Vinje & Middleton Engineering, Inc.

Geotechnical Investigation

Proposed 5-Lot Subdivision
1505 York Drive
San Diego County

(A.P.N. 184-012-12)

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I. INTRODUCTION

The subject property consists of a 3-acre, partially developed, residential property located at the referenced address in the County of San Diego. The property location is shown on a Vicinity Map attached to this report as Plate 1. The approximate site coordinates are 33.1825°N latitude and -117.2113°W longitude. We understand that the property is proposed for a split into 5 lots for residential purposes. Consequently, this investigation was initiated to determine soil and geotechnical conditions at the property and to ascertain their influence upon the future residential construction. Test pit digging, soil/rock sampling, and laboratory analysis were among the activities conducted in conjunction with this effort which has resulted in the grading and foundation recommendations provided in following sections.

II. SITE DESCRIPTION

The study property is shown on a Geotechnical Map, reproduced from a Tentative Subdivision Map by bHA, Inc., included as Plate 2. As shown, the site consists of a generally rectangular-shaped lot that descends westerly from York Drive at modest to gentle gradients that transition to nearly level terrain in the west areas of the property. An existing residential development is present in the southeast corner of the property and is designated as Lot 5. The existing development will remain, and therefore, was not investigated as part of this effort.

The northwest portions of the property have been locally modified by shallow cut-fill grading which has resulted in gentle stepped terraces. Below the terraces, to the west, is a level building pad that presently supports a storage building and adjacent concrete flatwork. Beyond the storage structure, in the northwest corner of the property, two storage containers are also present. Elsewhere, the study property is mantled by grass and local trees.

Outside of the short graded transition slopes, associated with the building pad and terraces, site terrain generally slopes at gradients that approach 5:1 maximum. Existing graded slopes are constructed at 2:1 gradients maximum and approach 3 feet high maximum. Documentation pertaining to previous grading or ground modifications in the study areas of the property are not available.

Site drainage sheetflows in a southwesterly direction to natural swale-like terrain along the west property line that drains in a southerly direction. Local shallow erosional ruts are present; however, excessive scouring or erosion is not in evidence.

III. PROPOSED DEVELOPMENT

As shown on the attached Plate 2, the property will be split into 5 lots and an interior cul-de-sac with associated improvements. Lots 1 through 4 are planned for future single-family residential developments. Lot 5 presently supports an existing residential development that will remain and is not a part of this investigation.

The existing storage structures and improvements will be removed / demolished to make room for the planned subdivision. Modest grading efforts will be required to achieve the planned design grades for building pads and improvement surfaces. Vertical cuts and fills approaching 10 feet maximum will be needed. Resulting graded slopes are programmed for 2:1 gradients maximum and will approach 20 feet high maximum.

Detailed construction plans are not presently available. The use of conventional wood-frame with exterior stucco building construction supported on shallow foundations with stem walls and slab-on-grade floors, or slab-on-ground with turn-down footings, is assumed herein for the purpose of this study.

IV. SITE INVESTIGATION

Geotechnical conditions at the study property were chiefly determined by the excavation of 6 exploratory test pits dug with a track-mounted caterpillar 305 excavator. All the test pits were logged by our project geologist who also directed the sampling of representative soil and bedrock material at selected locations and intervals for laboratory testing.

Test pit locations are shown on the Geotechnical Map, Plate 2. Logs of the trenches are included as Plates 3-7. Laboratory test results are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

The property is underlain by crystalline bedrock units at shallow to modest depths. Surficial soil includes shallow fills and colluvial soil that thicken downslope into alluvium. Approximate distribution of geologic units over the project property is mapped on the enclosed Geotechnical Map. Geologic Cross-Sections depicting subsurface conditions based on our exploratory test pits, existing grades, and proposed grades are included as Plate 8. Geologic instability that could preclude the proposed lot split for residential purposes is not indicated at the property. The following geotechnical conditions are apparent:

A. Earth Materials

Bedrock (Kgb) - The property is underlain at shallow to relatively modest depths by crystalline bedrock units which are rooted in the Southern California Batholith. As exposed, the underlying bedrock typically consists of fine to coarse-grained gabbroic rocks that were found in weathered and friable conditions near the surface becoming hard and very dense at depth. Project bedrock are stable, competent units which will provide excellent support for new fills, structures, and improvements.

Colluvium (Qcol) / Alluvium (Qal) - The underlying bedrock is mantled by colluvial soil in the upper easterly portion of the property that transitions into alluvium in the lower westerly areas of the property. Project colluvium / alluvium consists of silty sandy soil that occurs in highly porous and loose conditions overall. The colluvium / alluvium was found dry to damp in upper exposures becoming moist with depth. Due to the limits of the excavator, the thickness of the alluvium and depth to bedrock in the lower, westerly areas of the property is unknown. However, based on our cross-sections, the depth to bedrock below the alluvium is estimated to approach 20 feet.

Fill (af) - Shallow, undifferentiated fill soils associated with the short terraces and storage building locally mantle the site colluvium deposits, as mapped on Plate 2. Site existing fills are typically gravelly to rocky deposits that were found in a loose condition overall.

All existing fills and site colluvium / alluvium deposits are not suitable for structural support in their present condition, and should be removed and recompacted as part of the remedial grading operation.

B. Groundwater and Surface Drainage

Groundwater conditions were not encountered in our test pits to the depths explored and is generally not expected to impact the planned developments. However, groundwater or associated weeps may be expected in the deeper removals associated with the loose and porous lower alluvium deposits. Appropriate dewatering efforts, if required, should be suitable to the site conditions and effectively remove intruding water and allow safe excavating and filling to proceed. Gravel-filled sumps in the bottom of the removals with submersible pumps, may be considered. Temporary construction slope development, ground stabilization techniques, and remedial grading recommendations in the following sections are provided considering potential affects of possible groundwater intrusions and saturated ground condition.

Like all developed properties, the control of surface run-off and storm water is critical to the continuing stability of the property. Water should not be allowed to pond on pad surfaces and over-watering of site vegetation may create overly moist areas where shallow bedrock occurs.

C. Slope Stability

Natural sloping terrain at and near the project site is underlain by crystalline bedrock units which typically perform well in graded and natural slope conditions. Existing graded fill slopes will be removed as part of the site grading activities. New graded slopes will be grossly stable to design heights provided our slope development recommendations are adhered to during grading.

D. 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 13.6 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 0.0160g at the project site. The following list represents the most significant faults which commonly impact the region. Estimated ground acceleration data compiled from Digitized California Faults (Computer Program EQ Fault Version 3.00 updated) typically associated with the fault is also tabulated.

TABLE 1

FAULT ZONE	DISTANCE FROM SITE	MAXIMUM PROBABLE ACCELERATION (R.H.)
Newport-Inglewood Fault	12.7 miles	0.206g
Rose Canyon Fault	11.7 miles	0.143g
Elsinore-Julian Fault	17.7 miles	0.180g
Coronado Bank Fault	27.5 miles	0.151g

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 9.

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.

E. Seismic Ground Motion Values

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

F. Geologic Hazards

Geologic hazards are not presently indicated at the project site. Exposed slopes do not indicate gross geologic instability. The most significant geologic hazards at the property will be those associated with ground shaking in the event of a major seismic event. Liquefaction or related ground rupture failures are not anticipated.

G. Field and Laboratory Tests and Test Results

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

TABLE 2

Soil Type	Description
1	Tan silty to gravelly sand to clayey sand - Fill (af)
2	Brown silty fine to medium-grained sand - Colluvium (Qcol) / Alluvium (Qal)
3	Red brown fine to coarse-grained gabbroic rock - Bedrock (Kgb)

The following tests were conducted in support of this investigation:

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

TABLE 2

Location	Soil Type	Maximum Dry Density (γ_m-pcf)	Optimum Moisture Content (ω_{opt}-%)
TP-5 @ 4'	2	139.0	7.9

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

TABLE 3

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (γ_d-pcf)	Max. Dry Density (γ_m-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP-1 @ 3'	2	5	122.8	139.0	88	36
TP-2 @ 3'	2	3	107.4	139.0	77	14
TP-2 @ 5'	2	3	104.3	139.0	75	13
TP-2 @ 7'	2	10	119.2	-	-	66
TP-2 @ 8'	3	7	118.9	-	-	45
TP-3 @ 2'	2	2	105.5	139.0	76	9
TP-3 @ 4'	2	3	108.6	139.0	78	15
TP-3 @ 6'	2	3	105.9	139.0	76	14
TP-3 @ 8'	2	9	122.3	-	-	64
TP-3 @ 9'	3	8	119.2	-	-	53
TP-4 @ 3'	2	4	100.5	139.0	72	16
TP-4 @ 5'	2	4	104.5	139.0	75	18
TP-4 @ 7'	2	4	103.0	139.0	74	17
TP-4 @ 9'	2	4	108.1	139.0	78	19
TP-4 @ 11'	2	7	113.3	139.0	81	39
TP-4 @ 12'	2	7	114.7	139.0	83	40

Table 3 (continued)

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (Yd-pcf)	Max. Dry Density (Ym-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP-5 @ 4'	2	4	103.1	139.0	74	17
TP-5 @ 6'	2	4	105.3	139.0	76	18
TP-5 @ 8'	2	5	109.7	139.0	79	25
TP-5 @ 10'	2	10	110.2	139.0	79	51
TP-5 @ 12'	2	6	105.5	139.0	76	27

Note 1: Sample may be somewhat disturbed.
Assumptions And relationships:
In-place Relative Compaction = (Yd ÷ Ym) X100
Gs = 2.70
e = (Gs Yω ÷ Yd) - 1
S = (ω Gs) ÷ e

3. **Expansion Index Test:** One expansion index (EI) test was performed on a representative sample of Soil Type 2 in accordance with the ASTM D-4829. The test results are presented in Table 4.

TABLE 4

Sample Location	Soil Type	Molded ω (%)	Degree of Saturation (%)	Final ω (%)	Initial Dry Density (PCF)	Measured EI	EI 50% Saturation
TP-5 @ 4'	2	7.9	53.9	12.5	120.7	0	0

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

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

4. **Direct Shear Test:** One direct shear test was performed on a representative sample of Soil Type 2. The prepared specimen was soaked overnight, loaded with normal loads of 1, 2, and 4 kips per square foot respectively, and sheared to failure in an undrained condition. The test result is presented in Table 5.

TABLE 5

Sample Location	Soil Type	Sample Condition	Wet Density (Yw-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
TP-5 @ 4'	2	Remolded to 90% of YM @ % ωopt	136.0	33	161

5. **<u>pH and Resistivity Test:</u>** pH and resistivity of a representative sample of Soil Type 2 was determined using "Method for Estimating the Service Life of Steel Culverts," in accordance with California Test Method (CTM) 643. The test result is tabulated in Table 6.

TABLE 6

Sample Location	Soil Type	Minimum Resistivity (OHM-CM)	pH
TP-2 @ 3'	2	2688	7.3

6. **<u>Sulfate Test:</u>** A sulfate test was performed on a representative sample of Soil Type 2 in accordance with California Test Method (CTM) 417. The test result is presented in Table 7.

TABLE 7

Sample Location	Soil Type	Amount of Water Soluble Sulfate In Soil (% by Weight)
TP-2 @ 3'	2	0.014

7. **<u>Chloride Test:</u>** A chloride test was performed on a representative sample of Soil Type 2 in accordance with the California Test Method (CTM) 422. The test result is presented in Table 8.

TABLE 8

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
TP-2 @ 3'	2	0.002

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.

Laboratory test results performed on selected representative site samples indicate that the minimum resistivity is more than 1000 ohm-cm suggesting presence of low quantities of soluble salts. Test results further indicated pH less than 5.5, sulfate concentration less than 2000 ppm, and chloride concentration 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 0.014 percent by weight which is considered negligible according to ACI 318, Table 4.3.1. Portland cement Type II and concrete with minimum specified 28 days compressive strength (f'_c) of 2,500 psi and maximum water-cement ratio of 0.50 may be considered, unless otherwise specified (also see CBC Table 1808.8.1). Table 9 is appropriate based on the pH-Resistivity test result:

TABLE 9

Design Soil Type	Gage	18	16	14	12	10	8
2	Years to Perforation of Steel Culverts	30	39	48	66	84	102

VII. HYDRO MODIFICATIONS

Project stormwater quality treatment control Best Management Practices (BMP), if appropriate and as applicable, should be designed and constructed considering the site indicated geotechnical conditions. The implemented management practice(s) and water treatment control BMPs shall have no short and long term impacts on the site new building pads, graded embankments and natural surfaces, fills and backfills, structures, and onsite and nearby offsite improvements.

Bio-retention and filtration systems consisting of vegetated buffers or strips and self-contained retention/detention areas with impermeable liners on sides and bottom, special engineered sand filter media and perforated pipe(s) which discharge into an approved storm drain facility are typical methods consistent with the project geotechnical conditions for stormwater quality treatment control BMPs, if applicable. The bio-retention/detention areas should be sited adequately away from the site structures, improvements, retaining walls, foundations and top and toe of graded embankments, unless otherwise specifically approved.

The bio-retention/detention basins should be properly sized for adequate storage capacity with filtrations completed not more than 72 hours and vegetation carefully managed to prevent creating mosquito and other vector habitats. Additional and more specific recommendations should be provided by the project geotechnical consultant at the final plans review phase, if necessary.

VIII. CONCLUSIONS

Based upon the foregoing investigation, development of the project property for a 5-lot residential subdivision and associated improvements, substantially as proposed, is feasible from a geotechnical viewpoint. The site is underlain by stable, competent bedrock units that are mantled by a thin to moderately thick soil cover. The bedrock units are inherently stable and expected to perform well in graded slope embankments. The following geotechnical conditions are unique to the property and will most influence its development:

- Geologic hazards including faults or significant shear zones, ground failures or instability of natural terrain are not indicated at the project site. The most significant design factor will be ground motion during brief periods of seismic activity along distant active faults. The study property is not located near or within the Alquist - Priolo earthquake fault zone.

- Surficial soils at the property, including colluvial, alluvial, and all fill deposits (where encountered) are loose compressible deposits that are not suitable in their present condition for structural support. These soils should be removed and recompacted in accordance with recommendations given in a following section herein. Below, crystalline bedrock units will adequately support new compacted fills, structures, and improvements.

- The overall stability of the new graded pad and construction surfaces developed over the project site sloping terrain is most dependent upon adequate keying and benching of new fill into the undisturbed competent bedrock, as approved in the field, during the grading operations. At the project site, added care should be given to the proper construction of base keyway and benching excavations.

- Based upon our site observations and available subsurface exposures, project bedrock units are expected to excavate to planned design grades using medium to large size bulldozers (Caterpillar D-6 or equivalent) and track hoes.

- Soils generated from the site excavations and removals will predominantly consist of granular soils that will work well in new site fills and backfills. All trash debris, organic matter, and deleterious material generated from onsite demolition and brushing should be selectively removed and excluded from the site new fills to the satisfaction of the project geotechnical consultant.

- Natural groundwater is not expected to significantly impact project grading or the long term stability of the individual developed lots. However, local groundwater conditions and / or seeps may be anticipated within alluvial areas requiring aerating of wet soils and local dewatering efforts.

- Natural sloping terrain at the project site is geologically stable. Graded cut slopes planned in connection with site development will be grossly stable with respect to deep seated stability for the indicated maximum heights at 2:1 gradients, provided our slope development recommendations are followed during grading.

- Expansive soils will not be a factor in site development provided our recommendations are followed. Final bearing and subgrade soils are anticipated to consist of a silty sand mixture (SM) with very low expansion potential (expansion index less than 20), based on ASTM D-4829 classification.

- Site excavations, stripping and removals, trenching, remedial grading and earthwork construction shall not impact adjacent properties and public improvements. Adequate excavations and grading setbacks shall be observed and all temporary construction slopes completed as specified in the following sections. Added field recommendations, however, may also be necessary and should be given by the project geotechnical consultant, and should be anticipated.

- The proper control of surface drainage is an important factor in the stability of the graded pad. Ponding or concentrated runoff should not be allowed on graded surfaces, and over-watering of site vegetation should be avoided. The existing cut slope on the west margin of the study site should be provided with a concrete brow ditch. Runoff should not be allowed over graded slope surfaces.

- Post construction settlements are not expected to be a major geotechnical concern in the development of project property provided our remedial grading, pad construction, and foundation recommendations are followed. Post construction settlements after remedial grading operations, 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.

- Liquefaction, seismically induced settlements, and soil collapse are not considered a major geotechnical factor in the planned developments provided our remedial grading and foundation recommendations are followed.

IX. RECOMMENDATIONS

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

A. Grading and Earthworks

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

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

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

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

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

   All unsuitable materials generated from demolition and clearing efforts should be properly removed and disposed of from the site. Construction debris, trash, vegetation, and all deleterious material should not be allowed to occur in or contaminate new site fills and backfills. The prepared ground should be observed and approved by the project geotechnical consultant or his designated field representative prior to grading and earthwork.

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

   Removal depths will vary throughout the site and should be established by the project geotechnical engineer in the field at the time of earthwork operation based on actual exposures. Approximate removal depths are estimated to approach 20 feet in the lower west areas of the property. Hillside area removals are expected to be less than 10 feet maximum. Locally deeper removals may be necessary based on the actual exposures and should be anticipated. All existing fills, where encountered, should also be excavated and recompacted as a part of the project remedial grading operation.

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

4. **Cut-Fill Transitions and Undercuts:** Ground transition from excavated cut to compacted fills shall not be permitted underneath the proposed foundations, structures, and on-grade improvements. Buildings and on-grade improvements should be supported entirely on compacted fills or founded entirely on competent undisturbed dense natural bedrock units. Transition pads will require special treatment. The cut portion of the cut-fill pad plus 10 feet outside the perimeter, where possible, and as directed in the field, should be undercut to a sufficient depth

to provide for a minimum 4 feet of compacted fill mat below rough finish grades, or at least 12 inches of compacted fill beneath the deepest footing(s), whichever is more. In the roadways, driveway, parking, and on-grade slabs/improvement transition areas there should be a minimum of 12 inches of compacted soils below rough finish subgrade.

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

5. **Excavation Characteristics:** Site excavations to the anticipated removal and over-excavation depths are expected to be achieved with moderate efforts using medium bulldozers (Caterpillar D-6 or equivalent) and track hoes. Major specialized excavation techniques are not currently indicated. However, deeper excavations into the site crystalline bedrock may require locally concentrated and specialized efforts.

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

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

   Trenching, excavations and temporary slopes exposing competent undisturbed bedrock units may be constructed at near vertical gradients to a maximum height of 8 feet, unless otherwise directed. Larger temporary slopes developed within site bedrock units may be constructed at near vertical gradients within the lower 5-foot and laid back at ½:1 gradient maximum within the upper portions thereafter unless otherwise approved or directed in the field.

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

7. **Soil Properties, Fill, and Backfill Materials:** Excavations of site weathered bedrock units and site surficial soil deposits (colluvium and alluvium) will predominantly generate good sandy materials that may be reused as new fills and backfills. Site harder bedrock units may generate some rock debris that may include larger rock sizes. Generated rock laden materials may be considered for reuse as site new compacted fills provided they are properly processed and manufactured into an approved mixture as specified herein. Excavations of site surficial soils will largely generate sandy deposits that will work well in new site fills and backfills.

8. **Shrinkage, Bulking and Compaction:** Based on our analyses, site existing colluvial / alluvial deposits may be expected to shrink, on average, approximately 10% to 20%.

   Import soils, if needed to complete grading, should conform to the requirements of this report. Import soils should be clean sandy granular, non-corrosive deposits (SM/SW) with very low expansion potential (100% passing 1-inch sieve, more than 50% passing #4 sieve and less than 18% passing #200 sieve with expansion index less than 21). Import soils should be inspected, tested as necessary, and approved by the project geotechnical consultant prior to delivery to the site. Import soils should also meet or exceed engineering characteristics and soil design parameters specified in the following sections.

   Uniform and stable compacted fill support should be constructed underneath the project new building pads and site improvements by the remedial grading and earthwork operations. For this purpose, properly manufactured fills should be adequately processed, thoroughly mixed, moisture conditioned to slightly (2%-3%) above the optimum moisture levels, placed in thin (8 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. Fill soils placed within the lower alluvium removal areas subject to groundwater inundation should be mechanically compacted to at least 95% of the laboratory maximum dry density (ASTM D-1557) to 3 feet above the existing water table. Subgrade soils underneath asphalt paving surfaces should also be compacted to minimum 95% compaction levels within the upper 12 inches.

9. **Permanent Graded Slopes:** Project permanent graded cut slopes exposing bedrock may be constructed at 2:1 gradients maximum. Permanent graded cut

slopes exposing loose colluvium should be reconstructed as stability fill slopes, as directed in the field. Stabilization fill slopes should be supported by a minimum equipment width (12 feet) stabilization fill consisting of locally derived soils compacted to a minimum value of 90% of the laboratory maximum density value. The stabilization fill slope should be provided with a 2-foot deep keyway below the design pad grades and keyed and benched into the hillside. Stabilization fill slopes should be constructed in substantial compliance with the enclosed Plates 10 and 11.

Fill slopes should be programmed for 2:1 gradients maximum. Graded slopes constructed as recommended herein will be grossly stable with respect to deep seated and surficial failures for the anticipated design maximum vertical heights.

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

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

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

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

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

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

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

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

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

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

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

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

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

- Fill/backfill observation - After the fill/backfill placement is started but before the vertical height of fill/backfill exceeds 2 feet. A minimum of one test shall be required for each 100-lineal foot maximum in every 2 feet vertical gain, with the exception of wall backfills where a minimum of one test shall be required for each 30-lineal foot maximum. Onsite plastic clayey soils are not suitable for wall backfills and good quality sandy granular soils should be used for this purpose. Wall backfills should consist of minus 3-inch sandy granular materials and also be mechanically compacted to a minimum of 90% compaction levels unless otherwise specified or directed in the field. Finish rough and final pad grade tests shall be required regardless of fill thickness.

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

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

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

- Underground utility, plumbing and storm drain trench backfill observation - After the backfill placement is started above the pipe zone but before the vertical height of backfill exceeds 2 feet. Testing of the backfill within the pipe zone may also be required by the governing agencies. Pipe bedding and backfill materials shall conform to the governing agencies' requirements and project soils report if applicable. All trench backfills shall consist of minus 3-inch sandy granular materials and 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 should also be mechanically compacted and tested for a minimum of 90% compaction levels. Flooding or jetting techniques as a means of compaction method should not be allowed.

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

B. Foundations and Slab-on-Grades

The following recommendations are consistent with very low expansive (expansion index less than 21) silty sand (SM) foundation bearing soil and site specific geotechnical conditions including fill differential and fill thickness. Additional recommendations may also be required and should be given in the update report and at the plan review phase. All design recommendations should be further confirmed and/or revised at the completion of rough grading based on the expansion characteristics of the foundation bearing soil and as-graded site geotechnical conditions, and presented in the final as-graded compaction report. Individual building sites may require different foundations/slab recommendations and should be anticipated.

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

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

Continuous interior and exterior stemwall 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. Reinforcement details for spread pad footings should be provided by the project architect/structural engineer.

Interior slabs should be a minimum of 5 inches in thickness, reinforced with #4 reinforcing bars spaced 18 inches on center each way, placed mid-height in the slab.

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

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

Provide re-entrant corner reinforcement for all interior slabs. Re-entrant corners will depend on slab geometry and/or interior column locations. The enclosed Plate 12 may be used as a general guideline.

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

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

C. Soil Design Parameters

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

1. Design soil unit weight = 136.0 pcf.
2. Design angle of internal friction of soil = 33 degrees.
3. Design active soil pressure for retaining structures = 40 pcf (EFP), level backfill, cantilever, unrestrained walls.
4. Design at-rest soil pressure for retaining structures = 61 pcf (EFP), non-yielding, restrained walls.
5. Design passive soil resistance for retaining structures = 461 pcf (EFP), level surface at the toe.
6. Design coefficient of friction for concrete on soils = 0.40.
7. Net allowable foundation pressure (minimum 15 inches wide, embedded at least 18 inches into 90% compacted fill = 1500 psf.
8. Allowable lateral bearing pressure (all structures except retaining walls) = 150 psf/ft.

Notes:

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

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

- The indicated net allowable foundation pressure provided herein was determined based on minimum 15-inch wide by 18-inch deep footings and may be increased by 20% for each additional foot of depth and 20% for each additional foot of width to a maximum of 4500 psf. The allowable foundation pressures provided herein also apply to dead plus live loads and may be increased by one-third for wind and seismic loading.

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

D. Exterior Concrete Slabs / Flatworks

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

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

3. Provide “tool joint” or “softcut” contraction/control joints spaced 10 feet on center (not to exceed 12 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as slab will support weight, and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum of 1-inch but should not exceed 1¼-inches deep maximum. In case of softcut joints, anti-ravel skid plates should be used and replaced with each blade to avoid spalling and ravelings. Avoid wheeled equipments across cuts for at least 24 hours.

4. Joints shall intersect free-edges at a 90° angle and shall extend straight for a minimum of 1½ feet from the edge. The minimum angle between any two

intersecting joints shall be 80°. Align joints of adjacent panels. Also, align joints in attached curbs with joints in slab panels. Provide adequate curing using approved methods (curing compound maximum coverage rate = 200 sq. ft./gal.).

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

E. Asphalt and PCC Pavement Design

Specific pavement designs can best be provided at the completion of rough grading based upon R-value and expansion index tests of the actual finish subgrade soils; however, the following structural sections may be considered for cost estimating purposes only (not for construction).

1. **Asphalt Paving:** A minimum section of 4 inches asphalt on 6 inches Caltrans Class 2 aggregate base, or the minimum section required by the County of San Diego, whichever is more, may be considered. Actual design will also depend on the design TI and the approval of the County of San Diego.

   The Class 2 aggregate base shall meet or exceed the requirements set forth in the current California Standard Specification (Caltrans Section 26-1.02). Base materials should be compacted to a minimum 95% of the maximum dry density. Subgrade soils beneath the pavement base layer should also be compacted to a minimum 95% of the corresponding maximum dry density within the upper 12 inches.

2. **PCC Paving:** PCC driveways and parking supported on very low expansive (expansion index less than 21) subgrade soil should be a minimum of 5½ inches in thickness, reinforced with #3 reinforcing bars at 18 inches on centers each way placed at mid-height in the slab. Subgrade soil beneath the PCC driveways and parking should be compacted to a minimum 90% of the corresponding maximum dry density, unless otherwise specified.

   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 18 inches on centers placed mid-height in the slab.

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

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

3. **General Paving:** Base section and subgrade preparation per structural section design, will be required for all surfaces subject to traffic including roadways, travelways, drive lanes, driveway approaches and ribbon (cross) gutters. Driveway approaches within the public right-of-way should have 12 inches subgrade compacted to a minimum 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. Base section may not be required under curb and gutters, and sidewalks, in the case of very low to non-expansive subgrade soils (expansion index less than 21). Appropriate recommendations should be given in the final as-graded compaction report.

F. General Recommendations

1. The minimum foundation design and steel reinforcement provided herein are based on the anticipated 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 illegal

encroachments or unnecessary additional grading which will increase construction costs.

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

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

5. Expansive clayey soils should not be used for backfilling of any retaining structure. All retaining walls should be provided with a 1:1 wedge of granular, compacted backfill measured from the base of the wall footing to the finished surface and a well-constructed back drain as shown on the enclosed Plate 13.

6. 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.

7. Footings located on or adjacent to the top of slopes should be extended to a sufficient depth to provide a minimum horizontal distance of 7 feet or one-third of the slope height, whichever is greater (need not exceed 40 feet maximum) between the bottom edge of the footing and face of slope. This requirement applies to all site improvements and structures including fences, posts, etc. A minimum 10 feet horizontal setback should be considered for more sensitive structures and improvements which can not tolerate minor movement. Concrete, asphalt paving, and improvements should be provided with a thickened edge to satisfy the specified setback requirement.

8. Site drainage over the finished pad surfaces should flow away from structures onto the street in a positive manner. Care should be taken during the construction, improvements, and fine grading phases not to disrupt the designed drainage patterns. Roof lines of the buildings should be provided with roof gutters. Roof water should be collected and directed away from the buildings and structures to a suitable location.

9. Final plans should reflect preliminary recommendations given in this report. Final foundations and grading plans may also be reviewed by the project geotechnical

consultant for conformance with the requirements of the geotechnical investigation report outlined herein. More specific recommendations may be necessary and should be given when final grading and architectural/structural drawings are available.

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

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

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

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

    * Cure the concrete as long as practical.

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

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

X. GEOTECHNICAL ENGINEER OF RECORD (GER)

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

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

XI. LIMITATIONS

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

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

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

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

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

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

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

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

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

Once again, should any questions arise concerning this report, please do not hesitate to contact this office. Reference to our **Job #16-237-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

Ralph M. Vinje
GE #863

Steven J. Melzer
CEG #2362

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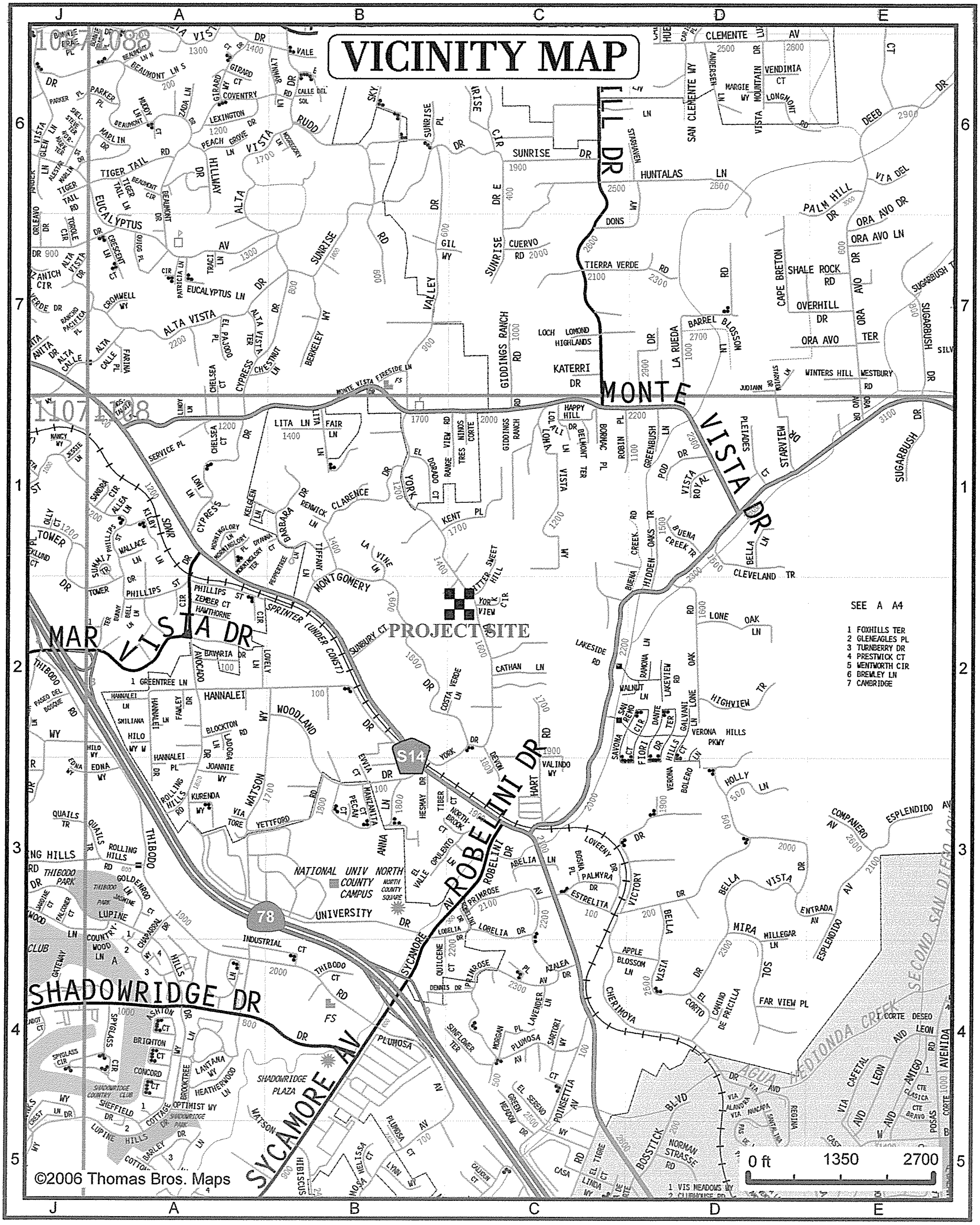
Appendix

Seismic Ground Motion Values



PROJECT SITE: 1108 - C2

PLATE 1
V&M JOB #16-237-P

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

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4

CLEAR SQAURE SIEVE OPENINGS: ¾", 3", 12"

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

RELATIVE DENSITY

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

CONSISTENCY

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

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

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

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

VINJE & MIDDLETON ENGINEERING, INC.

2450 Auto Park Way
Escondido, California 92029

KEY TO BORING / TEST PITS LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

December, 2013

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

PROJECT: Proposed Residential Subdivision **CLIENT:** Gary Van Eik

PROJECT NO.: 16-237-P **PROJECT LOCATION:** 1505 York Drive, Vista

Date Excavated: 12/15/16 **Bucket Size:** 18" **Logged By:** SJM

Equipment: Caterpillar 305 Excavator

Remarks: No caving. No groundwater.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
		SM-GM	**Fill (af):** Silty to gravelly fine to coarse sand. Tan color. Damp. Loose. ST-1					
2		SM	**Colluvium (Qcol):** Silty fine to medium sand. Brown color. Damp. Porous. Locally blocky. Loose to medium dense. ST-2	Chunk sample	5	122.8	88	36
4		GP	**Bedrock (Kgb):** Gabbroic rock. Fine to coarse grained. Red brown color. Blocky. Hard. Very dense. ST-3					

Bottom of test pit at 5.0 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

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

PROJECT: Proposed Residential Subdivision **CLIENT:** Gary Van Eik

PROJECT NO.: 16-237-P **PROJECT LOCATION:** 1505 York Drive, Vista

Date Excavated: 12/15/16 **Bucket Size:** 18" **Logged By:** SJM

Equipment: Caterpillar 305 Excavator

Remarks: No caving. No groundwater.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–2	SM-GM	**Fill (af):** Silty to gravelly fine to coarse sand. Includes up to 20% rock fragments to 12-inches in diameter. Tan color. Damp. Very loose. ST-1	Bulk				
2–7.5	SM	**Colluvium (Qcol):** Silty fine to medium sand. Brown color. Dry to damp. Porous. Locally blocky. Very loose to firm. ST-2	Bulk, Chunk	3	107.4	77	14
			Chunk	3	104.3	75	13
			Chunk	10	119.2	-	66
7.5–9	GP	**Bedrock (Kgb):** Gabbroic rock. Fine to coarse grained. Red brown color. Weathered. Blocky. Massive. Dense to very dense. ST-3	Chunk	7	118.9	-	45

Bottom of test pit at 9.0 feet.

BULK SAMPLE — CHUNK SAMPLE — DENSITY TEST — GROUND WATER

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

PROJECT: Proposed Residential Subdivision **CLIENT:** Gary Van Eik

PROJECT NO.: 16-237-P **PROJECT LOCATION:** 1505 York Drive, Vista

Date Excavated: 12/15/16 **Bucket Size:** 18" **Logged By:** SJM

Equipment: Caterpillar 305 Excavator

Remarks: No caving. No groundwater.

DEPTH (ft)	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
	GM-GC	**Fill (af):** Silty to claye sand with gravel. Grey color. Damp. Very loose. ST-1					
2	SM	**Colluvium (Qcol):** Silty fine to medium sand. Brown color. Dry to damp. Porous. Very loose. Locally blocky. ST-2	Chunk	2	105.5	76	9
4	SM	Loose to firm at 4 feet. Dry to damp.	Chunk	3	108.6	78	15
6	SM	Highly porous at 6 feet. Continues dry. Loose to very loose.	Chunk	3	105.9	76	14
8	SM	Blocky and relatively tight at 7.5 feet. Damp. Medium dense to dense. Appears to be a residual soil (weathered reflection of the underlying bedrock).	Chunk	9	122.3	-	64
	GP	**Bedrock (Kgb):** Gabbroic rock. Fine to coarse grained. Olive brown color. Massive. Blocky. Dense. ST-3	Chunk	8	119.2	-	53

Bottom of test pit at 9.5 feet.

BULK SAMPLE | CHUNK SAMPLE | DENSITY TEST | GROUND WATER

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

PROJECT: Proposed Residential Subdivision **CLIENT:** Gary Van Eik

PROJECT NO.: 16-237-P **PROJECT LOCATION:** 1505 York Drive, Vista

Date Excavated: 12/15/16 **Bucket Size:** 18" **Logged By:** SJM

Equipment: Caterpillar 305 Excavator

Remarks: No caving. No groundwater.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0		SM	**Colluvium (Qcol):** Silty fine to medium sand. Brown color. Damp. Porous. Locally blocky. Very loose to firm. ST-1					
3			Highly porous at 3 feet. Dry to damp. Very loose.	Chunk	4	100.5	72	16
5				Chunk	4	104.5	75	18
7			Continues dry to damp and highly porous at 7 feet. Very loose.	Chunk	4	103.0	74	17
9				Chunk	4	108.1	78	19
11			Somewhat blocky at 11 feet. Moist. Porous. Loose to medium dense.	Chunk	7	113.3	81	39
12				Chunk	7	114.7	83	40

End test pit at 12.5 feet - extent of excavator.

Bottom of test pit at 12.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

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

PROJECT: Proposed Residential Subdivision **CLIENT:** Gary Van Eik

PROJECT NO.: 16-237-P **PROJECT LOCATION:** 1505 York Drive, Vista

Date Excavated: 12/15/16 **Bucket Size:** 18" **Logged By:** SJM

Equipment: Caterpillar 305 Excavator

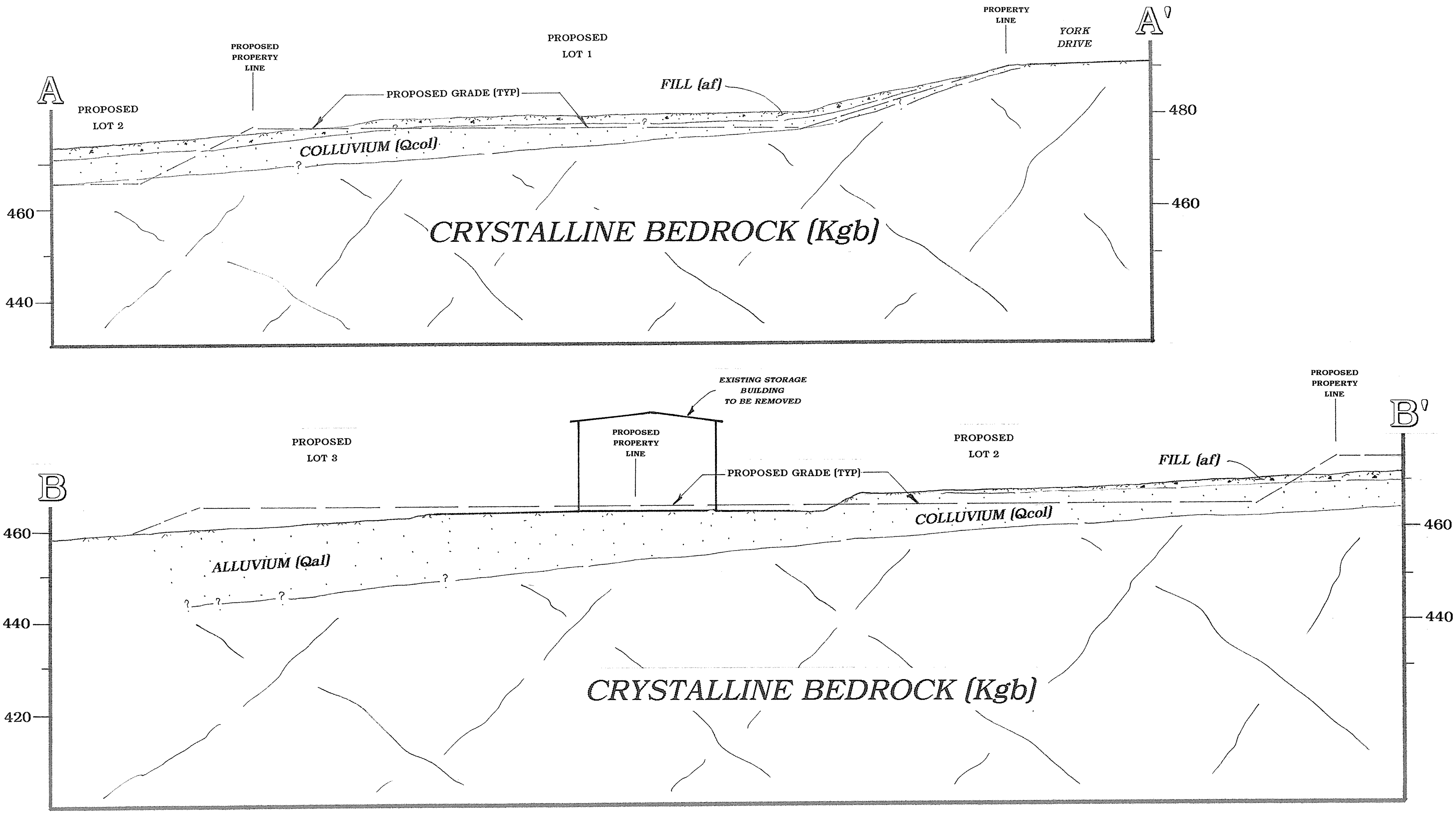
Remarks: No caving. No groundwater.

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
			Colluvium (Qcol):					
			Silty fine to medium sand. Brown color. Dry to damp. Somewhat blocky. Highly porous. Very loose. ST-2					
2								
4				Bulk, Chunk	4	103.1	74	17
6		SM	Continues highly porous and very loose at 6 feet. Dry to damp.	Chunk	4	105.3	76	18
8				Chunk	5	109.7	79	25
			Damp to moist at 9 feet. Highly porous. Loose to firm.					
10				Chunk	10	110.2	79	51
			Somewhat blocky at 11 feet. Moist. Highly porous. Loose.					
12			Blocky at 12 feet. Moist. Loose to firm.	Chunk	6	105.5	76	27

Bottom of test pit at 12.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

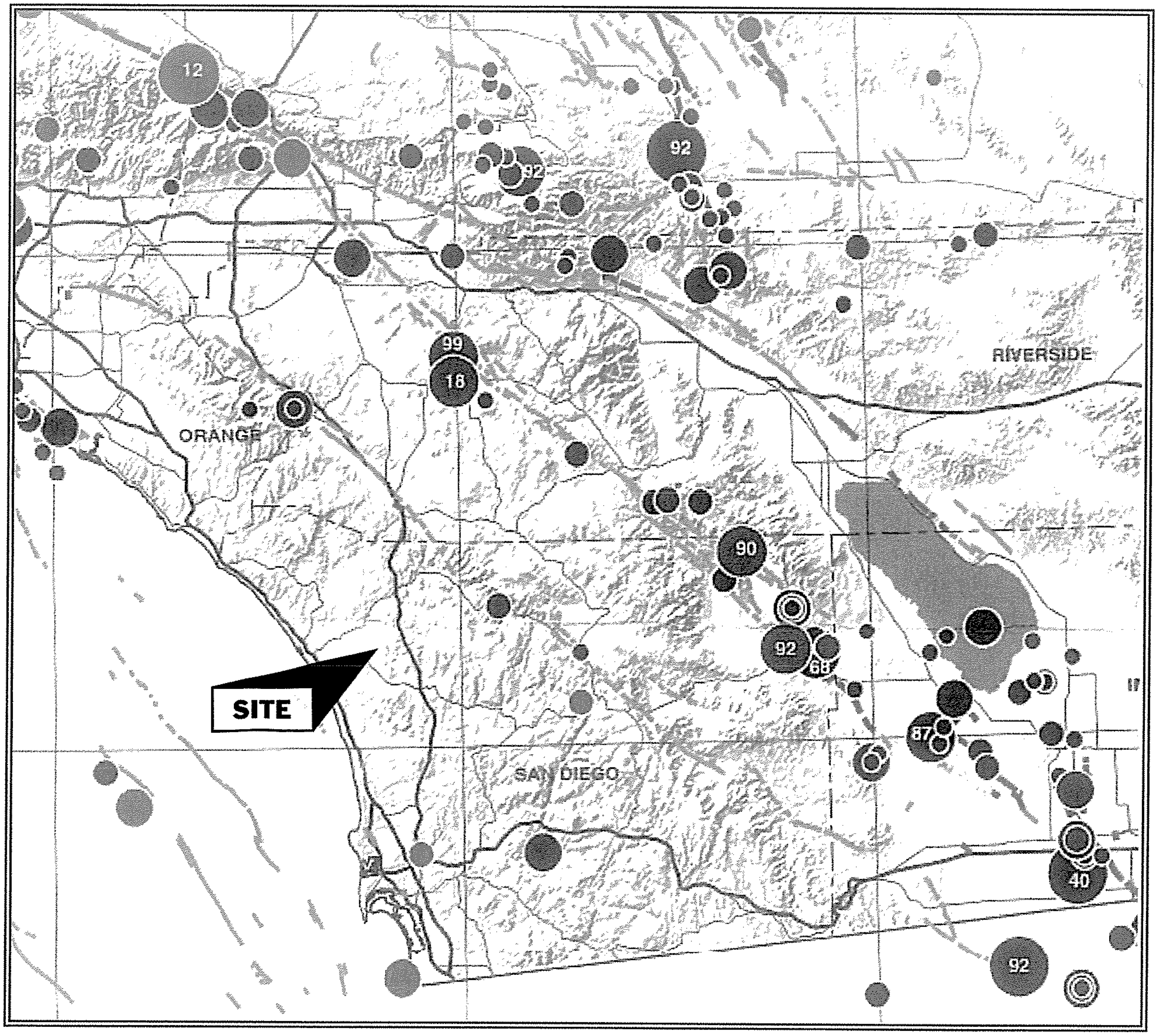
GEOLOGIC CROSS-SECTIONS



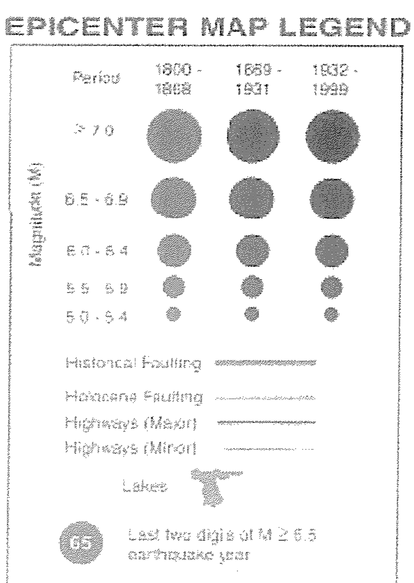
SCALE: 1" = 20'

PLATE 8
V&M JOB #16-237-P

FAULT-EPICENTER MAP
SAN DIEGO COUNTY REGION



Indicated Earthquake Events Through A 200 Year Period

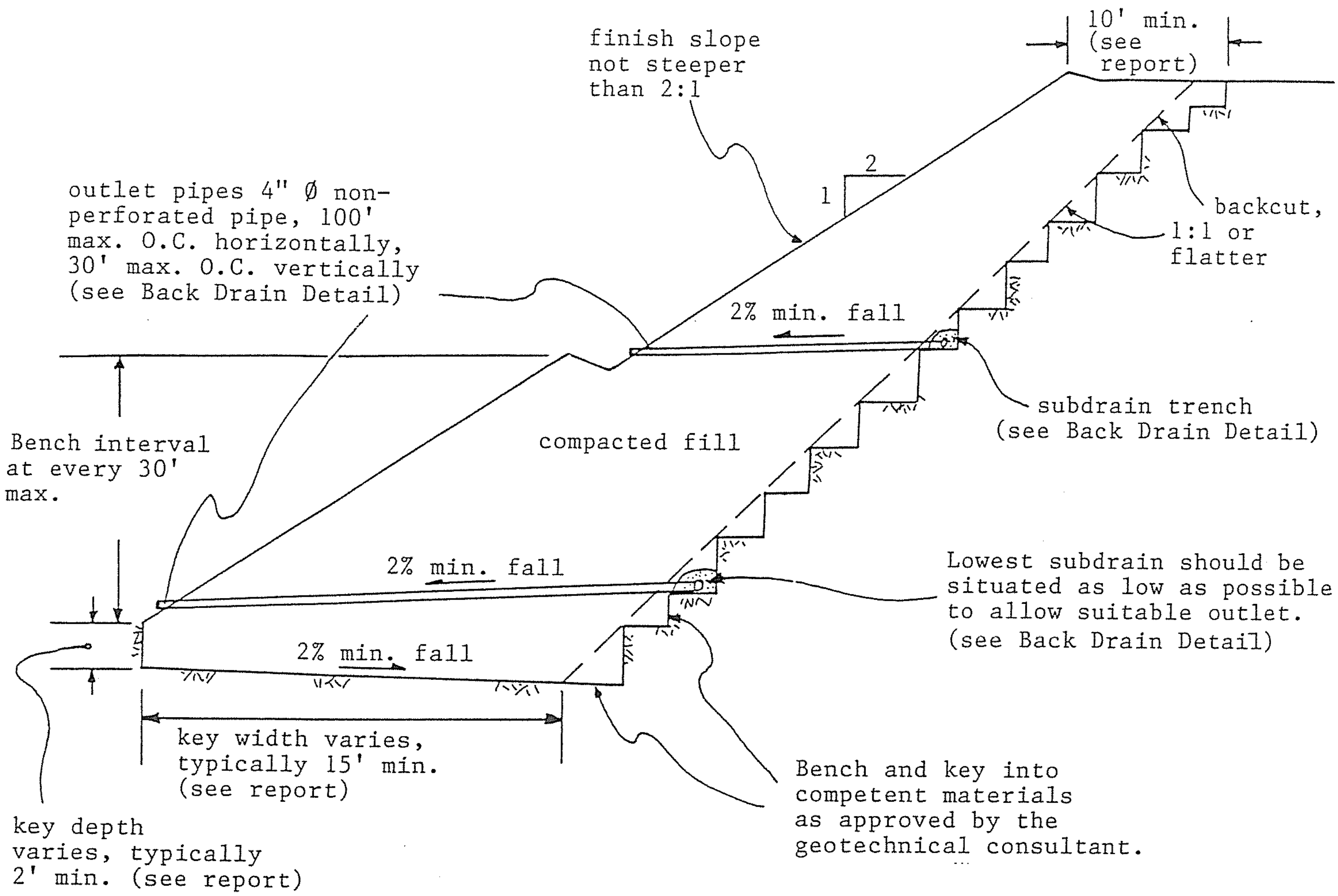


Map is reproduced from California Division of Mines and Geology, "Epicenters of / and Areas Damaged by M $\geq$ 5 California Earthquakes, 1800-1999".

PLATE 9
V&M JOB #16-237-P

TYPICAL STABILITY FILL / BUTTRESS DETAIL

Not to Scale

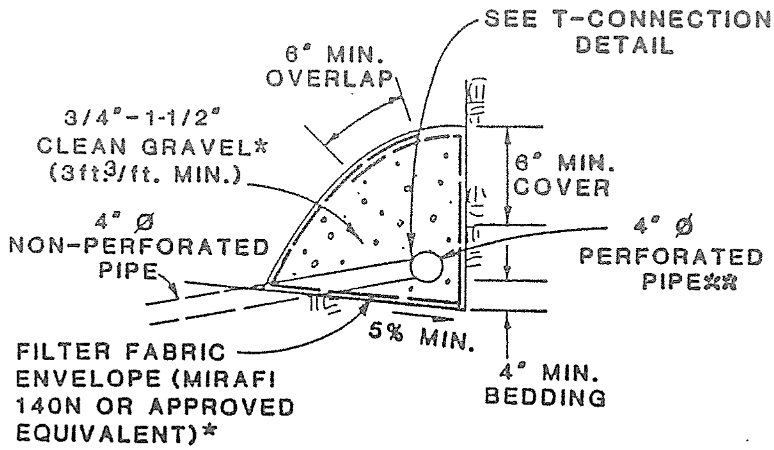


Notes: For buttress dimensions, see geotechnical report/plans. Actual dimensions may be changed by the geotechnical consultant based on field conditions.

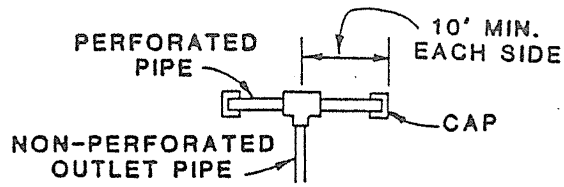
All backcuts will require to be field inspected at the time of grading by the project geotechnical consultant.

The need for backdrains to be determined in the field based on actual exposures.

BACK DRAIN DETAIL



SUBDRAIN TRENCH DETAIL



T-CONNECTION DETAIL

*If Caltrans Class II permeable material is used in place of 3/4"-1½" gravel, fabric filter may be deleted.

**SUBDRAIN TYPE - Subdrain type should be Acrylonitrile Butadiene Stryene (A.B.S.), Polyvinyl Chloride (PVC) or approved equivalent. Class 125, SDR 32.5 should be used for maximum fill depths of 35 feet. Class 200, SDR 21 should be used for maximum fill depths of 100 feet.

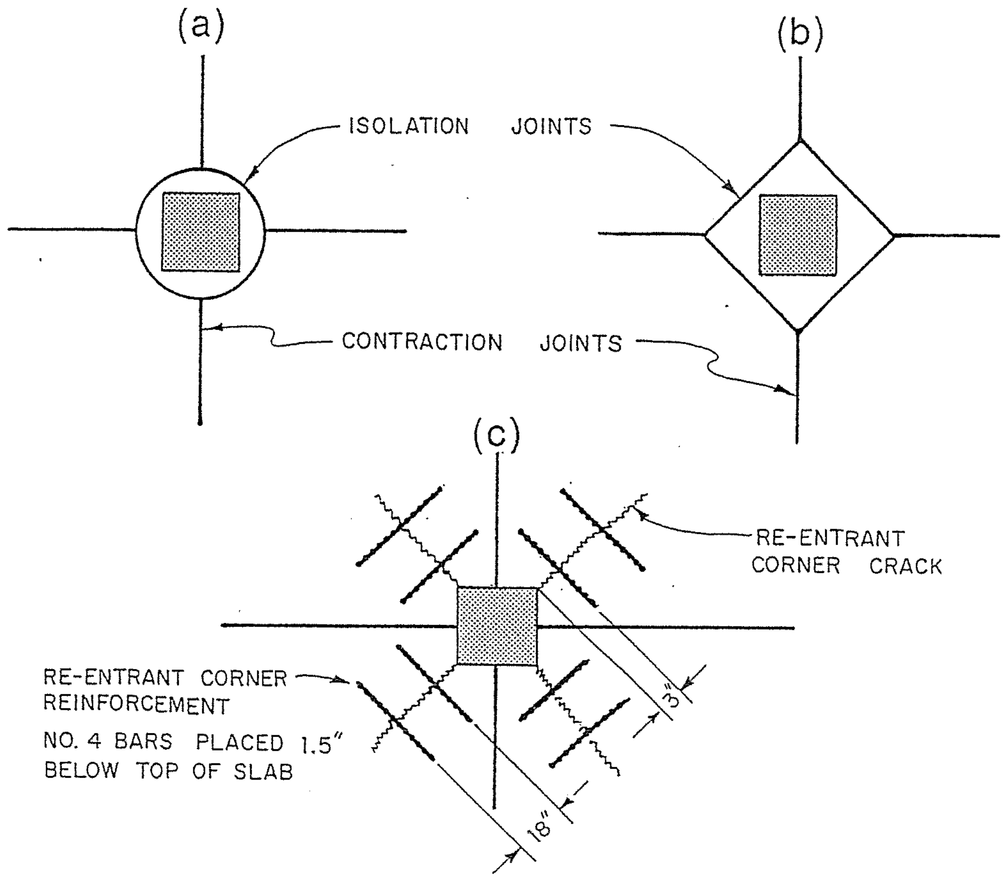
SPECIFICATIONS FOR CALTRANS CLASS II PERMEABLE MATERIAL

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

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.

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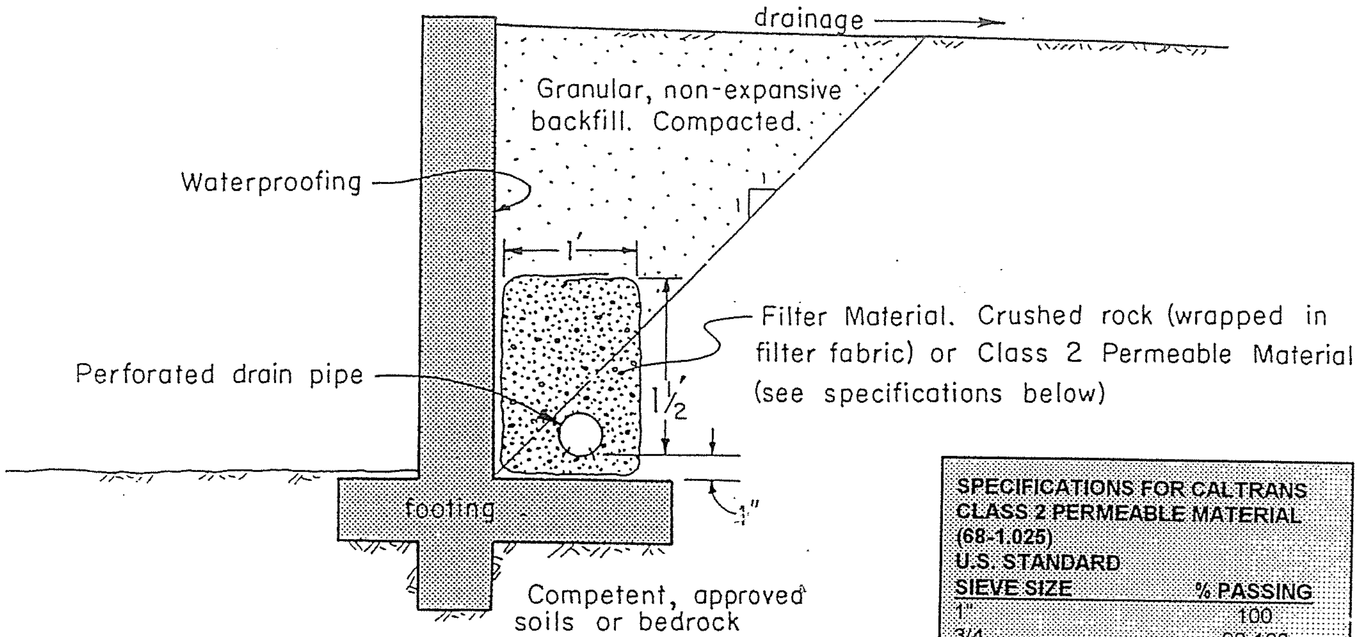
RETAINING WALL DRAIN DETAIL

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.

Appendix

Seismic Ground Motion Values

Design Maps Summary Report

User–Specified Input

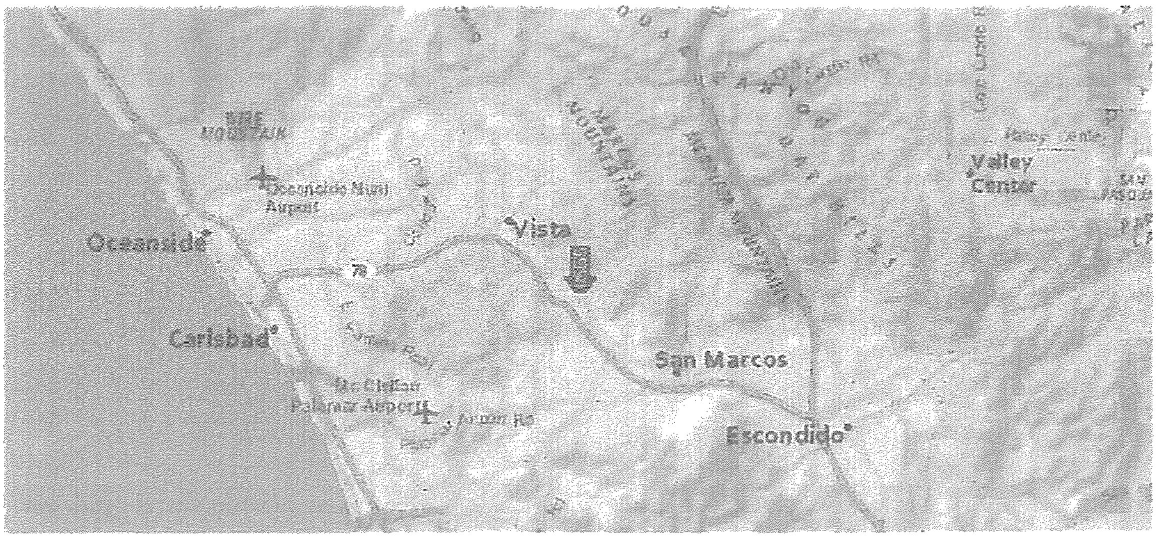
Report Title 16-237-P
Wed January 11, 2017 19:25:04 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.1825°N, 117.2113°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

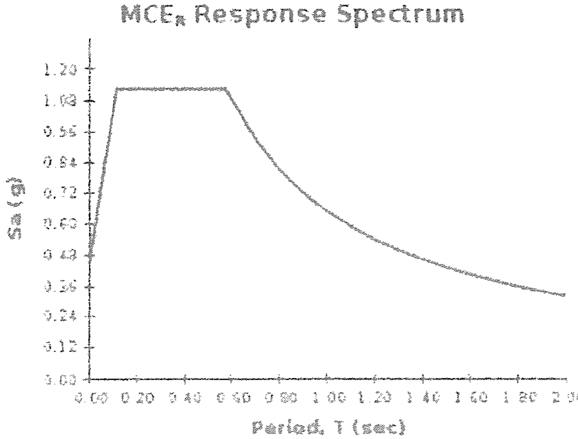


USGS–Provided Output

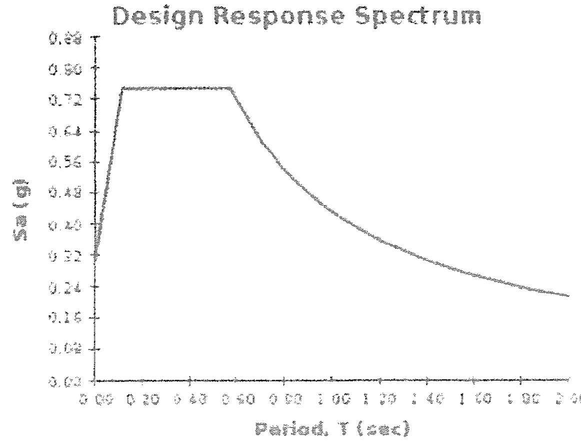
$S_S =$	1.035 g	$S_{MS} =$	1.124 g	$S_{DS} =$	0.749 g
$S_1 =$	0.406 g	$S_{M1} =$	0.647 g	$S_{D1} =$	0.431 g

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

MCE_R Response Spectrum



Design Response Spectrum



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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.1825°N, 117.2113°W)

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

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1] $S_S = 1.035$ g

From Figure 22-2 [2] $S_1 = 0.406$ g

Section 11.4.2 — Site Class

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

Table 20.3-1 Site Classification

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

For SI: 1ft/s = 0.3048 m/s 1lb/ft^2 = 0.0479 kN/m^2

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

Table 11.4-1: Site Coefficient F_a

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

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

For Site Class = D and S_S = 1.035 g, F_a = 1.086

Table 11.4-2: Site Coefficient F_v

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

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

For Site Class = D and S_1 = 0.406 g, F_v = 1.594

Equation (11.4-1): $S_{MS} = F_a S_S = 1.086 \times 1.035 = 1.124$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.594 \times 0.406 = 0.647$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

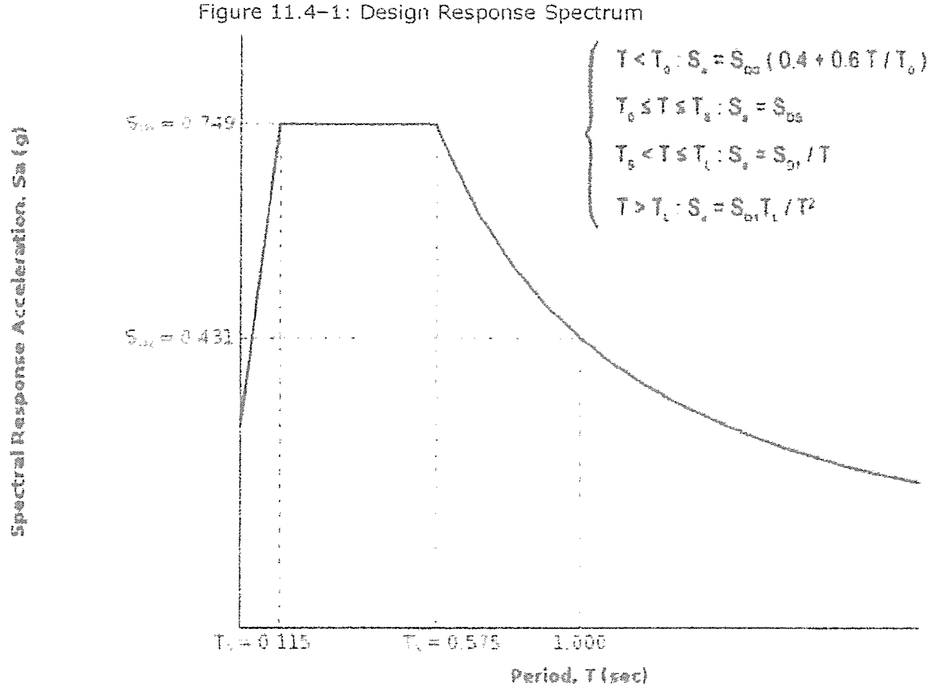
Equation (11.4-3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.124 = 0.749$ g

Equation (11.4-4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.647 = 0.431$ g

Section 11.4.5 — Design Response Spectrum

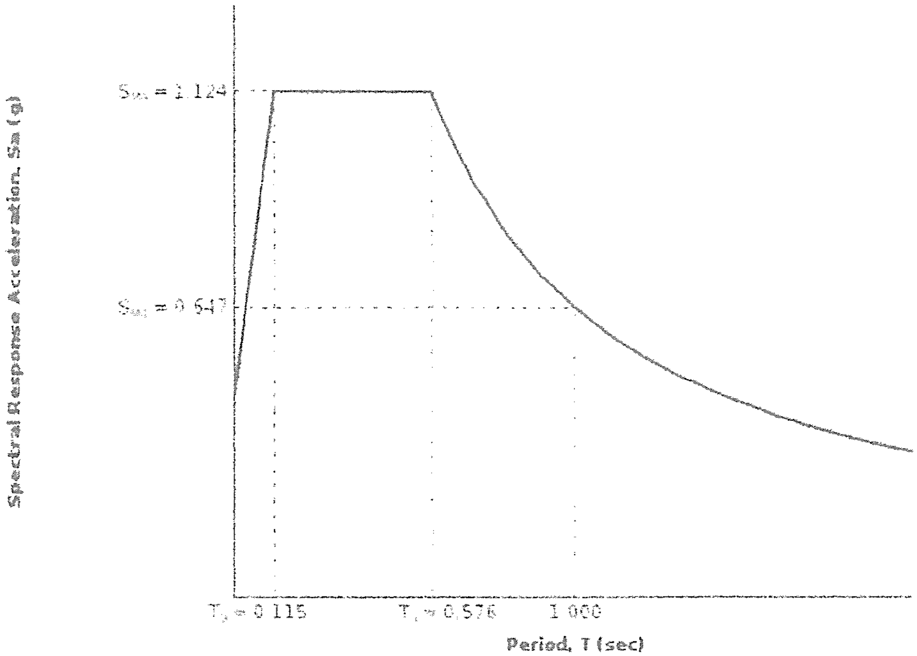
From Figure 22-12[3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



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

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



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

From Figure 22-7[4] PGA = 0.387

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.113 \times 0.387 = 0.431$ g

Table 11.8-1: Site Coefficient F_{PGA}

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

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

For Site Class = D and PGA = 0.387 g, F_{PGA} = 1.113

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

From Figure 22-17[5] C_{RS} = 1.021

From Figure 22-18[6] C_{R1} = 1.064

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 0.749 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.431 g, Seismic Design Category = D

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

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

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

References

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

GEOTECHNICAL MAP

TENTATIVE SUBDIVISION MAP

COUNTY OF SAN DIEGO TRACT NO. 5443 RPL 3

GENERAL NOTES:

1. THE INTERNAL STREET WILL BE A PRIVATE STREETS
2. GRADING AND IMPROVEMENTS SHALL BE IN ACCORDANCE WITH COUNTY STANDARDS.
3. EASEMENTS OF RECORD NOT SHOWN HEREON SHALL BE HONORED, ABANDONED AND/OR RELOCATED TO THE SATISFACTION OF ALL INTERESTED PARTIES, AND PUBLIC UTILITY EASEMENTS NECESSARY TO SERVE THIS PROJECT WILL BE COORDINATED WITH SERVING UTILITY COMPANIES.
4. LOT DIMENSIONS AND AREAS SHOWN HEREON ARE APPROXIMATE. THE DIMENSIONS MAY BE ADJUSTED TO BE CONSISTENT WITH THE FINAL MAP.
5. THIS PLAN IS PROVIDED TO ALLOW FOR FULL AND ADEQUATE DISCRETIONARY REVIEW OF A PROPOSED DEVELOPMENT PROJECT. THE PROPERTY OWNER ACKNOWLEDGES THAT THE ACCEPTANCE OR APPROVAL OF THIS PLAN DOES NOT CONSTITUTE AN APPROVAL TO PERFORM ANY GRADING SHOWN HEREON, AND AGREES TO OBTAIN VALID GRADING PERMISSIONS BEFORE COMMENCING SUCH ACTIVITY.
6. ZONING REQUIREMENTS:

	EXISTING
USE REGULATIONS:	RR .5
NEIGHBORHOOD REGS	J
DENSITY	2
LOT SIZE (AC)	.5 ACRE
BUILDING TYPE	–
MAX. FLOOR AREA	–
FLOOR AREA RATIO	–
HEIGHT	G
COVERAGE	–
SETBACK	G
OPEN SPACE	–
SPECIAL AREA REGS	–

TOTAL LOTS AND AREA:

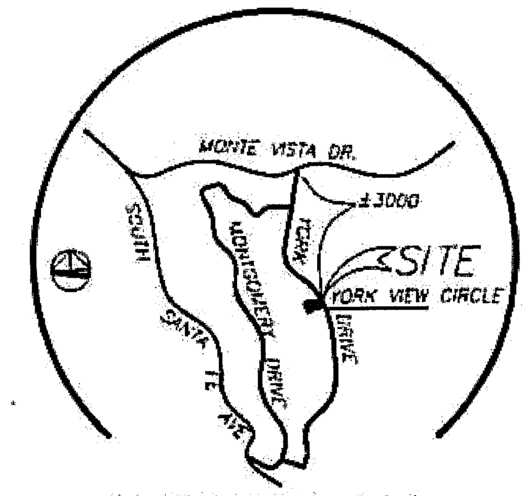
ACREAGE: 3.00 AC GROSS
2.82 AC. NET (EXISTING)
LOTS: (RESIDENTIAL) 5
MIN. LOT SIZE: 0.5 ACRE
NO. OF DWELLING UNITS: 5 UNITS
GENERAL PLAN DESIGNATION: RESIDENTIAL 3
REGIONAL CATEGORY: CUDA
COMMUNITY PLAN: NORTH COUNTY METRO

UTILITIES:

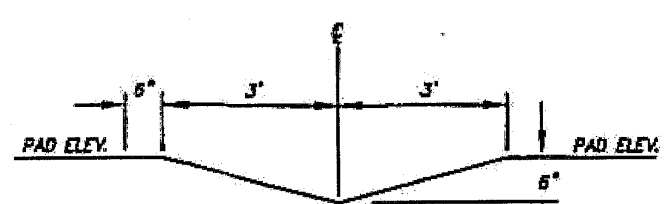
SEWER:	BUENA SANITATION DISTRICT
WATER:	VISTA IRRIGATION DISTRICT
SCHOOLS:	VISTA UNIFIED SCHOOL DISTRICT
FIRE:	VISTA FIRE PROTECTION DISTRICT
TELEPHONE:	SBC
GAS & ELECTRIC:	SAN DIEGO GAS & ELECTRIC CO.
STREET LIGHTING:	NO STREET LIGHTS ARE PROPOSED

LEGEND:

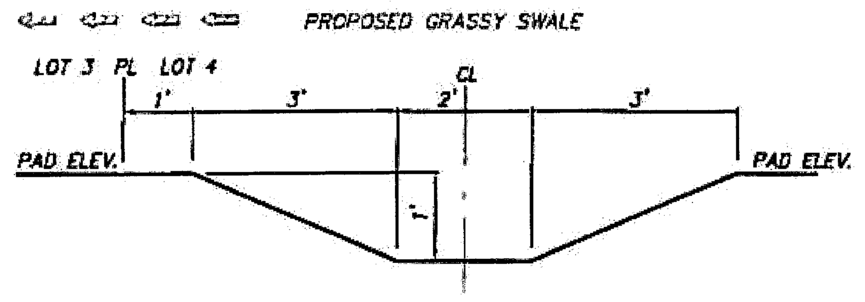
PROPOSED FIRE HYDRANT
EXISTING FIRE HYDRANT
EXISTING SEWER
EXISTING SEWER MANHOLE
2 PROPOSED LOT NUMBER
PE 465 PROPOSED PAD ELEVATION
PROPOSED SLOPE EMBANKMENTS (2:1 MAX) (FILL/CUT)
PROPOSED DAY LITE
470 EXISTING CONTOURS
470 PROPOSED CONTOURS
(W) EXISTING WATER LINE
EXISTING EASEMENT
EXISTING FENCE
EXISTING STRUCTURE
F/C FILL/CUT LINE
PROPOSED BROW DITCH
PROPOSED RCP PIPE
PROPOSED GRASSY SWALE



VICINITY MAP
NO SCALE
TB 1108 C-2



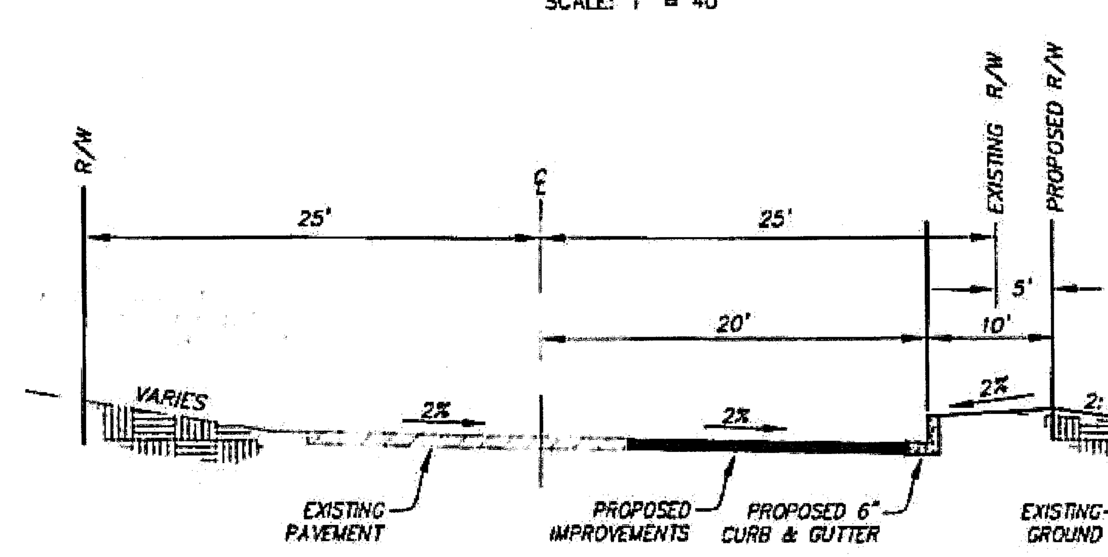
TYPICAL SECTION
GRASSY SWALE
NTS



SECTION A-A
GRASS SWALE – LOT 4
NTS

GEOTECHNICAL LEGEND

▬	Approx. Location of Test Pit
	Geologic Cross-Section
Qcol	Colluvium
Qal	Alluvium
Kgb	Crystalline Bedrock



TYPICAL SECTION
YORK DRIVE (PUBLIC)
NTS

OWNERS / SUBDIVIDER

WE HEREBY CERTIFY THAT WE ARE THE RECORD OWNERS OF THE PROPERTY SHOWN ON THIS TENTATIVE SUBDIVISION MAP AND THAT SAID MAP SHOWS OUR ENTIRE CONTIGUOUS OWNERSHIP (EXCLUDING SUBDIVISION LOTS). WE UNDERSTAND THAT PROPERTY IS CONSIDERED CONTIGUOUS EVEN IF IT IS SEPARATED BY ROADS, STREETS, UTILITY EASEMENTS OR RAILROADS RIGHTS-OF-WAY.

OWNERS

GARY VAN EIK
841 QUAILS TRAIL
VISTA, CA 92081

GARY VAN EIK AS OWNER

ENGINEER OF WORK:

bHA, Inc.
land planning, civil engineering, surveying
5115 AVENIDA ENCINAS
SUITE "L"
CARLSBAD, CA. 92008-4387
(760) 931-8700

REGISTERED PROFESSIONAL ENGINEER MICHAEL H. SMITH No. 65090 Exp. 09/30/09 CIVIL STATE OF CALIFORNIA

MICHAEL H. SMITH RCE 65090 EXP. 9-30-09 3/24/07 DATE

AERIAL TOPOGRAPHY

EARTHWORK QUANTITIES

CUT: 3,300 C.Y. IMPORT: 0 C.Y.
FILL: 3,300 C.Y.

ASSESSOR'S PARCEL NUMBERS

A.P.N. 184-012-12 TAX RATE AREA: 96036

LEGAL DESCRIPTION

PORTIONS OF LOTS 18 AND 19 OF KEW GARDENS, ACCORDING TO MAP NO. 2046, AS FILED ON THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, JULY 15, 1927 TOGETHER WITH A PORTION OF THE WESTERLY HALF OF CLARENCE DRIVE AS SHOWN ON SAID MAP NO. 2046, BEING IN THE COUNTY OF SAN DIEGO, STATE OF CALIFORNIA

SITE ADDRESS: 1505 YORK DRIVE, VISTA, CA

PARK LAND DEDICATION STATEMENT

NO PARKLAND DEDICATION IS BEING PROPOSED, PAYMENT OF FEES WILL BE IN LIEU OF DEDICATION

SPECIAL ASSESSMENT STATEMENT

NO SPECIAL ASSESSMENT ACT PROCEEDING IS REQUESTED FOR THIS PROJECT

SOLAR ACCESS STATEMENT

THIS IS A SOLAR SUBDIVISION AS REQUIRED BY SECTION 81.401 (n) OF THE SUBDIVISION ORDINANCE. ALL LOTS HAVE AT LEAST 100 SQUARE FEET OF UNOBSTRUCTED ACCESS TO SUNLIGHT ON THE BUILDABLE PORTION OF THE LOT.

STREET LIGHTING STATEMENT

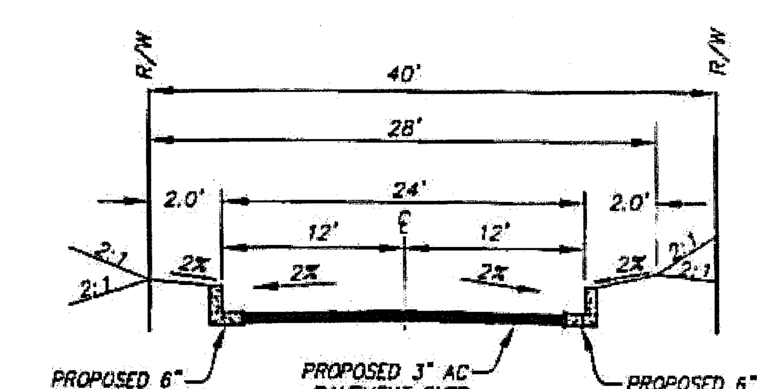
NO STREET LIGHTS ARE PROPOSED – PRIVATE STREETS

ACCESS

ACCESS IS FROM YORK DRIVE WHICH IS A PUBLICLY MAINTAINED ROAD. PROPERTY FRONTS YORK DRIVE.

EASEMENTS

AN EASEMENT TO THE BUENA SANITATION DISTRICT FOR SEWER LINES AND INCIDENTAL PURPOSES, RECORDED NOVEMBER 8, 1965 AS INSTRUMENT NO. 202758 OF OFFICIAL RECORDS.



TYPICAL SECTION
HAPPY DAZE LANE
NTS

PLATE 2
V&M JOB #16-237-P

DECEMBER 2016

K:\Land Projects 3\757-1019-400\dwg\tm\757-1019-400-TM-01.dwg 3/24/2009

JOB # 16-236-S
Test Site Location:
1505 York Drive,
Vista

Storm Water BMP
Percolation Data

12/15/2016

Infiltration BMP Type	Infiltration Basin
Test Method	Shallow Percolation Testing (Option 2)
Factor of Safety	FS=3
Drill Date	12/6/2016
Test Date	12/7/2016
Equipment Type	B-31 Mobile Drill Using Solid Stem Auger
Test Bore Diameter	8" inch
Observation Bore Diameter	6" inch
Groundwater Conditions	No Groundwater Encountered
Weather Conditions	Dry, Partley Sunny, 68° F

Test No.	Depth (*ft*)	Percolation Rate (*mpi*)	Infiltration Rate (in/hr)	
A	3'	5.0	0.86	**BAISN A**
B	3'	3.1	1.51	
C	3'	2.6	1.74	
D	5'	3.1	0.77	**BASIN B**
E	5'	2.1	1.16	
F	5'	2.1	1.16	
G	3'	8.6	0.45	**BASIN C**
H	3'	5.6	0.75	
I	3'	4.7	0.89	

Site Average = 4.1 mpi Site Average = 1.03 in/hr

Infiltration Rates Vary Slightly Due to Actual Depth of Hole In Inches

Depth (ft)	Observation Boring 1	12/6/2016
0 - 3'	Reddish Brown Loamy Sand	
3 - 15'	Brown Clayey Sand	
15 - 19'	Grey Silty Sand (D.G.)	
	End @ 19'	

Depth (ft)	Observation Boring 2	12/6/2016
0 - 3'	Reddish Brown Loamy Sand	
3 - 10'	Brown Clayey Sand	
10 - 19'	Grey Silty Sand (D.G.)	
	End @ 19'	

note: See Geotechnical Report For USCS Soil Classifications

REGISTERED PROFESSIONAL ENGINEER
RALPH MALCOLM VINJE
No. 863
Exp. 12-31-17
GEOTECHNICAL
STATE OF CALIFORNIA

Ralph M. Vinje GE # 863

Vinje & Middleton
Engineering, Inc.

2450 Auto Park Way
Escondido, CA 92029-1229
(760) 743-1214

JOB #16-236-S BASINS #1 & #2

1505 YORK DRIVE, VISTA

Categorization of Infiltration Feasibility Condition			Form I-5
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: SEE ATTACHED INFILTRATION RATE RESULTS WITH MAXIMUM INFILTRATION RATE = 1.74 INCHES/HOUR AND MINIMUM INFILTRATION RATE = 0.45 INCHES/HOUR AND SITE AVERAGE = 1.03 IN/HR. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: HAZARD POTENTIAL IS BASED ON PROPOSED INFILTRATION BASINS WITHIN CLOSE PROXIMITY TO ENGINEERED SLOPES BOTH CUT AND FILL. INFILTRATION RATES INDICATE RAPID WATER MOVEMENT THRU THE SOIL PROFILE. THE RATE AT WHICH WATER IS INFILTRATING THE SOIL WILL ALLOW WATER TO EXIT THE SLOPE FACE. THIS WILL CAUSE SOIL STABILITY ISSUES AS WELL AS SOIL EROSION. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Form I-5			
Criteria	Screening Question	Yes	No
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

SHALLOW GROUNDWATER CONDIDTIONS WERE NOT SEEN AT OBSERVATION BORINGS #1 & #2. BASED ON GROUNDWATER CONDITIONS BEING DEEPER THAN 19' FEET, STORMWATER CONTAMINANTS WILL HAVE ADEQUATED TIME FOR BIOREMEIDIATION WITHIN THE SOIL PROFILE BEFORE THEY ENCOUNTER NATIVE GROUNDWATER.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonally or ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

PROPOSED INFILTRATION OR BIOFILTRATION BASINS WILL INTERCEPT STORMWATER AND PROVIDE CONTAMINATE TREATMENT TO STORMWATER THAT WOULD OTHERWISE SHEET FLOW DIRECTLEY TO THE SEASONAL FLOWLINE ALONG THE SOUTH-WESTERN PROPERTY LINE UNTREATED.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	"NO"

Form I-5

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

SEE ATTACHED INFILTRATION RATE RESULTS WITH MAXIMUM INFILTRATION RATE = 1.74 INCHES/HOUR AND MINIMUM INFILTRATION RATE = 0.45 INCHES/HOUR AND SITE AVERAGE = 1.03 IN/HR AT 3' TO 5' FOOT DEPTH.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

HAZARD POTENTIAL IS BASED ON PROPOSED INFILTRATION BASINS WITHIN CLOSE PROXIMITY TO ENGINEERED SLOPES BOTH CUT AND FILL. INFILTRATION RATES INDICATE RAPID WATER MOVEMENT THRU THE SOIL PROFILE. THE RATE AT WHICH WATER IS INFILTRATING THE SOIL WILL ALLOW WATER TO EXIT THE SLOPE FACE. THIS WILL CAUSE SOIL STABILITY ISSUES AS WELL AS SOIL EROSION.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Form I-5			
Criteria	Screening Question	Yes	No
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

SHALLOW GROUNDWATER CONDIDTIONS WERE NOT SEEN AT OBSERVATION BORINGS #1 & #2. BASED ON GROUNDWATER CONDITIONS BEING DEEPER THAN 19' FEET, STORMWATER CONTAMINANTS WILL HAVE ADEQUATED TIME FOR BIOREMEIDIATION WITHIN THE SOIL PROFILE BEFORE THEY ENCOUNTER NATIVE GROUNDWATER.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

8	**Can infiltration be allowed without violating downstream water rights?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

DOWNSTREAM WATER RIGHTS VIOLATIONS ARE EXPECTED TO REMAIN UNCHANGED AND NON-EXISTING.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Part 2 Result*	If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	NO INFILTRATION

Form I-5 Certification

The Geotechnical Engineer certifies they completed Form I-5 except Criteria 4 & 8 (see Appendix C.4.3).

Professional Geotechnical Engineer's Printed Name:

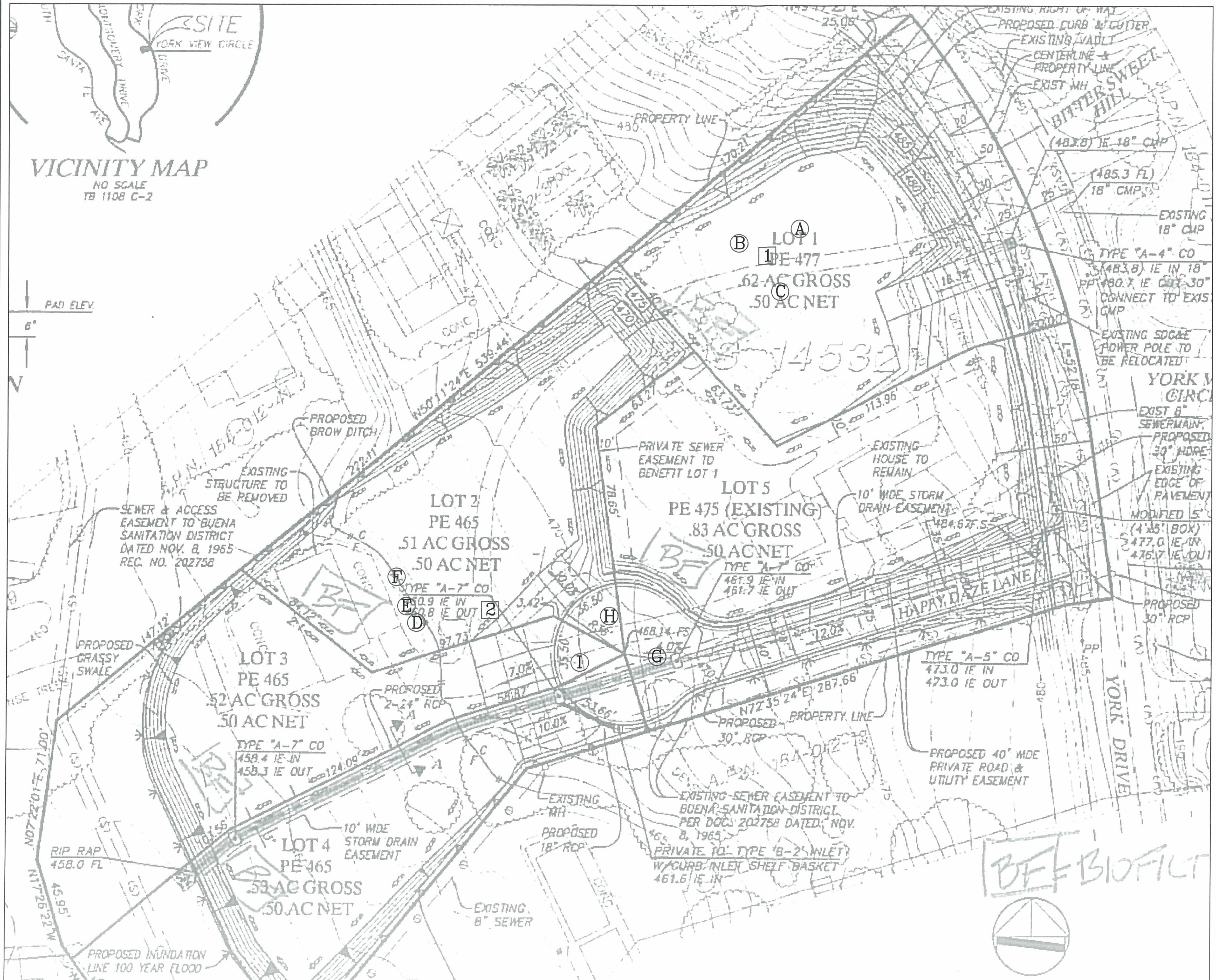
RALPH MALCOLM VINJE

Professional Geotechnical Engineer's Signed Name:

Date: 12-20-16

[SEAL]

REGISTERED PROFESSIONAL ENGINEER
RALPH MALCOLM VINJE
No. 863
Exp. 12-31-17
GEOTECHNICAL
STATE OF CALIFORNIA



VINJE & MIDDLETON ENGINEERING, INC.
2450 Auto Park Way
Escondido, CA 92029-1229
760-743-1214

Ralph M. Vinje GE #863

LOT INFORMATION

DATE: 12/16/2016
JOB No. 16-236-S
APN 184-012-12
ACRES 2.77

Owners Address	Site Location
Gary Van Eik 841 Quails Trail Vista, CA 92081 c. 858-453-1331	1505 York Drive, Vista

c/o: Bruce Rice @ 760-931-8700 ext. 236
brice@bhaincsd.com

LEGEND

Ⓐ ← PERC TEST LOCATION
1 ← OBSERVATION BORING
475 ～ CONTOUR LINE W/ 1' INTERVAL

LOT #1 = BASIN A
LOT #2 = BASIN B
LOT #5 = BASIN C

SCALE 1" = 50'
N

FIGURE 1

Infiltration Test Boring Location Map