APPENDIX J
GEOTECHNICAL INVESTIGATION – EASTERN PARCEL

Geotechnical Investigation

Proposed Multi-Family Residential Project
4665 North River Road
Oceanside, California

(A.P.N. 157-060-40)

January 18, 2016

Prepared For:
So Cal Ag Properties, Inc.
Mr. Jay Kwano
P.O. Box 4601
Oceanside, California 92052

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

Job #15-188-P

VINJE & MIDDLETON ENGINEERING, INC.

2450 Auto Park Way
Escondido, California 92029-1229

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

Job #15-188-P

January 18, 2016

So Cal Ag Properties, Inc.
Mr. Jay Kwano
P.O. Box 4601
Oceanside, California 92052

GEOTECHNICAL INVESTIGATION, PROPOSED MULTI-FAMILY RESIDENTIAL PROJECT, 4665 NORTH RIVER ROAD, OCEANSIDE, CALIFORNIA (A.P.N. 157-060-40)

Pursuant to your request, Vinje & Middleton Engineering, Inc. has completed the attached Update Geotechnical Investigation Report for the proposed family apartment housing project at the above-referenced site.

The following report summarizes the results of our research and review of previous pertinent geotechnical reports, maps and records, current field observations, field explorations, laboratory testing, and engineering analyses. Conclusions and recommendations based on this study and consistent with site geotechnical conditions are provided for the proposed future development, as understood. From a geotechnical engineering standpoint, it is our opinion that the site is generally suitable for the planned multi-family residential development 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 #15-188-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

RALPH MALCOLM VINJE
No. 863
Exp. 12-31-17

Ralph M. Vinje
GE #863

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL PROJECT
4665 NORTH RIVER ROAD
OCEANSIDE, CALIFORNIA
(A.P.N. 157-060-40)

I. INTRODUCTION

The property investigated herein includes a commercial packing plant development, a private residence and a vacant lot at the referenced location in the city of Oceanside. The site location is shown on a Vicinity Map attached to this report as Plate 1. Approximate site coordinates are 33.2445°N latitude and -117.3106°W longitude. We understand that the property is planned for redevelopment into a future condominium development.

Consequently, this investigation was initiated to determine soil and geotechnical conditions at the property and to ascertain their influence upon the proposed future development. Test pit digging, test borings, soil sampling and laboratory testing were conducted in support of this effort which has resulted in the remedial grading, bearing soil preparation and development recommendations provided in following sections.

II. SITE DESCRIPTION

A Geotechnical Map, reproduced from an As-Built survey map by Dask Land Survey (dated August 26, 2009), shows existing site conditions and is included as Plate 2. The project site is nearly a square-shaped property bordered by North River Road and Calle Joven to the north and south respectively, a commercial entity lies to the east, and a vacant lot to the west. Eastern portions of the property are presently occupied by a large, multi-story packing plant with nearby associated structures and improvements. Surface areas associated with the packing plant range from paved to dirt and are utilized for parking and storage. An older, single-family residential structure occupies the central portion of the property. The west portion of the property is characterized largely by natural dirt-covered surfaces that were previously used for agricultural purposes. The San Luis Rey River lies to the south in relatively close proximity to the site.

Level to gentle topography characterize much of the project site. Minor graded slopes are associated with a small elevated area east of the packing plant structures. Graded slopes are also present ascending to adjacent roadways along the west and south site margins. All graded slopes are constructed at gradients approaching 2:1 maximum and generally approach 5 feet high maximum.

Access to the site is provided by paved entries/exits that connect to North River Road to the north and Calle Joven to the south.

Site drainage within the developed portion of the site (central and east portions) is generally developed to flow away from structures and improvements to storm drains and then offsite. Drainage in the west portion of the property generally sheetdrains in a southerly direction to a dirt swale which drains into a storm drain in the southwest corner of the site. Excessive scouring or erosion is not in evidence.

III. PROPOSED DEVELOPMENT

Development plans are not yet available. However, we understand that existing structures and improvements will be removed to make room for a condominium development with associated interior roads, surface and subsurface improvements.

Minor grade alterations (less than 10 feet) are expected for the creation of level building pad surfaces. The construction of new large graded embankments are not anticipated in connection with the future site development.

Detailed foundation plans are also not yet available. However, planned condominium buildings are anticipated to consist of masonry or wood-frame structures with exterior stucco supported on shallow stiff foundations with stem walls and slab-on-grade floors, or slab-on-ground with turn-down footings.

IV. SITE INVESTIGATION

Subsurface conditions at the property were determined by the excavation of 5 test pits dug with a track-mounted 310 caterpillar excavator and 2 test borings drilled with a truck-mounted rotary drill rig. All the exploratory excavations were logged by our project geologist who also directed in-situ sampling at selected depths and locations for subsequent laboratory testing. The borings were permitted (Permit #LMWP 001990), per the County of San Diego DEH requirements. Logs of the excavations are included as Plates 3-11. Laboratory test results and engineering properties of tested samples are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

The northeast / east portions of the project property are underlain by Pleistocene age Terrace Deposits. Elsewhere, the study areas are underlain by alluvium deposits to the depths explored. Geologic Cross-Sections depicting subsurface conditions based on Plate 2 of our test excavations is attached as Plate 12.

A. Earth Materials

Terrace Deposit: The northeast and east portions of the study property are underlain at shallow depths by Pleistocene age Terrace Deposits. The Terrace Deposits, as exposed in our test pits, consist of red brown-colored sandstone that was found in cemented and dense conditions overall. Project Terrace Deposits are considered stable, competent rocks that will provide adequate support for future fills, structures, and improvements.

Alluvium: Younger alluvium deposits, associated with the nearby San Luis Rey River, occur in the central and western areas of the property, outside the Terrace Deposits. Project alluvium are typically silty fine to medium grained sandy deposits. Site alluvium generally occur in a loose condition near the surface and becomes more uniformly firm to locally dense at depth. Local areas of cohesionless "running sand" were exposed within our test pits. Based on our test borings, site alluvium is known to extend more than 50 feet beneath much of the alluvial portion of the property.

B. Groundwater and Surface Drainage

Groundwater was encountered in our deep boring at a depth of 26 feet to 27 feet below the ground surface. Indicated groundwater levels are expected to fluctuate depending upon seasonal rainfall conditions and annual storm events influencing flow levels within the nearby San Louis Rey River. Accurate historic high groundwater levels at the project property are also unknown. However, a very significant raise is generally not considered likely. Therefore, groundwater levels, as recorded during this study, are not expected to impact grading and construction work or directly impact future buildings and improvements. Project excavations may encounter some subsurface groundwater or local seeps depending on the seasonal conditions requiring appropriate dewatering efforts suitable to the site conditions. Ground stabilization technique and remedial grading recommendations outlined in the following sections are provided considering potential effects of possible groundwater intrusions and saturated ground conditions.

As with all developed properties, the proper control of flood waters and site surface drainage is a critical component to overall stability of the graded building pads. Surface water should not pond upon graded surfaces, and irrigation water should not be excessive. Over-watering of site vegetation may also create perched water and the creation of excessively moist areas at finished lot surfaces.

C. 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 17 miles from the project area. This event, which is thought to have occurred along an offshore fault, reached an estimated magnitude of 6.5 with estimated bedrock acceleration values of 0.136g 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 EQ Fault Version 3.00 updated) typically associated with the fault is also tabulated.

TABLE 1

FAULT ZONE	DISTANCE FROM SITE	MAXIMUM PROBABLE ACCELERATION (R.H.)
Newport-Inglewood Fault	9.2 miles	0.169g
Rose Canyon Fault	10.3 miles	0.156g
Elsinore-Julian Fault	19.5 miles	0.167g
Coronado Bank Fault	26.3 miles	0.156g

The location of significant faults and earthquake events relative to the study site are depicted on a Fault - Epicenter Map attached to this report as Plate 13.

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.

D. Seismic Ground Motion Values

Seismic ground motion values were determined as part of this investigation in accordance with Chapter 16, Section 1613 of the 2013 California Building Code (CBC) and ASCE 7-10 Standard using the web-based United States Geological Survey (USGS) ground motion calculator. Generated results including the Mapped (S_s, S_1), Risk-Targeted Maximum Considered Earthquake (MCE_R) adjusted for site Class effects (S_{Ms}, S_{M1}) and Design (S_{Ds}, S_{D1}) Spectral Acceleration Parameters as well as Site Coefficients (F_a, F_v) for short periods (0.20 second) and 1-second period, Site Class, Design and Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrums, Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration adjusted for Site Class effects (PGA_M) and Seismic Design Category based on Risk Category and the severity of the design earthquake ground motion at the site are summarized in the enclosed Appendix.

E. Geologic Hazards

Conditions which could result in potential geologic hazards are known in the areas of San Diego County. In accordance with the Uniform Building Code regulations and local codes and standards, the following geotechnical factors are herein evaluated:

1. **Seismicity** - The most significant geotechnical factor which could impact the project site relates to ground shaking during an earthquake event along an active fault. Moderate to locally heavy levels of ground shaking can be anticipated during rare events over the lifetime of the development. Details of the project's seismic environment are given in a preceding section.

2. **Faulting** - Faults or significant shear zones are not indicated within the project site. The project is not located in proximity to Alquist - Priolo earthquake fault zone areas associated with active faults discussed above.

3. **Flood Inundation Potential** - Flooding hazards at the project site were evaluated by a review of nearby drainage basins, and review of the appropriate Flood Insurance Rate Map, compiled by the Federal Emergency Management Agency (FEMA). The San Luis Rey River is a significant feature that drains a large portion of north San Diego county and is located south of the subject property. According to FEMA Map Panel 756 of 2375 enclosed with this report as Plate 14, the project site is situated within Zone X, designated as "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1-foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood."

4. **Liquefaction** - Soil liquefaction or related ground failures can adversely impact manmade structures and improvements at sites where subsoils consist of loose alluvial deposits inundated with groundwater. Liquefaction is the collapse of the soil structure in association with an increase in pore pressure during a seismic event. A liquefaction analysis of the underlying alluvial soil was conducted as part of this investigation. Findings and conclusions of the liquefaction analysis are detailed in a following section.

5. **Slope Stability** - No significant slopes are present. Anticipated future graded fill slopes will be grossly stable to design heights provided our grading recommendations are implemented during grading.

6. **Collapsible Soils** - Buildings and improvements founded on loose to very loose and dry sandy deposits may be damaged by soil collapse. Soil collapse is sudden and often large induced settlements when susceptible load bearing deposits become saturated after construction. Collapsible soils are typified by

low values of dry unit weight and natural water content. The amount of settlement depends on the applied vertical stresses and the extent of the wetting and availability of water.

Upper alluvial deposits at the site are in a dry and loose to very loose condition indicating a high potential for collapse. Collapsible soils within the project foundation bearing and subgrade soils should be removed and recompacted as controlled fill as recommended in following sections.

7. **Expansive Soils -** Site soils are predominantly sandy granular non to very low expansive deposits. Potentially expansive soils may locally occur at the site in minor quantities. Site potentially expansive soils, if encountered, should be selectively buried in deeper fills or thoroughly mixed with an abundance of available non expansive soils in order to manufacture a non to very low expansive mixture as recommended below.

8. **Settlements and Ground Subsidence -** Anticipated settlements after removal and recompaction of the upper alluvial soils as specified herein, are expected to be within the allowable tolerances on the order of 1 inch, which is expected to 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.

   Monitoring of site soils during and after remedial grading of upper alluvial soils by installing settlement plates and settlement monuments are recommended to confirm settlement characteristics of the underlying soils.

F. 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 & boring excavations, site soils have been grouped into the following soil types:

TABLE 2

Soil Type	Description
1	Grey silty fine to medium sand - Fill (af) / Alluvium (Qal)
2	Red brown fine to medium sand - Fill (af) / Terrace Deposit (Qt)

The following tests were conducted in support of this investigation:

1. **Standard Penetration Test:** Standard penetration tests (SPT) were performed at the time of borehole drilling in accordance with ASTM standard procedure D-1586 using the rope & cathead method. The procedure consisted of a standard 51 MM outside diameter sampler without liner, 457 MM in length and 35 MM in inside diameter driven with a 140-pound hammer, dropped 30-inches using 5-foot long AW drill rods. The bore hole was 200 MM (8 inches) in diameter and mud wash drilling method was used below the water table. The test results are indicated at the corresponding locations on the attached Boring Logs.

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 3.

TABLE 3

Location	Soil Type	Maximum Dry Density (Ym-pcf)	Optimum Moisture Content (ω_{opt}-%)
TP-1 @ 2'	1	127.1	10.0
TP-5 @ 4'	2	135.2	8.7

3. **Moisture-Density Tests (Undisturbed Chunk and Ring 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, and undisturbed ring samples using the weights and measurements test method. Results are presented in Table 4 and tabulated on the attached Test Trench and Boring Logs.

TABLE 4

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'	1	3	104.9	127.1	83	13
TP- 1 @ 4'	1	3	97.5	127.1	77	11
TP-1 @ 6'	1	3	96.6	127.1	76	11
TP-1 @ 8'	1	3	105.2	127.1	83	13
TP-1 @ 10'	1	9	94.1	127.1	74	30
TP-1 @ 12'	1	7	92.8	127.1	73	23

TABLE 4 (continued)

TP-1 @ 14'	1	11	85.5	127.1	67	31
TP-2 @ 3'	1	3	104.8	127.1	82	13
TP-2 @ 6'	1	3	102.6	127.1	81	13
TP-2 @ 10'	1	6	102.5	127.1	81	25
TP-2 @ 14'	1	4	99.7	127.1	78	16
TP-3 @ 2'	1	2	118.8	127.1	94	27
TP-3 @ 5'	1	12	102.9	127.1	81	51
TP-4 @ 2'	2	9	118.2	135.2	87	55
TP-4 @ 4'	1	3	108.7	127.1	86	15
TP-4 @ 6'	1	8	118.0	127.1	93	50
TP-4 @ 11'	1	2	119.3	127.1	94	13
TP-5 @ 2'	1	6	105.6	127.1	83	27
TP-5 @ 4'	2	7	118.7	135.2	88	43
TP-5 @ 6'	2	7	114.0	135.2	84	38
TP-5 @ 8'	2	7	117.2	135.2	87	42
TP-6 @ 2'	2	3	108.8	135.2	81	14
TP-6 @ 4'	2	4	118.4	135.2	88	24
TP-7 @ 5'	1	12	87.7	127.1	69	35
B-1 @ 5'	1	3	101.0	127.1	79	12
B-1 @ 10'	1	3	101.6	127.1	80	12
B-1 @ 15'	1	3	-	127.1	Sample Disturbed	10
B-1 @ 20'	1	7	-	127.1	Sample Disturbed	28

Note 1: Sample may be somewhat disturbed.
Assumptions And relationships:
In-place Relative Compaction = $(Y_d \div Y_m)$ X100
Gs = 2.70
$e = (Gs\ Y_\omega \div Y_d) - 1$
$S = (\omega\ Gs) \div e$

4. **Amount of Material in Soils Finer Than the No. 200 Sieve:** The amount of material in soils finer than No. 200 sieve tests were performed on selected representative samples of soil type 1 in accordance with the ASTM D-1140. Test results are tabulated in Table 5:

TABLE 5

Location	Original Dry Mass (g)	Dry Mass Retained after washing (g)	Percent of Material Finer Than No. 200 Sieve	Predominant Soil Type
B-2 @ 18'	352.4	335.2	5	SM
B-2 @ 48'	394.3	283.3	28	SM

5. **Liquid Limit, Plastic Limit and Plasticity Index:** Liquid limit, plastic limit, and plasticity index tests were performed on a representative sample of Soil Type 1 in accordance with the ASTM D-4318. Test results are tabulated in Table 6.

TABLE 6

Location	Soil Type	Liquid Limit (LL-%)	Plastic Limit (PL-%)	Plasticity Index (PI=LL-PL)
B-2 @	1	23	23	0 (Non-Plastic)

6. **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 @ 2'	1	9.4	50.1	16.4	111.8	0	0

(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

7. **Direct Shear Test:** One direct shear test was performed on a representative sample of Soil Type 1. The prepared specimen was 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 result is 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 @ 2'	1	Remolded to 90% of YM @ % ωopt	125	31	0

8. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 2 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-4 @ 2'	2	2148	7.9

9. **Sulfate Test:** A sulfate test was performed on a representative sample of Soil Type 2 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-4 @ 2'	2	0.006

10. **Chloride Test:** A chloride test was performed on a representative sample of Soil Type 2 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-4 @ 2'	2	0.003

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 testing performed on selected representative site samples indicate that the minimum resistivity is greater 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. 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 0.006 percent by weight which is considered negligible according to 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	34	44	54	74	95	115

VII. HYDRO MODIFICATIONS

Specific hydro modification designs for the project development are not currently available. The following are appropriate for the design and construction of sand filtration trenches, vegetated swales, buffers or strips and sedimentation ponds from a geotechnical engineering point of view.

- Water should not be allowed to penetrate and saturate structural fills, graded or natural slopes, bearing and subgrade soils and wall backfills. Consequently, filtration trenches, sedimentation ponds, and vegetated swales may not be suitable on fill sites, wall backfill zones and areas of problematic soils such as expansive/compressible (collapsible) soils, unless otherwise designed for and approved.

- Groundwater separation should be at least 10 feet from the trench invert to the measured (or historic high, whichever is higher) groundwater elevation, unless otherwise approved.

- Locations away from the buildings, embankments, pavements, walls, structures and improvements greater than 10 feet will be required unless otherwise approved. Closer systems may be permitted if specifically designed and protective structures and moisture protection measures are provided. Set back from wells greater than 100 feet should also be considered unless otherwise permitted.

VIII. LIQUEFACTION EVALUATION AND ANALYSIS

Liquefaction Potential: Soil liquefaction or related ground failures can adversely impact manmade structures and improvements at the sites where subsoils consist of loose sandy deposits inundated with groundwater. Liquefaction is the sudden loss of soil strength in response to ground shaking during an earthquake event. At the study site, the subsoil profile consist of ancient alluvial (Qal) deposits that are chiefly medium dense to dense deposits with satisfactory to relatively high SPT (uncorrected N) values (see attached Boring Logs). Dense alluvial deposits are generally less susceptible to liquefaction.

Static groundwater level was also established at the depth of 26 feet below the existing ground levels (BGS) and may be expected to fluctuate. In the absence of accurate records, the historic high groundwater (HHW) is assumed at the depth of 12 feet BGS for the purpose of our analysis.

In order to more accurately establish liquefaction potential at the project site, one of the exploratory borings (Boring B-2) was advanced to the depth of 52.5 feet (BGS) with frequent in-situ testing (SPT) as the drilling progressed. Analysis were subsequently performed on the collected subsoil data in order to assess liquefaction potential. For this purpose, field SPT values were first corrected and normalized to determine $N1_{60}$ values, as presented in the following table.

D (ft)	Nm	% (#200	Unit Wt. (Pcf)	∂'0 (tsf)	C_N	C_E	C_B	C_R	C_S	Silt Corr.	$N1_{60}$	Liq'n Poten'l
3	11		104	0.156	2.0	1.0	1.15	0.75	1.2	1.0	23	No**
6	11		104	0.312	1.83	1.0	1.15	0.75	1.2	1.0	21	No**
9	12		104	0.468	1.49	1.0	1.15	0.75	1.2	1.0	18	No**
12	20		104	0.624	1.29	1.0	1.15	0.85	1.2	1.0	30	No**
15	21		42*	0.687	1.23	1.0	1.15	0.85	1.2	1.0	30	No
18	24	5	42*	0.750	1.18	1.0	1.15	0.85	1.2	1.0	33	No
21	24		42*	0.813	1.13	1.0	1.15	0.95	1.2	1.0	35	No
24	22		42*	0.876	1.09	1.0	1.15	0.95	1.2	1.0	31	No
27	19		42*	0.939	1.05	1.0	1.15	0.95	1.2	1.0	26	Marginal
30	17		42*	1.000	1.02	1.0	1.15	0.95	1.2	1.0	23	Marginal
33	25		42*	1.065	0.99	1.0	1.15	1.0	1.2	1.0	34	No
36	33		42*	1.128	0.96	1.0	1.15	1.0	1.2	1.0	44	No
39	32		42*	1.191	0.94	1.0	1.15	1.0	1.2	1.0	42	No
42	20		42*	1.254	0.91	1.0	1.15	1.0	1.2	1.0	25	Marginal
45	24		42*	1.317	0.89	1.0	1.15	1.0	1.2	1.0	29	Marginal
48	27	28	42*	1.380	0.87	1.0	1.15	1.0	1.2	1.0	32	No
51	20		42*	1.443	0.85	1.0	1.15	1.0	1.2	1.0	33	No
* Buoyant Unit Weight. ** Above Groundwater.						Cat-head	8"-Dia.		No Liner	Avg. $N1_{60}$ = 30		

Typically, $N1_{60}$ of 30 or greater indicate the subsoil strata below groundwater table is not liquefiable. $N1_{60}$ of 30 between 15 and 30 indicate a marginal liquefaction potential. Liquefaction potential of saturated loose subsoil strata may exist and appropriate mitigation measures typically apply for $N1_{60}$ less than 15.

Further analyses were performed on the underlying subsoil layers considering the $N1_{60}$ values, design earthquake magnitude (M) of 6.5 and peak horizontal acceleration (a_{max}) of 0.437g. Results of our analyses are summarized in the following table:

Depth (ft)	% Clay	LL (%)	PI	MC (%)	0.9LL	Nm	$N1_{60}$	CRR	CSR	Safety Factor (SF)	Comments
3'				4		11	23	0.25	0.187	1.3	Above Water
6'						11	21	0.22	0.187	1.2	Above Water
9'						12	18	0.20	0.185	1.1	Above Water
12'				4		20	30	0.50	0.185	2.7	Non-Liquefiable
15'						21	30	0.50	0.204	2.4	Non-Liquefiable
18'				4		24	33	Inf.	0.226	Inf.	Non-Liquefiable
21'						24	35	Inf.	0.241	Inf.	Non-Liquefiable
24'				28		22	31	0.50	0.253	2.0	Non-Liquefiable
27'				33		19	26	0.30	0.260	1.2	Non-Liquefiable
30'						17	23	0.25	0.268	0.93	Liquefiable
33'				25		25	34	Inf.	0.271	Inf.	Non-Liquefiable
36'						33	44	Inf.	0.273	Inf.	Non-Liquefiable
39'				29		32	42	Inf.	0.277	Inf.	Non-Liquefiable
42'						20	25	0.28	0.276	1.0	Marginal/Liq'ble
45'				30		24	29	0.42	0.275	1.5	Non-Liquefiable
48'				28		27	32	Inf.	0.274	Inf.	Non-Liquefiable
51'		23〈35	0	24	〉21	20	23	0.32	0.271	1.2	Non-Liquefiable

Liquefaction Screening: CLAYEY soils may be liquefiable having ALL of the following criterial apply:
i) %clay content (0.005mm) 〈 15
ii) L.L. 〈 35%
iii) MC 〉 0.9L.L.

Assumptions and Explanations: Inf. = Infinite, High Groundwater Table Assumed at 12', a_{max} = 0.437g, MSF = 1.5, % Fines = 5% to 45' & 15% below 45'.

Our analysis, as summarized in the table above, generally indicate satisfactory factor of safety against potential liquefaction within the saturated subsoil layers overall, with the exception of 2 thin layers where liquefiable and marginally liquefiable soils occur at the depths of 30 and 42 feet respectively. A minimum safety factor of 1.1 or greater are considered satisfactory against liquefaction potential.

However, the potentially liquefiable (safety factors less than 1.1) thin layers occur at the depth (30 and 42 feet BGS), sandwiched between "non-liquefiable" layers on the top and bottom, and are approximately 3 feet thick. Consequently, liquefaction potential within these thin layers are not considered to be a major influencing factor on the overall subsoil profile.

Ground stabilization and remedial grading procedures specified herein consisting of 15 feet removal of the upper alluvial soils and recompaction to minimum 95% compaction levels are also provided to further improve foundation soils against liquefaction potential to very low levels of risk. In our opinion, liquefaction will not be a major geotechnical concern in the development of the project property provided our remedial grading and foundations recommendations are followed. The 95% compacted fills within the upper 15 feet will also mitigate soil collapse potential indicated in the upper loose and dry sandy alluvial deposits.

Seismically induced total and differential settlements are accepted to be on the order of 1.25 and 0.75 inches for total and differential settlements respectively, for a rare 7.5 magnitude earthquake event. Smaller seismically induced total and differential settlements, on the order of 1 and 0.5 inches, respectively, may be estimated for more frequent low magnitude seismic events.

IX. CONCLUSIONS

The redevelopment of the project site for multi-family residential purposes is generally feasible from a geotechnical viewpoint. The following conditions are unique to the study property and will most impact its redevelopment from a geotechnical viewpoint:

1. Landslides, faults, or significant shear zones are not present at the site and are not considered a geotechnical factor in the redevelopment of the project property.

2. Collapse of the upper dry, loose alluvial soils during a major seismic event along a nearby active fault is considered the most significant geotechnical concern at the portions of the project property underlain by young alluvial deposits. Consequently, added site specific analysis were performed to more accurately evaluate soil collapse and liquefaction potential to establish appropriate remedial grading ground stabilization techniques suitable for support of future buildings and improvements in these areas.

   Based on our analysis, remedial grading ground stabilization techniques are recommended in the following sections in order to construct a stable bearing soils profile based on the anticipated soil collapse and liquefaction susceptibility at the project alluvial areas.

3. Post construction settlements after removal and recompaction of the upper alluvial soils as specified herein, are anticipated to be within the allowable tolerances on the order of 1 inch, and are expected to occur below the heaviest loaded footings. 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. Monitoring during and after ground stabilization remedial grading is recommended to confirm settlement characteristics of the underlying soils.

4. The project site is relatively flat to a very gently sloping terrain and creation of larger graded slopes is not expected in connection with the proposed future site development. Consequently, slope stability will not be a major geotechnical factor in the development of the project property.

5. Site excavations will chiefly generate silty sand deposits which may be considered for reuse as new compacted fills as approved in the field. Attempts should be made to bury any clay-bearing soils (if encountered) in deeper fills. Higher compaction requirements typically require added processing, mixing, and grading efforts to manufacture suitable fill mixture and achieve the specified compaction levels.

6. Site soils are expected to shrink when compacted as specified herein and import soils may be required to achieve final design grades. Import soil should be good quality sandy (D.G.) deposits conforming to the requirements of this report as specified below.

7. Based on our field observations and available test results, site soils predominantly consist of silty sand (SM) deposits with very low expansion potential based on ASTM D-4829 classification. Actual classification and expansion characteristics of the finished grade soil should be confirmed in the final as-graded compaction report based on proper testing of foundation bearing and subgrade soils.

8. Based on our field explorations, groundwater conditions were recorded at the depths of 26-27 feet below the ground surface. Historic high groundwater (HHG) levels at the project property are unknown.

   Groundwater conditions at the project site are also expected to seasonally fluctuate. However, a significant rise with major impacts on remedial grading efforts of upper soils, as recommended herein, are not expected. Some subsurface groundwater or local seeps may impact site deeper excavations depending on seasonal conditions which may require dewatering efforts suitable to the site conditions. Commencing site excavations and remedial grading operations during dry seasons of the year is recommended.

9. Adequate site surface drainage control is a critical factor in the future stability of the developed property as planned. Drainage and storm water control facilities should be designed and installed for proper collection and disposal of surface runoff. Hydro modifications and stormwater management should be designed and constructed considering the site geotechnical conditions as outlined in this report.

10. Site excavations and proposed constructions should not impact the adjacent properties, structures and improvements. Adequate setbacks shall be maintained and temporary construction slopes developed or supported as specified in the following sections. Added or revised 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.

X. RECOMMENDATIONS

Recommendations given herein are based on economic feasibility and ease of construction. However, other ground stabilization methods and foundation systems are available and may be considered, if desired. Any techniques other than those specified herein, if considered, should be reviewed by the project geotechnical engineer and design consultants to assure conformance with the indicated site geotechnical conditions. Additional or amended recommendations may also be necessary and should be provided at the time of geotechnical plan review phase, as necessary:

A. Grading and Earthworks

Cut-fill and remedial grading techniques may be used in order to achieve final design grades and construct a safe and stable level surface for the support of the planned new structures and improvements. All excavations, grading, earthwork, construction, and bearing soil preparation should be completed in accordance with Chapter 18 (Soils and Foundations) and Appendix "J" (Grading) of the 2013 California Building Code (CBC), the Standard Specifications for Public Works Construction, City of Oceanside Grading Ordinances, the requirements of the governing agencies and following sections, wherever appropriate and as applicable:

1. Underground and Utility Mark-Up

All existing underground waterlines, sewer lines, storm drains, utilities, tanks, structures and improvements at or nearby the project construction site should be thoroughly potholed, identified and marked prior to the initiation of actual ground stabilization work, excavations, remedial grading operations, trenching, and earthwork. 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 grading and earthwork operations, and reinstalled upon completion of the constructions. 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/backfills, 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. All wells, if present, should be destroyed in conformance with the County of San Diego requirements

2. Site Preparation and Clearing

All existing structures, surface and subsurface improvements, asphalt, concrete, vegetation, trees, roots, stumps, construction debris, and all other unsuitable materials and deleterious matter should be removed from all areas of proposed new fills, improvements and structures as approved in the field.

Construction debris generated from the removals and demolition of the site existing structures, improvements, pavings, and abandoned underground facilities should also 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 ground should be inspected and approved by the project geotechnical consultant or his designated field representative prior to grading and earthwork.

3. Remedial Grading and Ground Stabilization - Alluvial Areas

Westerly and southerly areas of the site are underlain by a thick section of alluvial soils which vary in characteristic and in-situ conditions. Alluvial deposits range to more than 50 feet in thickness and generally occur in loose to soft and dry conditions near the surface become more uniformly consolidated with depth.

Special ground stabilization and remedial grading techniques will be required in order to mitigate soil collapse & liquefaction potential and construct safe and stable building pad surfaces as specified below. Actual over-excavation depths should be confirmed and approved by the project geotechnical engineer at the time of remedial grading operations. Deeper over-excavations may be necessary based on the actual field exposures and should be anticipated. Bottom of over-excavations exposing soft and yielding soils which may also require deeper excavations or placement of stabilization geogrid, as directed in the field.

a. **Over-Excavation and Recompaction:** Site existing upper loose to soft alluvial soils underneath all areas planned for new fills, embankments, structures, and improvements, plus a minimum horizontal distance of 10 feet outside the perimeter, where possible and as directed in the field, should be over-excavated to a minimum depth of 15 feet below design rough pad grades, or 15 feet below existing ground surfaces, whichever is more. There should be at least 15 feet of well-compacted fills below design rough pad grades. Should sandstone Terrace Deposits be encountered during removals, refer to Section 4 (Remedial Grading - Terrace Deposits).

   The over-excavated materials should then be properly processed and placed back as compacted fills in accordance with the requirements of this report. New fills should be compacted to minimum 95% of the corresponding maximum dry density (ASTM D-1557), unless otherwise approved.

b. **Stabilization of Bottom of Over-Excavations:** Bottom of all over-excavations should be stabilized by in-place moisture conditioning and recompaction to at least 90% compaction levels (ASTM D-1557) to a minimum depth of 12-inches, prior to fill placement. In the event minimum 90% compaction levels could not be achieved within the soft and yielding bottom exposures as specified herein, a layer of Tensar BX-1200 stabilization geogrid (or approved equal) should be neatly placed at the entire prepared bottom of over-excavations as directed in the field. Initial fill lifts can then be carefully placed over the geogrid and compacted as specified. Additional layers of geogrid may be required at 3-foot increments should yielding conditions continue, as determined by the project geotechnical consultant.

   Field conditions will control actual bottom of over-excavation stabilization procedures. Specific recommendations should be given by the project geotechnical engineer at the time of bottom of over-excavation inspections.

c. **Temporary Construction Slopes:** Top of temporary slopes should maintain adequate set back from existing on and offsite improvements and structures as approved and directed in the field. Undermining and/or damages to existing improvements, structures, underground utilities and within public right-of-way or adjacent easements and properties should be avoided. Face of temporary slopes should be protected from excessive runoff or rainfall and stockpiling the excavated materials near the top of construction embankments should be disallowed. Constructions should also be completed in a timely manner minimizing unsupported slope conditions for prolonged periods of time.

Temporary slopes and trenching excavations development within the site existing alluvial soils above the water table should be laid back at 1:1 gradients maximum unless otherwise directed or approved. Completing excavations in limited sections and considering proper staging and stockpiling areas may also be necessary. The new fills should then be properly benched and tightly keyed into the temporary slope as the backfill placement progresses and as directed in the field by the project geotechnical consultant. Revised temporary construction slope and trenching recommendations including flatter slope gradients, larger setbacks, and the need for temporary shoring/trench shield support may be necessary and should be anticipated. The project contractor shall also obtain appropriate permits, as needed, and conform to the CAL-OSHA and local governing agencies requirements for trenching/open excavations and safety of the workmen during construction.

d. **Dewatering:** Groundwater levels underneath the project site are established well below the specified over-excavation and removal depths. However, some water intrusion or local seeps may develop in the site excavations depending on seasonal conditions. Consequently, minor to local dewatering efforts may be expected. Any dewatering technique which can effectively remove the intruding water and allowing earthworks and constructions to proceed such as gravel-filled trench sumps with submersible pumps may be considered. If dewatering becomes necessary, a qualified contractor should be consulted in this regard. Completing site remedial grading and earthwork during the dry seasons of the year should be considered.

e. **Fill and Backfill Materials, Shrinkage and Import Soils:** Soils generated from the site over-excavations may be reused as new fills provided they are adequately processed and manufactured into a clean uniform mixture free of vegetation, organic matter, trash debris and unsuitable materials as approved in the field. Locally very moist to wet soils may be encountered from the deeper over-excavations requiring additional spreading, drying and processing work.

Site clayey soils, if encountered, should be buried in deeper fills a minimum of 5 feet below finish grades, and more sandy soils available from the onsite over-excavations placed within upper pad grades.

Based upon our analyses and experience with similar earth deposits, site soils may also be expected to shrink approximately 10% to 20% on a volume basis when compacted as specified herein.

f. **Fill and Backfill Processing, Placement, & Compaction:** Uniform and stable fill support should be constructed underneath the site alluvial areas by the ground stabilization, remedial grading, and earthwork operations. For this purpose, site soils should be adequately processed, thoroughly mixed, moisture conditioned to slightly (3% or as directed in the field) above the optimum moisture levels, placed in thin (6 inches maximum) uniform horizontal lifts and mechanically compacted to a minimum of 95% compaction levels per ASTM D-1557.

g. **Instrumentations and Monitoring:** Geotechnical instrumentation devices consisting of settlement plates and settlement monuments should be installed at the project site. The settlement plates should be placed at the bottom of the over-excavations to monitor settlement of the underlying surcharged natural alluvium. The settlement monuments should be installed near the rough finish pad grades to monitor the post grading characteristics of the compacted fill mass. Typical settlement plate and monument schematics are included as Plates 15 and 16 respectively. Geotechnical instrumentation sites should be installed at selected locations not to interfere with the grading and post grading construction phases.

Monitoring should be performed by means of field surveying shots periodically taken at each monitoring site as the fill and backfill placement progresses approximately once every 2 days. At the completion of remedial grading, monitoring should continue for both the settlement plate and settlement monument sites on a bi-weekly and/or monthly basis, per the monitoring schedule developed by the project geotechnical consultant. Surveying shots should be reduced (plotted versus time in days) by the project geotechnical consultant, to establish settlement patterns and soil compression characteristics with respect to surcharge loading pressures, compaction efforts and earthworks activities.

Actual locations for the proposed settlement plates and settlement monuments should be given by the project geotechnical engineer when detailed grading and development plans are available. Geotechnical instrumentations should be installed by or under direct supervision of the project geotechnical consultant and monitoring carried out by surveying methods provided by the project civil engineer or surveyor. Survey records of the instrumentations (vertical and horizontal positioning) should then be given to the project geotechnical consultant for interpretation.

Utility and foundation trenching can only begin after completion of primary soil compression and approval of the project geotechnical consultant (less than 0.01-foot or 0.12 inches between at least three consecutive post-grading readings per the monitoring schedule, unless otherwise noted or required by the project geotechnical consultant). Foundation and slab recommendations provided in the following sections should also be confirmed and / or revised based upon the settlement monitoring data compiled at the completion of monitoring period.

4. Remedial Grading - Terrace Deposits

Northeasterly and easterly areas of the site are underlain at shallow depths by competent sandstone Terrace Deposits, mantled by sandy surficial soils.

a. **Removals:** All site existing surficial soils (fill / topsoil) and upper weathered Terrace Deposits in all areas planned for new fills, embankments, structures, and improvements plus a minimum of 10 horizontal feet outside the perimeter, where possible and as directed in the field, should be stripped (removed) to the depth of the underlying dense and competent Terrace Deposits and placed back as properly compacted fills. All existing fills, where encountered, should also be removed extending to the underlying competent bedrock and recompacted as specified herein.

   Removal depths will vary. Actual depths should be established by the project geotechnical engineer or his designated field representative in the field at the time of remedial grading operations. Deeper removals and over-excavations may also be required as established in the field and should be anticipated.

b. **Cut - Fill Transitions and Undercuts:** Ground transition from excavated cut to compacted fills should not be permitted underneath future proposed structures and improvements. Building and structural foundations as well as on-grade improvements should be uniformly founded on undisturbed competent Terrace Deposits or supported entirely on compacted fills. Transition pads will require special treatment. The cut portion of the cut-fill pad plus 10 horizontal feet outside the perimeter, where possible and as directed in the field, should be undercut to a sufficient depth to provide for a minimum 4 feet of a compacted fill mat below rough finish grade, 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 12-inches of compacted soils below rough finish subgrade.

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

c. **Trenching and Temporary Construction Slopes:** Project excavations, trenching, and construction slopes are mostly expected to expose shallow surficial soils atop competent sandstone. Project excavations, trenching, and construction slopes exposing competent sandstone may be developed at near vertical gradients to 5 feet high maximum, unless otherwise specified or directed in the field. Temporary excavation slopes greater than 5 feet developed into site Terrace Deposits may be constructed near vertical gradients within the lower 5 feet and laid back at 1:1 gradients within the upper sections, unless otherwise noted.

Elsewhere, construction slopes and trenches excavated with the site existing surficial soils less than 3 feet in maximum height, may be constructed at near vertical gradients, unless otherwise approved or directed in the field. Trench and construction slopes greater than 3 feet high maximum developed within these deposits may be constructed at near vertical gradients in the lower 3 feet and laid back at 1:1 in the upper portions, as approved 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 geotechnical inspections during the excavation operation.

Specific recommendations should be given in the field by the project geotechnical consultant based on actual exposures. Revised temporary construction slope and trenching recommendations including flatter slope gradients, larger setbacks and the need for temporary shoring/trench shield support may be necessary and should be anticipated. 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.

d. **Fill-Backfill Materials and Compaction:** Soils generated from excavations of site surficial soils and weathered Terrace Deposits, will predominantly consist of good quality sandy material which will work well as new site fills. Excavations may also locally encounter some clayey soils which are expected to be minor in overall quantities. Minor clayey soils, if encountered, should be selectively buried in deeper fills at least 4 feet below rough finish pad grades.

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. Trench and wall backfills shall consist of a minimum of 3-inch particles and maintain the minimum specified fines to rock ratio. Rocks larger than 6-inches in maximum diameter should not be allowed within site fills.

Uniform bearing soils conditions should be constructed at the site Terrace Deposit locations by the grading operations. Site soils should be adequately processed, thoroughly mixed, moisture conditioned to slightly (2%) 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 laboratory maximum dry density per ASTM D-1557, unless otherwise specified.

5. Import Soils

Import soils, if required to complete remedial grading and achieve final design grades, should be good-quality, non-corrosive sandy granular (D.G.) deposits (100% passing 1-inch sieve, more than 50% passing #4 sieve and less than 18% passing #200 sieve with expansion index less than 21) tested and approved by the project soils engineer prior to delivery to the site. Import soils should also meet or exceed the engineering properties of site soils as specified in the following sections.

6. Engineering Observations and Testing

All ground stabilization work, grading, and earthwork operations including over-excavations, 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 daily field and final as-constructed reports. The recommended construction procedures and specifications should be field verified or modified as necessary at that time. 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 clearing limits have been staked but before grading/brushing starts.

- Over-excavation observation - After excavations are started but before the vertical depths are more than 5 feet. Local and Cal-OSHA safety requirements for open excavations apply.

- Bottom of over-excavation observation - After the bottom of over-excavation is exposed and prepared to receive new fills or the stabilization geogrid, but before fill or geogrid is placed.

- 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 maximum. Fills should be compacted to minimum specified (90% and 95%) compaction levels, 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 excavation, but before steel placement.

- Foundation bearing/slab subgrade soil 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 excavation, but before placement of pipe bedding or installation of the underground facilities. Local and Cal-OSHA safety requirements for open excavations apply. Inspection 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 good quality sand materials, as approved in the field and mechanically compacted to the specified minimum compaction levels. Plumbing trenches more than 12-inches deep maximum under the floor slabs should also be mechanically compacted and tested for minimum (95% or 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 Floor Slabs

Project pad construction may be anticipated to consist of silty sand (SM) deposits with very low expansion potential (expansion index less than 21) within upper pad grades.

The following minimum recommendations are consistent with the anticipated foundation bearing soil material and site specific geotechnical conditions. Other foundation support systems are also available and may be considered, if desired. However, any foundation system other than those specified herein, if considered, should be reviewed by the project geotechnical engineer to assure conformance with the indicated site geotechnical conditions. Additional recommendations may also be required and should be given at the final plan review phase. All design recommendations should also be further confirmed and/or revised at the completion of ground stabilization and remedial grading based on the engineering characteristics of the foundation bearing soils and as-graded site geotechnical conditions, and presented in the final stabilization and compaction report. Foundation trenching within site areas exposing alluvium after minimum over-excavations can only begin after data reduction of monitoring records collected during and after remedial grading works and approval by the project geotechnical consultant.

1. Alluvium Areas: New buildings may be supported on shallow stiff stem wall or turned-down footings and spread pad foundations with interconnecting grade beams and slab-on-grade floors. Building foundations should be uniformly embedded into approved minimum 95% compacted fills as specified in this report.

    - Continuous stem wall foundations, and turned-down footings should be sized at least 18 inches wide and 24 inches deep for one and two-story structures. Spread pad footings should be at least 36 inches square and 18 inches deep and interconnected to the continuous foundations with grade beams. Grade beams should be at least 12 inches wide by 18 inches deep. Specified depths are measured from the lowest adjacent ground surface. Exterior continuous foundations or turned-down footings should enclose the entire building perimeter.

    Continuous interior and exterior stem wall foundations should be reinforced with a minimum of four #5 reinforcing bars. Place 2-#5 bars 3 inches above the bottom of the footings and 2-#5 bars 3 inches below the top of the stem wall. Turned-down footings should be reinforced with a minimum of 2-#5 bars at the top and 2-#5 bars at the bottom. Interconnecting grade beams should also be reinforced with a minimum of 2-#4 bars top and bottom. Reinforcement details for spread pad footings should be provided by the project architect/structural engineer.

- All interior slabs should be a minimum of 5 inches in thickness, reinforced with #4 reinforcing bars spaced 18 inches on center each way, placed near the slab mid-height. Slabs should be underlain by 4 inches of clean sand (SE 30 or greater) which is provided with a minimum 10-mil plastic moisture barrier 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 1¼ -inches in depth but should not exceed 1½ -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 17 may be used as a general guideline.

- The slab subgrade and foundation bearing soils should not be allowed to dry prior to pouring the concrete or additional ground preparations, moisture re-conditioning and recompaction will be necessary as directed in the field. The required moisture content of the bearing soils is approximately 3% (or as directed in the field) over the optimum moisture content to the depth of 24 inches below slab subgrade. Attempts should be made to maintain as-graded moisture contents in order to preclude the need for added ground preparations and moisture reconditioning of the subgrade and bearing soils.

- 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.

2. Terrace Deposit Areas: New buildings may be supported on shallow stiff stem wall or turned-down footings and spread pad foundations with interconnecting grade beams and slab-on-grade floors. Building foundations should be

uniformly embedded into approved minimum 90% compacted fills as specified in this report.

- Continuous stem wall foundations, and turned-down footings should be sized at least 15 inches wide and 18 inches deep for single-story structures, and 18 inches wide and 24 inches deep for two-story structures. Spread pad footings should be at least 24 inches square and 18 inches deep. Specified depths are measured from the lowest adjacent ground surface. Exterior continuous foundations or turned-down 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.

- 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 near the slab mid-height. Slabs should be underlain by 4 inches of clean sand (SE 30 or greater) which is provided with a minimum 10-mil plastic moisture barrier 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 1¼ -inches in depth but should not exceed 1½ -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 17 may be used as a general guideline.

- The slab subgrade and foundation bearing soils should not be allowed to dry prior to pouring the concrete or additional ground preparations, moisture re-conditioning and recompaction will be necessary as directed in the field. The required moisture content of the bearing soils is approximately 3% (or as directed in the field) over the optimum moisture content to the depth of 24 inches below slab subgrade. Attempts should be made to maintain as-graded moisture contents in order to preclude the need for added ground preparations and moisture reconditioning of the subgrade and bearing soils.

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

1. Design wet unit weight = 125 pcf.
2. Design angle of internal friction = 31 degrees.
3. Design active soil pressure = 41 pcf (EFP), level backfill, cantilever, unrestrained walls.
4. Design at-rest soil pressure = 61 pcf (EFP), non-yielding, restrained walls.
5. Design passive resistance = 391 pcf (EFP), level surface at the toe.
6. Design coefficient of friction for concrete on soils = 0.40.
7. Design net allowable foundation pressure (minimum 18 inches wide footing embedded at least 24 inches into 95% compacted fill) = 1750 psf.
8. Design net allowable foundation pressure (minimum 15 inches wide footing embedded at least 18 inches into 90% compacted fill) = 1500 psf.
9. Allowable lateral bearing pressure = 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 was determined for footings having the indicated minimum widths and minimum depths for 95% compacted fill and 90% compacted fill. The indicated values may be increased by 20% for each additional foot of depth and each additional foot of width to a maximum of 5500 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 the designated value for each additional foot of depth to a maximum of 1500 pounds per square foot.

D. Exterior Concrete Slabs / Flatworks

1. All exterior slabs (walkways, patios) supported on very low expansive subgrade soils should be a minimum of 4 inches in thickness, reinforced with #3 bars at 16 inches on centers in both directions placed near the slab mid-height. The subgrade soils should be compacted to the minimum specified compaction levels at the time of fine grading and before placing the slab reinforcement.

   In order to enhance performance of exterior slabs and flatwork, a minimum 8 inches wide by 8 inches deep thickened edge reinforced with a minimum of 1-#4 continuous bar near the bottom should be considered along the slab perimeter. Tying the slab panels to adjacent curbs, where they occur, with #3 bars at 16 inches on centers, may also be considered.

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 inches long #3 dowels (dowel baskets) at 16 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 1¼ -inches in depth but should not exceed 1½ -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.).

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

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.

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

   - A minimum section of 3 inches asphalt on 4 inches Class 2 aggregate base or the minimum structural section required by the City of Oceanside, whichever is more, may be considered for the on-site asphalt paving surfaces outside the private and public right-of-way.

   - The Class 2 aggregate base shall meet or exceed the current Green Book Standard Specifications for Public Works Construction and Regional Supplement Amendments, 2003, Sections 400-2.3. 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 95% of the corresponding maximum dry density.

   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 maximum placed mid-height in the slab (9 inches on either side of the joint). In the areas where longitudinal grades exceed 15%, also provide a minimum 8 inches wide by 8 inches deep pavement anchors constructed 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 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 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.)

3. 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.

4. Base section and subgrade preparation 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 95% compaction levels, and provided with 95% compacted Class 2 base section per the 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 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 are critical factors 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. 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-functioning back drainage system as shown on the enclosed Plate 18. Planting large trees behind site building/basement retaining walls should be avoided.

7. All underground utility and plumbing trenches should be mechanically compacted to a minimum of 95% (or 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. Maintaining a uniform as-graded soil moisture during the post construction periods is essential in the future performance and stability of site structures and improvements. Excessive irrigation resulting in wet soil conditions should be avoided. Hydro modification design and location of associated drainage improvements should be completed considering characteristics of onsite soils. Surface water should not be allowed to infiltrate into the underlying bearing and subgrade soils, wall backfills, or impact graded embankments.

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.

10. Final plans should reflect preliminary recommendations given in this report. Final foundations and grading plans should 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 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 aggregate 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.

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.

XI. GEOTECHNICAL ENGINEER OF RECORD (GER)

Vinje & Middleton Engineering, Inc. will be 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, grading engineering observations, field inspections, 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 Oceanside for the assumption of responsibility in accordance with the applicable codes and standards (1997 UBC Section 3317.8).

XII. LIMITATIONS

The conclusions and recommendations provided herein have been based on available data obtained from the review of available reports and maps, subsurface exploratory excavations, engineering analysis, 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 by 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 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. The project geotechnical engineer should also be provided the opportunity to verify the foundations prior to the placing of concrete. 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 #15-188-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

RALPH MALCOLM VINJE
No. 863
Exp. 12-31-17

Ralph M. Vinje
GE #863

STEVEN JAY MELZER
No. 2362
Exp. 5-31-17

Steven J. Melzer
CEG #2362

REFERENCES

- Annual Book of ASTM Standards, Section 4 - Construction, Volume 04.08: Soil and Rock (I); D 420 - D 5611, 2005.
- Annual Book of ASTM Standards, Section 4 - Construction, Volume 04.09: Soil and Rock (II); D 5714 - Latest, 2005.
- Corrosion Guidelines, Caltrans, Version 1.0, September 2003.
- “Green Book” Standard Specifications For Public Works Construction, Public Works Standards, Inc., BNi Building News, 2003 Edition.
- California Department of Conservation, Division of Mines and Geology (California Geological Survey), 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, DMG Special Publication 117, 71p.
- California Department of Conservation, Division of Mines and Geology (California Geological Survey), 1986 (revised), Guidelines for Preparing Engineering Geology Reports: DMG Note 44.
- California Department of Conservation, Division of Mines and Geology (California Geological Survey), 1986 (revised), Guidelines to Geologic and Seismic Reports: DMG Note 42.
- EQFAULT, Ver. 3.00, 1997, Deterministic Estimation of Peak Acceleration from Digitized Faults, Computer Program, T. Blake Computer Services and Software.
- EQSEARCH, Ver 3.00, 1997, Estimation of Peak Acceleration from California Earthquake Catalogs, Computer Program, T. Blake Computer Services and Software.
- Tan S.S. and Kennedy, M.P., 1996, Geologic Maps of the Northwestern Part of San Diego County, California, Plate(s) 1 and 2, Open File-Report 96-02, California Division of Mines and Geology, 1:24,000.
- “Proceeding of The NCEER Workshop on Evaluation of Liquefaction Resistance Soils,” Edited by T. Leslie Youd and Izzat M. Idriss, Technical Report NCEER-97-0022, Dated December 31, 1997.
- “Recommended Procedures For Implementation of DMG Special Publication 117 Guidelines For Analyzing and Mitigation Liquefaction In California,” Southern California Earthquake center; USC, March 1999.
- “Soil Mechanics,” Naval Facilities Engineering Command, DM 7.01.
- “Introduction to Geotechnical Engineering, Robert D. Holtz, William D. Kovacs.
- “Introductory Soil Mechanics and Foundations: Geotechnical Engineering,” George F. Sowers, Fourth Edition.
- “Foundation Analysis and Design,” Joseph E. Bowels.
- Caterpillar Performance Handbook, Edition 29, 1998.
- Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology, Geologic Data Map Series, No. 6.
- Kennedy, M.P., 1977, Recency and Character of Faulting Along the Elsinore Fault Zone in Southern Riverside County, California, Special Report 131, California Division of Mines and Geology, Plate 1 (East/West), 12p.
- “Procedure To Evaluate Earthquake-Induced Settlements In Dry Sandy Soils,” Daniel Pradel, ASCE Journal Of Geotechnical & Geoenvironmental Engineering, Volume 124, #4, 1998.
- "Minimum Design Loads For Buildings and Other Structures," ASCE 7-05, American Society of Civil Engineers.

AS-BUILT SURVEY

EGEND

	BUILDING LINE	GA	GUY ANCHOR
	PROPERTY LINE	WV	WATER VALVE
	CENTERLINE	FSP	FIRE STAND PIPE
	LOT LINE	SLPB	STREET LIGHT PULL-BOX
	AERIAL ELECTRIC LINE	TMH	TELEPHONE MANHOLE
	ASPHALT	SMH	SEWER MANHOLE
	FENCE LINE	SDMH	STORMDRAIN MANHOLE
	WALL LINE	GV	GAS VALVE
	CONCRETE AREA	SCO	SEWER CLEAN OUT
	LIGHT	TC	TOP OF CURB ELEVATION
	POWER POLE	FL	FLOW LINE ELEVATION
	WATER METER	▲	FOUND CENTERLINE WELL MONUMENT
L/S	LANDSCAPED AREA	◉	FOUND 2" IRON PIPE
BSL	BUILDING SETBACK LINE		
GP	GUY POLE		

VICINITY MAP

NO SCALE

GENERAL NOTES

-ALL ADJACENT LOTS ARE CONTIGUOUS TO SAID LAND WITH NO INTERVENING GAPS OR GORES.
-SAID LAND IS CONTIGUOUS WITH AND HAS DIRECT ACCESS TO NORTH RIVER ROAD AND THERE ARE NO INTERVENING GAPS OR GORES.
-THE POINT OF ENTRY OR EXIT OF ALL UTILITIES AND SANITARY AND STORM SEWERS ARE THROUGH PUBLICLY DEDICATED STREETS UNLESS OTHERWISE SHOWN HEREON.

BENCHMARK

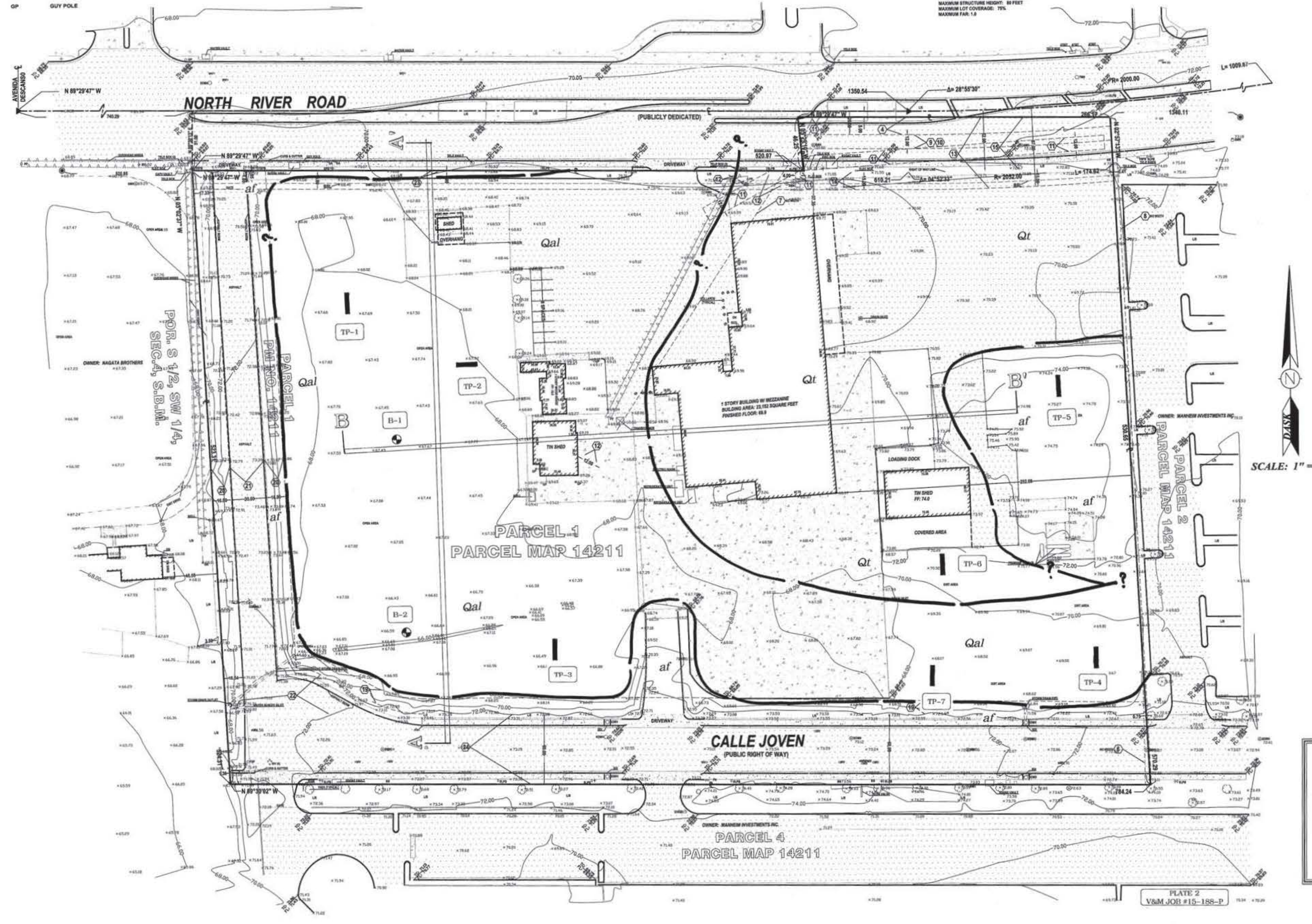
-CITY OF OCEANSIDE BENCHMARK #A-38, FOUND BRASS SCREW & WASHER STAMPED "COS BM A-38", TOP OF CURB, NORTHEAST RETURN OF NORTH RIVER ROAD & AVENIDA DESCANSO. EL= 68.847' (1983 ADJUSTED).

SITE ADDRESS: 4665 NORTH RIVER ROAD, OCEANSIDE, CA
PROPERTY AREA: 9.706 ACRES (422,793 SQUARE FEET)
ZONE: " IR " (INDUSTRIAL)
TOTAL PARKING: 8 SPACES (INCLUDING 0 HANDICAP SPACES)
FLOOD ZONE: " X " MAP NO.: 060 73C 0756, DATED: 01/19/2001
"SAID PROPERTY DOES NOT LIE WITHIN A FEMA DESIGNATED 100-YR FLOOD PLAIN"

GEOTECHNICAL MAP

ZONING NOTES

MINIMUM FRONT BUILDING SETBACK: 10 FEET
MINIMUM SIDE YARD BUILDING SETBACK: NONE
MINIMUM CORNER SIDE BUILDING SETBACK: 10 FEET
MINIMUM REAR BUILDING SETBACK: NONE
MAXIMUM STRUCTURE HEIGHT: 60 FEET
MAXIMUM LOT COVERAGE: 75%
MAXIMUM FAR: 1.0



SCALE: 1" = 40'

PLATE 2
V&M JOB #15-188-P

LEGAL DESCRIPTION

Real property in the City of Oceanside, County of San Diego, State of California, described as follows:

PARCEL 1 OF PARCEL MAP NO. 14211, IN THE CITY OF OCEANSIDE, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, RECORDED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, FEBRUARY 27, 1986.

APN: 157-090-48-00

EASEMENT NOTES

PER: First American Title Insurance Company
ORDER NO.: 3319614 (50)
DATED: June 09, 2009

③ An easement for open ditch, flume line and incidental purposes, recorded November 22, 1909 in Book 481 of Deeds, Page 41.
In Favor of: Hayes Land Co.
Affects: The land
The location of the easement cannot be determined from record information.
"SAID ITEM CANNOT BE PLOTTED"

④ An easement shown or dedicated on the Map as referred to in the legal description
For: Public road purposes and incidental purposes.

⑤ An easement for right of way for pipelines, motor, pump and incidental purposes, recorded June 30, 1943 in Book 1516, Page 62 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation of the Sixth Class
Affects: The route or location of said easement cannot be determined from the record
The location of the easement cannot be determined from record information.
"SAID ITEM CANNOT BE PLOTTED"

⑥ An easement for either or both pole lines, underground conduits and incidental purposes, recorded July 25, 1944 in Book 2180, Page 226 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land

⑦ An easement for either or both pole lines, underground conduits and incidental purposes, recorded May 16, 1949 in Book 3189, Page 204 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land

⑧ An easement for either or both pole lines, underground conduits and incidental purposes, recorded November 20, 1952 in Book 4661, Page 1 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land
"CENTERLINE OF SAID ITEM IS NOT WITHIN SAID PROPERTY"

⑨ An easement for right to construct, reconstruct, repair, replace, operate, maintain and use sanitary sewer and appurtenances and incidental purposes, recorded June 11, 1954 as Instrument No. 105495 of Official Records.
In Favor of: City of Oceanside
Affects: Said land

⑩ An easement for either or both pole lines, underground conduits and incidental purposes, recorded February 14, 1959 as Instrument No. 27637 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land

⑪ An easement for either or both pole lines, underground conduits and incidental purposes, recorded February 21, 1958 as Instrument No. 31879 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land

⑫ An easement for either or both pole lines, underground conduits and incidental purposes, recorded April 8, 1975 as Instrument No. 55953 of Official Records.
In Favor of: San Diego Gas and Electric Company
Affects: Said land

⑬ An easement for sewer, effluent water main and incidental purposes, recorded February 19, 1974 as Instrument No. 040519 of Official Records.
In Favor of: City of Oceanside
Affects: Said land

⑭ An easement for sewer, effluent water main and incidental purposes, recorded February 19, 1974 as Instrument No. 040520 of Official Records.
In Favor of: City of Oceanside
Affects: Said land

⑮ An easement for public highway, utility and incidental purposes, recorded October 11, 1994 as Instrument No. 94-595625 of Official Records.
In Favor of: City of Oceanside
Affects: Said land

⑯ The terms and provisions contained in the document entitled Public Improvements executed by and between SCHU Properties, a General Partnership and City of Oceanside recorded November 14, 1985 as Instrument No. 85-420345 of Official Records.
"SAID ITEM CANNOT BE PLOTTED"

⑰ The terms and provisions contained in the document entitled Street Improvements executed by and between Raymond Kawano, et al and City of Oceanside recorded January 13, 1986 as Instrument No. 86-014211 of Official Records.
"SAID ITEM CANNOT BE PLOTTED"

⑱ The Parcel Map referred to in the Legal Description herein contains various restrictions in improving or developing the property herein described. reference is made to said Parcel Map for further particulars.
"SAID ITEM CANNOT BE PLOTTED"

⑲ An easement for slopes and incidental purposes, recorded June 19, 1992 as Instrument No. 92-0381279 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

⑳ An easement for slopes and incidental purposes, recorded June 19, 1992 as Instrument No. 92-0381281 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

㉑ An easement for emergency access and utility and incidental purposes, recorded June 19, 1992 as Instrument No. 92-0381283 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

㉒ An easement for temporary turn around and incidental purposes, recorded June 19, 1992 as Instrument No. 92-0381284 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

㉓ An easement for public right of way purposes and incidental purposes, recorded June 19, 1992 as Instrument No. 92-0381286 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

㉔ An easement for public right of way purposes and incidental purposes, recorded June 29, 1992 as Instrument No. 92-0404303 of Official Records.
In Favor of: City of Oceanside, a Municipal Corporation
Affects: Said land

SURVEYORS CERTIFICATE

THIS IS TO CERTIFY THAT SAID MAP IS TRUE AND CORRECT AS SHOWN. THE POSITIONAL UNCERTAINTIES RESULTING FROM THE SURVEY MEASUREMENTS MADE ON THE SURVEY DO NOT EXCEED THE ALLOWABLE POSITIONAL TOLERANCES FOR AN URBAN SURVEY.

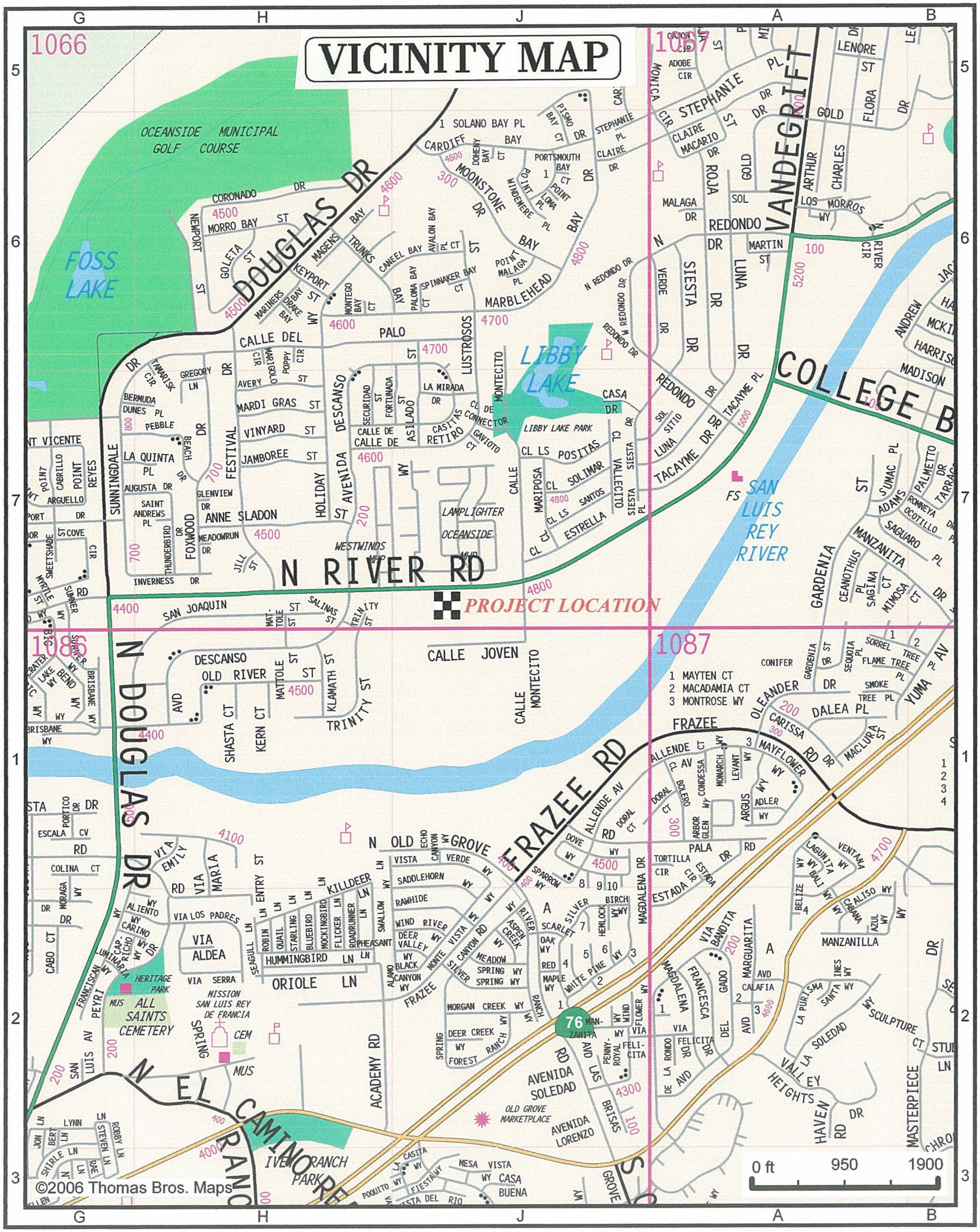
SIGNED: SOLOMON S. HANDY
REGISTRATION NO.: LS -7584
DATE: 6/24/09

GEOTECHNICAL LEGEND

▬	Approx. Location of Test Pit
◕	Approx. Location of Test Boring
⊔	Geologic Cross-Section
af	Fill (Approx.)
Qal	Alluvium (Approx.)
Qt	Terrace Deposit (Approx.)

REVISED: AUG. 26, 2009
JOB NO. 32-021-09

DASK
LAND SURVEYI
DASK LAND SURVEYINC
960 RANCHEROS DRIVE, SU
SAN MARCOS, CA 9206
PHONE: 760-740-1383
FAX: 760-740-1384



PROJECT LOCATION: 1066 - J7

PLATE 1
V&M JOB #15-188-P

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Claye sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS & CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LIQUID LIMIT IS MORE THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic soils
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat or other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE

CLEAR SQAURE SIEVE OPENINGS

200 40 10 4 ¾" 3" 12"

SILTS & CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS, GRAVELS & NON-PLASTIC SILTS	BLOWS / FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS & PLASTIC SILTS	STRENGTH	BLOWS / FOOT
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

1. BLOW COUNT: 140 POUND HAMMER FALLING 30-INCHES ON A 2-INCH DIAMETER O.D. SPLIT SPOON SAMPLER (ASTM D-1586)
2. UNCONFINED COMPRESSIVE STRENGTH PER SOILTEST POCKET PENETROMETER CL-700

▼ Sand Cone Test ■ Bulk Sample ☒ 2 4 6 Standard Penetration Test (SPT) - (ASTM D-1586) With Blow Counts Per 6-Inches

☐ Chunk Sample ○ Driven Rings ☒ 2 4 6 California Sampler With Blow Counts Per 6-Inches

VINJE & MIDDLETON ENGINEERING, INC.

2450 Auto Park Way
Escondido, California 92029

KEY TO BORING / TEST PITS LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

December, 2013

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-1**

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. Caving below 8 feet.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
1 2			**Alluvium (Qal):** Silty fine sand. Micaceous. Light grey color. Dry. Somewhat blocky. Loose. ST-1	■ ☐	3	104.9	83	13
3			Blocky and medium dense below at 3 feet.					
4				☐	3	97.5	77	11
5 6				☐	3	96.6	76	11
7 8		SM	Becomes loose at 8 feet. Slightly blocky. Continues dry. Local running sand. Caving below 8 feet.	☐	3	105.2	83	13
9 10			Damp at 10 feet. Continued caving (running sand).	☐	9	94.1	74	30
11 12				☐	7	92.8	73	23
13 14				☐	11	85.5	67	31
15			Extent of backhoe.					

Bottom of test pit at 15.0 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-2**

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. Caving below 8 feet.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
1			**Alluvium (Qal):**					
2			Silty fine sand. Micaceous. Light grey color. Dry. Somewhat blocky. Loose to medium dense. ST-1					
3								
				☐	3	104.8	82	13
4								
5								
6								
				☐	3	102.6	81	13
7								
		SM						
8			Damp at 8 feet. Locally blocky. Loose.					
9								
10			Damp to moist at 10 feet. Local running sand. Caving. Loose to very loose.	☐	6	102.5	81	25
11								
12								
13								
14								
				☐	4	99.7	78	16
15			Extent of backhoe.					

Bottom of test pit at 15.0 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

PLATE 4

VINJE & MIDDLETON ENGINEERING, INC. TEST PIT: TP-3

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. Significant caving.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
1			**Alluvium (Qal):**					
2			Silty fine sand. Micaceous. Dry. Blocky. Loose to medium dense. ST-1	☐	2	118.8	94	27
3			Loose to very loose at 3 feet. Significant caving below 3 feet. Local running sand.					
4		SM						
5				☐	12	102.9	81	51
6								
7								

Test pit ended at 7 feet due to sidwall caving.

Bottom of test pit at 7.0 feet.

▨ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-4**

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. Significant caving.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
1		SM-SP	**Fill (af):** Silty fine to medium sand. Brown to tan color. Blocky. Moderately compacted. ST-2					
2				■☐	9	118.2	87	55
3		SM	**Alluvium (Qal):** Silty fine sand. Micaceous. Light grey to brown color. Damp. Slightly blocky. Loose. ST-1 Running sand at 5 feet. Striated. Very loose. Damp. Significant caving.					
4				■☐	3	108.7	86	15
5								
6				☐	8	118.0	93	50
7								
8								
9								
10								
11				☐	2	119.3	94	13
12								

Test pit ended at 12 feet due to sidewall caving.

Bottom of test pit at 12.0 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. TEST PIT: TP-5

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. No caving.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
			Gravel-covered surface.					
1			**Fill (af):**					
2		SM	Silty fine sand. Slightly micaceous. Brown color. Damp. Moderately compacted. ST-1	☐	6	105.6	83	27
3								
4			**Terrace Deposit (Qt):**	■ ☐	7	118.7	88	43
5			Sandstone. Fine to medium grained. Red brown color. Blocky. Moderately cemented. Massive. Dense. ST-2					
6		SP-SW		☐	7	114.0	84	38
7								
8				☐	7	117.2	87	42

Bottom of test pit at 8.5 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-6**

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. No caving.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
1		SM	**Fill (af):** Silty fine sand. Brown color. Dry. Blocky. Medium dense. ST-1					
2				☐	3	108.8	81	14
3		SP-SW	**Terrace Deposit (Qt):** Sandstone. Fine to medium grained. Red brown color. blocky. Cemented. Massive. Dense. ST-2					
4				☐	4	118.4	88	24

Bottom of test pit at 4.5 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-7**

PROJECT: Proposed Residential Development **CLIENT:** So Cal Ag Properties, Inc.

PROJECT NUMBER: 15-188-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

Date Excavated: 8/26/15 **Logged By:** SJM

Equipment: Case 580 Backhoe

Remarks: No groundwater. Significant caving below 3 feet.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
			Gravel-covered surface.					
1 2		SM	**Fill (af):** Silty fine sand. Brown color. Damp. Firm. ST-1					
3 4 5 6		SM	**Alluvium (Qt):** Silty fine sand. Micaceous. Light grey color. Dry to damp. Striated. Very loose. Running sand. Significant caving. ST-1	☐	12	87.7	69	35

Test pit ended at 6 feet due to significant sidewall caving.

Bottom of test pit at 6.0 feet.

■ BULK SAMPLE ☐ CHUNK SAMPLE ▼ DENSITY TEST ∇ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **Boring: B-1**

PROJECT: Proposed Residential Development **CLIENT:** So Ca Ag Properties, Inc.

PROJECT NUMBER: 15-118-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

DATE LOGGED: 12/4/2015 **BOREHOLE DIA:** 8-Inch **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-Mounted Rotary Drill. Hollow Stem Auger.

SAMPLE METHOD: 140 LB. Hammer dropped 30-inches by rope & cathead. 5-Foot AW rods.

REMARKS: No Caving. No Groundwater. Trap Used in Sampler Due to Cohesionless Characteristic of Alluvium.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
			Alluvium (Qal):						
2			Silty fine sand. Slightly micaceous. Grey color. Dry to damp. Very loose. ST-1						
4									
			Loose at 5 feet. Continues dry to damp.	⧗	12-14	3	101.0	79	12
6									
8									
10									
		SM	Damp to locally moist below 10 feet. Loose.	⧗	14-16	3	101.6	80	12
12									
14									
			Medium dense at 15 feet. Continues damp to locally moist.	⧗	11-16	3	-	Sample Disturbed	10
16									
18									
20			Becomes relatively tight at 20 feet. Moist. Medium dense.	⧗	14-13	7	-	Sample Disturbed	28

Bottom of borehole at 21.0 feet.

⧗ STANDARD PENETRATION TEST ⧗ MODIFIED CALIFORNIA SAMPLER ■ BULK SAMPLE ▽ GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **BORING: B-2**

PROJECT: Proposed Residential Development **CLIENT:** So Ca Ag Properties, Inc.

PROJECT #: 15-118-P **PROJECT LOCATION:** 4665 North River Road, Oceanside

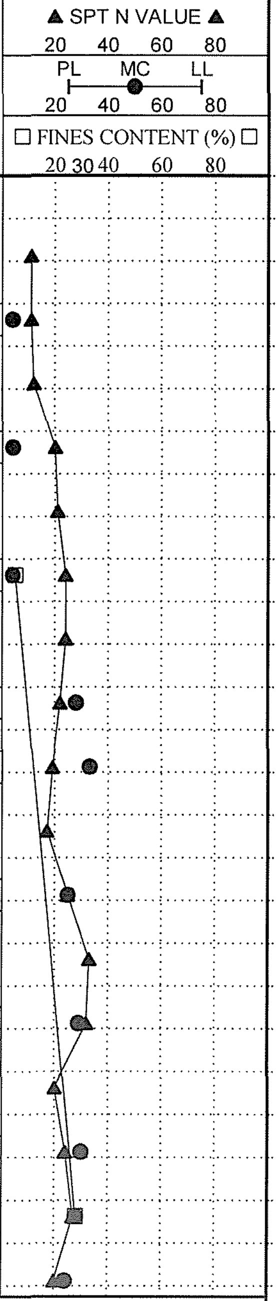
DATE DRILLED: 12/4/2015 **BOREHOLE DIA:** 8-Inch **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-Mounted Rotary Drill. Hollow Stem Auger.

SAMPLE METHOD: 140 LB. Hammer dropped 30-inches by rope & cathead. 5-Foot AW rods.

REMARKS: No Caving. Groundwater at 26-27 Feet. Trap Used in Sampler Due to Cohesionless Character of Alluvium.

DEPTH (ft) 0	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)
			Alluvium (Qal):			
			Silty fine sand. Slightly micaceous. Grey color. Dry to damp. Very loose. ST-1	⊠	4-6-5 (11)	
				⊠	5-5-6 (11)	4
10			Loose and damp at 9 feet. Locally fine to medium grained.	⊠	6-6-6 (12)	
			Becomes medium dense at 12 feet. Damp.	⊠	16-11-9 (20)	4
				⊠	7-9-12 (21)	
20			Two 1" - 2" thick clay seams in sampe at 21 feet. Clay is dark grey color, moist, plastic, and soft. Surrounding silty sand is loose to medium dense and damp to moist.	⊠	11-12-12 (24)	4
			Very moist to wet at 24 feet.	⊠	10-9-15 (24)	
		SM	▼ Static groundwater level measured at 26 feet below the surface prior to proceding with mud-drilling.	⊠	9-11-11 (22)	28
			▽ Groundwater at 27 feet at time of drilling. Saturated. Loose to medium dense.	⊠	6-10-9 (19)	33
30			Changed to mud-drilling method at 30 feet.	⊠	4-7-10 (17)	
			Locally clay-bearing sandy deposits below 30 feet. Saturated. Loose to medium dense.	⊠	4-11-14 (25)	25
			Becomes relatively tight and dense at 36 feet. Fine grained. Saturated.	⊠	10-16-17 (33)	
40				⊠	8-13-19 (32)	29
			Medium dense to locally dense below 40 feet.	⊠	8-8-12 (20)	
				⊠	6-9-15 (24)	30
50				⊠	7-10-17 (27)	28
			Continues medium dense to dense and saturated at 50 feet.	⊠	7-8-12 (20)	24



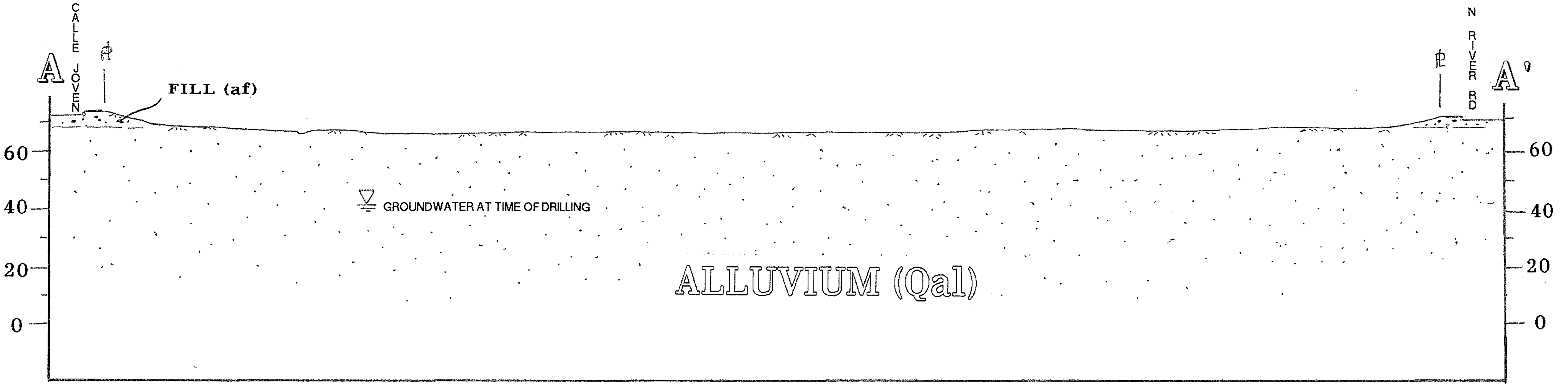
Bottom of borehole at 52.5 feet.

⊠ STANDARD PENETRATION TEST ⊠ MODIFIED CALIFORNIA SAMPLER ■ BULK SAMPLE ▽ GROUND WATER

PLATE 11

GEOLOGIC CROSS-SECTIONS

SCALE: 1" = 40'



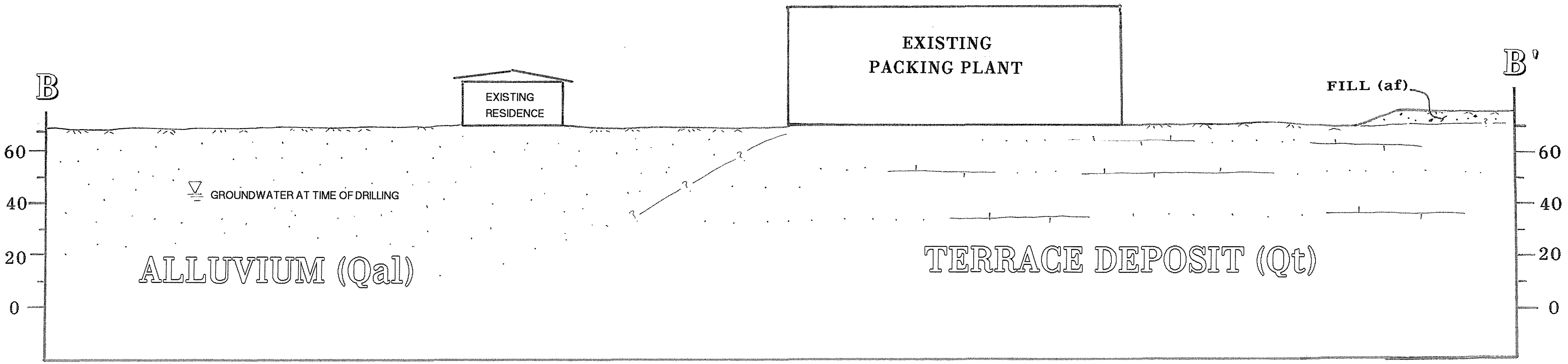
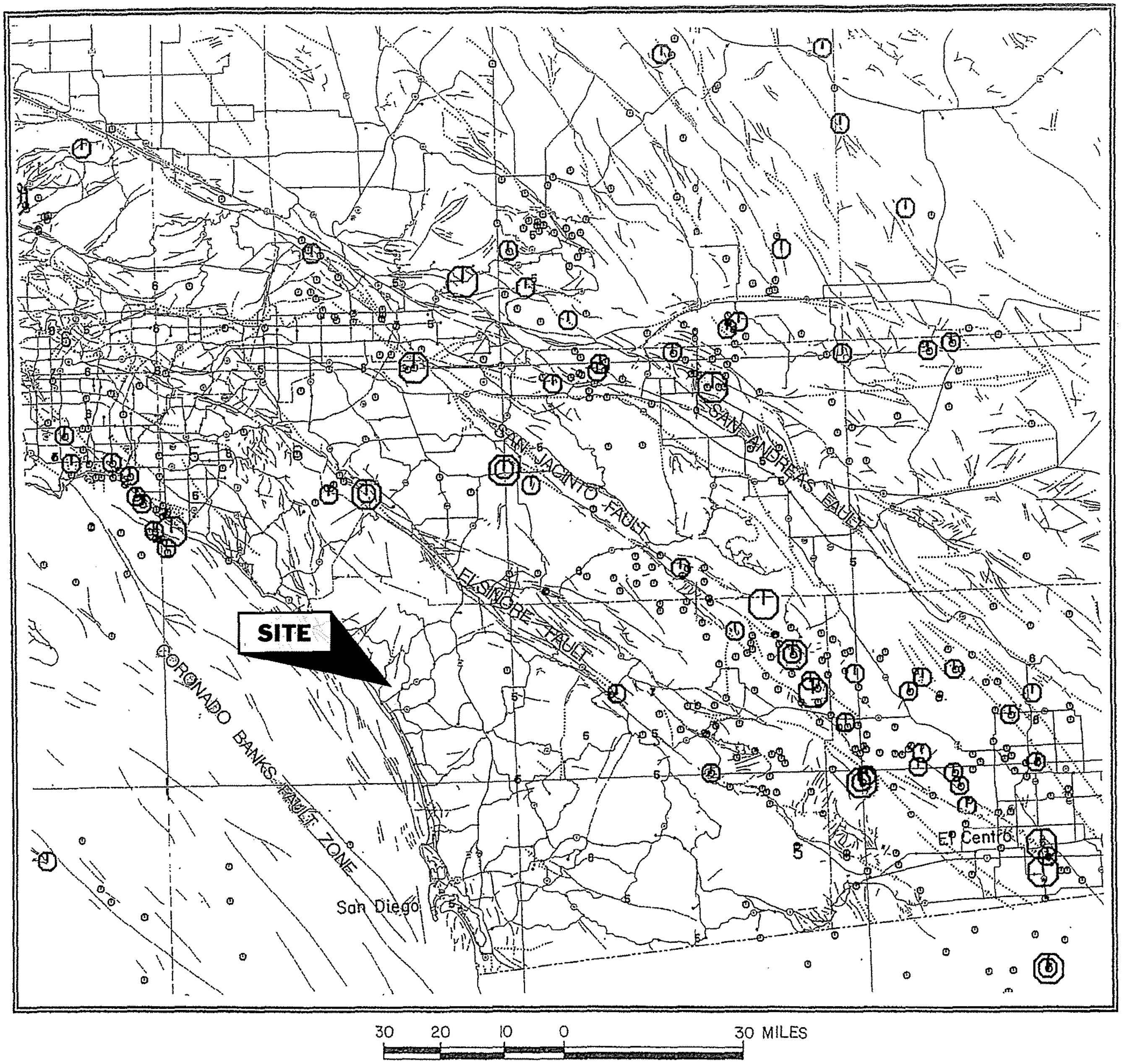


PLATE 12
V&M JOB #15-188-P

FAULT-EPICENTER MAP
SAN DIEGO COUNTY REGION



INDICATED EARTHQUAKE EVENTS THROUGH 75 YEAR PERIOD (1900-1974)

Map data is compiled from various sources including California Devision of Mines and Geology, California Institute of Technology and the National Oceanic and Atmospheric Administration. Map is reproduced from California Division of Mines and Geology, "Earthquake Epicenter Map of California; Map Sheet 39." 1978

MAGNITUDE

○ .... 4.0 TO 4.9
○ .... 5.0 TO 5.9
○ .... 6.0 TO 6.9

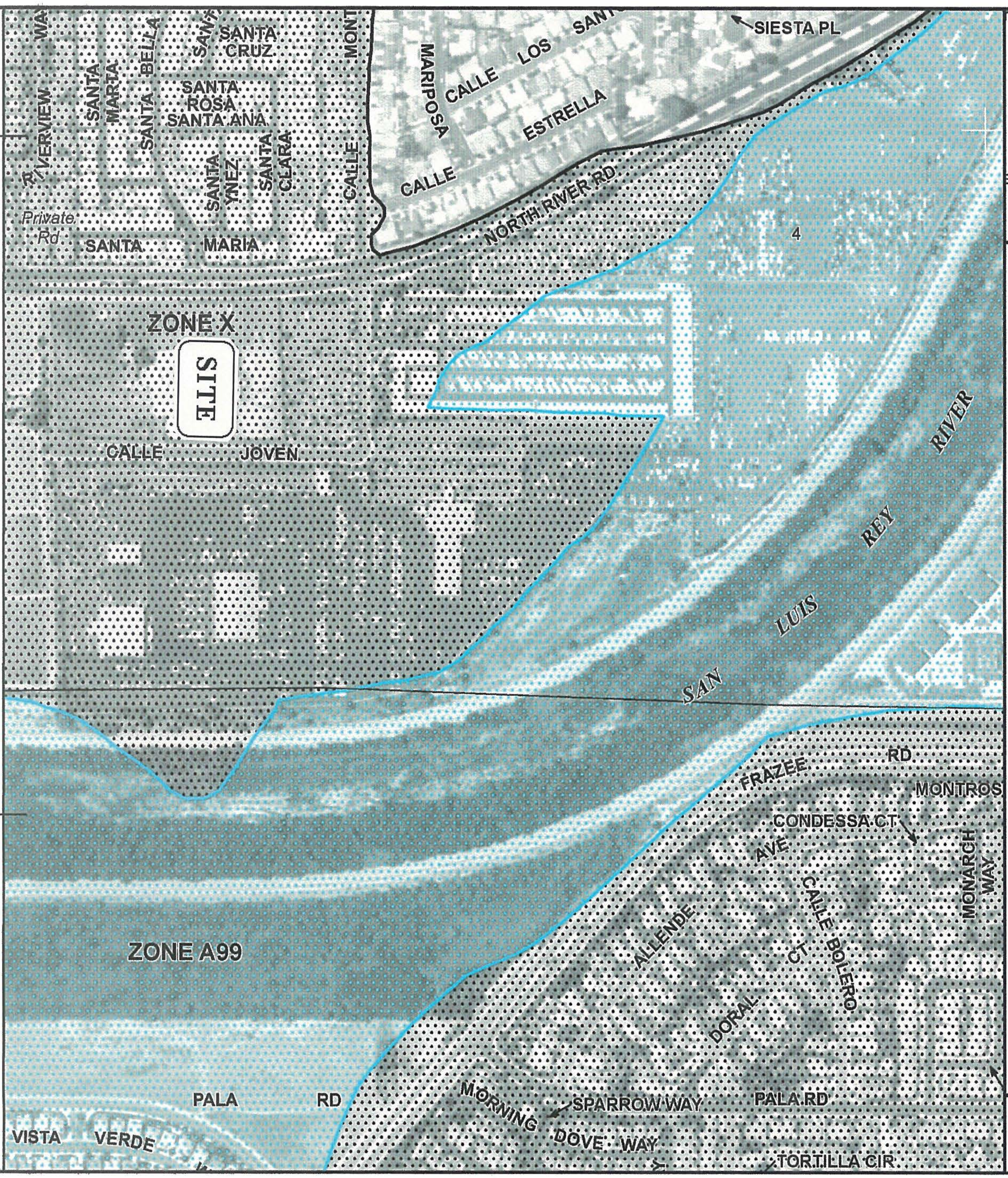
Fault

PLATE 13
V&M JOB #15-188-P

ance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

0 250 500 750 1,000 FEET



NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0756H

FIRM

FLOOD INSURANCE RATE MAP

SAN DIEGO COUNTY, CALIFORNIA

AND INCORPORATED AREAS

PANEL 756 OF 2375

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
OCEANSIDE, CITY OF	060294	0756	H

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06073C0756H

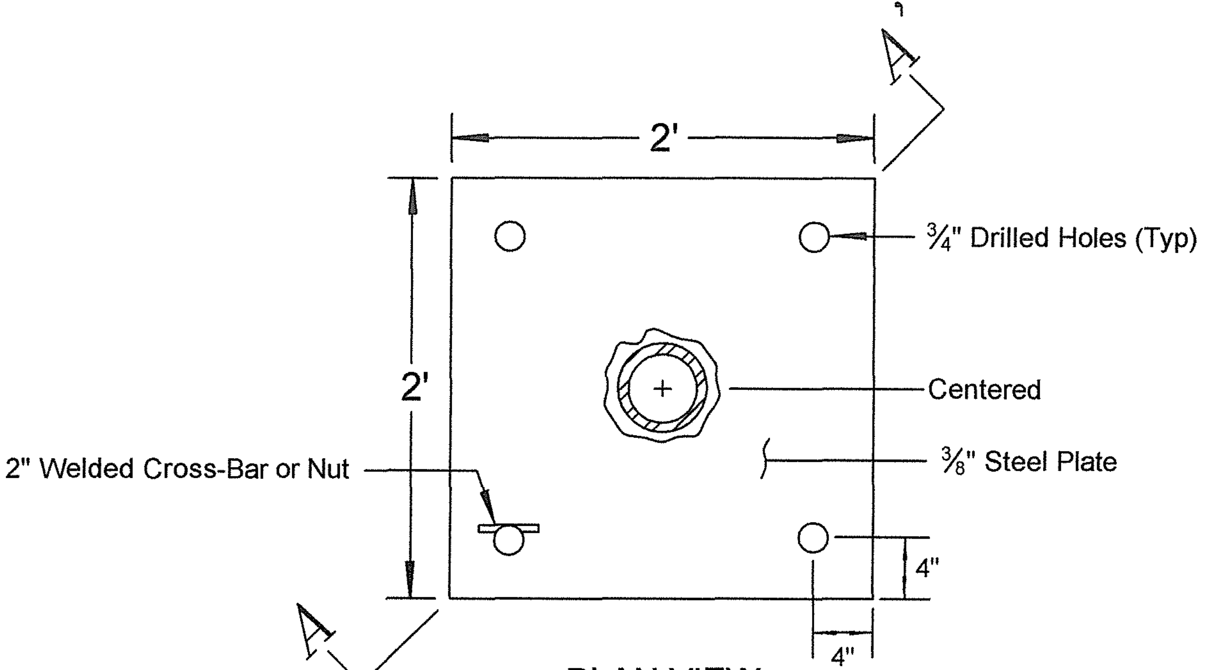
MAP REVISED
MAY 16, 2012

Federal Emergency Management Agency

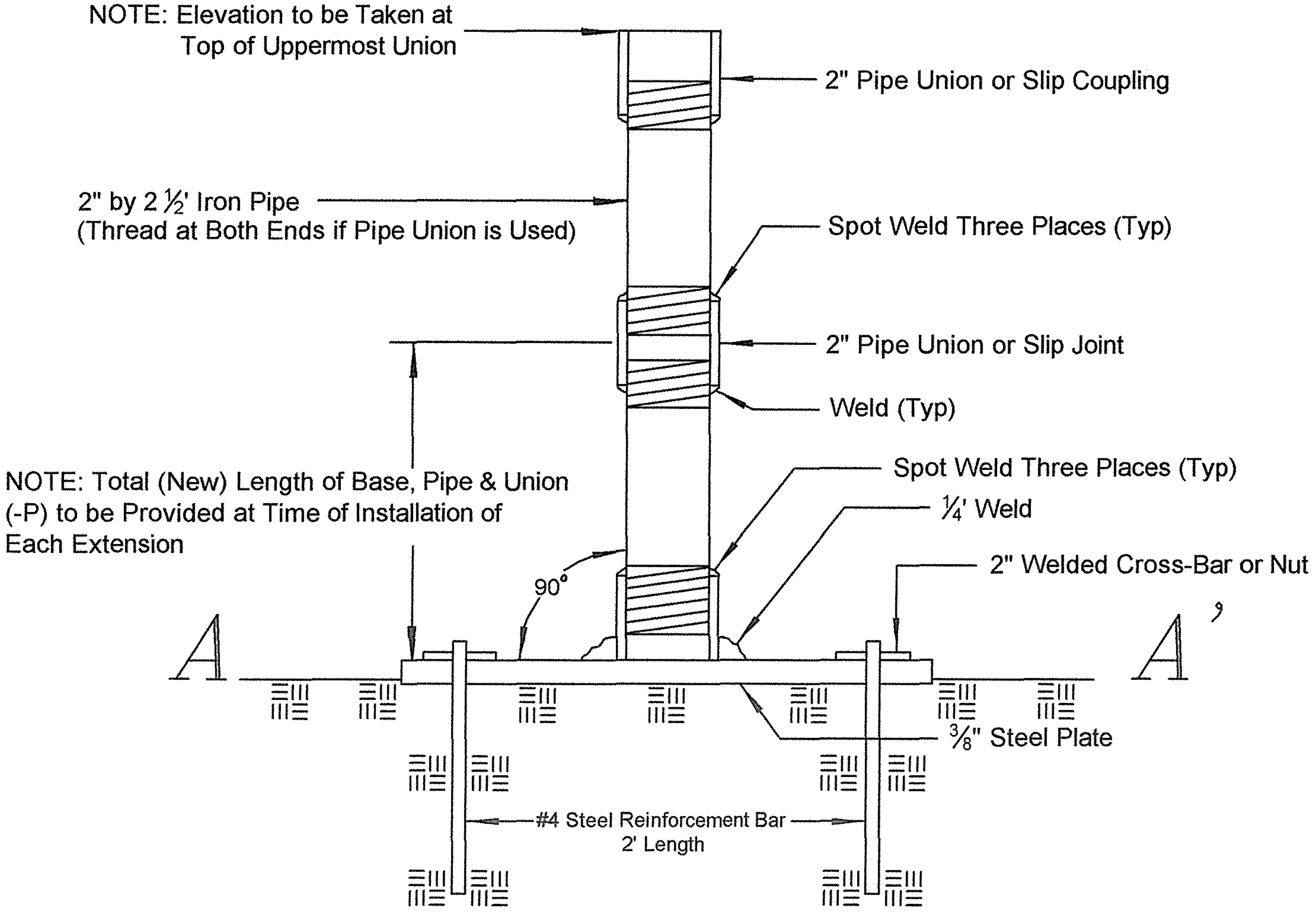
This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

PLATE 14
V&M JOB #15-188-P

SETTLEMENT PLATE SCHEMATIC
NO SCALE



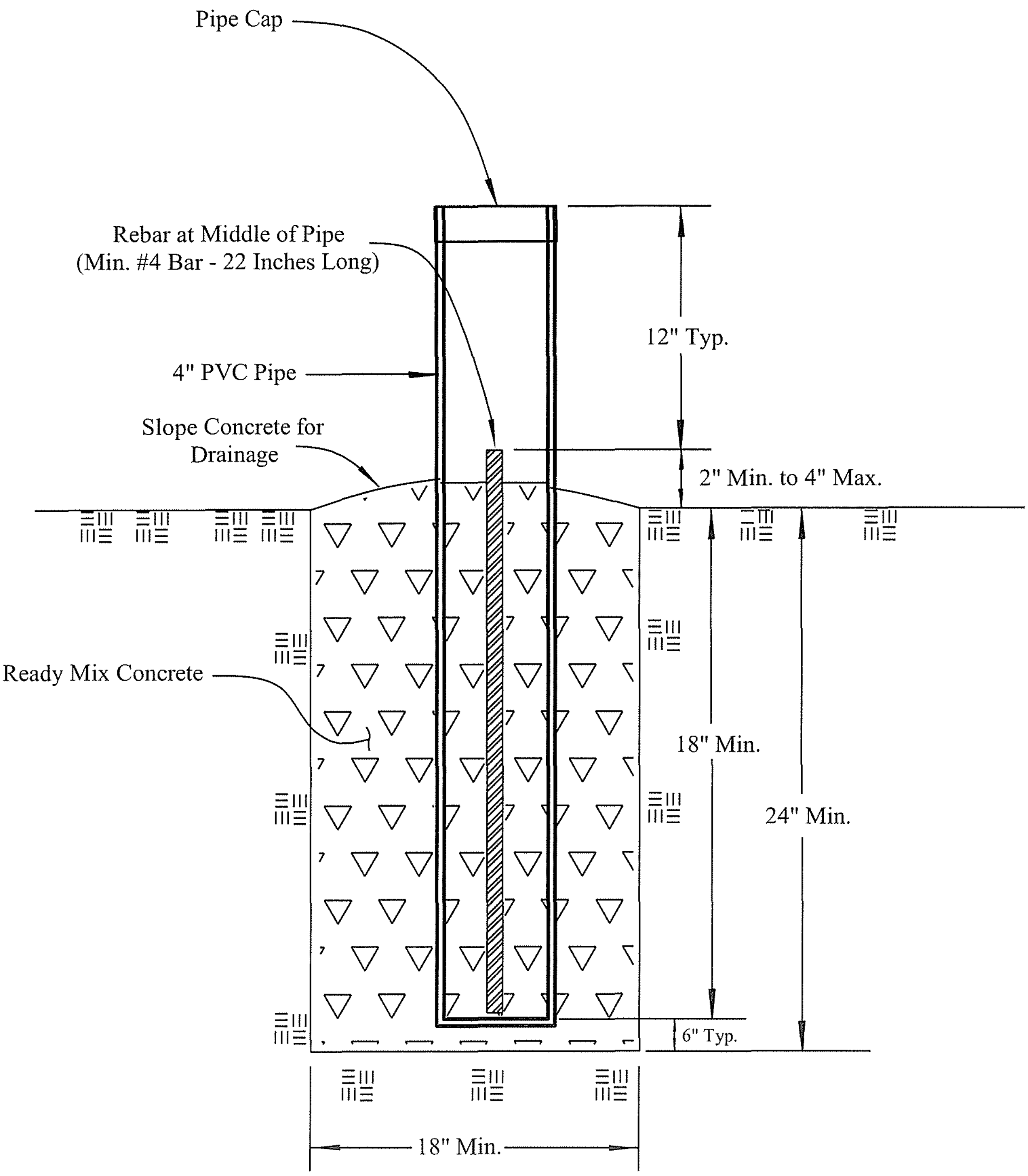
PLAN VIEW



CROSS-SECTION PROFILE

PLATE 15
V&M JOB #15-188-P

SETTLEMENT MONUMENT
SCHEMATIC
NO SCALE

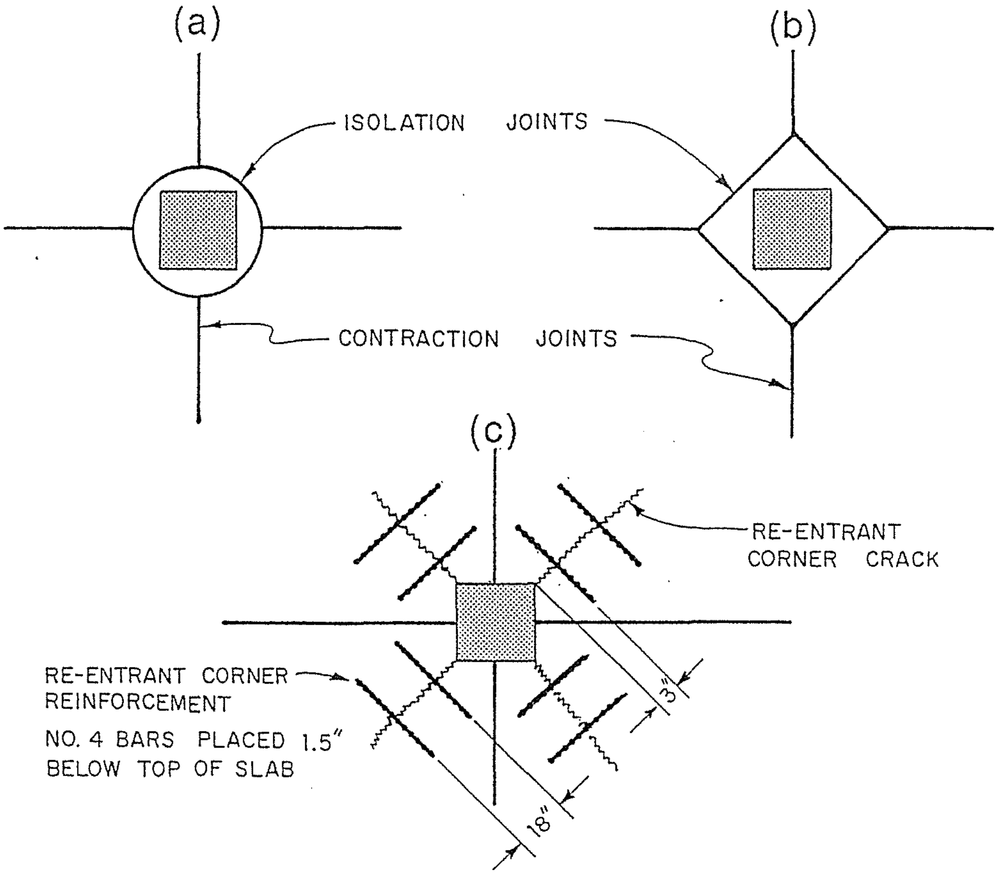


Vinje & Middleton Engineering, Inc.
2450 Auto Park Way
Escondido, California

PLATE 16
V&M JOB #15-188-P

ISOLATION JOINTS AND RE-ENTRANT CORNER REINFORCEMENT

Typical - no scale



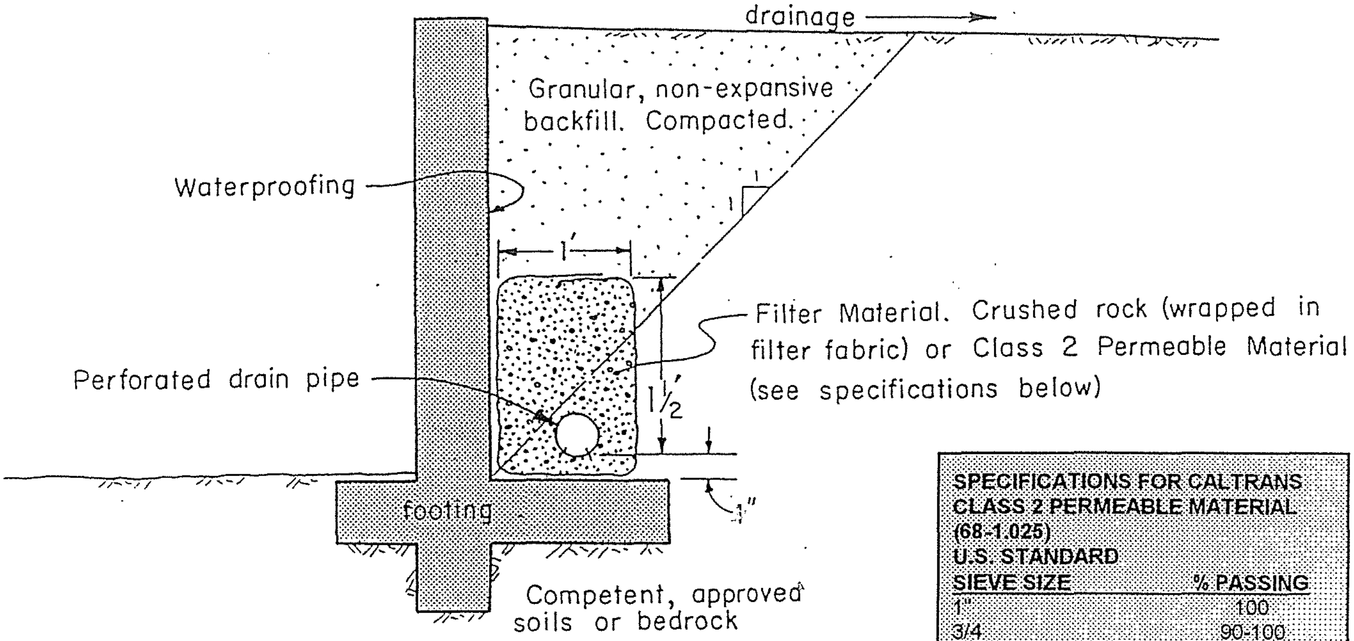
NOTES:

1. Isolation joints around the columns should be either circular as shown in (a) or diamond shaped as shown in (b). If no isolation joints are used around columns, or if the corners of the isolation joints do not meet the contraction joints, radial cracking as shown in (c)may occur (reference ACI).
2. In order to control cracking at the re-entrant corners (±270° corners), provide reinforcement as shown in (c).
3. Re-entrant corner reinforcement shown herein is provided as a general guideline only and is subject to verification and changes by the project architect and/or structural engineer based upon slab geometry, location, and other engineering and construction factors.

VINJE & MIDDLETON ENGINEERING, INC.

RETAINING WALL DRAIN DETAIL

Typical - no scale



SPECIFICATIONS FOR CALTRANS CLASS 2 PERMEABLE MATERIAL (68-1.025)

U.S. STANDARD

SIEVE SIZE	% PASSING
1"	100
3/4	90-100
3/8	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Sand Equivalent > 75

CONSTRUCTION SPECIFICATIONS:

1. Provide granular, non-expansive backfill soil in 1:1 gradient wedge behind wall. Compact backfill to minimum 90% of laboratory standard.

2. Provide back drainage for wall to prevent build-up of hydrostatic pressures. Use drainage openings along base of wall or back drain system as outlined below.

3. Backdrain should consist of 4" diameter PVC pipe (Schedule 40 or equivalent) with perforations down. Drain to suitable outlet at minimum 1%. Provide ¾" - 1½" crushed gravel filter wrapped in filter fabric (Mirafi 140N or equivalent). Delete filter fabric wrap if Caltrans Class 2 permeable material is used. Compact Class 2 material to minimum 90% of laboratory standard.

4. Seal back of wall with waterproofing in accordance with architect's specifications.

5. Provide positive drainage to disallow ponding of water above wall. Lined drainage ditch to minimum 2% flow away from wall is recommended.

* Use 1½ cubic foot per foot with granular backfill soil and 4 cubic foot per foot if expansive backfill soil is used.

VINJE & MIDDLETON ENGINEERING, INC.

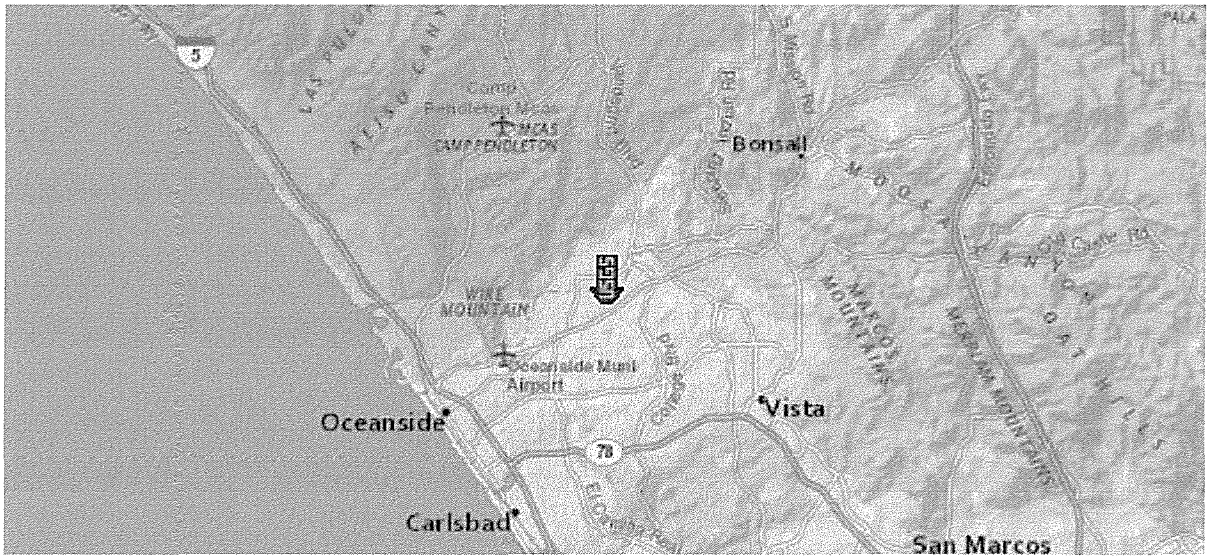
PLATE 18
V&M JOB #15-188-P

APPENDIX

Design Maps Summary Report

User–Specified Input

Report Title	15-188-P 4665 North River Road, Oceanside Sat December 19, 2015 17:15:40 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.2445°N, 117.3106°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

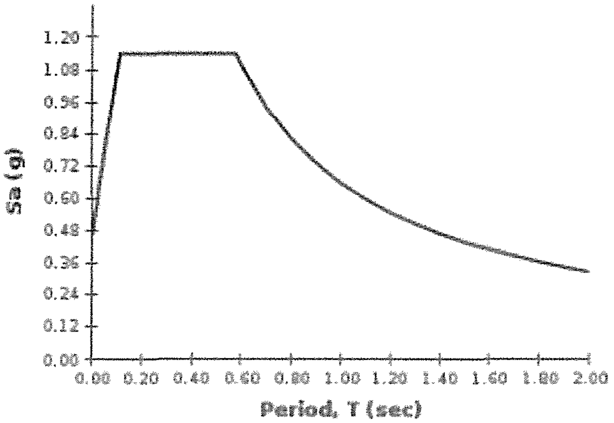


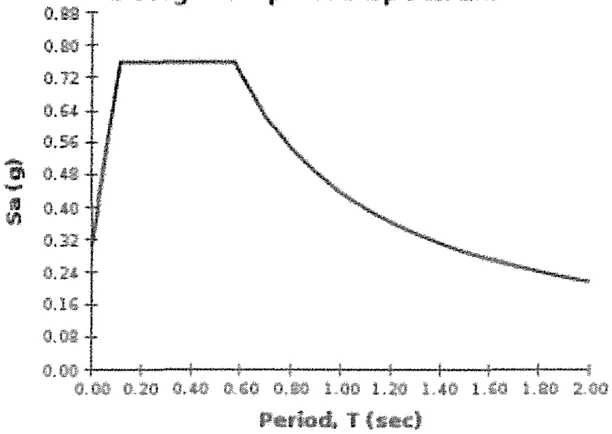
USGS–Provided Output

$S_S =$ 1.057 g $S_{MS} =$ 1.139 g $S_{DS} =$ 0.759 g

$S_1 =$ 0.413 g $S_{M1} =$ 0.656 g $S_{D1} =$ 0.437 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.


MCE_R Response Spectrum


Design Response Spectrum

For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.2445°N, 117.3106°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.057$ g

From Figure 22-2 [2] $S_1 = 0.413$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ **or** $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.057 g, F_a = 1.077

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.413 g, F_v = 1.587

Equation (11.4-1): $S_{MS} = F_a S_S = 1.077 \times 1.057 = 1.139$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.587 \times 0.413 = 0.656$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

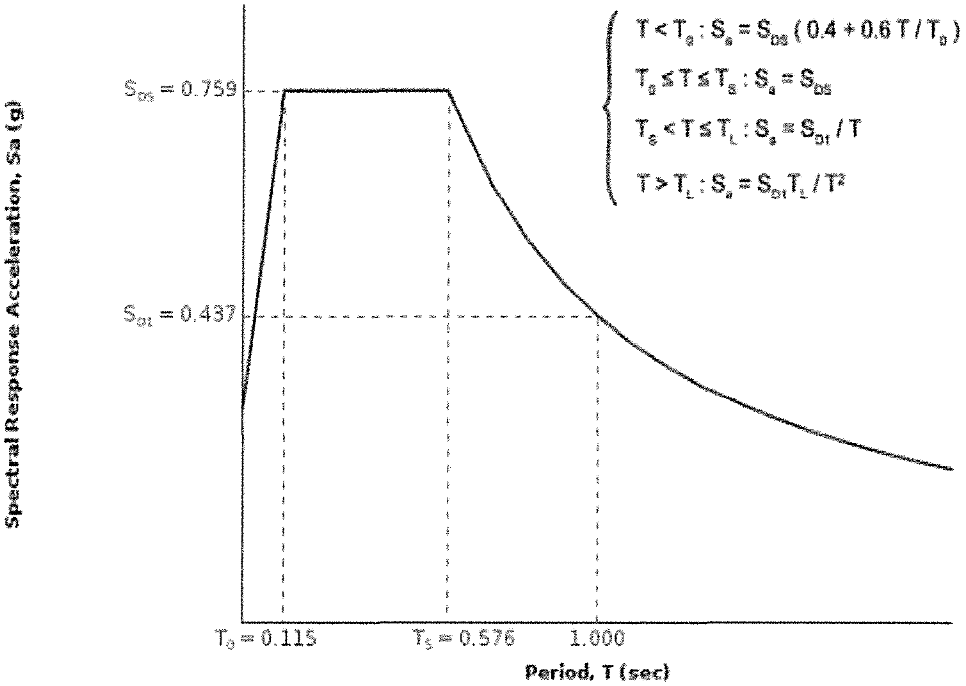
Equation (11.4-3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.139 = 0.759$ g

Equation (11.4-4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.656 = 0.437$ g

Section 11.4.5 — Design Response Spectrum

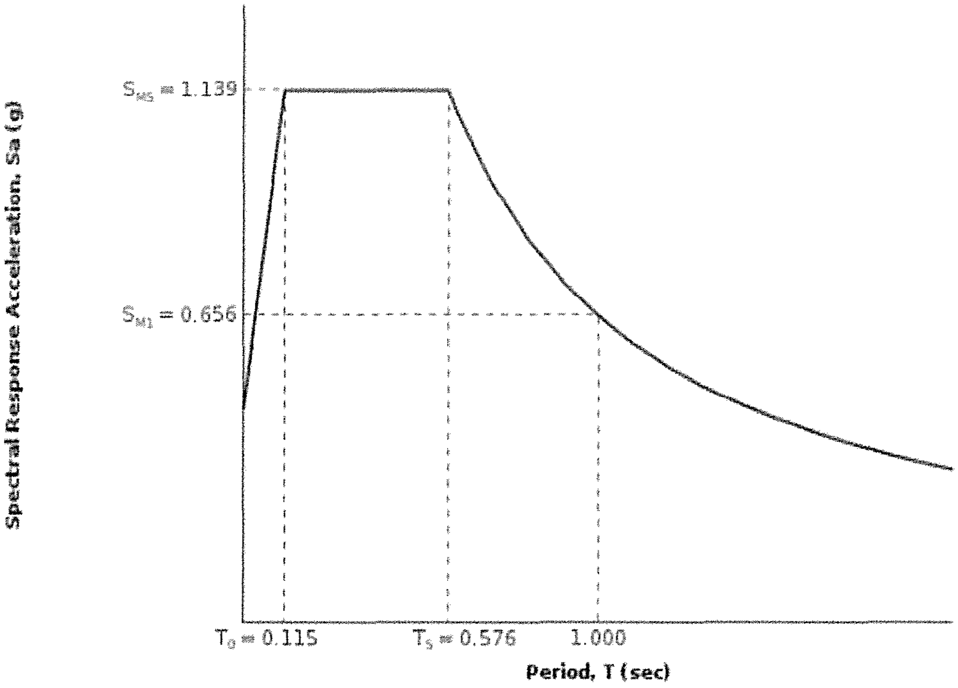
From Figure 22-12[3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7[4] $PGA = 0.395$

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.105 \times 0.395 = 0.437$ g

---

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.395 g, F_{PGA} = 1.105

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17[5] $C_{RS} = 1.015$

From Figure 22-18[6] $C_{R1} = 1.062$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.759 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.437 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf