expansion index less than 21) gravelly silty sand to sandy silty gravel (GM/GP) foundation bearing and subgrade soils and anticipated as graded conditions. Final foundation and slab design will depend on expansion characteristics of final foundation bearing soils and actual fill differential thickness underlying individual building pads. Modified or more specific recommendations may also be required and should be given at the plan review phase. All design recommendations should be further confirmed and/or revised as necessary 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:

1. Planned residential buildings may be supported on shallow stiff concrete foundations. The shallow foundations should be uniformly founded on certified compacted fills or founded entirely on undisturbed competent bedrock. Acceptable building foundations may include a system of spread pad and strip or turned-down footings with slab-on-grade floors.

2. Continuous strip stem wall and turned-down footings should be sized at least 15 inches wide and 18 inches deep with no, or less than 15 feet of fill differential thickness and 18 inches wide by 24 inches deep for fill differential thicknesses of greater than 15 feet for single and two-story buildings. Isolated pad footings should be at least 30 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 stem wall foundations and turned-down footings should enclose the entire building perimeter. Exterior isolated pad footings should also be interconnected with perimeter foundations with grade beams for lots where fill differential thicknesses exceed 15 feet maximum.

   Continuous interior and exterior stem wall foundations should be reinforced by at least four #4 reinforcing bars for lots with no, or less than 15 feet fill differential thickness and four #5 reinforcing bars for lots with more than 15 feet fill differential thickness. Place a minimum of two #4 (or #5) bars 3 inches above the bottom of the footings and a minimum of two #4 (or #5) bars 3 inches below the top of the stem wall. Turned-down footings should be reinforced with a minimum of two #4 (or #5) bars at the top and two #4 (or #5) bars at the bottom. Reinforcement details for spread pad footings should be provided by the project architect/structural engineer.

3 All interior slabs for lots with no, or less than 15 feet fill differential thickness should be a minimum 4 inches in thickness reinforced with #3 reinforcing bars spaced 16 inches on center each way placed mid-height in the slab. Interior slabs for lots with more than 15 feet of fill differential thickness should be at least 5 inches thick reinforced with #4 bars at 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 15-mil plastic) placed mid-height in the sand.

   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 raveling. 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 23 may be used as a general guideline.

4. 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 concrete.

C. Exterior Concrete Slabs / Flatworks

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

2. 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 the 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 raveling. Avoid wheeled equipments across cuts for at least 24 hours.

3. All exterior slab designs should be confirmed in the final as-graded compaction report.

4. 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.

D. Soil Design Parameters

The following preliminary soil design parameters are based on the tested representative samples of on-site earth deposits. Onsite poor to marginal quality plastic clayey soils should not be used for wall and trench backfills. All parameters should be re-evaluated when the characteristics of the final as-graded soils have been specifically determined:

* Design unit weight of soil = 125 pcf.
* Design angle of internal friction of soil = 31 degrees.
* Design active soil pressure for retaining structures = 40 pcf (EFP), level backfill, cantilever, unrestrained walls.
* Design active soil pressure for retaining structures = 63 pcf (EFP), 2:1 sloping backfill, cantilever, unrestrained walls.
* Design at-rest soil pressure for retaining structures = 61 pcf (EFP), non-yielding, restrained walls.
* Design passive soil resistance for retaining structures = 391 pcf (EFP), level surface at the toe.
* Design coefficient of friction for concrete on soils = 0.38.
* Net allowable foundation pressure for compacted fills (minimum 15 inches wide by 18 inches deep footings) = 1500 psf.
* Net allowable foundation pressure for competent undisturbed bedrock (minimum 15 inches wide by 18 inches deep footings) = 2500 psf.
* Allowable lateral bearing pressure (all structures except retaining walls) = 150 psf/ft .

Notes:

* Use a minimum safety factor of 1.5 for wall over-turning and sliding stability. However, because large movements must take place before maximum passive resistance can be developed a safety factor of 2 may be considered for sliding stability 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 net allowable foundation pressure provided herein were determined for footings having a minimum width of 15 inches and embedded at least 18 inches into approved foundation soils. The indicated values may be increased by 20% for each additional foot of depth and 20% for each additional foot of width to a maximum of 4500 psf, if needed. The allowable foundation pressure provided herein also applies to dead plus live loads and may be increased by one-third for wind and seismic loading.

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

E. Asphalt And PCC Pavement Design

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 may be considered for initial planning phase cost estimating purposes only (not for construction):

A minimum section of 3 inches asphalt on 6 inches Caltrans Class 2 aggregate base or the minimum structural section required by the City of San Marcos, whichever is more, may be considered for the onsite 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 AC thickness for each 2% increase in grade or portions thereof. PCC paving should be considered for longitudinal grades greater than 15% maximum. Actual design will also depend on the design TI and approval of the City of San Marcos.

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.

PCC Paving: PCC driveway and parking supported on very low expansive subgrade soils should be a minimum of 5 inches in thickness, reinforced with #3 reinforcing bars at 18 inches on centers each way, placed 2 inches below the top of slab.

Subgrade soils beneath the PCC parking and driveway should be compacted to a minimum 90% of the corresponding maximum dry density within the upper 6 inches, unless otherwise specified.

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 feet intervals maximum. The pavement anchors should be poured monolithically with the concrete paving surfaces.

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 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 raveling. 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 methods (curing compound maximum coverage rate = 200 sq. ft. / gal.).

General Paving: Subgrade and basegrade 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.

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 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 21). Appropriate recommendations should be given in the final as-graded compaction report.

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 is 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. Inadequate staking and/or lack of grading control may result in unnecessary additional grading which will increase construction costs.

3. Footings located on or adjacent to the top of slopes should be extended to a sufficient depth to provide a minimum horizontal distance of 7 feet or one-third of the slope height, whichever is greater (need not exceed 40 feet maximum) between the bottom edge of the footing and face of slope. This requirement applies to all improvements and structures including fences, posts, pools, spas, etc. Concrete and AC improvements should be provided with a thickened edge to satisfy this requirement.

4. 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 form the face of such footing.

5. 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.

6. 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.

7. 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 24. Planting large trees behind site building/basement retaining walls should be avoided.

8. All underground utility and plumbing trenches should be mechanically compacted to a minimum 90% (or 95%) 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.

9. 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. Planter areas adjacent to building foundations should be provided with impermeable liners and subdrain, if determined appropriate.

10. 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.

11. 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.

12. The amount of shrinkage and related cracks that occurs 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 aggregate usually requires 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.

13. 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.

IX. 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 be 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.

The property owner(s) should be aware that the development of cracks in all concrete surfaces such as floor slabs and exterior stucco are associated with normal concrete shrinkage during the curing process. These features depend chiefly upon the condition of concrete and weather conditions at the time of construction and do not reflect detrimental ground movement. Hairline stucco cracks will often develop at window/door corners, and floor surface cracks up to ⅛-inch wide in 20 feet may develop as a result of normal concrete shrinkage (according to the American Concrete Institute).

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 development 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 to ensure 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 soils 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. The project soils engineer should also be provided the opportunity to verify the foundations prior the placing of concrete. If the project soils 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 #07-420-P** will help to expedite our response to your inquiries.

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