Geotechnical Investigation

Proposed Commercial Redevelopment
999 North Broadway
Escondido, California

(A.P.N.'s 229-121-09, -10, -11, -12, -13, & -14)

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I. INTRODUCTION

The project site investigated herein consists of a developed automobile dealership property which is now abandoned. The property is located on the northwest corner of Highway 78 and North Broadway within the limits of City of Escondido. The general site location is shown on a Vicinity Map attached to this report as Plate 1. The approximate site coordinates are 33.1330°N latitude and 117.0881°W longitude.

We understand that the property is planned for a commercial redevelopment. The new development will include complete demolition and removal of existing site structures and improvements, and construction of a large grocery store and three smaller commercial buildings, with the associated paving and underground improvements. Consequently, the purpose of this investigation was to determine soil and geotechnical conditions at the project site and to ascertain their influence upon the proposed redevelopment. Research and review of ready available pertinent reports and documents, test borings, in-situ testing and sampling, and laboratory testing were among the activities conducted in conjunction with this effort which has resulted in the remedial grading and foundation recommendations presented herein.

II. SITE DESCRIPTION / BACKGROUND

The project property consists of 6 contiguous lots characterized by nearly level graded surfaces that currently support abandoned structures, improvements, and paved parking areas that were previously occupied by an automobile dealership. The site was largely developed to its current configuration approximately 40 years ago. Plans and documents pertaining to the original site development are not readily available. Some subsequent paving for a shop entrance and new west parking areas were carried out in 1987 and 1994 respectively. Geotechnical engineering services for the 1987 and 1994 paving improvements were provided by Vinje & Middleton Engineering, Inc. and MV Engineering, Inc.. The following technical reports and documents were published by this office in connection with the subject paving improvements:

A. Fill control testing and observation for the west parking lot construction and adjacent Lincoln Avenue, Job #94-203-F. Field reports prepared by Vinje & Middleton Engineering, Inc. dated between June 10, 1994 and June 14, 1995.

B. "R-Value Test Result, Pavement/Base Thickness and Sub/Basegrade Recommendations for a Portion of Lincoln Avenue, City of Escondido Drawing #P-2140," Job #94-203-F, prepared by Vinje & Middleton Engineering, Inc., dated December 20, 1994.

C. "Preliminary Soils Investigation, Proposed Parking Lot Facility, 225-227 West Lincoln Avenue, Escondido, California," Job #1099-93, prepared by MV Engineering, Inc., dated August 10, 1993.

D. R-Value laboratory result dated April 6, 1987, Job #1129-87, performed by MV Engineering, Inc. No letter or report issued.

The referenced reports are on file with our firm and copies can be obtained upon request.

An Existing Conditions Map depicting current site conditions is attached to this report as Plate 2. As shown, the easterly potion of the site currently supports existing structures that were utilized for automobile showrooms, offices, and mechanic/shop bays. The remainder of the site consists of asphalt paved parking surfaces with intervening block walls. Some landscaping improvements are present along the property margins.

Current site drainage is developed to flow away from structures and improvements to local drain inlets and ribbon gutters and then to off-site locations. Scouring or erosion is not in evidence.

III. PROPOSED DEVELOPMENT

The property is proposed for a commercial redevelopment as shown on a Proposed Development Plan included herein as Plate 3. All existing structures and improvements will be demolished and removed to accommodate the new redevelopment. The new development includes the construction of a large 30,300 sq. ft. market building in the property westerly margins, a 2,911 sq. ft. fast food building, a 1,700 sq. ft. coffee shop building, and a 3,500 sq. ft. bank building in the east portion of the site. Associated improvements include interior asphalt parking stalls and drive lanes, a trash enclosure, receiving truck ramp, underground utilities, drainage structures, and bio-filtration areas. Off-site improvements may include new access driveways and sidewalks along the frontage streets.

Major ground modifications or constructions of large new graded slopes is not anticipated in connection with the project development. However, minor grade alterations are anticipated for achieving final design grades and the development of new level building pad and improvement surfaces.

Construction details are unknown. However, building construction is anticipated to consist of a block wall or wood-frame type 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. Parking areas are expected to consist of asphalt pavement surfaces with concrete trash enclosure slab and receiving truck ramp, and local landscaped islands.

IV. SITE INVESTIGATION

Subsurface conditions were chiefly determined by the excavation of 11 exploratory test borings drilled with truck-mounted drill rigs. Six exploratory borings (B-1 through B-6) were drilled in planned parking and improvement areas using a solid stem auger rig for shallow subsurface evaluations and bulk sample collection. The remaining five exploratory borings (B-7 through B-11) were drilled with a hollow-stem auger in the planned building areas for deep subsurface evaluations, in-situ testing and sampling. The deep test borings were permitted through the County of San Diego Department of Environmental Health (Permit #LMON108359), as required. Exploratory boring locations are shown on both the Existing Conditions Map and Proposed Development Plan (Plates 2 and 3). Boring locations were limited by the existing structures and improvements. Logs of the borings are included as Plates 4-14. Laboratory results and engineering properties of selected samples are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

Crystalline bedrock units occur beneath the project site at depth and are overlain by natural ancient alluvial soils. The ancient alluvium deposits underlie existing shallow fills at the project property. Instability or adverse geotechnical conditions which could preclude proposed site redevelopment is not indicated. A Geologic Map depicting mapped geologic units at the site and surrounding areas is attached to this report as Plate 15. Geologic Cross-Sections depicting subsurface conditions based on our exploratory test borings are included as Plate 16.

A. Earth Materials

Bedrock (Kgb) - Crystalline bedrock units which are rooted in the Southern California Batholith underlie the project site at depth. Underlying bedrock, as encountered in our exploratory test borings, typically consists of dark-colored gabbroic rocks which chiefly occur in hard and very dense conditions overall. Project bedrock units will provide suitable support for overlying consolidated ancient alluvium deposits.

Ancient Alluvium (Qoa) - Ancient alluvium deposits, which occur locally within the City of Escondido, were encountered at shallow depths in our test borings. Project ancient alluvium typically consists of interbedded clayey fine sand to sandy clay deposits that were largely found in loose to soft and medium dense conditions near shallow surface exposures, becoming tight to very tight and dense to very stiff with depth, at the explored locations. Tight and dense to very stiff ancient alluvium below the shallow surface exposures are well-consolidated deposits that will provide adequate support for proposed new fills, structures and improvements as currently planned.

Fill (af) - Shallow fills mantle site ancient alluvium deposits and occur directly below the asphalt pavement surfaces at the project site. Engineering observations and compaction testing records for most of the site fills are not readily available. Based our subsurface exposures and limited sampling and testing, site existing fills chiefly range from imported sand to locally derived silty clayey sands to sandy clay deposits which mainly occur in a moist to very moist and soft to moderately compacted conditions overall.

Existing asphalt surfaces at the project site range from 2½ to 4½ inches thick as measured at our test boring locations. All site surface pavements will be removed as part of site demolition.

B. Groundwater and Surface Drainage

Shallow perched water was locally encountered at the depth of 3 feet in Boring B-1. The noted condition is thought to be the result of runoff from recent rain storms and not a representation of actual shallow groundwater conditions.

Groundwater conditions were encountered in three (Borings B-7, B-9, & B-10) of the five deeper borings at depths ranging from 13 to 21 feet below the surface. Static water levels measured at the completion of the drilling ranged from 15 to 17 feet below the existing ground surface. The groundwater encountered in Boring 9 (B-9) is also thought to likely represent a discontinuous perched water zone that may be trapped in a sandy soil layer lying atop a clay-rich deposit. Elsewhere, the groundwater is mostly occurring in more sandy deposits that lie atop the underlying crystalline bedrock. Historic High Groundwater (HHG) level at the project site is not known, but assumed herein as 10 feet below existing ground surfaces for the purpose of our engineering analysis.

The noted groundwater is expected to fluctuate with seasonal conditions but is not expected to impact near surface remedial grading and earthwork operations as specified herein. Shallow local perched water conditions may be expected to create some isolated wet soils requiring added grading efforts potentially with some minor dewatering for removals of intruding water. However, significant grading difficulties resulting from shallow perched water conditions are not anticipated.

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 15 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 0.146g at the project site. The following list represents the most significant faults which commonly impact the region. Estimated ground acceleration data compiled from Digitized California Faults (Computer Program EQFAULT VERSION 3.00 updated) typically associated with the fault is also tabulated.

TABLE 1

FAULT ZONE	DISTANCE FROM SITE	MAXIMUM PROBABLE ACCELERATION (R.H.)
Elsinore-Julian Fault	16.1 miles	0.193g
Rose Canyon Fault	15.8 miles	0.176g
Newport-Inglewood Fault	19.8 miles	0.149g
Coronado Bank Fault	30.6 miles	0.139g

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 17.

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

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

In recent years, the Rose Canyon Fault has received 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

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

TABLE 2

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	1.090	0.406	1.064	1.594	1.160	0.648	0.773	0.432
According to Chapter 16, Section 1613 of the 2010 California Building Code.								

Explanation:

S_S: Mapped MCE, 5% damped, spectral response acceleration parameter at short periods.
S_1: Mapped MCE, 5% damped, spectral response acceleration parameter at a period of 1-second.
F_a: Site coefficient for mapped spectral response acceleration at short periods.
F_v: Site coefficient for mapped spectral response acceleration at 1-second period.
S_{MS}: The MCE, 5% damped, spectral response acceleration at short periods adjusted for site class effects ($S_{MS}=F_aS_s$).
S_{M1}: The MCE, 5% damped, spectral response acceleration at a period of 1-second adjusted for site class effects ($S_{M1}=F_vS_1$).
S_{DS}: Design, 5% damped, spectral response acceleration parameter at short periods ($S_{DS}=⅔S_{MS}$).
S_{D1}: Design, 5% damped, spectral response acceleration parameter at a period of 1-second ($S_{D1}=⅔S_{M1}$).

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

E. Geologic Hazards

Conditions which could result in potential geologic hazards are known in the areas of the project site. The following geotechnical hazards are herein evaluated:

1. **Seismicity and Faulting -** Faults or significant shear zones were not evidenced at or nearby the project site. The project site is not located within an Alquist - Priolo Earthquake Fault Zone established by the State of California. The most significant geotechnical factor which could impact the project site relates to ground shaking during an earthquake event along nearby active faults. 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 detailed in a preceding section.

2. **Liquefaction -** Liquefaction and associated secondary effects were a potential geotechnical concern at the project property and evaluated herein as part of this study. Soil liquefaction and related ground failures can adversely impact manmade improvements at sites where subsoils consist of loose soil deposits inundated with groundwater. Liquefaction is the sudden loss of soil strength in response to ground shaking during an earthquake event. Factors significant to the occurrence of liquefaction are soil types, intensity of ground shaking, in-situ soil densities, duration of ground shaking, initial confining pressures and groundwater levels.

   In order to accurately evaluate site liquefaction potential, a detail subsurface in-situ testing program was carried out during the exploratory boring excavations. Based on the subsoil data generated from our boring explorations, saturated ancient alluvium with variable in-situ characteristics occur beneath the site. Consequently, liquefaction potential screening and analysis were performed based on subsoil data collected in Boring B-9 in order to estimate safety factors of susceptible subsoil layers against liquefaction. Our analysis indicated that liquefaction potential is not a geotechnical factor in the development of the project property. A summary of our liquefaction analysis is presented in the following sections.

3. **Flood Inundation -** Potential flood hazards at the property were also examined. The site is not located in or near proximity to a water reservoir storage facility and is removed from potential flood areas as indicated on a Fema Flood Map enclosed herein as Plate 18.

4. **Slope Stability -** The project property consists of nearly level to very gently sloping ground surfaces. Significant slopes are not present nor are any planned in connection with the project planned redevelopment. Consequently, slope stability is not considered a significant geotechnical concern in development of the project property as currently planned.

5. **Expansive Soils -** Based upon our field observations and laboratory testing, onsite near surface soils range from very low to medium expansive. Expansive soils will require special designs and ground mitigation if they occur near finish grade levels. Onsite potentially expansive soils should be selectively buried in deeper fills or thoroughly mixed with an abundance of low to very expansive sandy soils generated from the project excavations as recommended in the following sections.

6. **Settlements and Ground Subsidence -** Dynamic settlement of saturated soils, dynamic compaction of dry soils, ground subsidence and lateral spread are not considered a significant geotechnical factor at the project site. Anticipated settlements after removal and recompaction of the upper fil/topsoils and ancient alluvial deposits as specified herein, are expected to be within the allowable tolerances on the order 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.

F. Field and Laboratory Tests and Test Results

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

TABLE 3

Soil Type	Description
1	Brown to grey fine to coarse sand (Fill)
2	Red brown sandy silt to clayey fine sand (Fill / Ancient Alluvium)
3	Red brown fine sandy silty clay (Fill / Ancient Alluvium)
4	Brown fine to medium sand (Ancient Alluvium)
5	Red brown clayey fine to coarse sand w/ rock fragments (Ancient Alluvium)
6	Grey to "salt & pepper" fine to coarse sand (Bedrock)

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 an automatic trip-hammer. 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, mechanically dropped 30 inches using 5-foot long AW drill rods. The bore hole was 200 MM (8 inches) in diameter and drill fluid or water was not necessary to aid drilling. The test results are presented at the corresponding locations on the attached Boring Logs.

2. **Grain Size Analysis:** Grain size analyses were performed on representative samples of site soils. The test results are presented in Table 4 and graphically presented on the enclosed Plate 19.

TABLE 4

Sieve Size		½"	#4	#10	#20	#40	#200
Location	**Soil Type**	**Percent Passing**					
B-7 @ 3'	2	100	97	92	85	78	47
B-7 @ 18'	2	100	100	99	98	95	53
B-10 @ 3'	2	100	100	98	92	85	49

3. **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 representative selected samples of onsite soils in accordance with the ASTM D-1140. The results are tabulated below:

TABLE 5

Location	Soil Type	Original Dry Mass (g)	Dry Mass Retained after washing (g)	Percent of Material Finer Than No. 200 Sieve	Predominant Soil Type
B-7 @ 9'	2	328.4	149.1	55	ML/CL
B-7 @ 15'	2	312.0	146.1	53	ML
B-7 @ 24'	4	561.1	467.5	17	SP
B-7 @ 30'	5	299.2	227.5	24	GC

4. **Liquid Limit, Plastic Limit and Plasticity Index:** Liquid limit, plastic limit and plasticity index tests were performed on representative sample of Soil Types 2 and 3 in accordance with the ASTM D-4318. The results are tabulated in Table 6.

TABLE 6

Location	Soil Type	Liquid Limit (LL-%)	Plastic Limit (PL-%)	Plasticity Index (PI=LL-PL)
B-7 @ 3'	2	33	14	19
B-7 @ 18'	2	27	19	8
B-8 @ 1½'	3	31	14	17

5. **Maximum Dry Density and Optimum Moisture Content:** The maximum dry density and optimum moisture content of Soil Types 2 and 3 were determined in accordance with ASTM D-1557. The test results are presented in Table 7.

TABLE 7

Location	Soil Type	Maximum Dry Density (γ_m-pcf)	Optimum Moisture Content (ω_{opt}-%)
B-8 @ 1½'	3	129.6	11
B-10 @ 3'	2	134.5	10

6. **Moisture-Density Test (Undisturbed Ring Samples):** In-place dry density and moisture content of representative soil deposits beneath the site were determined from relatively undisturbed ring samples using the weights and measurements test method. The test results are presented in Table 8 and tabulated at corresponding locations on the attached Boring Logs.

TABLE 8

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 (%)
B-8 @ 3'	2	13	117.5	134.5	87	78
B-8 @ 6'	2	15	119.9	134.5	89	96
B-8 @ 15'	2	12	122.5	134.5	91	83
B-9 @ 3'	2	13	121.8	134.5	91	87
B-9 @ 6'	2	12	118.5	134.5	88	73
B-9 @ 18'	3	25	104.9	129.6	81	100
B-9 @ 25'	2	15	119.0	134.5	88	94
B-10 @ 5'	2	13	-	134.5	Sample Disturbed	-
B-10 @ 10'	2	17	117.9	134.5	88	100
B-10 @ 20'	3	23	103.7	129.6	80	97
B-11 @ 3'	2	17	117.9	134.5	88	92
B-11 @ 5'	2	14	120.6	134.5	90	92
B-11 @ 15'	2	16	119.6	134.5	89	100

Assumptions and relationships:
In-place Relative Compaction = $(Y_d \div Y_m)$ X100
Gs = 2.70
$e = (Gs\ Y_\omega \div Y_d) - 1$
$S = (\omega\ Gs) \div e$

7. **Expansion Index Test:** Three expansion index (EI) tests were performed on representative samples of Soil Types 2 and 3 in accordance with the ASTM D-4829. The results are presented in Table 9.

TABLE 9

Sample Location	Soil Type	Molded ω (%)	Degree of Saturation (%)	Final ω (%)	Initial Dry Density (PCF)	Measured EI	EI 50% Saturation
B-8 @ 1½'	3	10	51	20	109.4	43	43
B-8 @ 6"	3	10	52	20	110.0	59	62
B-10 @ 3'	2	9	56	14	118.1	7	10

(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

8. **<u>Direct Shear Test</u>:** Two direct shear tests were performed on representative samples of Soil Types 1 and 2. The prepared specimens were soaked overnight, loaded with normal loads of 1, 2, and 4 kips per square foot respectively, and sheared to failure in an undrained condition. The results are presented in Table 10.

TABLE 10

Sample Location	Soil Type	Sample Condition	Wet Density (Yw-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
B-8 @ 1½'	3	remolded to 90% of Y_m @ % ωopt	130.2	22	390
B-10 @ 3'	2	remolded to 90% of Y_m @ % ωopt	132.8	33	300

9. **<u>Permeability Test</u>:** A permeability test was performed a representative sample of site Soil Type 2 using the Falling Head test procedure in accordance with the California Test Method (CTM) 220. The sample was remolded to near 100% of the in-situ conditions. Selected results are presented in Table 11 below:

TABLE 11

Location	Sample Condition	Initial Unit Weight (pcf)	Final Unit Weight (pcf)	Q (cmEE3)	t (min.)	K (mm/sec)
B-8 @ 6'	Remolded	135.0	136.8	No Tail Water	17270	Less Than 1X10EE-4*

* No tail water was captured in 287 hours and 50 minutus (17270 minutus).

10. **R-Value Test**: One R-value test was performed on a representative sample of Soil Type 2 in accordance with the California Test Method 301. The results are presented in Table 12. The results are supplemented by prior tests completed as part of previous work performed by this office for related paving improvements at the project site (see referenced reports).

TABLE 12

Location	Soil Type	Description	R-Value
B-1 @ 1½'	3	Red brown fine sandy silty clay	8
Lincoln Avenue*	2	Orangish brown silty sand	52
Import Soil**	1	Grey fine to coarse sand	70
Surface Soil***	2	Brown silty clayey fine sand	27

* R-Value Test Result letter dated December 20, 1994 (Reference A)
** Preliminary Soils Investigation dated August 10, 1993. (Reference C)
*** Laboratory test result dated April 6, 1987.

11. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 3 was determined using "Method for Estimating the Service Life of Steel Culverts," in accordance with the California Test Method (CTM) 643. The result is tabulated in Table 13.

TABLE 13

Sample Location	Soil Type	Minimum Resistivity (OHM-CM)	pH
B-8 @ 1½'	3	1512	7.2

12. **Sulfate Test:** A sulfate test was performed on a representative sample of Soil Type 3 in accordance with the California Test Method (CTM) 417. The result is presented in Table 14.

TABLE 14

Sample Location	Soil Type	Amount of Water Soluble Sulfate In Soil (% by Weight)
B-8 @ 1½'	3	0.005

13. **Chloride Test:** A chloride test was performed on a representative sample of Soil Type 3 in accordance with the California Test Method (CTM) 422. The result is presented in Table 15.

TABLE 15

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
B-8 @ 1½'	3	0.004

14. **Consolidation Test:** Consolidation tests were performed on representative undisturbed samples of on-site Soil Type 3. The results are graphically presented in the enclosed Plate 20.

VI. SITE CORROSION ASSESSMENT

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

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

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

Results of limited laboratory tests performed on selected representative site samples indicate that the minimum resistivity is greater less than 1000 ohm-cm suggesting presence of low quantities of soluble salts. Test results further indicated that 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 0.005 percent by weight which is considered negligible according to ACI 318, Table 4.3.1. Portland cement Type II may be used. Table 16 is appropriate based on the pH-Resistivity test result:

TABLE 16

Design Soil Type	Gage	18	16	14	12	10	8
3	**Years to Perforation of Steel Culverts**	25	32	40	55	70	85

VII. HYDRO MODIFICATIONS

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

Bio-retention and filtration systems consisting of vegetated buffers or strips and filtration trenches with special engineered soil filter media and perforated pipes which discharge the treated water into the public stormwater facility are typical methods for the stormwater Best Management Practices (BMP). Self-contained vegetated bio-retention/detention area with impermeable liners on sides and bottom may also be appropriate based on site geotechnical conditions, if applicable. The following are appropriate to the site conditions:

1. Site soils are mostly silty to clayey plastic deposits with very low coefficient of permeability (hydraulic conductively). New structures and improvements also occupy almost the entire property. Consequently, infiltration trenches with direct infiltration into the soil matrix are not suitable systems for the project stormwater management.

2. Based on the project proposed redevelopment scheme and site indicated geotechnical conditions, self-contained vegetated bio-retention/detention area with impermeable liners on the sides and bottom, engineered soils filter medial and perforated pipes surrounded with aggregates or crushed rocks, and header collection and outlet pipes should be considered. Treated water from the bio-retention/detention areas should be captured and discharged via a new stormdrain pipe connected the existing stormwater facilities.

3. Water form the bio-retention/detention area should not be allowed to cause flooding, impact or penetrate and saturate structural fills, graded or natural terrain, bearing and subgrade soils and wall backfills.

4. Locations away from the buildings, embankments, pavements, walls, structures and improvements greater than 10 feet are recommended, unless otherwise approved. Closer bio-retention/detention areas may be permitted if specifically designed and protective structures and moisture protection measures are provided.

5. Design should consider the removal of water within the bio-retention/detention areas, grassed swales, sedimentation ponds and buffer strips not more than 72 hours and vegetation carefully managed to prevent creating mosquito and other vector habitats.

VIII. LIQUEFACTION EVALUATIONS AND ANALYSIS

Added detail analysis were performed as a part of this study to estimate liquefaction potential at the project property. Groundwater conditions were measured between 13 to 21 feet below existing ground surfaces during our boring explorations. Historic High Groundwater (HHG) level at the project site is not known, but assumed herein as 10 feet below existing ground surfaces for the purpose of our engineering analysis.

Liquefaction potential at the project property was analyzed based on a representative underlying subsoil profile as recorded in the exploratory boring B-7 and site-specific design seismic parameters, as presented in this report. Typically, a minimum safety factor of 1.1 is considered adequate for residential constructions, however, a safety factor of 1.3 or greater is desired for flow failure potential or projects within or in close proximity of known fault zones and larger magnitude earthquake events. Potential liquefaction was evaluated to indicated maximum depths to bedrock (31 feet). A summery of results are presented in the following tables:

TABLE 17
SUMMARY OF LIQUEFACTION SCREENING ANALYSIS

Boring No.	Depth (ft)	Nm	N1(60)	Comments
B-7	3'	17	32	No-Liquefaction, Above Water Table
B-7	6'	25	37	No-Liquefaction, Above Water Table
B-7	9'	30	36	No-Liquefaction, Above Water Table
B-7	12'	35	43	N1(60) 〉 30, No-Liquefaction
B-7	15'	55	63	N1(60) 〉 30, No-Liquefaction
B-7	18'	33	35	N1(60) 〉 30, No-Liquefaction
B-7	21'	25	28	N1(60) 〈 30, Check for Liquefaction
B-7	24'	27	29	N1(60) 〈 30, Check for Liquefaction
B-7	27'	87	89	N1(60) 〉 30, No-Liquefaction
B-7	30'	68	66	N1(60) 〉 30, No-Liquefaction
Assumed HHG at 10 feet below ground surfaces. N1(60) = Nm CN CE CB CR CS.				

Safety factor for liquefaction potential of susceptible layers from the depths of 21 to 24 feet were then checked with the results summarized in the following table:

TABLE 18
SUMMARY OF LIQUEFACTION SAFETY FACTOR ANALYSIS

Boring No.	Depth (ft)	N1(60)	CRR	CSR	Safety Factor (SF)	Comments
B-7	21'	28	Infinite	0.233	Infinite	SF 〉 1.3, No-Liquefaction
B-7	24'	29	Infinite	0.235	Infinite	SF 〉 1.3, No-Liquefaction

Based on our analysis of representative subsoil profile as summarized above, underlying soils beneath the project property are not susceptible to liquefaction potential, and liquefaction and related ground failures will not be a significant geotechnical factor in the site redevelopment.

IX. CONCLUSIONS

Based upon the foregoing investigation, the planned commercial redevelopment, substantially as currently proposed, is feasible at the project site from a geotechnical viewpoint. The following geologic and soils conditions are unique to the study site and will most influence the design and construction phase of the project:

1. Adverse soils and geologic conditions including landslides, faults or significant shear zones are not present at or in the close proximity of the project property and are not considered a geotechnical factor in the planned redevelopment. The study site is not located near or within the Alquist - Priolo earthquake fault zone established by the State of California.

2. Liquefaction and associated secondary effects such as seismically induced settlements and lateral spreading are not a significant geotechnical concern at the project property

3. Project site terrain are relatively flat laying surfaces and new graded slopes are not anticipated. Stability of natural or large graded slopes at or near the project site will not be a geotechnical factor in the planned new constructions.

4. Significant grade alterations are not expected and final grades are expected to be very near the existing ground elevation. Minor ground modifications and fine grading efforts, however, are expected to construct new level building pads and achieve final design grades.

5. The project site is directly underlain by a relatively shallow mantle of surficial soils. Below, ancient alluvial deposits occur in soft conditions near the upper exposures becoming stiff and very dense to very tight with depth. Regrading of site soil mantle and upper exposures of ancient alluvial deposits will be required in order to construct safe and stable building and improvement surfaces as specified in the following sections. Underlying stiff and very dense to very tight ancient alluvium will adequately support the upper graded fills and planned new structures and improvements.

6. Regrading of the unsuitable soil mantle and upper exposures of the underlying ancient alluvium will be required over the entire site with all new foundations, structures and improvements supported on well compacted fills, as recommended herein. Consequently, cut-fill transition daylight condition is not expected to be a factor in the redevelopment of the project property.

7. Demolition works at the project site will consist of removal of all existing structures, improvements and asphalt surfaces. All demolitions and asphalt removals at the site should be completed in accordance with the approved demolition plans. Trash and construction debris generated from the site demolitions should be properly removed and disposed of.

8. Soils generated from the site excavations will include silty to clayey deposits with high fines content. Generated deposits may be considered for reuse as new compacted fills as approved in the field. Attempts should be made to bury the site more plastic silty to clay soils in deeper fills and place better quality sandy deposits within upper pad grades using select grading techniques. Silty to clayey soils with higher fines content are also typically moisture sensitive deposits which characteristically require added processing, mixing and grading efforts to manufacture suitable fill mixture and achieve the specified compaction levels.

   Import soil, if required to complete grading or necessary for achieving final design grades, should be good quality sandy (D.G.) deposits conforming to the requirements of this report as specified below. Import soils should be placed within the upper pad grades providing a cap of good quality sandy soils in the foundation zones.

9. Site near surface soils predominantly consist of clayey sand to sandy silty clay deposits (SC/CL) ranging to medium expansion potential (expansion index less than 91) based on ASTM D-4829 classification. Adverse affects of the site potentially expansive soils should be considered in the project designs and effective mitigation measures implemented during the construction of the project as specified in the fallowing sections. Placing sandy soils available from the site excavations and considering sandy (D.G.) import soils within the upper pad grades will help to mitigate adverse affects of site expansive soils and improve engineering properties of foundation bearing and subgrade soil.

   Actual classification and expansion characteristics of the finish pad grade soil mix should be conformed in the as-graded compaction report based upon proper testing of final bearing soils when rough finish grades are achieved.

10. Groundwater was encountered in three exploratory borings (B-7, B-9, and B-10) at depths of 13 to 21 feet below the existing ground surfaces at the time of our field investigation. Groundwater conditions at the project site are expected to seasonally fluctuate but significant rise impacting the recommended remedial grading operations or the long term performance of the new buildings and site improvements is not expected

Local shallow perched water, thought to be the result of recent rain storms, was also encountered at the depth of 3 feet in boring B-1. The noted condition may be expected to create some isolated wet soils requiring added grading efforts. Some minor dewatering for removals of intruding water may also become necessary and can not be ruled out. However, significant grading difficulties resulting of the shallow perched water conditions are not anticipated.

11. As with all developed sites, the proper control of surface drainage and storm water is a critical component to overall site and building performance. Run off 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 surfaces and should be avoided.

    Stormwater and drainage control facilities should be designed and installed for proper control and disposal of surface water as shown on the approved grading or drainage improvement plans. Stormwater Best Management Practices (BMP) should considered site geotechnical conditions with the associated bio-retention/detention area, filtration trenches, sedimentation ponds, detention basins, grassed swales and vegetated buffer strips appropriately sited in the areas of property with no impacts on the site structures and improvements.

12. Added care will be required to avoid any damages to the nearby existing improvements and underground utilities, off-site properties, and public right-of-way due to earthwork grading and construction works. Adequate excavation set-backs should be observed and temporary excavation slopes constructed as specified in the following sections.

13. Post construction settlement after completion of remedial grading works, as specified herein, is not expected to exceed approximately 1-inch and should occur below the heaviest loaded footing(s). The magnitude of post construction differential settlements, as expressed in terms of angular distortion, is not anticipated to exceed ½-inch between similar adjacent structural elements.

14. Soil collapse will not be a factor in the redevelopment of the project property provided our remedial grading recommendations are followed.

X. RECOMMENDATIONS

The following recommendations are consistent with the indicated geotechnical conditions at the project site. All recommendations provided herein should be confirmed and/or revised as necessary at the time of the geotechnical plan review phase when final design grades are known and detailed civil, architectural, and structural foundation plans are available. Additional or amended recommendations may be necessary and should also be provided at that time, as appropriate:

A. Remedial Grading and Earthworks

Planned construction areas are underlain by a relatively thin section of surficial soils which generally occur in soft and moist to very moist conditions overall. Below, ancient alluvium deposits occur in a soft condition within the upper exposures becoming increasingly stiff and very dense to tight with depth. Regrading of the soft to loose upper soil mantle and upper exposures of the underlying ancient alluvium will be required as specified below. All excavations, grading, earthworks, constructions, bearing and subgrade soil preparations should be completed in accordance with Chapter 18 (Soils And Foundations) and Appendix "J" (Grading) of the 2010 California Building Code (CBC), the Standard Specifications for Public Works Construction, City of Escondido Ordinances, the requirements of the governing agencies and following sections, wherever appropriate ans as applicable:

1. **Existing Underground Utilities:** All existing underground utilities including power, gas, sewer, drainage, water lines, buried tanks and structures within and near the planned new construction areas should be thoroughly pot-holed, identified and marked prior to the initiation of the actual earth works. In the event of conflict between the recommended extent or depths of removals and existing underground facilities or utilities, this office should be notified to provide modified recommendations as appropriate.

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

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

2. **Demolitions, Asphalt Removal, and Site Clearing:** Site demolition of existing structures and improvements should be completed as shown on the approved plans.

   Remove all trash, construction debris and other unsuitable/deleterious materials from all areas of proposed new fills, improvements, and structures plus a minimum 10 feet outside the perimeter where possible, and as directed in the field by the project geotechnical engineer or his designated representative. Existing asphalt surfaces may be pulverized or ground up to smaller than 1½-inch particles and used as a subbase layer under the project new asphalt improvements placed and compacted in accordance with the requirements of this report.

Demolition debris generated from the removal of existing structures, trash, vegetation, and debris should not be allowed to occur or contaminate new site fills and backfills, and should be removed from the site and properly disposed of. The prepared ground should be inspected and approved by the project geotechnical engineer or his designated representative.

3. **Stripping and Removals:** Site surficial compressible soil mantle and upper exposures of the underlying ancient alluvium in the areas of planned new fills, buildings, structures and improvements plus 10 feet outside the perimeter, where possible and as directed in the field, should be stripped (removed) to the depth of stiff and very dense to tight ancient alluvial deposits and placed back as properly compacted fills.

   Actual stripping depths should be established by the project geotechnical consultant based on field observations of subsurface exposures developed during the remedial grading operations. Based on the limited available subsurface explorations, stripping depths are expected to be on the order of 3 to 5 feet below existing surfaces, or should be extended at least 12 inches below the bottom of deepest footing(s), whichever is more. There should be a minimum of 12 inches of compacted fills below the bottom of the deepest footing(s). Deeper removals may also be necessary based on the actual field exposures and should be anticipated. Bottom of all removals should be additionally prepared, ripped and recompacted in-place to a minimum depth of 8 inches as directed in the field. All grounds steeper than 5:1 receiving fills/backfills should also be properly benched and keyed as directed in the field.

   Obtaining special permit(s) from the adjacent property owner(s) or public agencies may be required for any off-site grading or work within the public right-of-way.

4. **Setbacks and Temporary Construction Slopes:** Excavations and removals adjacent to existing structures, improvements and off-site public and private properties should be performed under observations of the project geotechnical engineer. Undermining off-site properties, right-of-ways, existing improvements and structures by the excavations and removal operations shall not be allowed. Temporary construction slopes should be adequately setback from the existing structures and improvements as approved or directed in the field.

   Temporary trenching and excavation slopes less than 5 feet high maximum may be constructed at near vertical gradients, unless otherwise directed in the field. Trench and excavation slopes greater than 5 feet should constructed at 1:1 gradients, unless otherwise approved. The remaining wedge of soil should then be properly benched and new fill tightly keyed-in as the backfill placement progresses.

Vertical excavations and trenching more than 5 feet high maximum should be provided with adequate shoring and trench shield support, unless otherwise approved or specified. Protection of all existing pipes, utilities, conduits, underground improvements and nearby structures, including those in the public right-of-way located within the zone of influence of temporary excavations and trenching will also be required by the project contractor.

All temporary construction slopes require continuous geotechnical engineering observations during the construction operations. Additional recommendations including revised temporary slope gradients, set-backs, completing earthwork constructions in limited sections and the need for temporary shoring or trench shield support should be given at that time, as 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.

5. **Fill and Backfill Materials, Shrinkage and Import Soils:** In general, soils generated from the site stripping, excavations and removals may be reused as new fills provided they are adequately processed and manufactured into a clean uniform mixture free of vegetation, organic matter, trash, construction debris and unsuitable materials as approved in the field. However, site soils include plastic clayey to silty deposits which typically require additional processing, mixing and moisture conditioning efforts in order to manufacture a uniform homogeneous mixture suitable for reuse as new compacted fills. Plastic clayey to silty soils are not suitable for wall and trench backfills and good quality sandy granular import soils should be used for this purpose. Locally some wet soils should also be expected requiring added aerating, drying and processing to achieve near optimum water contents.

   Attempts should be made to burry the site plastic clayey to silty soils with high fines content in deeper fills and place more sandy materials available for the onsite excavations within the upper pad grades, using select grading techniques. For this purpose, some stockpiling of site sandy soils for later placement within upper grades may be required.

   Based upon our analyses and experience with similar earth deposits, site soil mantle and upper exposures of ancient alluvial deposits recommended herein for regrading may be expected to shrink approximately 5% to 15% on a volume basis when compacted as specified herein. Import soil if required to complete remedial grading and achieve final design grades, or used as wall and trench backfills 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.

Import sandy soils should be placed within the upper grades and to cap the building pads.

6. **Fill and Backfill Processing, Placement, and Compaction:** Uniform and stable fill support should be constructed underneath the project new building pads and improvement areas by the remedial grading and earthwork operations. For this purpose, site soils should be adequately processed, thoroughly mixed, moisture conditioned to slightly (2%-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 90% of corresponding laboratory maximum dry density per ASTM D-1557, unless otherwise specified.

   Onsite plastic clayey to silty soils are not considered suitable for wall and trench backfills and good quality sandy granular import soils should be considered. Wall and trench backfills should also be adequately processed, moisture conditions to near optimum levels and mechanically compacted to a minimum of 90% compaction levels (per ASTM D-1557), unless otherwise specified or directed in the field.

7. **Drainage and Erosion Control:** A critical element to the continued stability of the graded building pads and improvements is adequate surface drainage. This can most effectively be achieved by installation of appropriate flood and drainage control systems. Flood and stormwater control structures should be installed per the project drainage improvement plans. Building pad surface run-off should be collected and directed away from the planned buildings and improvements to a selected location in a controlled manner. Over-watering of the site vegetation should also not be allowed. Only the amount of water to sustain vegetation should be provided. Area drains should be installed.

   Stormwater Best Management Practices (BMP) should be designed and constructed considering site specific geotechnical conditions and requirements of this report. Temporary erosion control facilities and silt fences should be installed during the construction phase periods and until landscaping is fully established and as indicated and specified on the approved project grading/erosion improvement plans.

8. **Engineering Observations:** All grading and earthworks operations including stripping, trenching, excavations and removals, suitability of earth deposits used as compacted fills and backfills, processing, spreading and placement, and

compaction procedures should be continuously observed and tested by the project geotechnical consultant and presented in the final as-graded compaction report. The nature of finished bearing and subgrade soils should be confirmed in the final compaction report at the completion of grading.

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

* Initial observation - After the clearing limits have been staked but before demolition or grading works starts.

* Stripping and bottom of excavation observation - After stripping and exposure of bottom of excavations receiving new fill or backfill but before fill or backfill is placed.

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

* Fill/backfill observation - After the fill/backfill placement is started but before the vertical height of fill/backfill exceeds 2 feet. A minimum of one test shall be required for each 100 lineal feet maximum in every 2 feet of vertical gain, with the exception of wall and trench backfills where a minimum of one test shall be required for each 30 lineal feet maximum. Onsite expansive silty to clayey soils are not considered suitable for wall backfills and good quality sandy import soils should be considered. Wall backfills should also be mechanically compacted to a minimum of 90% compaction levels unless otherwise specified or directed in the field. Finish rough and final pad grade tests shall be required regardless of fill thickness.

* Foundation trench and slab subgrade soils observation - After the foundation trench excavations but before steel placement for adequate footing embedment, and proper moisture and specified compaction levels.

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

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

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

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

B. Foundations and Floor Slabs

Project pad constructions may be anticipated to develop clayey sand to sandy silty clay deposits (SC/CL) deposits ranging to medium expansion potential (expansion index less than 91) within upper pad grades. The following minimum recommendations are consistent with the anticipated foundation bearing soil materials 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 remedial grading based on the engineering characteristics of the foundation bearing soils and as-graded site geotechnical conditions, and presented in the final compaction report.

1. 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.

2. Continuous stem wall foundations, and turned-down footings should be sized at least 18 inches wide and 24 inches deep. 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.

3. Interior floor slabs for commercial type buildings 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 moisture barrier (Stego) placed mid-height in the sand. Alternatively, a 4-inch thick base of compacted ½-inch clean aggregate provided with the vapor barrier (minimum 10-mil Stego) in direct contact with (beneath) the concrete may also be considered provided a concrete mix which can address bleeding, shrinkage and curling is used.

   Provide "softcut" contraction/control joints consisting of sawcuts spaced 10 feet on centers each way for all interior slabs. Cut as soon as the slab will support the weight of the saw and operate without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. The sawcuts should be a minimum of 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 21 may be used as a general guideline.

4. 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 2% to 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.

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

C. Soil Design Parameters

The following soil design parameters are based upon 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:

* Design soil unit weight = 133 pcf.
* Design angle of internal friction of soil = 33 degrees.
* Design active soil pressure for retaining structures = 40 pcf (EFP), level backfill, cantilever, unrestrained walls.
* Design at-rest soil pressure for retaining structures = 60 pcf (EFP), non-yielding, restrained walls.
* Design passive soil resistance for retaining structures = 400 pcf (EFP), level surface at the toe.
* Design coefficient of friction for concrete on soils = 0.30.
* Net allowable foundation pressure (minimum 18 inches wide by 24 inches deep footings) = 2000 psf.
* Allowable lateral bearing pressure (all structures except retaining walls) = 200 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 minimum safety factor of 2 may be considered for sliding stability particularly where sensitive structures and improvements are planned near or on top of retaining/basement walls.

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

* The indicated net allowable foundation pressure provided herein was determined based on a minimum 18 inches wide by 24 inches deep footings and may be increased by 20% for each additional foot of depth and 10% for each additional foot of width to a maximum of 5500 psf. The allowable foundation pressures provided herein also apply to dead plus live loads and may be increased by one-third for wind and seismic loading.

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

D. Exterior Concrete Slabs and Flatworks

1. All exterior slabs (walkways, flatworks, patios) supported on potentially expansive soils should be a minimum 4 inches in thickness reinforced #3 bars placed at 15 inches on center each way carefully placed at mid-height in the slab. Final designs should be confirmed and/or revised, as necessary, based on actual expansion potential of final subgrade soils. The finish subgrade soils should be compacted (or recompacted as necessary) to minimum 90% compaction levels at the time of final subgrade preparations and prior to steel placement.

2. Slab reinforcement laying on subgrade will be ineffective and soon corrode due to lack of adequate concrete cover. Slab reinforcements should also extend through the construction (cold) 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 15 inches on center maximum placed mid-height in the slab (9 inches on either side of the joint).

   In order to enhance performance of exterior slabs and flatworks supported on expansive and moisture sensitive subgrade soils, a minimum 8 inches wide by 8 inches deep thickened edge reinforced with a minimum of 1-#3 continuous bar near the bottom should be considered along the slab perimeter.

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 the slab will support weight and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi

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

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 and/or revised as necessary 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. Pavement Design

Onsite paving improvements are expected to include asphalt parking stalls and drive lanes as well as PCC trash enclosure pads, receiving truck ramp and driveways. The following are appropriate:

1. **Asphalt Pavements:** Specific pavement designs can best be provided at the completion of remedial grading work based on R-value testing of the actual finish subgrade soils. However, the following structural section may be utilized for initial cost estimating purposes based on a tested R-Values of 8 and 27. The minimum structural sections tabulated below, or the minimum structural section required by the City of Escondido, whichever is more, should be considered. Actual design will also depend on the design TI and approval of the City of Escondido.

TABLE 19

Design R-value	Design Traffic Index (TI)			
	4.5 (Parking Stalls)	**5.0** (Drive Lanes)	**6.0** (Light Trucks)	**7.0** (Fire Lane/Heavy Trucks)
27	3" AC over 6" AB	3" AC over 6" AB	3" AC over 9" AB	4" AC over 10" AC
8	3" AC over 7.5" AB	3" AC over 9.5" AB	4" AC over 11" AB	4" AC over 15" AB
Aggregate base (AB) materials shall meet or exceed Caltrans Class 2 specifications.				

Final pavement design should be confirmed ans/or revised as necessary based on actual R-value testing of the final subgrade soil mixture performed at the completion of subgrade preparations. Revised structural sections should be anticipated.

Aggregate base materials should be compacted to a minimum of 95% of the corresponding maximum dry density (ASTM D-1557). Subgrade soils beneath all asphalt pavings including roadway, drive lanes, parking stalls and driveways should also be compacted to a minimum 95% of the corresponding maximum dry density within the upper 12 inches.

2. **PCC Pavings, Concrete Pads and Receiving Truck Ramps:** Commercial PCC pavings supported on potentially expansive subgrade soils should be a minimum of 5½ inches in thickness, reinforced with #3 reinforcing bars at 15 inches on centers each way placed near the slab mid-height. PCC pavements supported on expansive subgrade should also be provided with a minimum 8 inches wide by 8 inches deep thickened edge reinforced with minimum 1-#3 continuous bar placed near the bottom along the perimeter. Subgrade soils beneath the PCC paving surfaces should be recompacted to the minimum 90% compaction levels within the upper 6 inches at the time subgrade fine grading works.

   The trash enclosure concrete pads should be a minimum of 5½ inches in thickness and reinforced with #4 reinforcing bars spaced 18 inches on center each way, placed near the slab mid-height. The concrete pad should also be provided with a minimum 12 inches wide by 12 inches deep thickened edge at the opening reinforced with minimum 1-#4 bar top and bottom.

   PCC receiving truck ramp and loading dock pavement section should consist of a minimum of 6 inches PCC over 6 inches of 95% compacted Class 2 crushed aggregate base (AB) over 90% compacted subgrade. PCC receiving truck ramp and loading dock pavements should be reinforced with #4 reinforcing bars at 18 inches on centers each way, near the slab mid-height. The PCC ramp/loading dock pavement should be provided with a minimum of 12 inches wide by 12 inches deep thickened edge reinforced with minimum 1-#4 continuous bar placed near the top and bottom at grade breaks as well as the beginning and end perimeters, as appropriate.

   Slab reinforcement lying on subgrade will be ineffective and corrode soon after construction. Reinforcing bars should be correctly placed extending through the construction (cold) 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 (9 inches on either side of the joint) #3 dowels (dowel baskets) at 15 inches on centers placed near the slab mid-height. Use minimum 560-C-3250 concrete per Standard Specifications for Public Works Construction (Green Book) for all onsite PCC pavings.

Provide "tool joint" or "softcut" contraction/control joints spaced 10 feet on center (not to exceed 15 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as the slab will support the weight and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum 1¼-inch 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.).

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

   Base layer under curb and gutters should be compacted to a minimum of 95%, while subgrade soils under curb and gutters, and base and subgrade under sidewalks should be compacted to a minimum of 90% compaction levels, unless otherwise specified. Additional and/or more specific recommendations should be given in the final as-graded compaction report.

F. <u>General Recommendations</u>

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

2. Adequate staking and grading control is a critical factor in properly completing the recommended remedial and site grading operations. Grading control and staking should be provided by the project grading contractor or surveyor/civil engineer, and is beyond the geotechnical engineering services. Staking should apply the required setbacks shown on the approved plans and conform to setback requirements established by the governing agencies and applicable codes for off-site private and public properties and property lines, utility easements, right-of-ways, nearby structures and improvements, leach fields and septic systems, and graded embankments. Inadequate staking and/or lack of grading control may result in 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 22. 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 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. Onsite soils are potentially expansive and include moisture sensitive silty to clayey soils. These deposits can experience movements and undergo volume changes upon wetting and drying. Consequently, 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 designs and location of associated drainage improvements should be completed considering characteristics of onsite potentially expansive soils. Surface water should not be allowed to infiltrate into the underlying bearing and subgrade soils, wall backfills or impact graded improvement surfaces and 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. shall be considered 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 Escondido 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 pertinent reports and plans, current subsurface exploratory investigations and 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 in 20 feet may develop as a result of normal concrete shrinkage (according to the American Concrete Institute).

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

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

The project 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 #12-134-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

S. Mehdi S. Shariat
GE #2885

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

Steven J. Melzer
CEG #2362

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

Distribution: Addressee (3, e-mail)
Excel Engineering; Mr. Mike Levin (3, e-mail)

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.

- Highway Design Manual, Caltrans. Fifth Edition.

- Corrosion Guidelines, Caltrans, Version 1.0, September 2003.

- California Building Code, Volumes 1 & 2, International Code Council, 2010.

- "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.

REFERENCES (continued)

- "Soil Mechanics," Naval Facilities Engineering Command, DM 7.01.

- "Foundations & Earth Structures," Naval Facilities Engineering Command, DM 7.02.

- "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.

- Kennedy, M.P. and Peterson, G.L., 1975, Geology of the San Diego Metropolitan Area, California: California Division of Mines and Geology Bulletin 200, 56p.

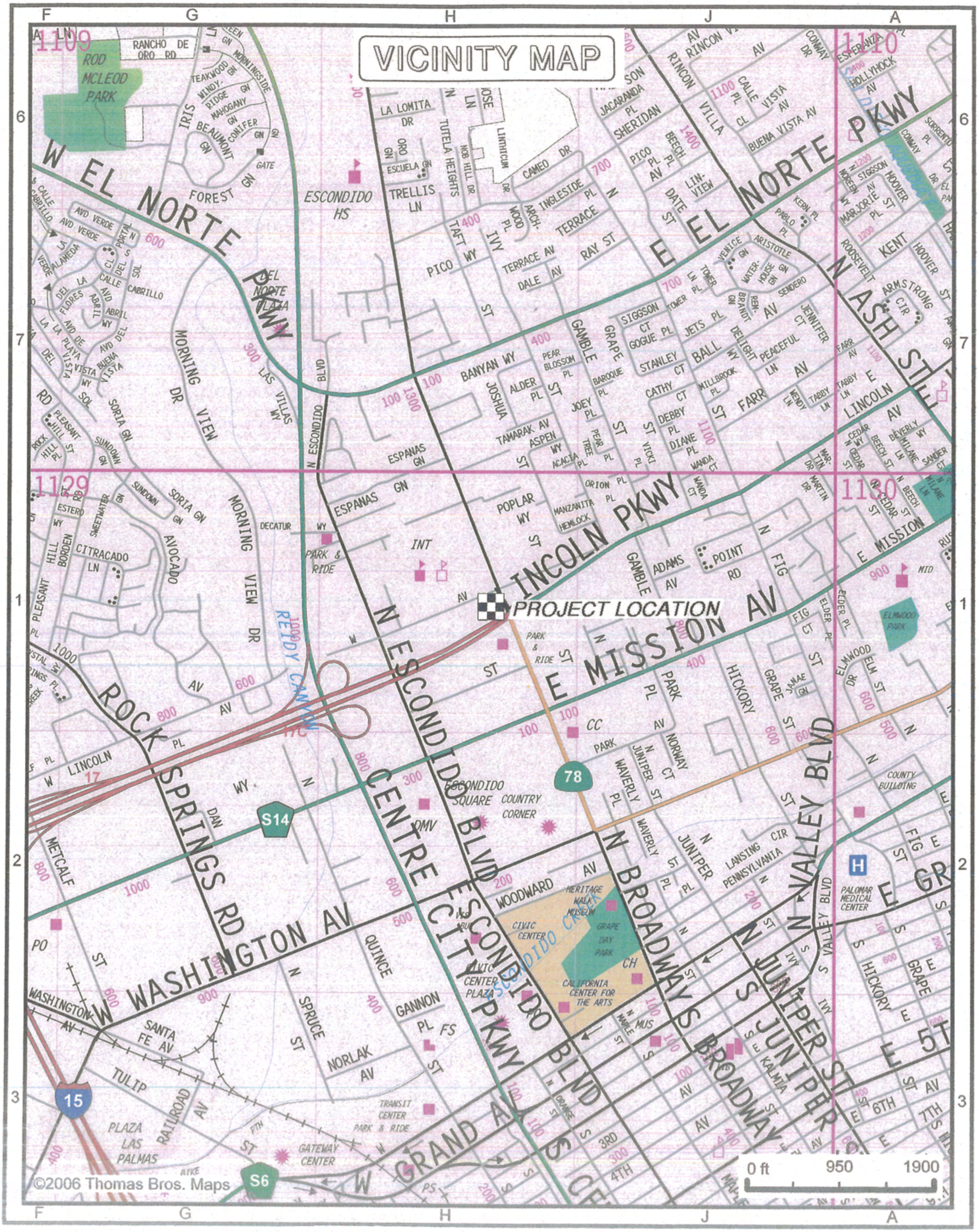
- Kennedy, M.P. and Tan, S.S., 1977, Geology of National City, Imperial Beach and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California, Map Sheet 24, California Division of Mines and Geology, 1:24,000.

- Kennedy, M.P., Tan, S.S., Chapman, R.H., and Chase, G.W., 1975, Character and Recency of Faulting, San Diego Metropolitan Areas, California: Special Report 123, 33p.

- "An Engineering Manual For Slope Stability Studies," J.M. Duncan, A.L. Buchignani and Marius De Wet, Virginia Polytechnic Institute and State University, March 1987.

- "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.



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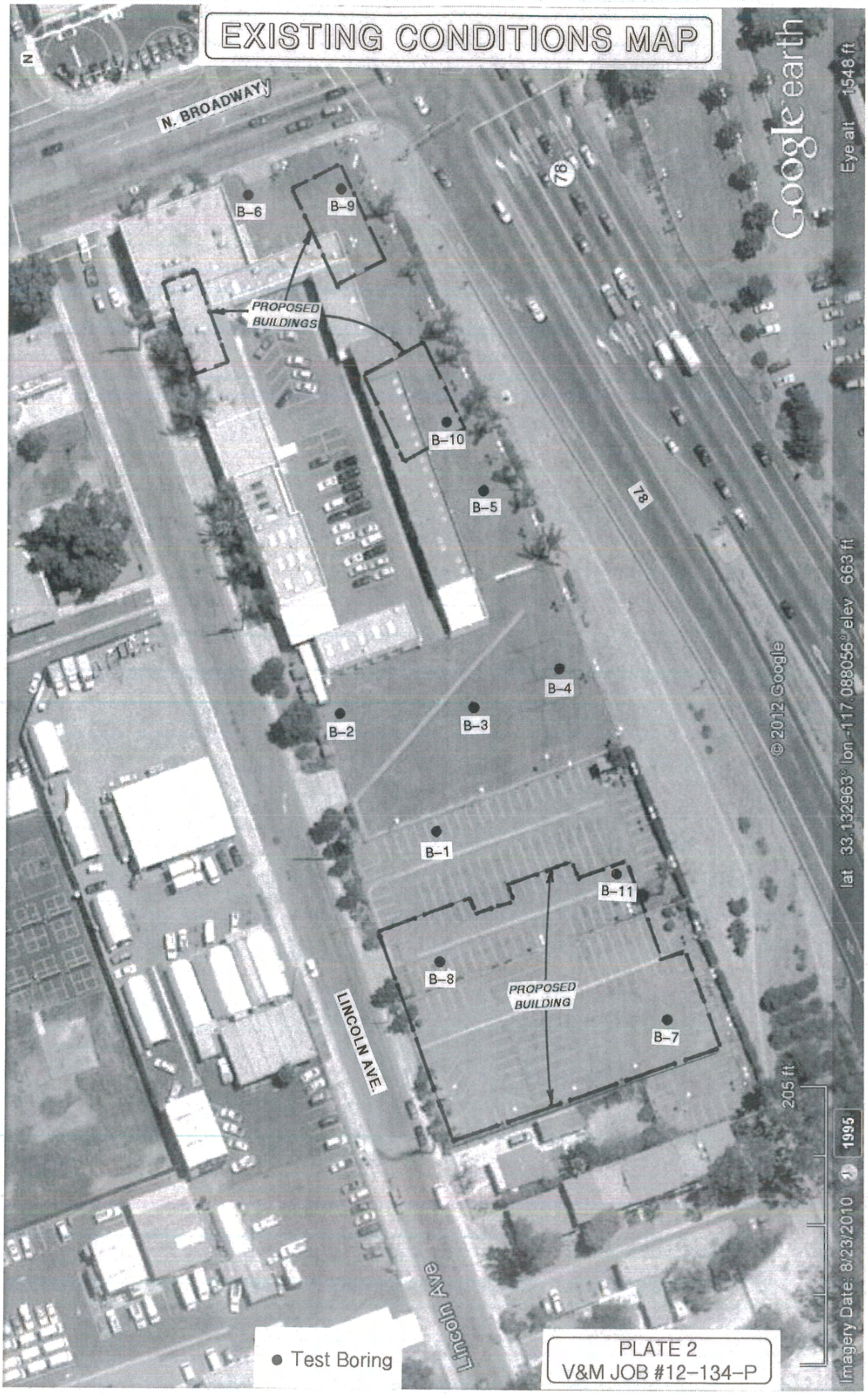
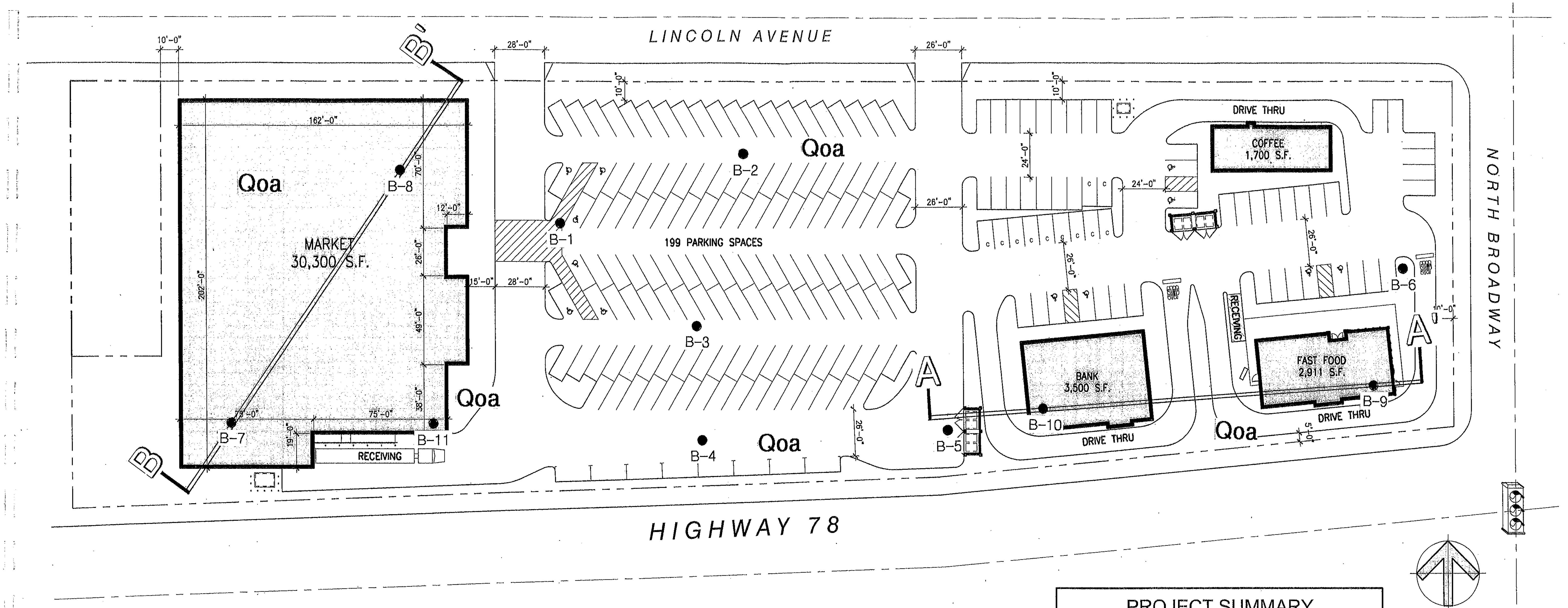
A.P.N.'s 229–121–09, 10, 11, 12, 13, & 14

PLATE 1



APPROX. SCALE: 1" = 100'



EXPLANATION

- ● Test Boring
- Geologic Cross-Section
- Qoa Ancient Alluvium

PROJECT SUMMARY

LOT AREA:	APPROX. 160,989 S.F.
BUILDING AREAS	
MARKET:	30,300 S.F.
BANK WITH DRIVE THRU:	3,500 S.F.
FOOD WITH DRIVE THRU:	2,911 S.F.
SPECIALTY COFFEE WITH DRIVE THRU:	1,700 S.F.
TOTAL BUILDING AREA:	38,411 S.F.
TOTAL PARKING REQUIRED @ 1/200:	192 SPACES
TOTAL PARKING PROVIDED:	199 SPACES
PARKING RATIO PROVIDED:	1 SPACE/193 S.F.

PACIFIC DEVELOPMENT PARTNERS, LLC
393.4141 501 SANTA MONICA BLVD
393.4103 SANTA MONICA, CA 90401

GATEWAY CENTER

990 N BROADWAY
ESCONDIDO, CA 92026

AN EXCITING NEW MULTI RETAIL CENTER AT THE GATEWAY TO THE HIGHWAY 78

PROPOSED DEVELOPMENT PLAN

SCHEME
B
2012. JAN. 23

310. 838. 9766
PERUZZI ARCHITECTS

PLATE 3
V&M JOB #12-134-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 FRACTION 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.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION 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 or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND 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 AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

GRAIN SIZES U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS, GRAVELS AND 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 AND 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 2 inch 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, CA 92029-1229

KEY TO EXPLORATORY BORING LOGS
Unified Soil Classification System (ASTM D-2487)

PROJECT NO.		KEY
12-136-P		

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

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: **No caving. Perched water at 3 feet.**

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 (%)
			2.5-Inches asphalt pavement.						
		SP	**FILL: (af):** Fine to coarse sand. Brown color. Moist. Firm. ST-1						
1									
		SC-CL	Fine sandy silty clay. Red brown color. Very moist to wet. Low to medium plsatic. Very soft. ST-3						
2									
3			Perched groundwater at 3 feet (contact).						
		CL-CH	**ANCIENT ALLUVIUM (Qoa):** Fine sandy to silty clay. Red brown color. Moist to very moist. Medium to high plastic. Stiff. ST-3						
4									

Bottom of borehole at 4.0 feet.

STANDARD PENETRATION TEST · MODIFIED CALIFORNIA SAMPLER · BULK SAMPLE · GROUND WATER

PLATE 4

VINJE & MIDDLETON ENGINEERING, INC.

BORING: B-2

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: No caving. No groundwater.

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 (%)
			2.5-Inches asphalt pavement.						
		SP	**FILL: (af):** Fine to coarse sand. Brown color. Moist. Firm. ST-1						
1 2		SC-CL	Clayey silty fine sand to fine sandy clay. Red brown color. Moist. Soft. ST-2						
3		CL-CH	**ANCIENT ALLUVIUM (Qoa):** Silty to sandy clay. Red brown color. Moist. Medium to high plastic. Stiff. ST-3						
4									

Bottom of borehole at 4.0 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | ∇ GROUND WATER

PLATE **5**

VINJE & MIDDLETON ENGINEERING, INC.

BORING: B-3

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: No caving. No groundwater.

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 (%)
			2.5-Inches asphalt pavement.						
1		SP	**FILL (af):** Fine to coarse sand. Grey color. Moist. Firm. ST-1						
2		SC-CL	**ANCIENT ALLUVIUM (Qoa):** Silty to clayey fine sand. Red brown color. Moist. Firm. ST-2						
3									

Bottom of borehole at 3.0 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 6

VINJE & MIDDLETON ENGINEERING, INC. **BORING: B-4**

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: No caving. No groundwater.

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 (%)
			2.5-Inches asphalt pavement.						
1		SP	**FILL (af)** Fine to coarse sand. Grey color. Slightly moist to moist. Firm. ST-1						
2 3		SC-CL	**ANCIENT ALLUVIUM (Qoa):** Silty clayey fine sand. Red brown color. Very moist to wet. Low plastic. Soft. ST-2						
4		CL-CH	Silty clay. Red brown color. Moist. Medium to high plastic. Stiff. ST-3						

Bottom of borehole at 4.0 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 7

VINJE & MIDDLETON ENGINEERING, INC.

BORING: B-5

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: **No caving. No goundwater.**

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 (%)
			4-Inches asphalt pavement.						
1, 2, 3		SC-CL	**FILL (af):** Clayey silty fine sand. Red brown color. Very moist to wet. Low plastic. Very soft. ST-2						
4		CL-CH	**ANCIENT ALLUVIUM (Qoa):** Fine sandy clay. Red brown color. Moist to very moist. Medium to high plastic. Firm. ST-3	Bulk					

Bottom of borehole at 4.5 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 8

VINJE & MIDDLETON ENGINEERING, INC. **BORING: B-6**

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/2/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Daisy Drilling **DRILL METHOD:** Truck-mounted rotary drill. Solid-stem auger.

SAMPLE METHOD:

REMARKS: No caving. No groundwater.

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 (%)
			4-Inches asphalt pavement.						
1		SP	**FILL (af):** Fine to coarse sand. Grey color. Moist. Firm. ST-1						
2, 3, 4		SC	**ANCIENT ALLUVIUM (Qoa):** Silty to clayey fine sand. Red brown color. Moist. Loose to firm. ST-2						

Bottom of borehole at 4.0 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 9

VINJE & MIDDLETON ENGINEERING, INC. BORING: B-7

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/14/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-mounted rotary drill. Hollow stem auger.

SAMPLE METHOD: 140 LB. Hammer droppped 30-inches by automatic trip-hammer. 5-Foot AW rods.

REMARKS: No caving. Groundwater at 19 feet (static level at completion of drilling: 15 feet). Backfill: well grout w/ concrete cap.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
	SC-CL	3-Inches asphalt pavement over approximately 8-inches aggregate base.					
5	SC-CL	**FILL (af):** Sandy clay to clayey sand. Red brown color. Moist. Medium plastic. Firm. ST-2	5-8-9 (17)				
	CL-CH	**ANCIENT ALLUVIUM (Qoa):** Silty clayey fine sand. Dark brown color. Moist. Medium dense to dense. ST-2	8-8-17 (25)				
10		Fine sandy to silty clay. Red brown color. Moist to very moist. Very stiff to hard. High plastic. ST-3	6-13-17 (30)				
	SC-ML	Sandy Silt to clayey fine sand. Red brown color. Some rust-colored staining. Moist. Dense to very dense. ST-2	7-13-22 (35)				
15		Static groundwater at completion of drilling: 15 feet. At 15 feet becomes dense to very dense. Slightly micaceous.	12-18-37 (55)				
		Groundwater at 19 feet.	6-14-19 (33)				
20	SP	Fine to medium sand. Slightly micaceous. Brown / red brown color. Wet to saturated. Medium dense to dense. ST-4	6-11-14 (25)				
25		Grades fine to coarse grained at 25 feet. Includes local pebbles. Tight. Continued wet to saturated.	14-13-14 (27)				
		Blow counts for sample at 27 feet inflated due to rock fragments near the lower contact.	16-43-44 (87)				
30	GC	Clayey fine to coarse sand with rock fragments. Red brown color. Moist. Tight to very tight. ST-5	16-21-47 (68)				
	GP	**BEDROCK (Kgb):** Gabbroic rock. Coarse grained. Grey color. Weathered. Hard. Very dense. ST-6	50/6"				

Bottom of borehole at 33.5 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 10

VINJE & MIDDLETON ENGINEERING, INC. **BORING: B-8**

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/14/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-mounted rotary drill. Hollow stem auger.

SAMPLE METHOD: 140 LB. Hammer droppped 30-inches by automatic trip-hammer. 5-Foot AW rods.

REMARKS: No caving. No groundwater. Backfill: well grout w/ concrete cap.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
	SP	2.5-Inches asphalt pavement.						
	CL-CH	**FILL (af):** Fine to coarse sand. Grey color. Slightly moist. Firm. ST-1	Bulk					
		Silty sandy clay. Red brown color. Moist to very moist. Medium plastic. Very firm to hard. ST-3	Modified California	27-50/3"	13	117.5	87	78
5	SC	**ANCIENT ALLUVIUM (Qoa):** Silty clayey fine sand. Red brown color. Some rust-colored staining. Slightly moist. Locally cemented to very tight. Dense to very dense. ST-2						
		Continued very tight at 6 feet. Slightly moist. Somewhat slow drilling.	Modified California / Bulk	10-31	15	119.9	89	96
10		At 10 feet moist. Very tight. Very dense.	SPT	26-50/2"				
15			Modified California	28-50/3"	12	122.5	91	83
	GP	**BEDROCK (Kgb):** Gabbroic rock. Fine to coarse grained. Grey color. Weathered. Hard. Very dense. ST-6						
			SPT	50/5"				

Bottom of borehole at 19.0 feet.

STANDARD PENETRATION TEST — MODIFIED CALIFORNIA SAMPLER — BULK SAMPLE — GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **BORING: B-9**

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/14/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-mounted rotary drill. Hollow stem auger.

SAMPLE METHOD: 140 LB. Hammer droppped 30-inches by automatic trip-hammer. 5-Foot AW rods.

REMARKS: No caving. Groundwater at 21 feet (static level at completion of drilling: 15 feet). Backfill: well grout w/ concrete cap.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0	SP	3-Inches asphalt pavement.					
	SC-CL	**FILL (af):** Silty fine to coarse sand. Grey color. Slighly moist. Firm. ST-1	5-8	13	121.8	91	87
5	SC	Silty clayey sand to fine sandy clay. Red brown color. Moist. Loose to soft. ST-2	50/6"	12	134.5	88	73
	CL-CH	**ANCIENT ALLUVIUM (Qoa):** Clayey silty fine sand. Red brown color. Some rust-colored staining. Dry to slightly moist. Locally cemented to very tight. Very dense. ST-2					
10	SC	Fine sandy to silty clay. Red brown color. Moist. Stiff. Medium plastic. ST-3	5-9-15 (24)				
	CL-CH	Silty clayey fine sand. Slightly micaceous. Brown / red brown color. Moist. Tight. Medium dense to dense. ST-2					
15		Fine sandy silty clay. Brown color. Moist. Very stiff to indurated. Medium to high plastic. ST-3 Static groundwater level at completion of drilling: 15 feet.	12-19				
				25	104,9	81	100
20	SC	Silty clayey fine sand. Slightly micaceous. Red brown color. Moist. Medium dense. ST-2 Groundwater at 21 feet.	5-7-11 (18)				
25			28-50/4"	15	119.0	88	94
	SP-GP	**BEDROCK (Kgb):** Gabbroic rock. Coarse grained. "Salt & pepper" color. Hard. Very dense. ST-6	50/6"				

Bottom of borehole at 28.5 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE 12

VINJE & MIDDLETON ENGINEERING, INC. BORING: B-10

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/15/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-mounted rotary drill. Hollow stem auger.

SAMPLE METHOD: 140 LB. Hammer droppped 30-inches by automatic trip-hammer. 5-Foot AW rods.

REMARKS: No caving. Perched groundwater at 13-16 feet (static level at completion of drilling: 17 feet). Backfill: well grout w/ concrete cap.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0		4.5-Inches asphalt pavement.					
	SC-CL	**FILL (af):** Silty clayey sand to fine sandy clay. Red brown color. Moist to very moist. Loose to soft. ST-2					
		ANCIENT ALLUVIUM (Qoa):	6-22-50/6"				
5	SC-ML	Sandy silt to clayey fine sand. Red brown color. Some rust-colored staining. Very moist to 4 feet. Soft. Low plastic. ST-2 Slightly moist below 4 feet. Tight. Very dense.	50/6"	13	Sample Disturbed	-	-
		Becomes moist at 8 feet. Slightly micaceous. Medium dense to dense.					
10			11-16	17	117.9	88	100
		Perched groundwater at 13 to 16 feet (perched atop clay deposit at 16 feet).					
15			2-4-6 (10)				
	CL-CH	Silty to fine sandy clay. Brown / red brown color. Moist. Very stiff. High plastic. ST-3 Static groundwater at completion of drilling: 17 feet.					
20		At 20 feet moist to very moist. Continued very sfiff and high plastic.	7-9	23	103.7	80	97
	SP	Silty fine to medium sand. Brown color. Moist to very moist. Tight. Dense. ST-4	9-12-18 (30)				
25	SP-GP		24-29-44 (73)				
		BEDROCK (Kgb): Gabbroic rock. Coarse grained. "Salt & pepper" color. Weathered. Very dense. ST-6					

Bottom of borehole at 26.5 feet.

STANDARD PENETRATION TEST · MODIFIED CALIFORNIA SAMPLER · BULK SAMPLE · GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. BORING: B-11

PROJECT: Proposed Commercial Development **CLIENT:** Pacific Development Partners, LLC

PROJECT NUMBER: 12-136-P **PROJECT LOCATION:** 999 North Broadway, Escondido, CA.

DATE DRILLED: 3/15/2012 **BOREHOLE DIA:** 8" **LOGGED BY:** SJM

CONTRACTOR: Scott's Drilling **DRILL METHOD:** Truck-mounted rotary drill. Hollow stem auger.

SAMPLE METHOD: 140 LB. Hammer droppped 30-inches by automatic trip-hammer. 5-Foot AW rods.

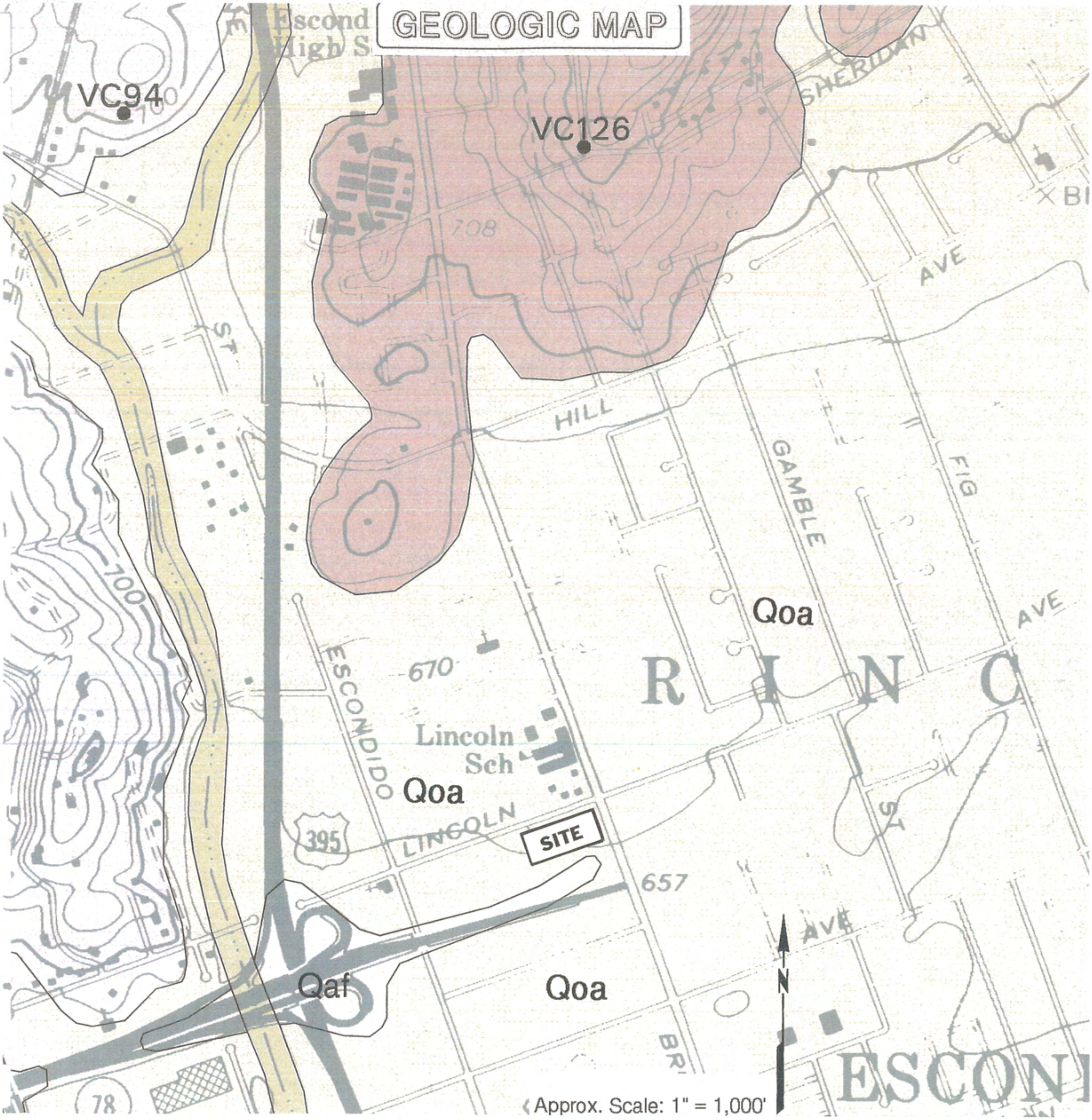
REMARKS: No caving. No groundwater. Backfill: well grout w/ concrete cap.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
	SP	3-Inches asphalt pavement.						
	SC	**FILL (af):** Silty fine to coarse sand. Brown color. Slightly moist. Firm. ST-1						
	CL-CH	Silty clayey sand. Red brown color. Very moist. Loose to soft. ST-2	Modified California Sampler; Bulk Sample	8-13	17	117.9	88	92
5		**ANCIENT ALLUVIUM (Qoa):** Fine sandy to silty clay. Red brown color. Moist. Very stiff. Medium to high plastic. ST-3	Modified California Sampler	14-22	14	120.6	90	92
	SC	Silty clayey fine sand. Red brown color. Some rust-colored staining. Moist. Dense. ST-2						
10		At 10 feet cemented to very tight. Somewhat slow drilling. Dense to very dense.	Standard Penetration Test	13-24-37 (61)				
15			Modified California Sampler	19-39	16	119.6	89	100
		Some rock fragments near the lower contact.						
	SP-GP	**BEDROCK (Kgb):** Gabbroic rock. Fine to coarse grained. Grey color. Weathered. Very dense. ST-6	Modified California Sampler	50/6"				

Bottom of borehole at 18.5 feet.

STANDARD PENETRATION TEST | MODIFIED CALIFORNIA SAMPLER | BULK SAMPLE | GROUND WATER

PLATE **14**

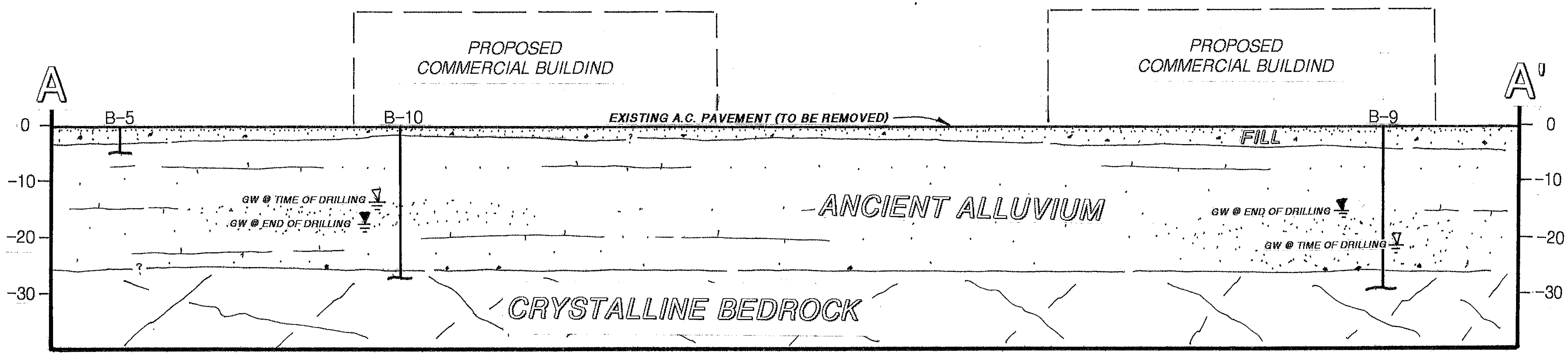


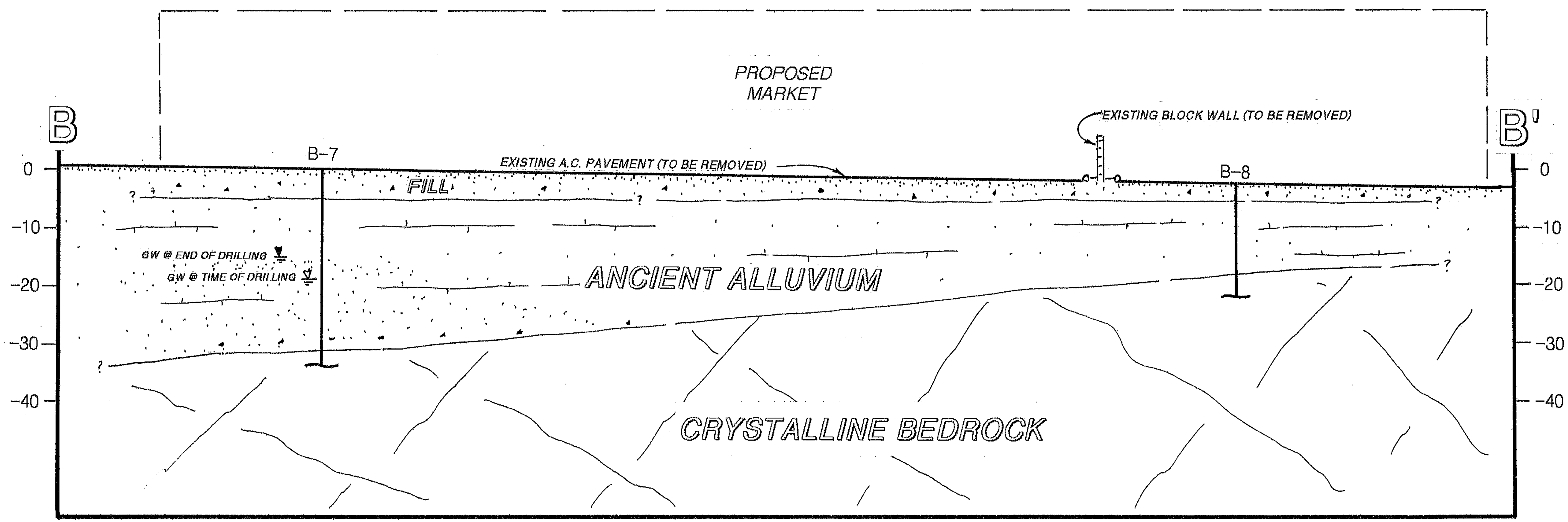
Qoa	Older alluvial flood plain deposits (Pleistocene, younger than 500,000 years) - Mostly moderately well consolidated, poorly sorted , permeable flood plain deposits.

Reprinted from the Geologic Map of the Valley Center 7.5' Quadrangle, San Diego County, California: A Digital Database, Version 1.0 by Michael P. Kennedy (Digital Database by Kelly R. Ruppert and Anne G. Kennedy) 1999.

PLATE 15
V&M JOB #12-136-P

GEOLOGIC CROSS-SECTIONS

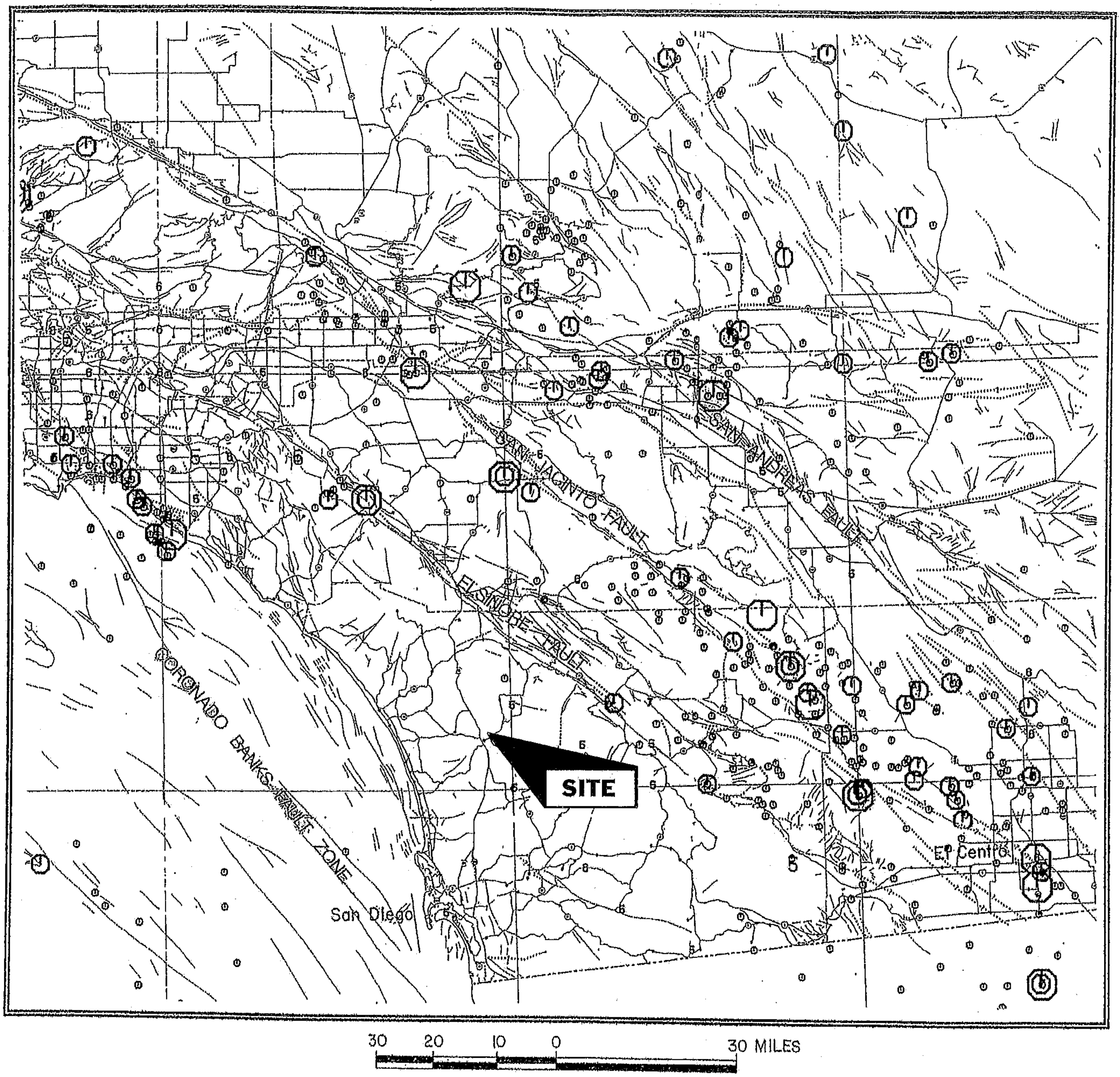




SCALE: 1" = 20'

PLATE 16
V&M JOB #12-136-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

PROJECT SITE: 999 N. BROADWAY
ESCONDIDO

FLOOD MAP

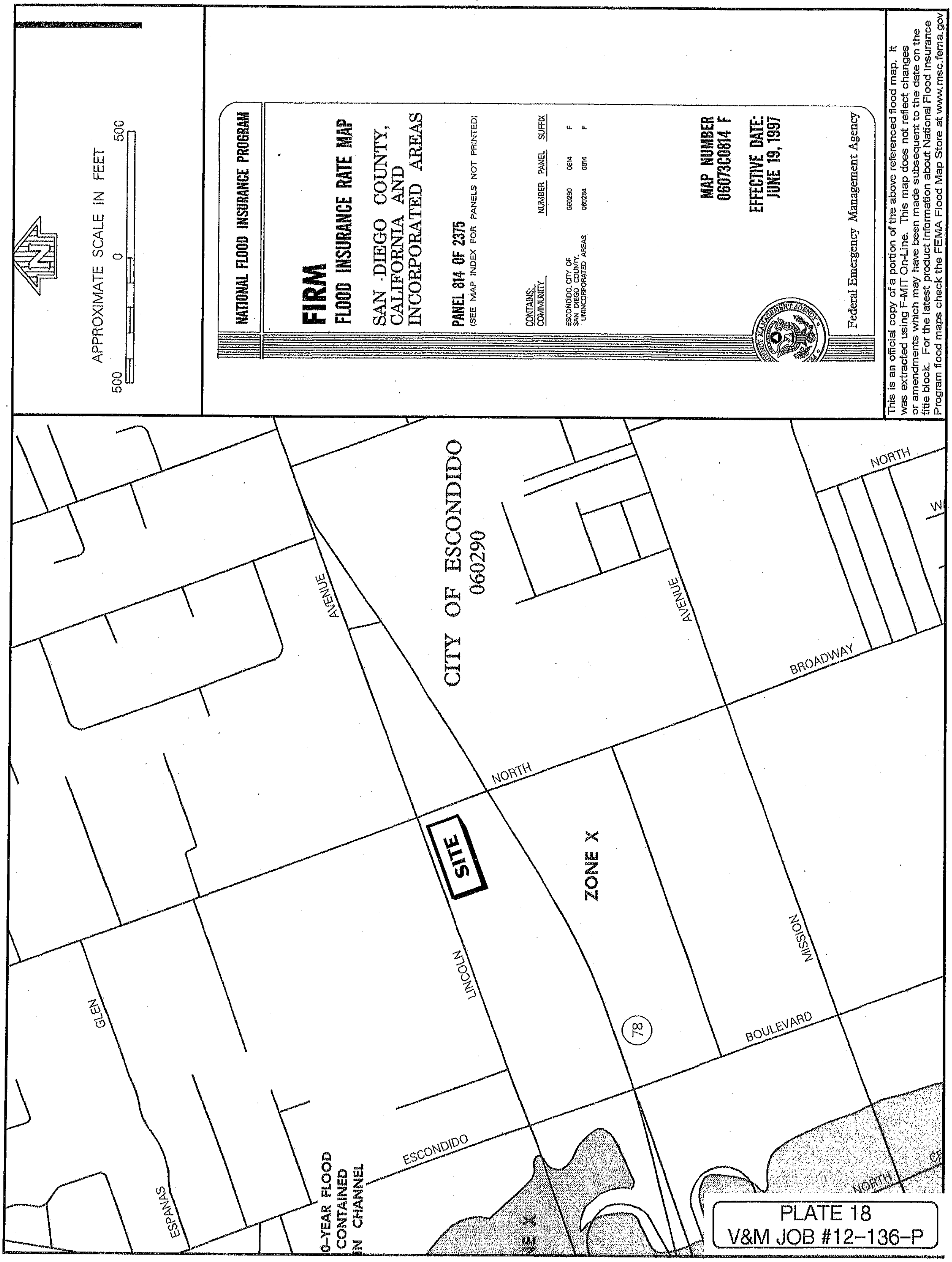
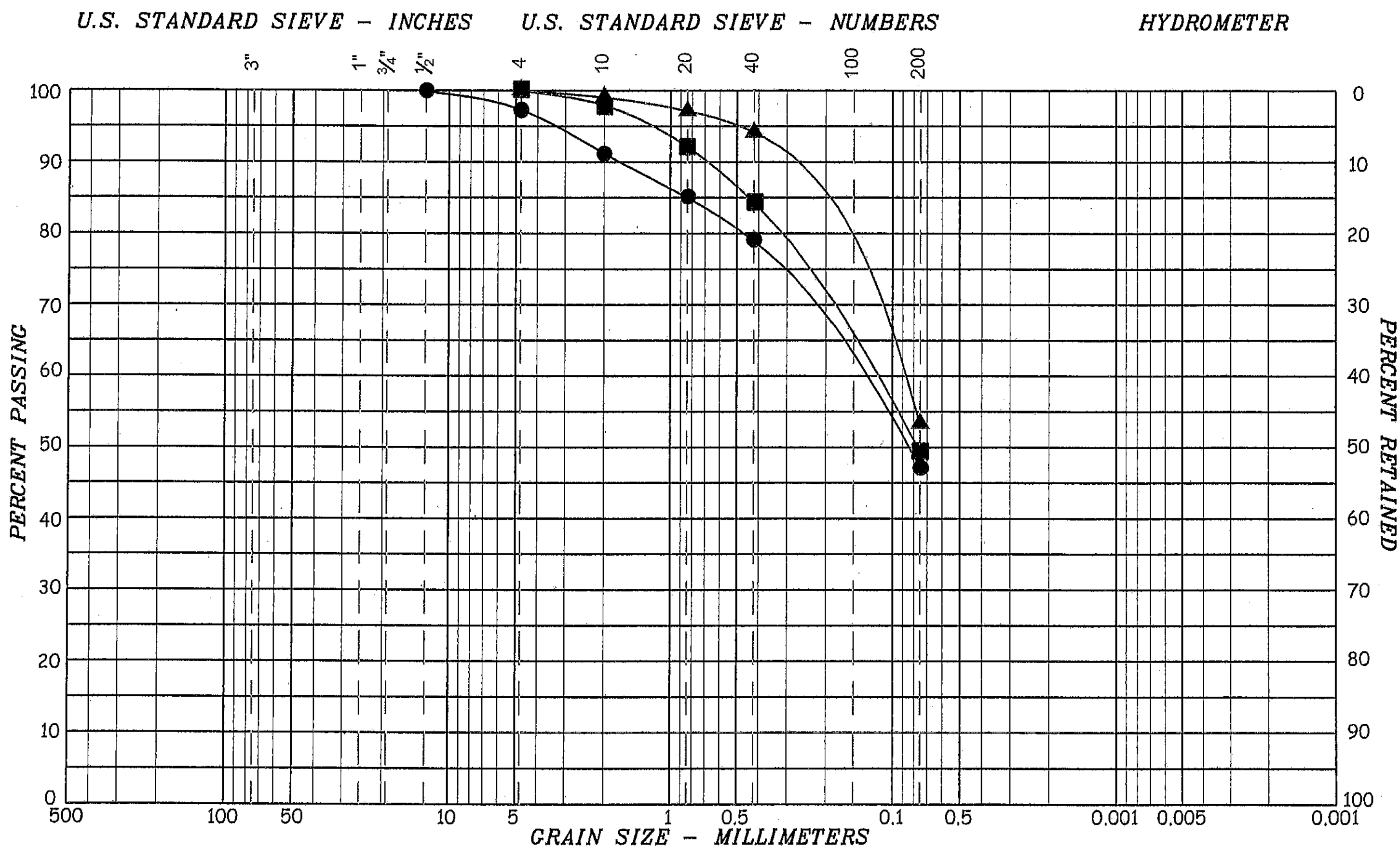


PLATE 18
V&M JOB #12-136-P

VINJE & MIDDLETON ENGINEERING

DSA FILE #		DSA APPL. #	
DSA/LEA #		Job # 12-136-P	Soil Type(s): 2
Project: Pacific Development Partners, LLC		Location: 999 N. Broadway, Escondido	
ASTM Test Method: D 422		Date: April 2011	Tech:
Supervising Lab Tech: Ray Fox		NICET 129713	Exp. Date: 7/1/2013
Supervising Lab Manager: S. Mehdi S. Shariat		RCE # 46174	Exp. Date: 12/31/2012



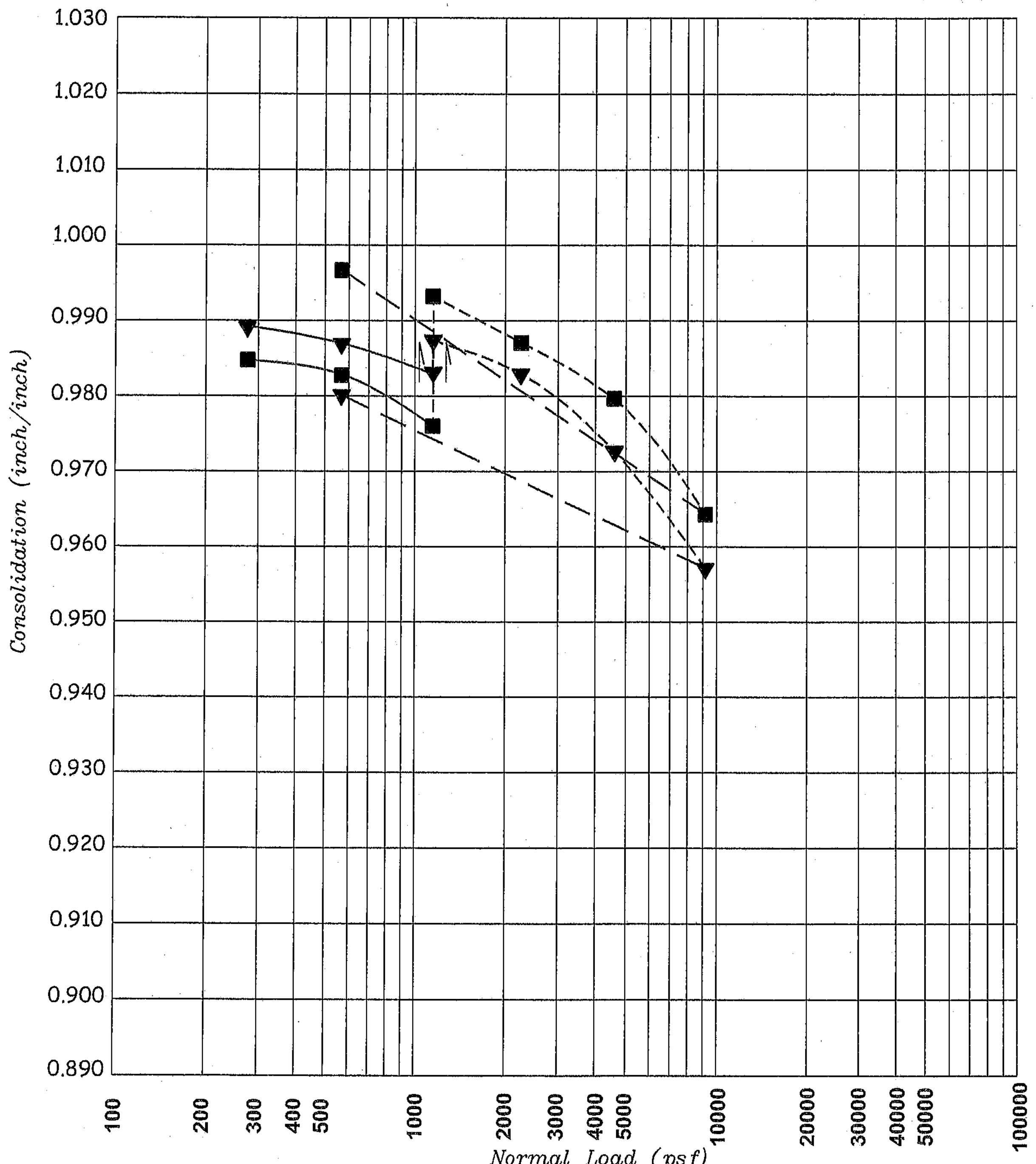
Cobbles	Gravel		Sand		SILT OR CLAY
	Coarse	Fine	Coarse to medium	Fine	

SAMPLE #	DEPTH (FT)	SYMBOL	USGS	NAT. ω%	LL	PL	PI	Cu (D_{60}/D_{10})	Cc ($D_{30}^2/D_{60}D_{10}$)
B-7	3	●	SC/CL	11	33	14	19		
B-7	18	▲	ML/SC	--	27	19	8		
B-10	3	■	SC/ML	13	--	--	--		

cc: ____Project Architect ____Structural Engineer ____Project Inspector ____DSA Regional Office

DSA FILE #	DSA APPL. #	DSA / LEA #	Test Method: ASTM D4186
Job # 12-136-P	Date: April 2012	Job Name: Pacific Development Partners, LLC	

Sample Location	Depth (ft.)	Sample Symbol	Sample Condition	Explanation
B-9	18	■	In-Place	FIELD MOISTURE
B-10	20	▼	In-Place	SAMPLE SATURATED
				REBOUND



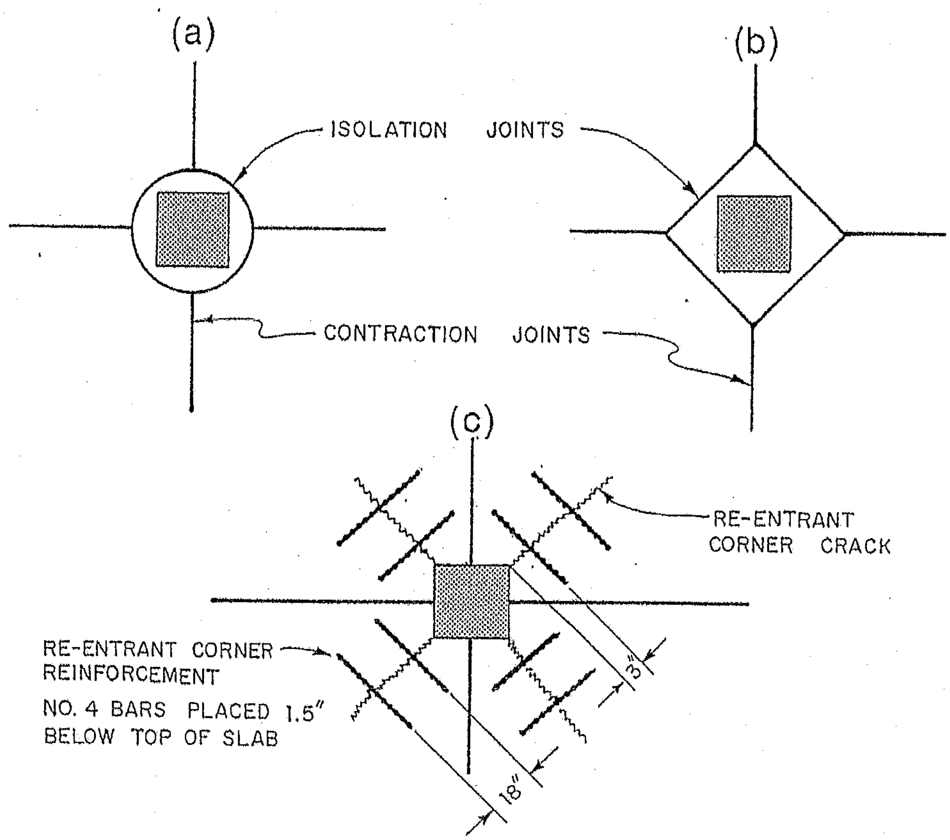
Supervising Lab Tech: Ray Fox Supervising Lab Manager: S. Mehdi S. Shariat

NICET: 129713 Exp. Date: 7/1/2013 RCE # 46174 Exp. Date: 12/31/2012

cc: ___Project Architect ___Structural Engineer ___Project Inspector ___DSA Regional Office

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).

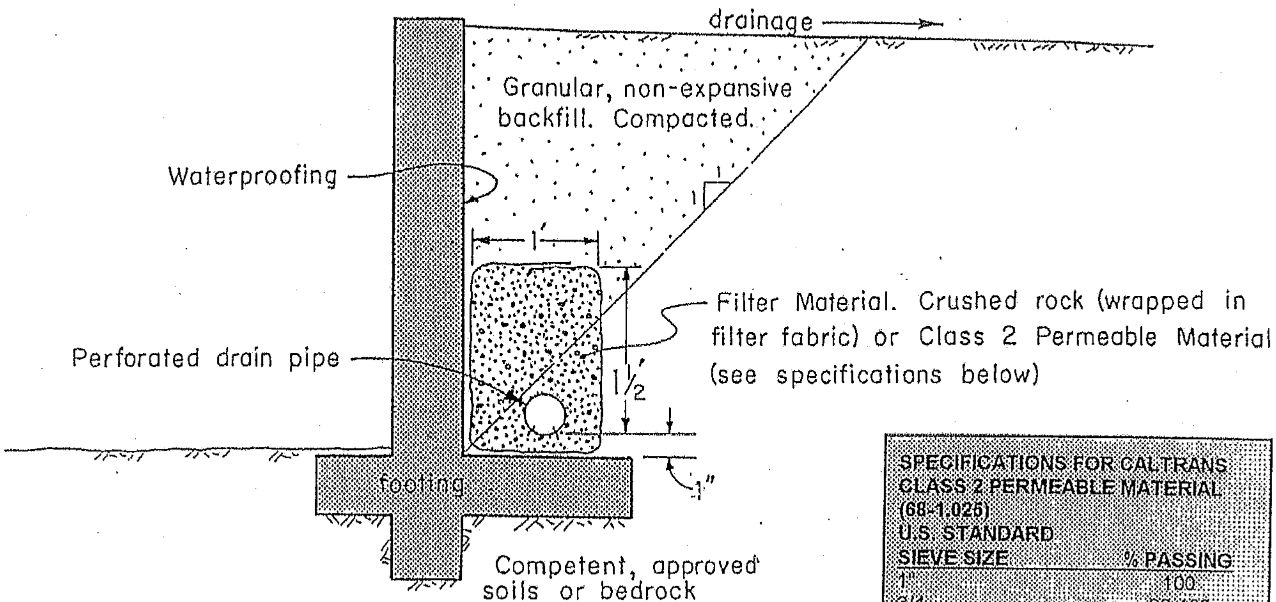
2. In order to control cracking at the re-entrant corners (±270° corners), provide reinforcement as shown in (c).

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.