Appendix J Geotechnical Investigation Proposed Road Improvements (September 2016)

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Vinje & Middleton Engineering, Inc.

Geotechnical Investigation

Proposed Road Improvements
Bear Valley Parkway
Escondido, California

September 27, 2016

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GEOTECHNICAL INVESTIGATION
PROPOSED ROAD IMPROVEMENTS
BEAR VALLEY PARKWAY
ESCONDIDO, CALIFORNIA

I. INTRODUCTION

Portions of Bear Valley Parkway, adjacent to 661 Bear Valley Parkway, are planned for widening and improvements in association with a multi-family residential development at 661 Bear Valley Parkway. A Vicinity Map showing the study roadway section is included with this report as Plate 1. This investigation was initiated to determine soil and geotechnical conditions along the planned roadway improvement areas and to ascertain their influence upon the proposed improvements. Test pit digging, geologic mapping, soil sampling, and laboratory testing were among the activities conducted in conjunction with this effort which has resulted in the grading and paving recommendations presented in the following sections.

Geotechnical conditions at the proposed residential development, adjacent to the proposed roadway improvements, were previously studied by this office with our findings, conclusions and recommendations summarized in the following published technical report:

Geotechnical Investigation
Proposed Residential Subdivision
661 Bear Valley Parkway
Escondido, California
Job No. 13-116-P, dated April 3, 2013

The referenced report was reviewed as part of this effort. Portions of the referenced report pertinent to this effort are reproduced herein where applicable and appropriate.

II. SITE DESCRIPTION

Improvement Plans for the project road sections showing the existing roadway and the proposed widening and improvements are included as Plates 2 through 5. The study portion of Bear Valley Parkway is between Choya Canyon Road to the north and Sunset Drive to the south.

Topographically, the existing Bear Valley Parkway descends in a southerly direction with approximately 135 feet in relief along the 3,500 feet of the project alignment. More recent improvements, including concrete curb & gutter and sidewalks, are present along a 500 foot section adjacent to Zlatibor Ranch Road.

Noted areas include:

- Choya Ranch Road to Zlatibor Ranch Road: Natural hilly areas and remnants of an old mine (spoils) are present along the westerly side of Bear Valley Parkway from Choya Ranch Road to the improved areas north at Zlatibor Ranch Road. The east side of Bear Valley Parkway mostly consists of gentle to modest natural terrain.

- Zlatibor Ranch Road to Encino Drive: The westerly side consists of natural terrain with local shallow fills that extend to approximately 500 feet from Encino Drive. Between this location and the entrance to Encino Drive, a local canyon flowline has been filled to allow the continuation of Bear Valley Parkway. The canyon road fill embankments are steep and heavily overgrown. Based on our field observations, the fill slopes generally approach 20 feet high maximum, and appear to locally approach 1:1 gradients.

  On the easterly side of Bear Valley Parkway, between Zlatibor Ranch Road and the filled-in canyon crossing, are existing cut slopes that approach near vertical and range to approximately 12 feet high. Much of the face of the slopes expose natural colluvial soils which are impacted by erosional ruts due to uncontrolled upslope runoff.

- Encino Drive to Sunset Drive: The westerly side is marked by developed residential properties that access Bear Valley Parkway. Ground surfaces appear generally natural with some minor modifications.

  The north portion of the easterly side of Bear Valley Parkway is marked by fill embankments that descend up to 20 feet, at locally oversteepened 1:1 gradients, to the flowline that was traversed at Encino Drive. The south portion of this section transitions into gentle to modest natural terrain, locally marked by erosional ruts. The apparent contact between the north and south sections is an access drive that was constructed across the flowline. An inlet and outlet with a connecting culvert allow the flowline runoff to continue southward under the access drive. Much of this entire length of roadway is heavily overgrown with limited to no access.

Documentation pertaining to the construction of Bear Valley Parkway along the study alignment is not available for review.

III. PROPOSED DEVELOPMENT

The entire length of the study portion of Bear Valley Parkway is planned for widening with new curb & gutter and sidewalk improvements outside of the existing improved section adjacent to Zlatibor Ranch Road.

New road construction will include the following:

- A large cut slope for roadway widening is proposed on the westerly side of Bear Valley Parkway, north of Zlatibor Ranch Road. The cut slope is programmed for 2:1 gradients maximum and will daylight into natural terrain to the north and south. The cut slope will approach 25 feet high maximum in the central portion of the new graded embankment.

- Ground transitioning for roadway widening at the existing canyon fill slope constructed across the flowline at Encino Drive and the fill slope ascending from the flowline along east side of Bear Valley Parkway between Encino Drive and Sunset Drive will be provided by a variable height retaining walls that will approach 13 feet high maximum.

- Existing oversteepened cut slopes will be reconstructed to 2:1 gradients along the easterly side of Bear Valley Parkway between Encino Drive and Zlatibor Ranch Road to allow for roadway widening.

- Elsewhere, roadway widening elevations will be very near to existing grades adjacent to Bear Valley Parkway.

- New driveways and accesses will be constructed for existing residential properties along Bear Valley Parkway.

IV. SITE INVESTIGATION

Geotechnical conditions along the study roadway were chiefly determined by a review of relevant test pits excavated near Bear Valley Parkway in connection with the referenced Geotechnical Report and the excavation of 5 additional test pits dug with a track-mounted Caterpiller hoe. Test pit locations were limited by existing graded embankments, existing roadway and private improvements, and underground utilities. All the excavations were logged by our project geologist who also retained representative soil / rock samples for laboratory testing. Test Pit locations are shown on the attached Improvement Plans, Plates 2-5. Logs of the recently excavated test pits and copies of the pertinent test pits from the referenced report are included as Plates 5 through 14. Laboratory test results and engineering properties of selected samples are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

Northern portions of the project roadway improvement areas are characterized by natural nearly level to modest terrain underlain at depth by crystalline bedrock units and mantled by ancient colluvial soils. The central and southern portions of the roadway include oversteepened graded cuts and fills that generally approach 20 feet high. These areas are also expected to be underlain by crystalline bedrock units which are mantled by fills, ancient colluvium soil, and young alluvial soils. Landslides or areas of existing slope instability are not in evidence along the planned roadway improvements. The following earth materials are recognized:

A. Earth Materials

Bedrock (Kgb): Crystalline bedrock units underlie all of the studied areas at or very near the surface. Exposures are typically fine to coarse grained granitic to gabbroic rocks that occur in a weathered and friable condition near the surface and grade to a hard condition quickly. Bedrock at the project site also include corestone units. These are spherical boulders of harder rock scattered throughout the surrounding bedrock. Larger corestones, if encountered during grading can create removal and disposal problems. Project bedrock are competent units which will adequately support new fills and road improvements.

Colluvium (Qcol): Much of the existing and proposed roadway areas are underlain directly by a modest to thick mantle of colluvial soils. As exposed, site colluvium consists largely of fine to medium grained sandy deposits. Developed exposures appear ancient and consolidated at depth. Overall, upper colluvium deposits were found in damp and loose conditions that grade more uniformly medium dense to dense at depth.

Alluvium (Qal): Local alluvium deposits are present within the local flowline in the south areas of the property. Due to inaccessibility of the flowline, test pits were not excavated in the alluvium. However, based on the previous geotechnical investigation, site alluvium is anticipated to occur in moist and loose to very loose conditions overall.

Fill (af): As exposed, site fill deposits consist of silty sand that locally includes trash debris. The fill was found in damp and loose to medium dense conditions overall.

The approximate distribution of earth deposits at the site are shown on the enclosed Improvement Plans, Plates 2-4. Details of site earth materials are given on the enclosed test pit logs, Plates 5-14. The subsurface relationship of site earth materials is depicted on Geologic Cross-Sections enclosed with this report as Plate 15.

B. Slope Stability

Project natural hillsides are underlain by competent crystalline bedrock units which typically perform well in natural and graded slope conditions. Slope instability is not indicated along the project natural hillside areas. Oversteepened graded fill slopes associated with ground transitioning along (or across) the existing flowline are heavily overgrown and appear to be currently performing well with no indication of instability. However, consideration should be given to reconstructing the slopes at 2:1 gradients minimum as part of the project development / street improvements.

Future graded cut embankments exposing crystalline bedrock are expected to be grossly stable to anticipated design heights. Graded cut slopes exposing natural colluvium or surficial soil should be track-walked with heavy construction equipment and compacted to 90% minimum on the slope face. Mine spoil, if exposed, should be entirely removed and reconstructed with compacted fills, as directed in the field by the project geotechnical engineer. All graded slopes should be constructed as recommended herein and provided with well-developed brow ditches. Runoff should not be allowed to occur in concentrated flow conditions or flow over slope faces.

C. Surface Flow and Subsurface Groundwater

Subsurface water was not encountered in project test excavations to the depths explored. However, portions of the flowline adjacent to the project roadway has intermittent surface flow. The tributary flowline will be subject to seasonal conditions and varying degrees of flow conditions. Storm water control is critical to the stability of project new road widening. Uncontrolled runoff should not be allowed to flow over graded surfaces. Storm water runoff control facilities should be designed and installed as necessary and appropriate. Rock-lined sidewalls or armor protection for the flowline crossing or fill slopes adjacent to the flowline may be necessary to limit erosion.

Project retaining walls should be provided with an adequate back drain system which should outlet into approved locations.

D. Seismic Ground Motion Values

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

E. Laboratory Testing / 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 1

Soil Type	Description
1	Brown / red brown silty sand - Road Fill (Raf) / Colluvium (Qcol)
3	Grey fine to coarse sand - Bedrock (Kgb)

The following tests were conducted in support of this investigation:

1. **Sand Equivalent Test (S.E.):** Sand equivalent tests were performed on representative samples of Soil Type 1 in accordance with California Test 217. The test results are presented in Table 2.

TABLE 2

Location	Soil Type	Description	Sand Equivalent (S.E.)
TP-101 @ 2'	1	Brown silty sand	16
TP-104 @ 2'	1	Brown silty sand	18

2. **Maximum Dry Density and Optimum Moisture Content:** The maximum dry density and optimum moisture content of Soil Type1 was determined in accordance with ASTM D-1557. The results are presented in Table 3.

TABLE 3

Location	Soil Type	Maximum Dry Density (γ_m-pcf)	Optimum Moisture Content (ω_{opt}-%)
TP-104 @ 2'	1	136.5	8.8

3. **Moisture-Density Tests (Undisturbed Chunk Samples):** In-place dry density and moisture content of representative soil deposits beneath the site were determined from relatively undisturbed chunk samples using the water displacement test method. Results are presented in Table 4 and tabulated on the attached Test Pit Logs.

TABLE 4

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (Yd-pcf)	Max. Dry Density (Ym-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP-101 @ 2'	1	3	117.6	136.5	86	18
TP-101 @ 4'	1	7	119.0	136.5	87	44
TP-102 @ 2'	1	7	119.0	136.5	87	44
TP-102 @ 3'	1	7	124.2	136.5	91	51
TP-103 @ 2'	1	3	113.7	136.5	83	16
TP-103 @ 4'	1	7	123.0	136.5	90	48
TP-104 @ 2'	1	6	107.7	136.5	79	28
TP-104 @ 4'	1	7	119.3	136.5	87	44
TP-104 @ 6'	1	8	122.3	136.5	90	55
TP-105 @ 4'	1	4	111.8	136.5	82	21
Note 1: Sample may be somewhat disturbed. Assumptions And relationships: In-place Relative Compaction = (Yd ÷ Ym) X100 Gs = 2.75 e = (Gs Yω ÷ Yd) - 1 S = (ω Gs) ÷ e						

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

TABLE 5

Sample Location	Soil Type	Sample Condition	Wet Density (Yw-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
TP-104 @ 2'	1	remolded to 90% of YM @ % ωopt	133	28	68

5. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 1 was determined using "Method for Estimating the Service Life of Steel Culverts," in accordance with 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-104 @ 2'	1	2912	6.3

6. **Sulfate Test:** A sulfate test was performed on a representative sample of Soil Type 1 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-104 @ 2'	1	0.006

7. **Chloride Test:** A chloride test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 422. The test result is presented in Table 8.

TABLE 8

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
TP-104 @ 2'	1	0.002

8. **R-Value Test:** Two R-value test were performed on representative samples of Soil Type 1 in accordance with the California Test Method 301. The test results are presented in Table 9.

TABLE 9

Location	Soil Type	Description	R-Value
TP-102 @ 2'	1	Brown silty sand	37
TP-104 @ 2'	1	Red brown silty sand	64

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 greater 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 0.006 percent by weight which is considered negligible according to ACI 318, Table 4.3.1. Portland cement Type II may be used. Table 10 is appropriate based on the pH-Resistivity test result:

TABLE 10

Design Soil Type	Gage	18	16	14	12	10	8
1	Years to Perforation of Steel Culverts	17	22	27	37	47	57

VII. CONCLUSIONS

Based upon the foregoing investigation, subgrade and bearing soil underlying the planned roadway improvements range from loose silty sand fill / surficial soil deposits to dense ancient colluvium and very dense crystalline bedrock units. The following geotechnical factors will most impact the planned road improvements and associated construction costs:

- The existing roadway along the study areas are scheduled to remain. The existing road will be widened to include new travel lanes, curb & gutter, and sidewalk along the entire length of the study areas outside of the existing improved areas adjacent to Zlatibor Ranch Road.

- Documentation pertaining to existing fills, backfills, and graded embankments occurring along the existing roadway alignment is not available for review. Site fills are relatively thick deposits and were generally placed atop natural topsoils. Existing loose fills and upper natural soils in planned improvement areas should be regraded as a part of project earthwork operations as recommended below.

- Underlying bedrock and dense natural soils are competent units which will adequately support the planned improvements.

- Gross geologic instability is not indicated or expected within the project natural terrain.

- New roadway graded cut slopes are programmed at 2:1 gradients with an estimated maximum vertical height approaching 25 feet. New cut embankments exposing surficial soil deposits should be track-walked with heavy construction equipment to enhance surficial stability.

- Existing fill embankments constructed as transition slopes adjacent to Bear Valley Parkway, and canyon fill slopes are constructed mostly at oversteepened gradients that approach 1:1. Documentation for the existing fill slope construction or fill placement is not available for review. These fills are generally not deemed suitable for support of new transition walls and may need re-grading. Deep foundations for the walls on oversteepened fill slopes may also be appropriate.

  The overall stability of graded embankments and roadway surfaces developed over sloping terrain is most dependent upon adequate keying and benching of fill into the undisturbed dense colluvium or bedrock during the grading operations. At the project site, added care should be given to the proper construction of fill slope keyways and subsequent hillside benching.

- Based on our observation and subsurface exploratory excavations, moderate ripping utilizing medium to large dozers (Caterpillar D-8 or equivalent) will likely be required to achieve final design grades in the deeper cut areas of the project. The need for specialized ripping and / or rock breakers are not anticipated.

- Surface drainage and storm water control are very important in the future performance of the project roadway improvements. Storm water control facilities should be constructed as shown on the project civil drawings.

VIII. RECOMMENDATIONS

The following recommendations are consistent with the indicated geotechnical conditions along the project roadway alignment and should be considered for designs and implemented during the construction phase. All excavations, grading, earthwork, construction, and bearing soil preparation should be completed in accordance with Chapter 18 (Soils and Foundations) and Appendix "J" (Grading) of the 2013 California Building Code (CBC), the Standard Specifications for Public Works Construction, City of Escondido Grading Ordinances, the requirements of the governing agencies and following sections, wherever appropriate and as applicable. Added or modified recommendations may also be appropriate as directed by the project geotechnical consultant in the field at the time of grading and construction and should be anticipated:

A. Remedial Grading and Earthworks

1. **Clearing and Grubbing:** Surface vegetation, deleterious materials and debris should be removed from the roadway improvement areas, plus 5 feet outside the perimeter where possible, and as directed in the field. Ground preparation should be inspected and approved by the project geotechnical engineer or his designated field representative prior to remedial grading.

   Existing underground utilities in the roadway construction areas should be pot-holed, identified and marked prior to the actual work.

   Inactive lines should be properly removed or abandoned as approved. Abandoned underground structures should also be removed and the generated voids properly backfilled with compacted soils in accordance with the recommendations provided herein.

2. **Over-excavations and Removals:** The most effective method to mitigate loose and compressible subgrade and bearing soils will utilize removal and recompaction remedial grading techniques. The existing upper fills and loose colluvium in planned roadway, retaining walls, and underground utilities plus a minimum of 5 feet outside the perimeter where possible, and as directed in the field, should be over-excavated to well-compacted fills, dense colluvial deposits, or competent bedrock and placed back as properly compacted fills. Bottom of all removals should be additionally prepared and recompacted to a minimum depth of 6 inches. Bottom of all trenches should be over-excavated to a minimum depth of 12 inches and reconstructed to design invert elevations with compacted fills.

   Based on our exploratory excavations, removal depths will vary along the roadway alignment and are expected to be on the order of 2 feet to more than 7 feet. However, locally deeper removals may be necessary particularly near the project flowline areas and should be anticipated. Fills/backfills can only be placed on level horizontal surfaces. Consequently, all ground surfaces steeper than 5:1 maximum receiving fills/backfills shall be horizontally benched and keyed as recommended herein and as directed in the field.

In the event over-excavations to suitable material cannot be achieved, ground stabilization techniques using Geogrid reinforcement may be used. For this purpose, the upper fills/topsoils should be removed as directed by the project geotechnical engineer. Removal depths will be determined in the field based upon actual exposures. A layer of Tensar BX-1200 (or greater from the same series) should be placed at the prepared bottom of over-excavation as directed in the field. Backfill placement can then proceed atop the Geogrid. Additional layers of Tensar BX-1200 may also be necessary within the compacted fill mass to construct stable and non-yielding conditions as directed in the field and should be anticipated. The upper most layer of geogrid should be at least 12 inches below the bottom of the deepest utility.

3. **Excavation Characteristics:** Project cuts and undercuts will likely excavate with moderate efforts. Harder rock conditions within deeper cut areas may require heavy ripping using large bulldozers (Caterpillar D-8 or equal) or excavators. Local corestones or large boulders may also be encountered during cut and remedial grading, and over-excavations which will require specialized and concentrated excavation efforts.

4. **Preparation of Wall Foundation Bearing Soils:** Over-excavation for treatment of bearing soil under the proposed wall foundations should be extended to well compacted fills or dense native ground and placed back as a properly compacted fill. Removals should extend a minimum of 3 feet laterally outside the wall foundations on both sides, where possible, and as directed in the field by the project geotechnical engineer. Benches slightly heeled back into the hillside as the wall backfill placement progresses will be necessary. There should be a minimum of 10 feet to daylight between the bottom outside edge of the wall footing and face of slope.

   New retaining walls are planned along existing 1:1 fill slopes. Oversteepened fill embankments are generally considered unsuitable for retaining wall support. Consideration should be given to reconstructing the fills slopes to more conventional 2:1 slopes. Alternatively, wall foundations may be supported on deep foundations embedded into the underlying competent ancient colluvium or crystalline bedrock. Additional recommendations can be given when specific wall details are known.

5. **Wall Back Drainage System:** A well-functioning back drainage system should be constructed behind all retaining wall type foundations. The wall back drainage system should consist of a minimum 4-inch diameter, Schedule 40 (SDR 35)

perforated pipe surrounded with a minimum of 1.5 cubic feet per foot of ¾-crushed rocks (12 inches wide by 18 inches deep) installed at the depths of the wall foundation level and wrapped in Mirafi 140N filter fabric. If Caltrans Class 2 permeable aggregate is used in lieu of the crushed rocks, the filter fabric can be deleted.

The wall back drain should be installed at suitable elevations to allow for adequate fall via a 4-inch diameter non-perforated solid pipe (SDR 35) to an approved outlet. A typical wall back drain system is depicted on the enclosed Plate 16. Provide adequate waterproofing as indicated on the approved project drawings. Protect pipe outlet(s) as appropriate. Wall back drains and outlet locations should also be shown on the project final as-build plans.

6. **Non-uniform Subgrade Soil Transitioning:** Foundation bearing and subgrade soil transitioning from excavated cut to placed fills should not be permitted underneath the proposed roadway and improvements. The cut portion of foundation bearing and subgrade transition areas should be undercut to a minimum depth of 12 inches below the bottom of footing/keyway or finish subgrade and reconstructed to design elevations as compacted fills. In the underground utility trenches there should also be a minimum of 12 inches of compacted fills below the pipe inverts.

7. **Trenching and Temporary Construction Slopes:** Excavations and removals adjacent to the existing underground pipes, utilities and improvements should be done under inspection of the project geotechnical engineer. Undermining existing improvements and structures by the removal operations shall not be allowed. Temporary construction slopes should be set back a minimum of 1-foot from the existing pipes, structures and improvements unless otherwise specified.

   Temporary trench excavations/embankments less than 5 feet high maximum may be constructed at near vertical gradients unless otherwise specified. Temporary trench side walls and construction slopes greater than 5 feet and less than 15 feet may be constructed at near vertical gradients within the lower 5 feet and laid back at 1:1 gradient within the upper portions with the remaining wedge of soil benched out and new backfills tightly keyed-in as the backfill placement progresses. Temporary trench and construction slopes greater than 5 feet maximum constructed at near vertical gradients will require shoring/trench shield support. In all cases groundwater, if encountered, shall be lowered at least 2 feet below the bottom of temporary trench/excavation slopes.

   All temporary construction slopes require continuous geotechnical inspection during the grading operations. Additional recommendations including revised slope gradients, setbacks and the need for temporary shoring support should be given at that time as necessary. The project contractor shall also obtain appropriate permits, as needed, and conform to the Cal-OSHA and local governing agencies requirements for trenching / open excavations and safety of the workmen during construction.

8. **Fill Materials and Compaction:** On-site weathered bedrock and fill / colluvial excavations will predominantly generate good quality sandy deposits suitable for reuse as compacted site fills. Local trash debris may be expected within site existing fills.

   Project fills shall be clean deposits free of trash, debris, organic matter and deleterious materials consisting of minus 6-inch particles and include at least 40% finer than #4 sieve materials by weight. Rocks more than 6-inches should be properly disposed of from the site.

   Site fill should be adequately processed, thoroughly mixed, moisture conditioned to slightly (2%) above the optimum moisture levels, placed in thin uniform horizontal lifts and mechanically compacted to a minimum of 90% of the corresponding laboratory maximum dry density per the ASTM D-1557, unless otherwise specified. Fills and backfills placed within the project flowline areas, and where subject to potential saturations or flood inundations, should be mechanically compacted to a minimum 95% of the laboratory maximum dry density (ASTM D-1557).

   Subgrade soils below the asphalt pavement base layer should be compacted to at least 95% of the corresponding laboratory maximum dry density within the upper 12 inches. Additionally, all fills/backfills placed within areas subject to potential flooding/inundations should be compacted a minimum of 95% levels.

9. **Permanent Road Embankment Slopes:** Permanent road fill and cut embankment slopes should be constructed at 2:1 gradients maximum. Road embankment slopes constructed as recommended herein, will be grossly stable with respect to deep seated and surface failures for the anticipated maximum design heights.

   All embankment fill slopes shall be provided with a lower keyway. The keyway should maintain a minimum depth of 2 feet into the dense natural soil or competent bedrock with a minimum width of 15 feet. The keyway should expose dense natural soil or competent bedrock throughout with the bottom heeled back a minimum of 2% into the natural hillside and inspected and approved by the project geotechnical engineer. In the flowline areas where dense natural soil or bedrock depths can not be achieved and yielding bottom of keyway excavations are encountered, a layer of Tensar BX-1200 (or greater from the same series) stabilization Geogrid should be placed at the prepared bottom as directed in the field. Additional layers of Geogrid may be required within the compacted fill mass and should be anticipated.

Additional level benches should be constructed into the natural, or graded, hillside as the fill slope construction progresses. Fill slopes should also be compacted to 90% (minimum) of the laboratory standard out to the slope face. Over-building and cutting back to the compacted core, or backrolling at maximum 4 feet vertical increments and "track-walking" at the completion of grading is recommended for site fill slope construction. Geotechnical engineering inspection and testing will be necessary to confirm adequate compaction levels within the fill slope face.

Based on our observations and engineering analyses, gross instability is not indicated or expected within the planned 2:1 cut embankments exposing competent crystalline rocks or dense colluvium. However, some erosion, and sediment washout as well as minor debris fall-outs from above and face of the graded slope, cannot be ruled out and may be anticipated. A raised curb along the edge of pavement and a drainage ditch should be considered along the toe of the project cut embankments. In some cases, a debris fence constructed along the toe of project cut slopes may also become necessary as determined in the field by project geotechnical consultant at the time of slope construction. Graded embankment cut slopes should also be inspected and approved by the project geotechnical consultant during the grading to confirm stability. Additional and/or revised recommendations will be given at that time, if necessary.

All graded cut slopes exposing natural soil should be track-walked with heavy construction equipment to achieve 90% minimum compaction on the face of slope, as directed by the project geotechnical engineer.

10. **Embankment Cut Slope Toe Drainage:** Graded cut embankments developed into the site bedrock units may discharge up-slope drainage along the toe. Nearby pavements and improvements can best be protected by a toe drainage constructed along the base of the slope. Slope toe drains, if required, should consist of a minimum 4-inch diameter, Schedule 40 (SDR 35) perforated pipe surrounded in a minimum of 2.25 cubic feet, per foot, of ¾-inch crushed rocks (1½ feet by 1½ feet trench), wrapped in filter fabric (Mirafi 140-N or equivalent), or Caltrans Class 2 permeable aggregate. Filter fabric can be eliminated if Caltrans Class 2 permeable material is used. The subdrain shall be installed at suitable elevation to ensure positive drainage into an approved drainage facility. The location of the proposed subdrain, if necessary, should be provided by the project geotechnical engineer in the field at the time of grading.

11. **Surface Flow, Subsurface Water and Dewatering:** A critical element to the continued stability of the roadways and graded embankments is adequate surface drainage and storm water control. This can most effectively be achieved by adequate pavement surface cross-fall and the installation of drainage and storm water control facilities. Excessive or concentrated sheetflow over project

embankment slopes will cause erosion and shall not be allowed. Drainage swales should be constructed along the top of all graded slopes. Surface run-off should be collected and directed to a selected location in a controlled manner.

Inundation or flooding of pavement surfaces due to heavy rains and major storm events should be prevented by establishing final pavement surfaces at suitable elevations. Roadway surfaces and face of graded embankments should be protected from storm waters by the construction of flood control structures and slope face rip-rap armor as appropriate. Flood/storm water control facilities should be installed per approved drainage plans.

Slopes constructed in areas subject to inundation should be compacted to a minimum of 95% and may require armor / rip-rap protection to limit erosion and protect the lower portions of the graded embankments.

Special ground stabilization and remedial grading techniques may be required for the grading and construction / improvement within or near the project flowlines.

Surface flow can be expected within the flowline adjacent to Bear Valley Parkway. Actual surface flow conditions within the site flowline are expected to be largely seasonal. Surface water within the site flowlines, if occurring, may impact remedial grading and earthwork construction.

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

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

B. Soil Design Parameters

The following soil design parameters are based upon tested representative samples of on-site soils. All parameters should be re-evaluated when the characteristics of the final backfill and bearing soils have been specifically determined:

* Design wet density of soil = 133 pcf.
* Design angle of internal friction of soil = 28 degrees.
* Design active soil pressure for retaining structures = 48 pcf (EFP), level backfill, cantilever, unrestrained walls.
* Design at-rest soil pressure for retaining structures = 70 pcf (EFP), non-yielding, restrained walls.
* Design passive soil pressure for retaining structures = 368 pcf (EFP), level surface at the toe.
* Design coefficient of friction for concrete on soils = 0.34.
* Design net allowable foundation pressure (minimum 12 inches wide by 12 inches deep footings) = 1000 psf.
* Allowable lateral bearing pressure (all structures except retaining walls) for certified on-site soils = 100 psf/ft .

Notes:

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

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

* The indicated net allowable foundation pressures provided herein were determined based on a minimum 12 inches wide by 12 inches 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 2500 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 of 2000 pounds per square foot.

C. Pavement Structural Section Design

The following pavement structural sections are based on tested subgrade R-values of 37 and 64 for Soil Type 1 materials and indicated assumed traffic Index (TI). A copy of the calculations is attached as Appendix B. A minimum section of 3 inches asphalt (AC) over 4 inches of Caltrans Class 2 aggregate base (AB) will be required and is specified herein when a lesser pavement section is indicated by design calculations:

TABLE 11

Design R-value	Design Traffic Index (TI)			
	4.5	5.0	6.0	7.0
37	3" AC over 4" AB	3" AC over 4" AB	4" AC over 4" AB	4" AC over 8" AC
64	3" AC over 4" AB	3" AC over 4" AB	4" AC over 4" AB	4" AC over 4" AC
Class 2 aggregate base (AB) materials shall meet or exceed Caltrans specifications.				

Final pavement sections will depend on the actual R-Value test results performed on finish subgrade soils, design TI and approval of the City of Escondido. All design sections should be confirmed and/or revised as necessary at the completion of rough pavement grading. In the areas where the longitudinal grades exceed 10%, ½-inch asphalt should be added to the design asphalt thickness for each 2% increase in grade or portion thereof. PCC paving should be considered for longitudinal grades over 15%.

Base materials should be compacted to a minimum of 95% of the maximum dry density. Subgrade soils beneath the pavement base layer should also be compacted to a minimum of 95% of the corresponding maximum dry density within the upper 12 inches. Base materials and subgrade soils should be tested for proper moisture and minimum 95% compaction levels and be approved by the project geotechnical consultant prior to the placement of the base or asphalt layers.

Base section and subgrade preparations per structural section design will be required for all surfaces subject to traffic including roadways, travelways, drive lanes, driveway approaches and ribbon (cross) gutters. Driveway approaches within the public right-of-way should have 12 inches subgrade compacted to a minimum of 95% compaction levels and provided with a 95% compacted Class 2 base section per the structural section design. Base layer under curb and gutters should be compacted to a minimum of 95%, while subgrade soils under curb and gutters, and base and subgrade under sidewalks should be compacted to a minimum of 90% compaction levels. Base section may not be required under curb and gutters, and sidewalks in the case of non-expansive subgrade soils (expansion index less than 21). Appropriate recommendations should be given in the final as-graded compaction report.

D. Re-Surfacing Asphalt

Existing pavements may be improved by resurfacing with an asphalt overlay to upgrade pavement performance. Pavement fabric should be placed over areas impacted by the more prominent surface cracks prior to resurfacing as directed in the field by the project geotechnical engineer. Pavement fabric (petromat or approved equal) should be placed per the manufacturer's recommendations.

The surface of the existing pavement should be prepared to the satisfaction of the project geotechnical engineer prior to the placement of the tack coat and pavement fabric. All materials and construction procedures should conform with the Standard Specifications for Public Works Construction (Green Book) standards.

IX. GENERAL RECOMMENDATIONS

1. All remedial grading including over-excavations, suitability of earth deposits used as compacted fills and backfills, and compaction procedures should be continuously inspected and tested by the project geotechnical consultant and presented in the final as-graded compaction report.

2. Adequate staking and grading control are critical factors in properly completing the recommended remedial grading operations. Grading control and staking should be provided by the project grading contractor or surveyor/civil engineer, and is beyond the geotechnical engineering services. Inadequate staking and/or lack of grading control may result in unnecessary additional grading which will increase construction costs.

3. All underground utility trenches should be compacted to a minimum of 90% of the maximum dry density unless otherwise specified. The upper 1-foot under the pavement base layer and in the areas subject to potential flooding or inundations shall be compacted to a minimum of 95% compaction levels. 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.

4. Expansive clayey soils should not be used for backfilling of any site 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 as shown on Plate 16.

5. Final road improvement and drainage plans should reflect preliminary recommendations given in this report. Final 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.

6. A preconstruction meeting between representatives of this office, the property owner or planner, and the grading contractor/paver is recommended in order to discuss grading/paving details associated with the site development.

X. LIMITATIONS

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

Of necessity, we must assume a certain degree of continuity between exploratory excavations and / or natural exposures. It is necessary, therefore, that all observations, conclusions, and recommendations 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 or changing drainage patterns which occur subsequent to issuance of this report.

This report should be considered valid for a period of one year and is subject to review by our firm following that time. If significant modifications are made to your tentative development plan, especially with respect to the height and location of cut and fill slopes, this report must be presented to us for review and possible revision. Vinje & Middleton Engineering, Inc., warrants that this report has been prepared within the limits prescribed by our client with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

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

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

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GE #863

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STATE OF CALIFORNIA

Steven J. Melzer
CEG #2362

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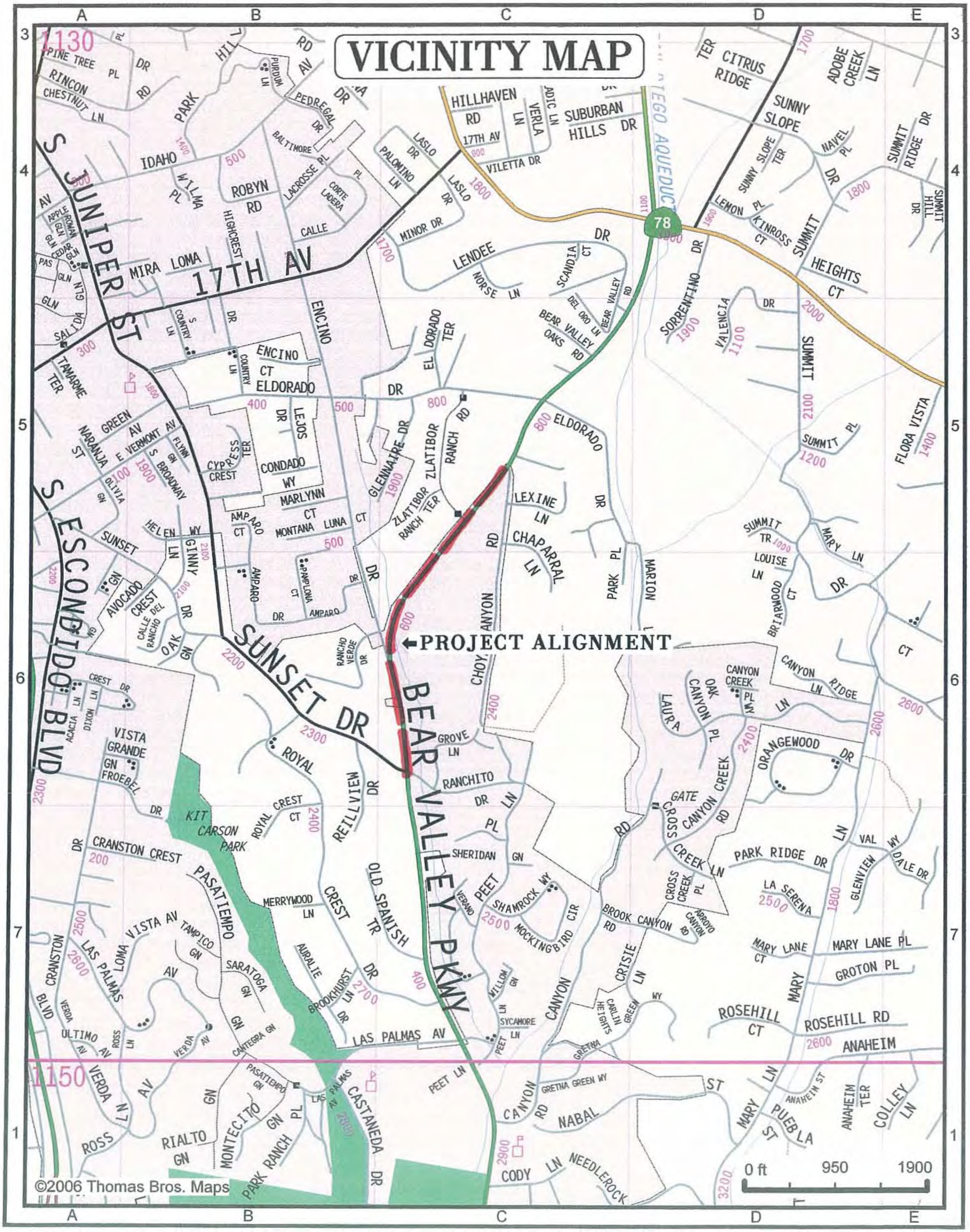

PLATE 1
V&M JOB #13-116-P

THIS AREA IS TO BE DEDICATED TO CITY OF ESCONDIDO AS PART OF PROJECT

THIS AREA TO BE ACQUIRED BY OTHERS & DEDICATED TO CITY OF ESCONDIDO IN FUTURE (TIMEFRAME UNKNOWN)

GEOTECHNICAL LEGEND

- Approx. Location of Exploratory Test Pit (T-1 Excavated 2/28/13; T-101 Excavated 9/13/16)
- Geologic Cross-Section
- *MSaf* — Mine Spoil Deposit
- *Qcol* — Colluvium (Ancient Soil)

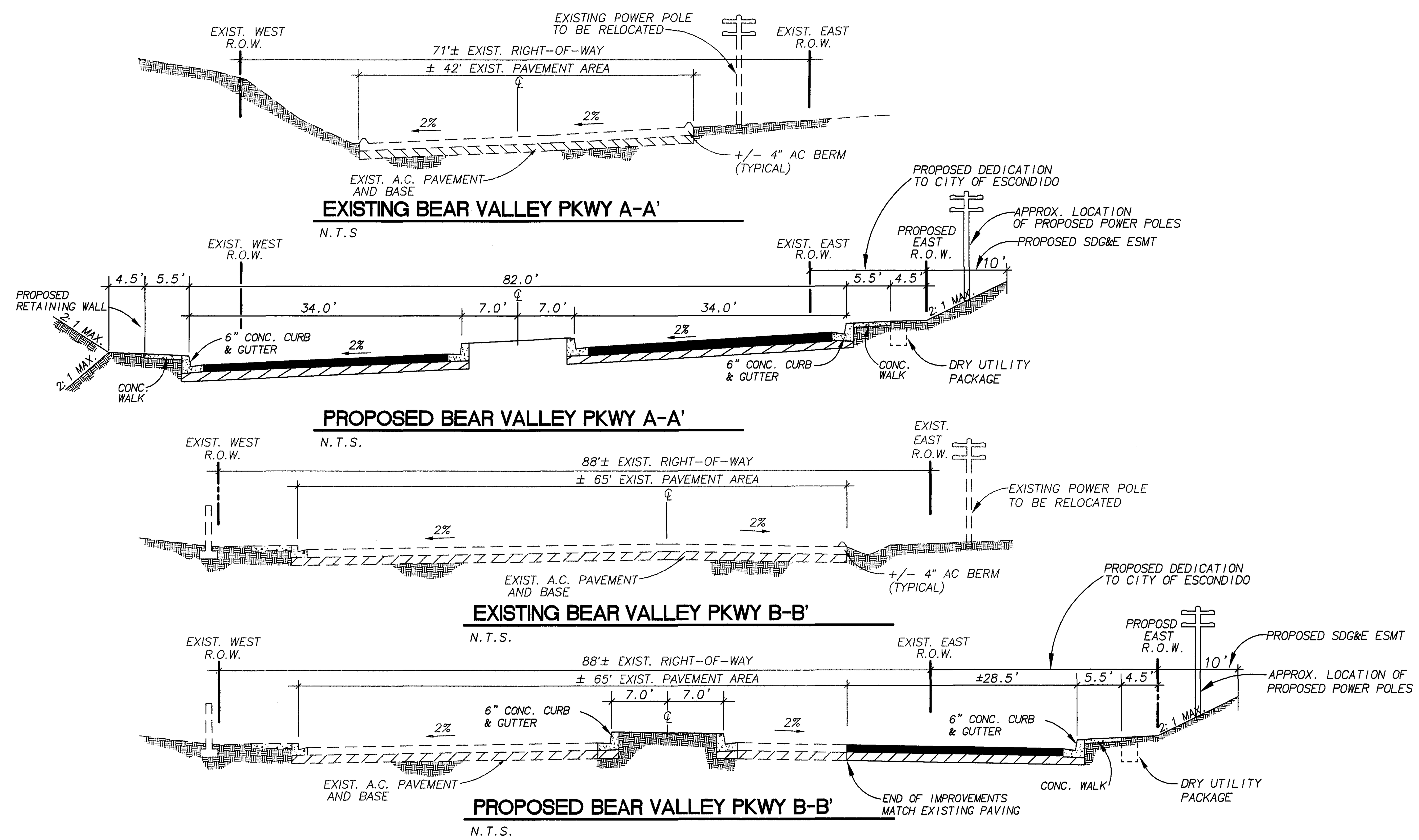

PLATE 2
V&M JOB #13-116-P

CONSTRUCTION RECORD	REFERENCES	DATE	BY	REVISIONS	APP'D	DATE	BENCH MARK	SCALE		DESIGNED BY	DRAWN BY	CHECKED BY		CITY OF ESCONDIDO — DEPT. OF PUBLIC WORKS	Drawing No.
CONTRACTOR							BENCH NUMBER	HORIZONTAL AS NOTED	OFFICE	SRC	SRC	RM	APPROVED	SPECIFIC PLAN FOR THE ALIGNMENT OF:	XX-XXXX
INSPECTOR							DESCRIPTION	VERTICAL AS NOTED	FILMED	PLANS PREPARED UNDER SUPERVISION OF		DATE	BY	BEAR VALLEY PARKWAY (FULL WIDTH)	Sheet 2 of 4
DATE COMPLETED							ELEV. ?		TRAFFIC			R.C.E. NO.	DEPUTY DIRECTOR OF ENGINEERING SERVICES		

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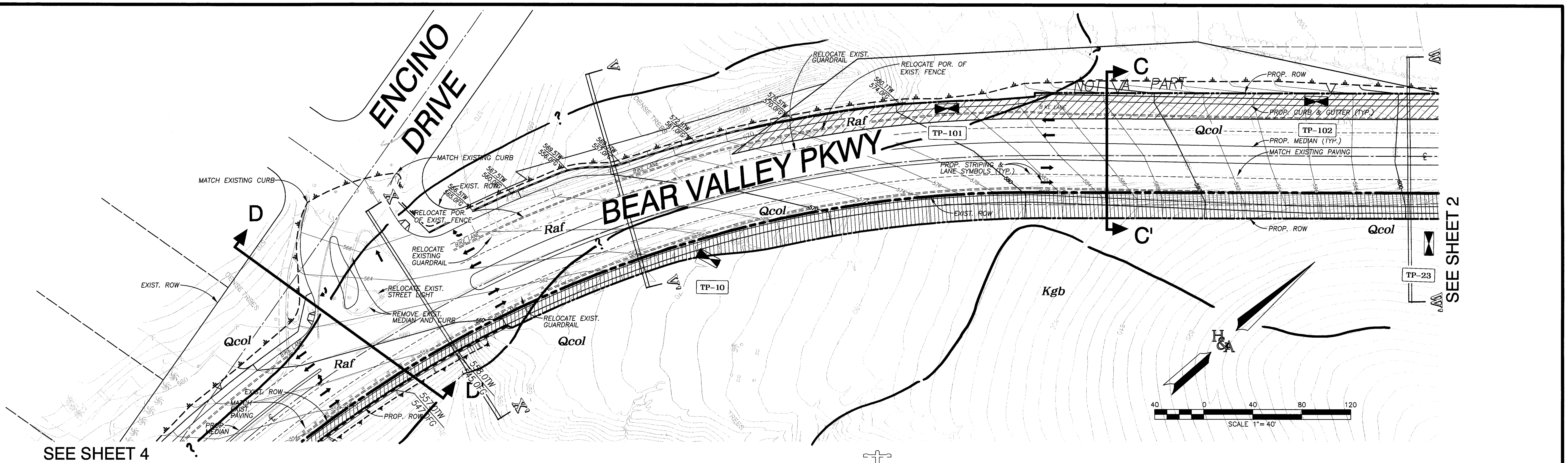

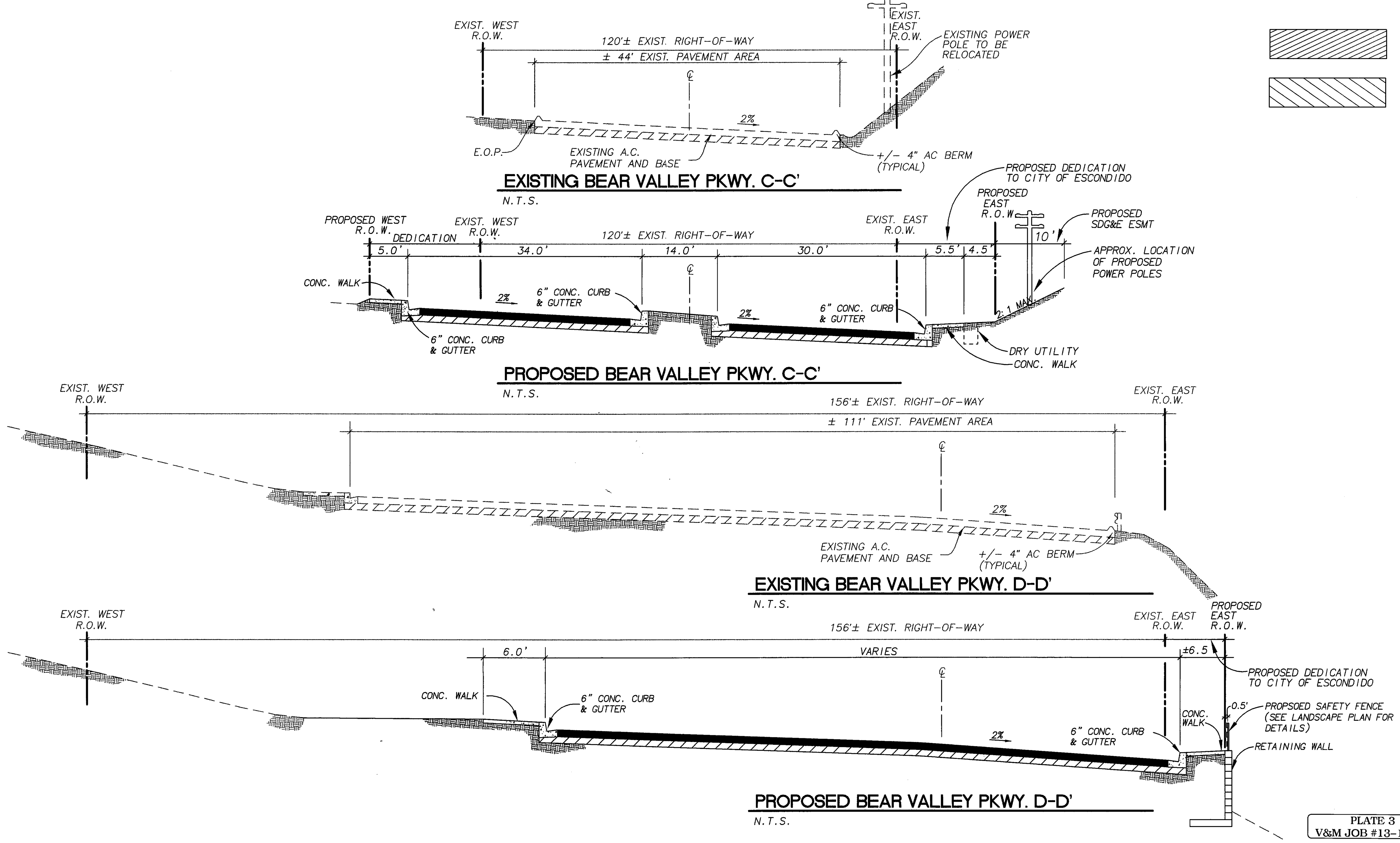

THIS AREA IS TO BE DEDICATED TO CITY OF ESCONDIDO AS PART OF PROJECT

THIS AREA TO BE ACQUIRED BY OTHERS & DEDICATED TO CITY OF ESCONDIDO IN FUTURE (TIMEFRAME UNKNOWN)

GEOTECHNICAL LEGEND

(test pit symbol)	Approx. Location of Exploratory Test Pit (T-1 Excavated 2/28/13; T-101 Excavated 9/13/16)
(section symbol)	Geologic Cross-Section
Raf	Road Fill
Qcol	Colluvium (Ancient Soil)
Kgb	Crystalline Bedrock

PLATE 3
V&M JOB #13-116-P

CONSTRUCTION RECORD	REFERENCES	DATE	BY	REVISIONS	APP'D	DATE	BENCH MARK	SCALE	OFFICE	DESIGNED BY	DRAWN BY	CHECKED BY	APPROVED	CITY OF ESCONDIDO	DEPT. OF PUBLIC WORKS	Drawing No.
CONTRACTOR							BENCH NUMBER	HORIZONTAL AS NOTED	FILMED	SRC	SRC	RM		SPECIFIC PLAN FOR THE ALIGNMENT OF:		XX-XXXX
INSPECTOR							DESCRIPTION	VERTICAL AS NOTED	TRAFFIC	PLANS PREPARED UNDER SUPERVISION OF	DATE	R.C.E. NO.	BY DEPUTY DIRECTOR OF ENGINEERING SERVICES	BEAR VALLEY PARKWAY (FULL WIDTH)		Sheet 3 of 4
DATE COMPLETED							ELEV. ?									

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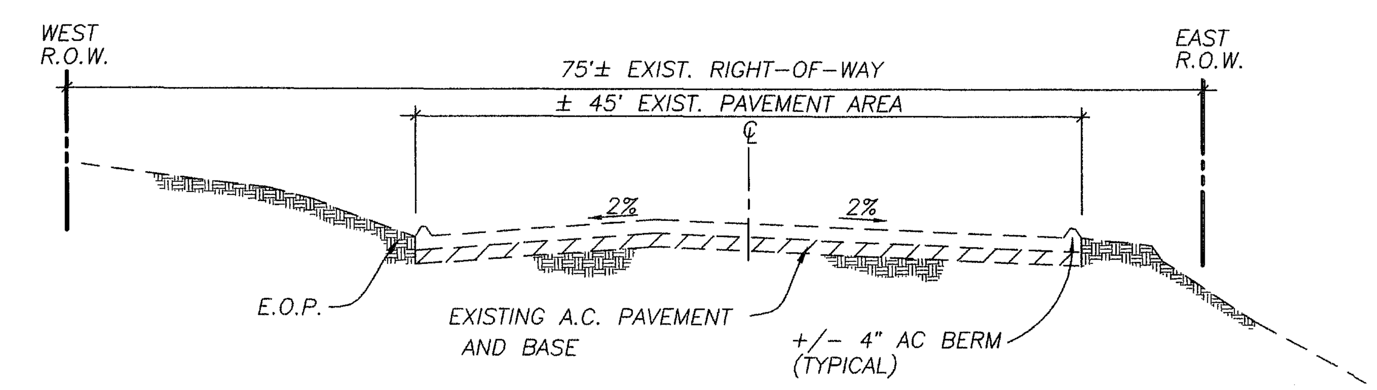
THIS AREA IS TO BE DEDICATED TO CITY OF ESCONDIDO AS PART OF PROJECT

THIS AREA IS TO BE VACATED AS PART OF PROJECT

THIS AREA TO BE ACQUIRED BY OTHERS & DEDICATED TO CITY OF ESCONDIDO IN FUTURE (TIMEFRAME UNKNOWN)

GEOTECHNICAL LEGEND

	Approx. Location of Exploratory Test Pit (T-1 Excavated 2/28/13; T-101 Excavated 9/13/16)
	Geologic Cross-Section
Raf	Road Fill
Qcol	Colluvium (Ancient Soil)
Qal	Alluvium



EXISTING BEAR VALLEY PKWY. F-F'

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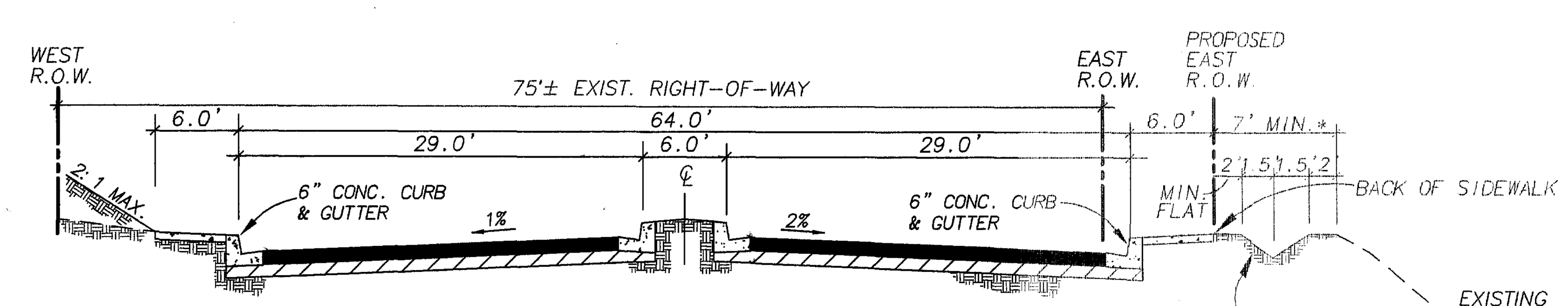
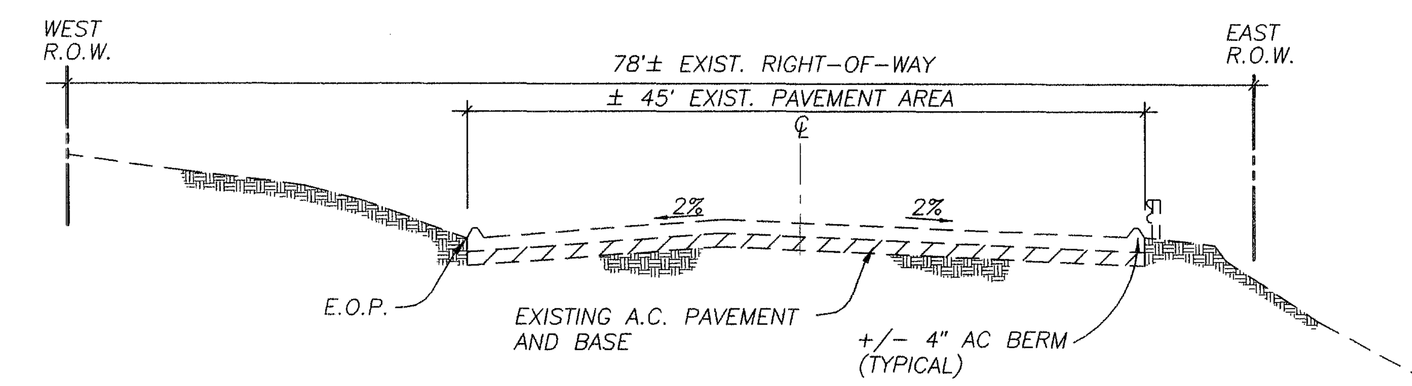


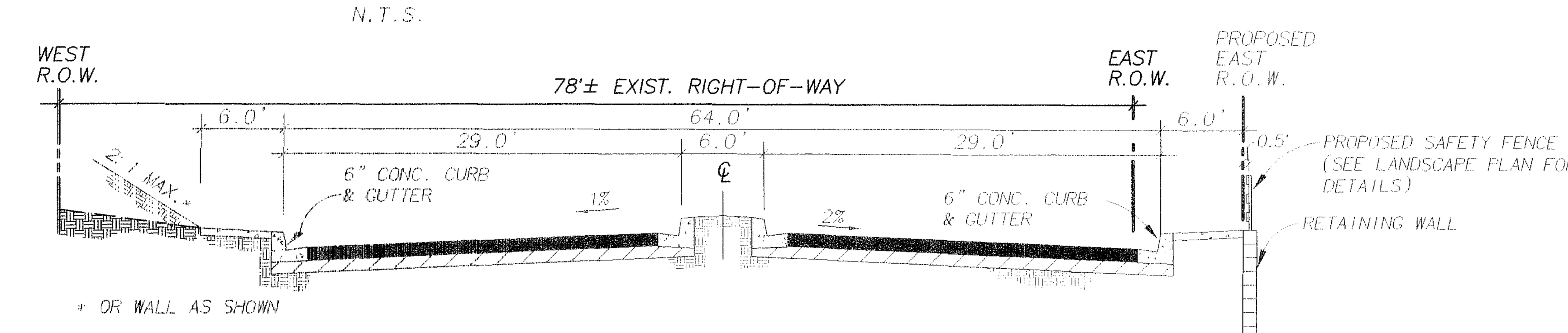
PROPOSED BEAR VALLEY PKWY. F-F'

N.T.S.



EXISTING BEAR VALLEY PKWY. E-E'

N.T.S.



PROPOSED BEAR VALLEY PKWY. E-E'

N.T.S.

PLATE 4
V&M JOB #13-116-P

CONSTRUCTION RECORD	REFERENCES	DATE	BY	REVISIONS	APP'D	DATE	BENCH MARK	SCALE		DESIGNED BY	DRAWN BY	CHECKED BY		CITY OF ESCONDIDO DEPT. OF PUBLIC WORKS	Drawing No.
CONTRACTOR							BENCH NUMBER DESCRIPTION	HORIZONTAL AS NOTED	OFFICE	SRC	SRC	RM	APPROVED	SPECIFIC PLAN FOR THE ALIGNMENT OF:	XX-XXXX
INSPECTOR								VERTICAL AS NOTED	FILMED	PLANS PREPARED UNDER SUPERVISION OF	DATE		BY	BEAR VALLEY PARKWAY (FULL WIDTH)	Sheet 4 of 4
DATE COMPLETED							ELEV. ?		TRAFFIC		R.C.E. NO.		DEPUTY DIRECTOR OF ENGINEERING SERVICES		

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VINJE & MIDDLETON ENGINEERING, INC. — TEST PIT: TP-101

PROJECT: Bear Valley Parkway Road Improvements **CLIENT:** Spieth & Wohlford, Inc.

PROJECT NO.: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Parkway, Escondido

Date Excavated: 9/13/16 **Bucket Size:** 24" **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 (%)
0–1			**Fill (af):**					
1			Silty fine to medium sand. Brown color. Dry to damp. Relatively tight. Includes chunks of asphalt up to 3 feet long (piece of curb). Loose. ST-1					
2		SM	Also includes scattered trash debris consisting of paper, plastic, glass, and metal.	Bulk, Chunk	3	117.6	86	18
3								
4			Loose to medium dense at 4 feet. Difficult to excavate due to large chunks of concrete.	Chunk	7	119.0	87	44

Bottom of test pit at 4.5 feet.

BULK SAMPLE | CHUNK SAMPLE | DENSITY TEST | GROUND WATER

PLATE 5

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-102**

PROJECT: Bear Valley Parkway Road Improvements **CLIENT:** Spieth & Wohlford, Inc.

PROJECT NO.: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Parkway, Escondido

Date Excavated: 9/13/16 **Bucket Size:** 24" **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 (%)
1			**Topsoil:**					
		SM	Silty fine to medium sand. Brown to red brown color. Damp. Loose at the surface. Blocky at 1-foot. Local white carbonate specks. Appears to be an ancient soil. Medium dense to dense. ST-1					
2				☐	7	119.0	87	44
3			Becomes dense at 3 feet. Moist.	☐	7	124.2	91	51

Bottom of test pit at 3.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-103**

PROJECT: Bear Valley Parkway Road Improvements **CLIENT:** Spieth & Wohlford, Inc.

PROJECT NO.: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Parkway, Escondido

Date Excavated: 9/13/16 **Bucket Size:** 24" **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 (%)
0–1		SM	**Fill (af):** Silty fine to medium sand. Brown color. Dry. Very loose. ST-1					
1–4.5		SM	**Topsoil:** Silty fine to medium sand. Brown to red brown color. Dry to damp. Locallly porous. Blocky. Appears ancient. Loose to very loose. ST-1					
2				Chunk	3	113.7	83	16
4			Grades dense at 4 feet. Moist.	Chunk	7	123.0	90	48

Bottom of test pit at 4.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

PLATE 7

VINJE & MIDDLETON ENGINEERING, INC. — TEST PIT: TP-104

PROJECT: Bear Valley Parkway Road Improvements **CLIENT:** Spieth & Wohlford, Inc.

PROJECT NO.: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Parkway, Escondido

Date Excavated: 9/13/16 **Bucket Size:** 24" **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 (%)
0–1.5		SW	**Fill (af):** Silty fine to coarse sand (D.G.). Olive grey color. Dry. Loose. ST-3					
1.5–6.5		SM	**Topsoil:** Silty fine to medium sand. Red brown color. Damp to moist. Loose to very loose. Porous. ST-1	Bulk, Chunk	6	107.7	79	28
4			Becomes blocky at 4 feet. Moist. Locally porous. Appears ancient. Medium dense to dense. ST-1	Chunk	7	119.3	87	44
6			Becomes dense at 6 feet. Moist.	Chunk	8	122.3	90	55

Bottom of test pit at 6.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC. TEST PIT: TP-105

PROJECT: Bear Valley Parkway Road Improvements **CLIENT:** Spieth & Wohlford, Inc.

PROJECT NO.: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Parkway, Escondido

Date Excavated: 9/13/16 **Bucket Size:** 24" **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 (%)
0–1		SM	**Colluvium (Qcol) / Fill (MSaf):** Silty fine to medium sand. Brown color. Dry to damp. Porous. Very Loose. ST-1					
4		SM	Becomes somewhat blocky at 4 feet. Damp. Continues loose.	Chunk sample	4	111.8	82	21
7		SW-GW	**Bedrock (Kgb):** Gabbroic to granitic rock. Fine to coarse grained. Red brown to grey color. Weathered. Excavates gravelly. Dense. ST-3					

Bottom of test pit at 7.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC.

TEST PIT: TP-1

PROJECT: Proposed Residential Subdivision **CLIENT:** Speith & Wohlford, Inc.

PROJECT NUMBER: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Pkwy, Escondido (APN's 237-131-01 & 02)

Date Excavated: 2/28/13 **Logged By:** SJM

Equipment: Caterpiller 420 Backhoe

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	**COLLUVIUM (Qcol):** Silty fine to medium sand. Brown color. Damp. Loose. ST-1					
2			Sandy silt to sandy clay. Red brown color. Moist. Soft to loose. Plastic. ST-2	Bulk / Chunk	14	104	79	56
4		CL-ML						
6		SW-GP	**BEDROCK (Kgb):** Gabbroic rock. Fine to medium grained. Red brown color. Weathered. Friable. Massive. ST-3					
			Becomes blocky at 7 feet. Dense.	Chunk	12	-	Sample Disturbed	-

Bottom of test pit at 7.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

VINJE & MIDDLETON ENGINEERING, INC.

TEST PIT: TP-4

PROJECT: Proposed Residential Subdivision **CLIENT:** Speith & Wohlford, Inc.

PROJECT NUMBER: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Pkwy, Escondido (APN's 237-131-01 & 02)

Date Excavated: 2/28/13 **Logged By:** SJM

Equipment: Caterpiller 420 Backhoe

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 (%)
0–3		SM	**COLLUVIUM (Qcol):** Clayey sand. Red brown color. Moist. Soft to loose. Low plastic. ST-2					
3–5		SC-CL	Clayey sand / sandy clay (residual soil). Red brown color. Moist. Firm to stiff. Low - medium plastic. ST-4	Bulk				
5–6		SW-GP	**BEDROCK (Kgb):**					
6–12			Gabbroic rock. Fine to coarse grained. Red brown color. Weathered. Blocky. Dense. ST-3	Chunk	6	138.7	100+	61
			A mining excavation was encountered at 4-6 feet below the surface. The mining excavation appears to be an adit and measures approximately 7 feet wide and approximately 6 feet high. The excavation is trending N75E, and may be descending slightly in a northeast direction.					
12–13		SW-GP	Gabbroic rock. Dense.					

Bottom of test pit at 13.0 feet.

BULK SAMPLE | CHUNK SAMPLE | DENSITY TEST | GROUND WATER

PLATE 11

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

PROJECT: Proposed Residential Subdivision CLIENT: Speith & Wohlford, Inc.

PROJECT NUMBER: 13-116-P PROJECT LOCATION: 661 Bear Valley Pkwy, Escondido (APN's 237-131-01 & 02)

Date Excavated: 2/28/13 **Logged By:** SJM

Equipment: Caterpiller 420 Backhoe

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 (%)
0–7		SC	**COLLUVIUM (Qcol):** Clayey sand. Red brown color. Moist. Soft to loose. Low plastic. ST-2					
2.5			Becomes somewhat blocky and medium dense at 2.5 feet.					
~3				Bulk / Chunk	13	112.7	86	64
7–8		SM	Silty fine to medium sand. Brown color. Moist. Somewhat blocky. Medium dense. ST-1	Chunk	15	116.7	84	82
8–10		GP	**BEDROCK (kgr):** Granitic rock. Fine grained. Tan to reddish color. Fractured. Includes quartz veins. Local polished surfaces. Dense. Excavates blocky to gravelly. ST-5	Chunk	14	119.6	91	81

Bottom of test pit at 10.0 feet.

BULK SAMPLE — CHUNK SAMPLE — DENSITY TEST — GROUND WATER

PLATE 12

VINJE & MIDDLETON ENGINEERING, INC. TEST PIT: TP-10

PROJECT: Proposed Residential Subdivision CLIENT: Speith & Wohlford, Inc.

PROJECT NUMBER: 13-116-P PROJECT LOCATION: 661 Bear Valley Pkwy, Escondido (APN's 237-131-01 & 02)

Date Excavated: 2/28/13 **Logged By:** SJM

Equipment: Caterpiller 420 Backhoe

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	**COLLUVIUM (Qcol):** Silty fine to medium sand. Brown color. Dry at the surface, damp at 2 feet. Somewhat blocky. Medium dense. ST-1					
2				Bulk				
4			Very tight and blocky at 4 feet. Moist. Slow digging. Appears to be an ancient colluvium. Dense.	Chunk	11	120.4	87	65
6				Chunk	10	118.0	85	56
8								
10			Continues very tight and blocky. Dense. Backhoe refusal at 10.5 feet.	Chunk	11	120.3	87	65

Bottom of test pit at 10.5 feet.

BULK SAMPLE CHUNK SAMPLE DENSITY TEST GROUND WATER

PLATE 13

VINJE & MIDDLETON ENGINEERING, INC. **TEST PIT: TP-23**

PROJECT: Proposed Residential Subdivision **CLIENT:** Speith & Wohlford, Inc.

PROJECT NUMBER: 13-116-P **PROJECT LOCATION:** 661 Bear Valley Pkwy, Escondido (APN's 237-131-01 & 02)

Date Excavated: 3/1/13 **Logged By:** SJM

Equipment: Caterpiller 420 Backhoe

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 (%)
0–6		SM	**COLLUVIUM (Qcol):** Silty fine to medium sand. Red brown color. Damp to moist. Loose. ST-1					
6–7		SW	**BEDROCK (Kgb):** Gabbroic rock. Fine to coarse grained. Grey color. Weathered. Friable. Massive. Medium dense to dense. ST-3	Chunk	14	113.4	87	70

Bottom of test pit at 7.0 feet.

BULK SAMPLE · CHUNK SAMPLE · DENSITY TEST · GROUND WATER

GEOLOGIC CROSS-SECTIONS

SCALE: 1" = 40'

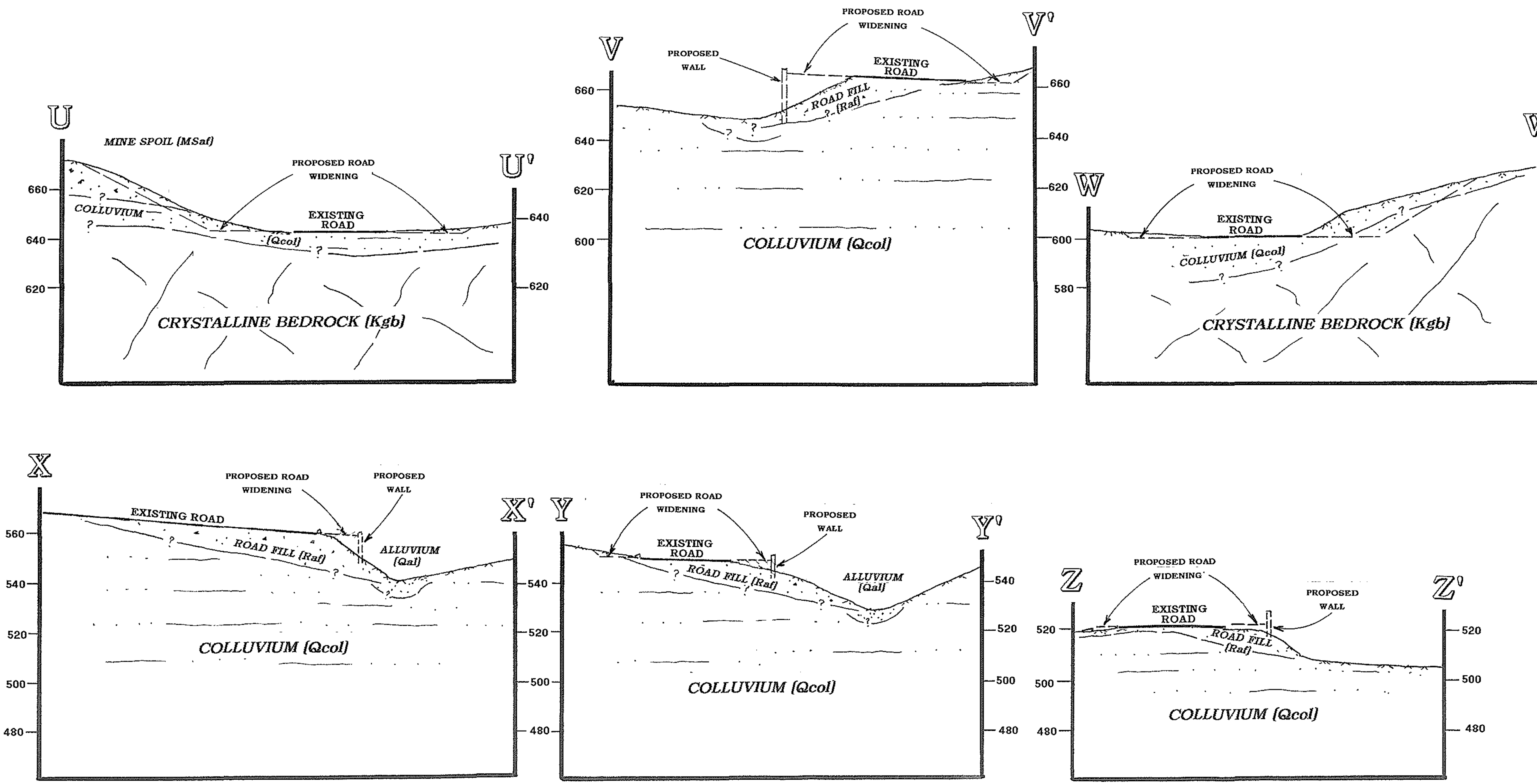
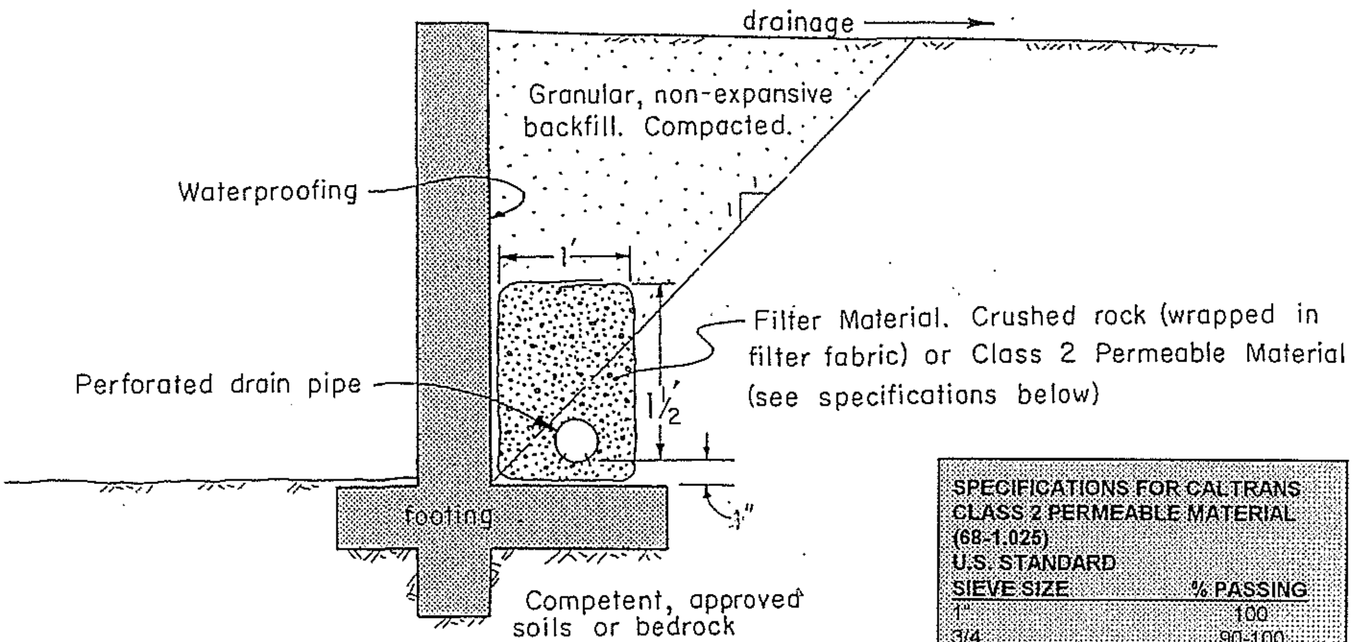


PLATE 15
V&M JOB #13-116-P

RETAINING WALL DRAIN DETAIL

Typical - no scale



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

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

Sand Equivalent > 75

CONSTRUCTION SPECIFICATIONS:

1. Provide granular, non-expansive backfill soil in 1:1 gradient wedge behind wall. Compact backfill to minimum 90% of laboratory standard.
2. Provide back drainage for wall to prevent build-up of hydrostatic pressures. Use drainage openings along base of wall or back drain system as outlined below.
3. Backdrain should consist of 4" diameter PVC pipe (Schedule 40 or equivalent) with perforations down. Drain to suitable outlet at minimum 1%. Provide ¾" - 1½" crushed gravel filter wrapped in filter fabric (Mirafi 140N or equivalent). Delete filter fabric wrap if Caltrans Class 2 permeable material is used. Compact Class 2 material to minimum 90% of laboratory standard.
4. Seal back of wall with waterproofing in accordance with architect's specifications.
5. Provide positive drainage to disallow ponding of water above wall. Lined drainage ditch to minimum 2% flow away from wall is recommended.

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

VINJE & MIDDLETON ENGINEERING, INC.

PLATE 16
V&M JOB #13-116-P

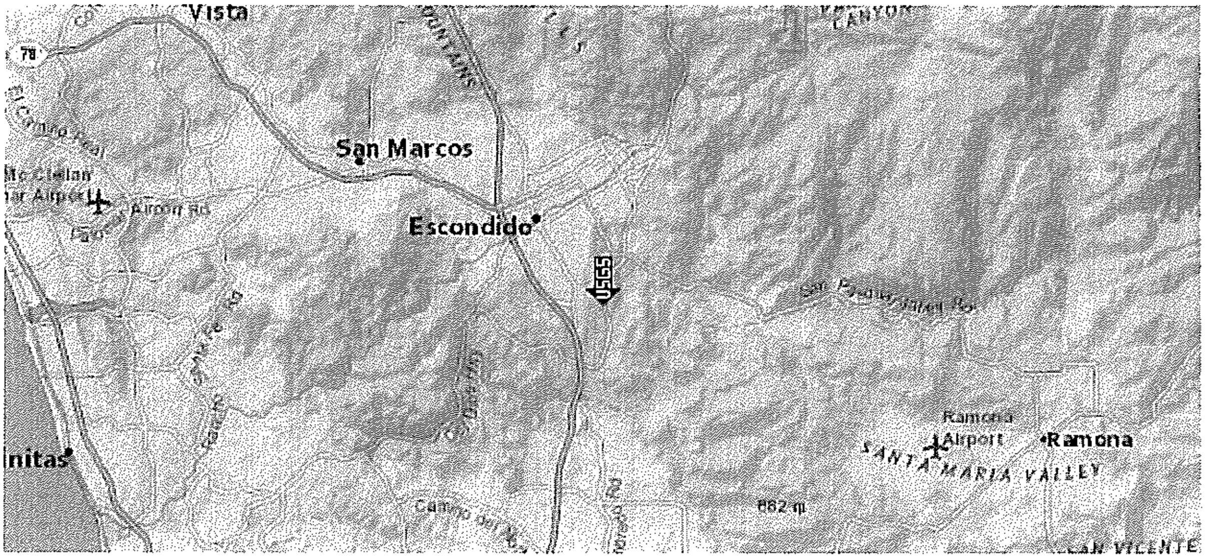
APPENDIX A

Seismic Ground Motion Values

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.1°N, 117.06°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

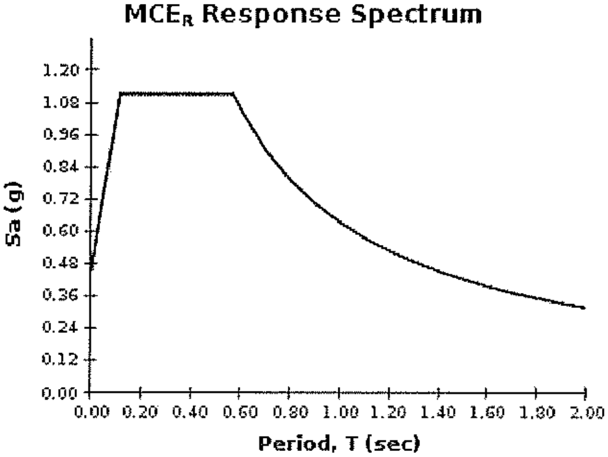


USGS–Provided Output

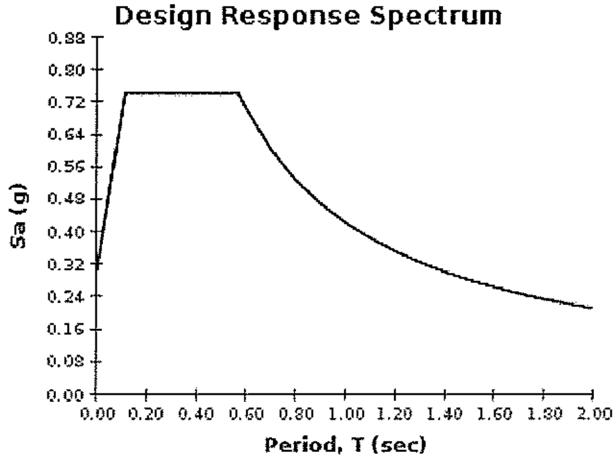
S_S =	1.016 g	S_{MS} =	1.111 g	S_{DS} =	0.741 g
S_1 =	0.391 g	S_{M1} =	0.633 g	S_{D1} =	0.422 g

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 (33.1°N, 117.06°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] $S_s = 1.016$ g

From Figure 22-2 [2] $S_1 = 0.391$ 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/ft² = 0.0479 kN/m²

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

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
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 = 1.016 g, F_a = 1.094

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 = 0.391 g, F_v = 1.617

Equation (11.4–1): $S_{MS} = F_a S_S = 1.094 \times 1.016 = 1.111$ g

---

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.617 \times 0.391 = 0.633$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.111 = 0.741$ g

---

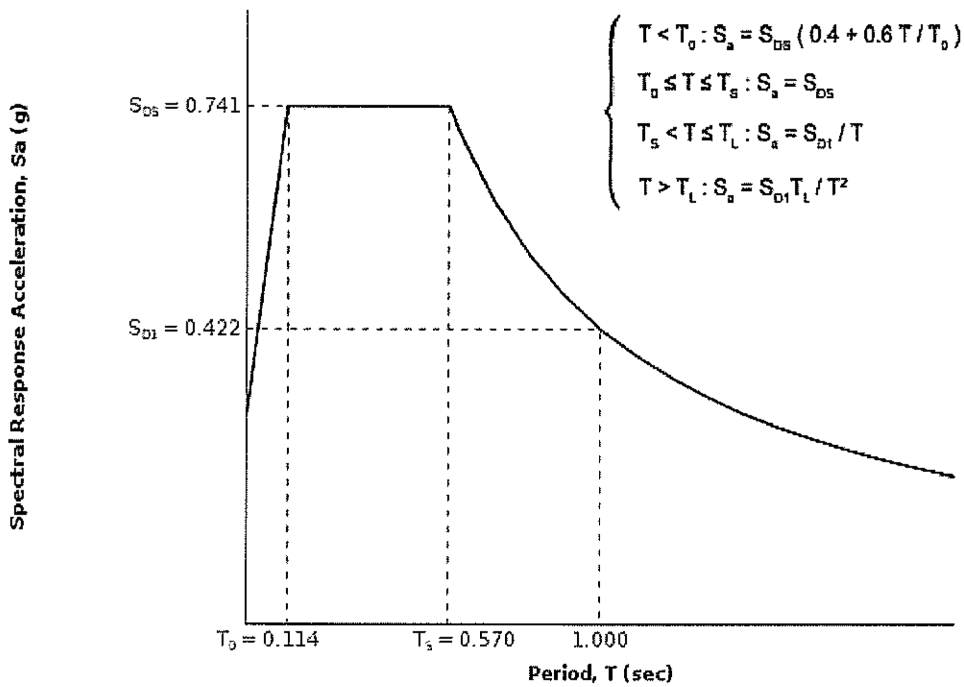
Equation (11.4–4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.633 = 0.422$ g

---

Section 11.4.5 — Design Response Spectrum

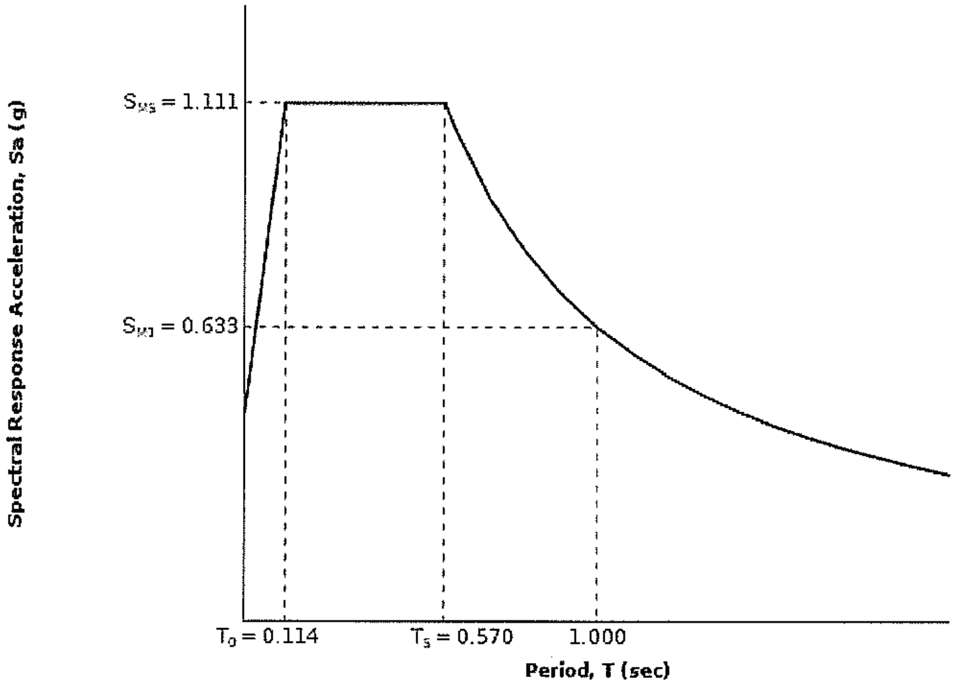
From <u>Figure 22-12</u>[3] $T_L = 8$ seconds

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 = 0.378

Equation (11.8–1): $PGA_M = F_{PGA} PGA = 1.122 \times 0.378 = 0.424$ 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 = 0.378 g, F_{PGA} = 1.122

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

From Figure 22-17 [5] $C_{RS} = 1.038$

From Figure 22-18 [6] $C_{R1} = 1.079$

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} = 0.741 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} = 0.422 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

Appendix B:

R-Value Calculation Sheet

R-VALUE CALCULATION SHEET

Job #13-116-P

Asphalt over Class 2 Aggregate Base Design:

Data: Subgrade ***R = 37***; ***TI = 4.5***; Minimum Section = 3" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 4.5 \times (100 - 37) = 0.91$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.54
$GE_{AC} = (T_{AC} \div 12) \times GF = 3 \div 12 \times 2.54 = 0.63$
$GE_{AB} = GE_{TOT} - GE_{AC} = 0.91 - 0.63 = 0.28$
$T_{AB} = (GE_{AB} \div GF_{AB}) \times 12 = (0.28 \div 1.1) \times 12 = 3"$

Use 3 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 37***; ***TI = 5.0***; Minimum Section = 3" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 5.0 \times (100 - 37) = 1.01$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.54
$GE_{AC} = (T_{AC} \div 12) \times GF = 3 \div 12 \times 2.54 = 0.63$
$GE_{AB} = GE_{TOT} - GE_{AC} = 1.01 - 0.63 = 0.38$
$T_{AB} = (GE_{AB} \div GF_{AB}) \times 12 = (0.38 \div 1.1) \times 12 = 4"$

Use 3 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 37***; ***TI = 6.0***; Minimum Section = 4" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 6.0 \times (100 - 37) = 1.21$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.32
$GE_{AC} = (T_{AC} \div 12) \times GF = 4 \div 12 \times 2.32 = 0.77$
$GE_{AB} = GE_{TOT} - GE_{AC} = 1.21 - 0.77 = 0.44$
$T_{AB} = (GE_{AB} \div GF_{AB}) \times 12 = (0.44 \div 1.1) \times 12 = 4"$

Use 4 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 37***; ***TI = 7.0***; Minimum Section = 4" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 6.5 \times (100 - 37) = 1.41$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.14
$GE_{AC} = (T_{AC} \div 12) \times GF = 4 \div 12 \times 2.14 = 0.71$
$GE_{AB} = GE_{TOT} - GE_{AC} = 1.41 - 0.71 = 0.70$
$T_{AB} = (GE_{AB} \div GF_{AB}) \times 12 = (0.70 \div 1.1) \times 12 = 8"$

Use 4 inches AC over 8 inches Caltrans Class 2

Job #13-116-P

Asphalt over Class 2 Aggregate Base Design:

Data: Subgrade ***R = 64***; ***TI = 4.5***; Minimum Section = 3" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 4.5 \times (100 - 64) = 0.52$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.54
$GE_{AC} = (T_{AC} \div 12) \times GF = 3 \div 12 \times 2.54 = 0.63$
$GE_{AB} = GE_{TOT} - GE_{AC} = 0.52 < 0.63$

Use 3 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 64***; ***TI = 5.0***; Minimum Section = 3" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 5.0 \times (100 - 64) = 0.58$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.54
$GE_{AC} = (T_{AC} \div 12) \times GF = 3 \div 12 \times 2.54 = 0.63$
$GE_{AB} = GE_{TOT} - GE_{AC} = 0.58 < 0.63$

Use 3 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 64***; ***TI = 6.0***; Minimum Section = 4" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 6.0 \times (100 - 64) = 0.69$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.32
$GE_{AC} = (T_{AC} \div 12) \times GF = 4 \div 12 \times 2.32 = 0.77$
$GE_{AB} = GE_{TOT} - GE_{AC} = 0.69 < 0.77$

Use 4 inches AC over 4 inches Caltrans Class 2

Data: Subgrade ***R = 64***; ***TI = 7.0***; Minimum Section = 4" A.C. over 4" Cl. 2 base.

Calculations:

$GE_{TOT} = 0.0032 \times TI \times (100 - R) = 0.0032 \times 6.5 \times (100 - 37) = 0.81$
GF: From Cal Trans 1990 Edition, Table #608.4B = 2.14
$GE_{AC} = (T_{AC} \div 12) \times GF = 4 \div 12 \times 2.14 = 0.71$
$GE_{AB} = GE_{TOT} - GE_{AC} = 0.81 - 0.71 = 0.10$
$T_{AB} = (GE_{AB} \div GF_{AB}) \times 12 = (0.10 \div 1.1) \times 12 = 1.1"$

Use 4 inches AC over 4 inches Caltrans Class 2