ARAGÓN GEOTECHNICAL, INC.
Consultants in the Earth & Material Sciences

PRELIMINARY GEOTECHNICAL INVESTIGATION
McCALL BOULEVARD WIDENING PROJECT, CIP 22-03
CITY OF MENIFEE, RIVERSIDE COUNTY, CALIFORNIA

FOR
KOA CORPORATION2141 W. ORANGEWOOD AVENUE, SUITE A
ORANGE, CALIFORNIA 92868

PROJECT NO. 4811-SFPV
AUGUST 3, 2022

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August 3, 2022
Project No. 4811-SFPV

KOA Corporation
2141 West Orangewood Avenue, Suite A
Orange, California 92868

Attention: Mr. Matt Stepien

Subject: Preliminary Geotechnical Investigation Report
McCall Boulevard Widening Project (CIP 22-03)
City of Menifee, Riverside County, California

Mr. Stepien:

In accordance with our services proposal dated October 29, 2021, Aragón Geotechnical Inc. (AGI) has completed a design Geotechnical Investigation for the referenced arterial street improvements. The accompanying preliminary report details AGI's findings, opinions, and recommendations developed as a result of surface field inspections, subsurface exploration including soil borings and seismic refraction line spreads, laboratory testing, and engineering and geological analyses.

McCall Boulevard was built as a two-lane County road in 1966. Repaving and discontinuous widening projects have been completed over the intervening decades. The present project seeks to create 4 through lanes and a raised median between Oakhurst Avenue and Menifee Road, a total length of approximately 4,000 feet. Future work could further expand the segment into a 6-lane urban arterial design.

Eight soil borings were drilled within the public street right-of-way. Findings indicated the alignment contained multiple generations of man-made fill, very old native soil deposits, and two different bedrock units. The original (concealed) pavement was still present, even in later-improved street segments. A deep bedrock cut near the project midpoint appeared to have generated very stony soil-rock mixes that were subsequently used for roadway fills east and west of the cut. Refraction survey results indicated that the bedrock cut at street grade includes materials with average seismic velocities over 5,600 feet per second. Hollow-stem auger tools met immediate refusal after passing through 18 to 30 inches of overbreak fill in the rock cut. At the western end of the project, a mid-slope retaining wall would be founded atop the rocky 1960's fill. The latter has been judged unsuitable for foundation loads without overexcavation and replacement as engineered fill.

Local well data and the soil borings indicated future construction should be entirely "in the dry". Geologic hazard risks from earthquake fault ground rupture, liquefaction and related ground deformation phenomena, settlement, or landsliding appear to be very low to nil. The existing cut slopes have performed well for more than 56 years and are assessed as globally stable. However, maintenance scaling is recommended. Additionally, catchment provisions should be designed above the toe wall on the north-side slope to prevent small tumbling stones from reaching the traffic lanes. The harder bedrock masses are judged non-rippable for conventional excavator buckets, but should break fairly easily with 3,000-5,000 foot-pound rated hydraulic hammers (breakers or "hoe rams").

Based on the results of the field investigation, laboratory testing, and professional experience, it is our opinion that native soils, the bedrock units, and existing/new fills will be competent for support of asphalt pavement structural sections pending recommended subgrade preparation actions. All alignment soil materials are considered suitable for reuse in utility trench backfills above defined pipe zones if free of coarse broken rock.

In addition to preliminary recommended bearing capacities and design values for loads on retaining structures, this report presents recommendations for mass grading, slope designs, concrete mix designs suited to local conditions, temporary excavations, and for geotechnical observation with compaction quality control tests. It is recommended that project plans be prepared for bid only after final geotechnical plan reviews including slope-specific stability analyses are performed, and the preliminary recommendations of this report are verified as suitable for construction.

Thank you very much for this opportunity to be of service. Please do not hesitate to call if you should have any questions.

Very truly yours,

Mark G. Doerschlag
Engineering Geologist, CEG 1752

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Geotechnical Engineer, G.E. 2994

MGD/CFA:mma

Distribution: (6) Addressee

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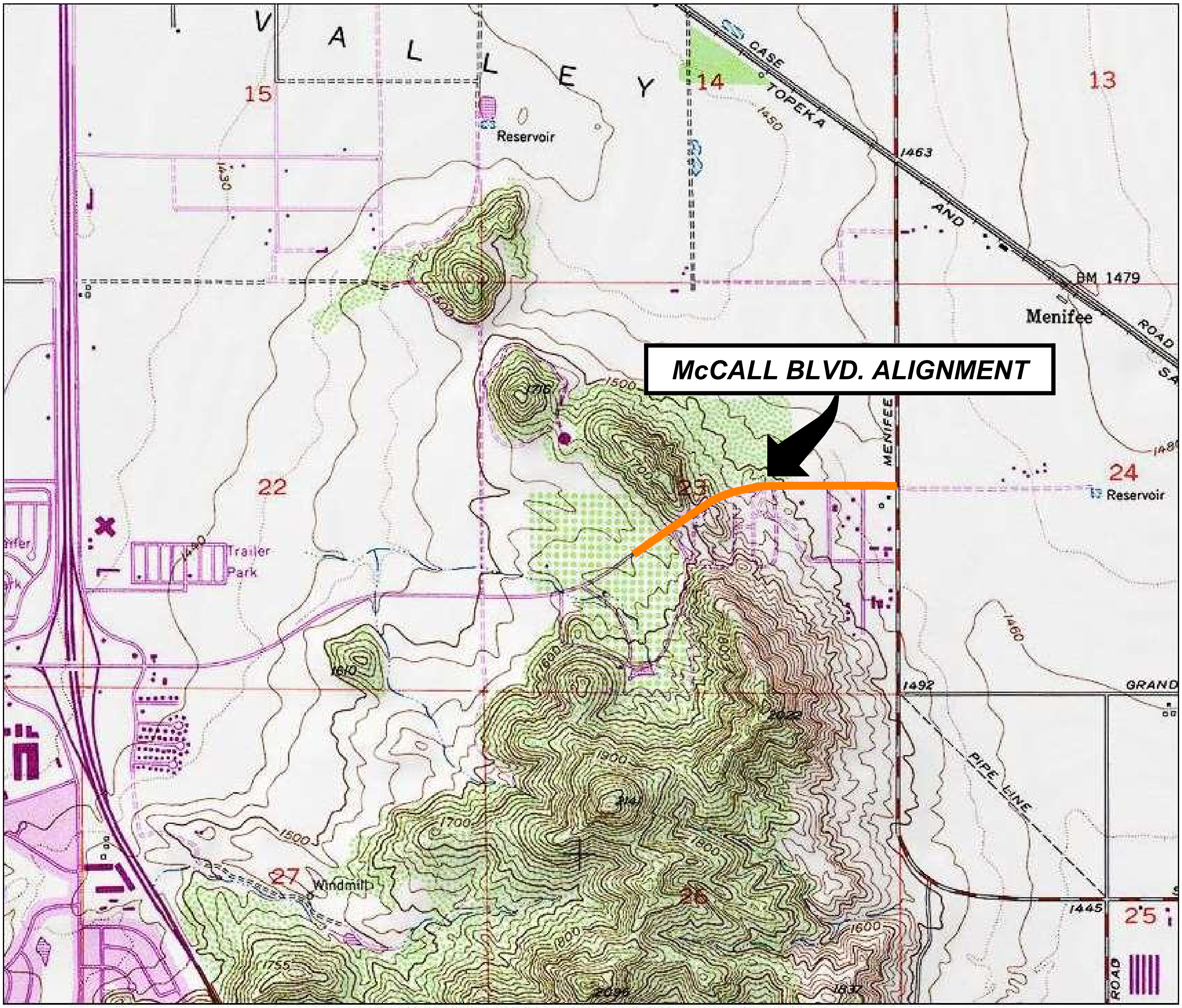
1.0 INTRODUCTION

In cooperation with the prime design consultant KOA Corporation (KOA), this report presents the results of geological and soils engineering evaluations conducted by Aragón Geotechnical, Inc. (AGI) for the City of Menifee capital improvement project named above. Roughly 4,000 linear feet of McCall Boulevard is slated to be expanded from an original 2-lane County rural road to a 4-lane arterial configuration between Oakhurst Avenue and Menifee Road. Per the 2013 City General Plan, we understand the boulevard may later be widened to a final 6-lane thoroughfare. This would be based on available funding and special engineering provisions needed for street passage through a deep bedrock cut. The design 130-foot-wide street right-of-way (ROW) will locally require property acquisitions. Planned construction will include minor added cuts, several fairly large fills, and probably a mix of partial-width and full-street-width pavement reconstruction. Figure 1 on the following page illustrates the general location of the street segment on a 1:24,000-scale topographic base map. Although out-of-date with respect to the rapid urbanization of the surrounding City of Menifee, the older map series was selected for clearer depictions of ground slope, drainage patterns, and historical adjacent land parcel improvements.

The primary objectives of our study were to determine the nature and engineering properties of the subsurface materials along the alignment, and supply needed grading, pavement design, installation, and construction inspection recommendations. Additionally, project concepts include several retaining walls, and site condition evaluations were geared to identifying optional structural design types for these features. Accordingly, AGI's scope included a surficial reconnaissance of the project alignment and surrounding areas, geologic mapping, geologic literature research, subsurface soil borings, recovery of representative soil samples, laboratory testing, and geotechnical analyses. Excavatability assessments for bedrock near the toe of existing roadcut slopes were based on limited seismic refraction geophysical spreads to measure average P-wave velocities. Historical aerial image analyses were also employed to understand pre-development conditions, and help refine the mapped limits of existing roadway fills and bedrock excavations.

Other investigation tasks included assessments of potential geological constraints such as landslide potential, surface faulting, seismicity, and groundwater. Our assessments relied on various published resources cited later in this report, conclusions drawn from field



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Reference: U. S. Geological Survey 7½-Minute Series Topographic Map, Romoland Quadrangle (1979).

AGI	**SITE LOCATION MAP**
	McCALL BLVD. WIDENING PROJECT, CITY OF MENIFEE, CALIF.
	PROJECT NO. 4811-SFPV / DATE: 8/3/22 / **FIGURE 1**

exploration and mapping results, and historical slope performance. However, environmental research for purposes of establishing whether toxic or hazardous substances had been generated, used, stored, or disposed of close to the future widened street, and chemical testing of air, soil, or groundwater found along the project alignment were beyond the scope of this study.

2.0 PROPOSED CONSTRUCTION

Conceptual drawings on aerial image base maps supplied by the City of Menifee Engineering Department (Sheets 1-3, edition date October 14, 2021), were referenced for preliminary design information and used for exploration site selection. An updated KOA-authored digital alignment exhibit (April 27, 2022) was modified for this report to illustrate local geology and AGI soil boring locations. The exhibit included cultural features, major trees, and ground surface elevations at one-foot contour intervals, but lacked subsurface utility alignments. Proposed street improvements will involve both eastbound and westbound traffic lanes. AGI exploration site locations included both travel directions, and were usually referenced to street intersections shown on the various concept plans.

The western project endpoint is at Oakhurst Avenue. McCall Boulevard ascends to a high point about 1,175 feet east of Oakhurst Avenue at an average gradient of ~2.2 percent. The southern side of the street along this stretch borders a 2005-era housing tract and is at final-design width. No alterations to existing south-side concrete curbs and sidewalks are shown on drawings. Most construction will occur along the westbound side where a new embankment fill slope and a mid-slope retaining wall up to 8 feet high are shown. The fill slope may be inclined at 1.5:1 (horizontal:vertical). Fill slopes steeper than 2:1 normally must incorporate soil reinforcement to meet minimum stability criteria. Options to delete the mid-slope wall are geotechnically much more favorable, and are discussed in detail later in this report.

About 700 linear feet of the alignment near the project high point is located in bedrock cut through a prominent ridge. Squeezing 4 traffic lanes and a center median strip into the existing cut will necessitate a low toe wall up to 4 feet high on the north side. An asphalt curb and concrete sidewalk will be relocated several feet southward along the eastbound side. No grading of existing rock slopes above the north-side toe wall or along the south side appears to be proposed. These slopes are up to 62 feet high. Considerations for rockfall protection are deemed important for public safety, however, and need to be addressed in project design.

Cut, fill, and “at-grade” as-built roadway construction is discernable in the existing boulevard east of the deep rock cuts. New fills will locally be needed. One planned fill slope would approach 35 feet tall, and may also be inclined 1.5:1. Per the KOA construction exhibit, most local road intersections will not require significant improvements other than signal modifications. The eastern half of the alignment features a mix of newer and original asphalt pavements, and some completed curbs and sidewalks next to a new school. Approaching the eastern project endpoint at Menifee Road, construction will entail widening of both sides of the boulevard, signal and power pole relocations, ramps, curbs, and sidewalks. Storm drain improvements could also be required at this major intersection.

Construction will be impacted by numerous man-made impediments such as underground utilities, traffic signal and utility poles, and existing street improvements. We have assumed that the present average street elevations will not change. Interfering above-grade features would be relocated. Data in this report should guide decisions concerning which pavements to retain in place and what areas should be considered for full-depth removal and reconstruction to the current City standard structural section for a 6-lane arterial boulevard. Beyond any saved pavement zones, the creation of dense, unyielding subgrades for new pavement would be a project goal.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Drilled site explorations for this study were completed by AGI on June 7, 2022. Eight soil borings were attempted at sites preliminarily selected by AGI to represent areas of special geotechnical concern. Actual boring locations were based on utility avoidance issues, opportunities to sample existing fills, and interpretations of “least-favorable” localities such as hard-rock spots or locations that were formerly in agricultural fields. The 8-inch-diameter borings were advanced with a Mobile Drill B-61 truck-mounted hollow-stem auger rig to terminal depths of up to 26.0 feet. Four borings encountered auger refusals above programmed terminal depths. All refusals were located within or close to bedrock terrain.

Four auxiliary pavement samples were obtained by AGI technicians with a diamond core barrel. The auxiliary sites included pavements of varying ages, but focused on the original County roadway. The asphalt core and any aggregate base layer were field measured for thickness. All drilled or cored pavement penetrations were restored with tinted non-shrink cement grout per City Public Works standards.

The drill cuttings and discrete soil samples were visually/manually examined and classified according to the Unified Soil Classification System, and observations made of relative porosity, cementation, soil plasticity, and past or present groundwater conditions. Bedrock borings were monitored primarily for auger penetration rate. Rock descriptive information was based on ISRM terminology for hardness, weathering, strength, and discontinuity spacing. Continuous logs of the subsurface conditions encountered in the borings were recorded by AGI's senior Engineering Geologist, and the results are presented on the Field Boring Logs in the accompanying Appendix A. Pavement structural sections were field-measured at each soil boring and noted on the logs. The approximate locations of the borehole explorations are illustrated on the KOA exhibit supplied to our office, presented as Plate No. 1 at the back of this report.

At increments of 2 to 5 feet in the borings, relatively undisturbed soil samples were recovered by driving a 3.0-inch O.D. "California modified" ring-lined split-barrel sampler. Fill containing abundant rock fragments was evaluated via Standard Penetration Tests (SPTs) conducted using an unlined 2.0-inch O.D. split-barrel spoon. All sampler driving was done using rods and a mechanically actuated automatic 140-pound hammer free-falling 30 inches. Disturbed bag samples judged representative of future compacted fill or subgrade soils were additionally collected from designated intervals. At every exploratory location, pertinent *in situ* materials properties were judged from machine behavior, penetration resistance of the barrel samplers, and soil texture.

Recovered soil samples were transported to our Riverside soils laboratory for analysis. Tests were conducted to evaluate index and engineering properties such as natural dry density and water content, soil strength, grain size distribution, sand equivalent value, compaction characteristics, and soil corrosivity. R-value tests were assigned to selected bulk samples of existing fill derived from bedrock cut, and from the far siltier natural alluvial soils that typify the alignment closer to Menifee Road. Detailed discussions of the laboratory test standards used and the test results are presented in Appendix B.

Given proposed excavations for a toe wall at the base of the tallest bedrock cut slope, and drilling tool refusals at 30 inches or less next to the cut, AGI added two 70-foot-long seismic refraction line spreads to evaluate bedrock rippability. Average seismic wave velocities and not layer models in the existing deep cut were the objectives. A 12-channel seismograph operated in 3-channel mode timed the compressional wave arrivals generated by a sledgehammer impulsive source at shotpoint-to-geophone distances of 30, 50, and 70 feet parallel to the slope toe. Two different bedrock types were evaluated by the line spreads shown as SL-1 and SL-2 on the Plate No. 1 geotechnical map exhibit. Seismic waves move quickly in hard, unfractured rock and more slowly in loose, highly weathered, or broken materials. Empirical correlations have been developed between measured wave velocities and rippability, both by equipment manufacturers and by local consultants for particular geological units, which in turn allow general predictions to be made about expected equipment performance.

4.0 ALIGNMENT GEOTECHNICAL CONDITIONS

4.1 Present & Past Area Uses

McCall Boulevard was built by cut-and-fill methods as a two-lane County road in about the year 1966. The road extended eastward from the newly established Sun City development and old U.S. 395 [Interstate 215] to Menifee Road. Other than the bedrock ridge transected by the rural road, land uses along the alignment had been limited to dry-farmed grain crops. The road did not follow a preexisting right-of-way and did not require demolition of any structures.

Subsequent years saw development slowly creep east of U.S. 395. In 1989, next to groves of citrus trees located north and south of McCall Boulevard near Oakhurst Avenue, the new “Menifee Valley Medical Center” hospital was opened. The western end of the widening project will border one of the relict groves. Orchards south of the boulevard were mass graded for a residential tract in 2004-2005. At the same time, construction was finishing up on a new elementary school northwest of the intersection of Junipero Road and McCall Boulevard. This work included localized boulevard widening at the site of AGI's soil boring B-5. Few changes other than eastbound-side asphalt curbing and a concrete pedestrian sidewalk through the deep cut and ending at Junipero Road seem to have been added since the mid-2000's. Based on our historical aerial image interpretations and field observations, the project alignment is not adjacent to past or present commercial sites, industrial-zoned property, or retail fuel stations.

4.2 Surface Conditions

Alignment-parallel utilities included potable water, gas, electrical, and telecommunications conduits. New service laterals into a future commercial development being built north of McCall Boulevard between Junipero Road and Menifee Road lie transverse to the traffic lanes. The eastern terminus at Menifee Road will cross multiple major high-pressure gas and water transmission lines.

McCall Boulevard is a designated urban arterial road that has received piecemeal improvements, patches, and surface restoration over more than 40 years. Existing asphalt pavement is in subjectively "good" to "poor" condition. Younger and better-engineered sections (thicker hot mix asphalt and provision of aggregate base layers), and areas excluded from regular traffic by painted islands, correlate to pavements with minimal visible distress. "Original" McCall Boulevard pavements penetrated in borings consisted of two lifts of conventional hot-mix asphaltic concrete totaling from ~5 to 5½ inches in thickness. Reflection block cracking and some subsidence with alligator cracking in wheel paths usually make it easy to pick the old road alignment from newer surfaces to either side. Notably, AGI's explorations encountered only a few inches of a select decomposed granite sand material below the original pavement. Graded aggregate was not found.

Descending fill slopes and ascending 1:1 rock slopes partly border the paved roadway. The former include oversteepened and very rocky embankment fills interpreted to have been derived from the ridgeline bedrock cut in 1966. Angular rock fragments are embedded in loose sandy matrix. The rocks can exceed 3 feet in size. Slope inclinations approach 1:1. Sparse scrub vegetation is present. The presumably undocumented fills would be buried by new fill along the north side of the boulevard beginning at the hospital entry opposite Oakhurst Avenue and extending about 900 feet east. Another short segment of original fill slope lies opposite Vine Street (a paper ROW) and just east of the topographic ridge. This fill may consist of more than 50 percent boulder-size rocks and has added dumped rock near the slope toe. A third stretch of rocky original fill slope will be concealed along the south side of the project between Durant Street and Junipero Road. Large landscaped fill slopes east of the bedrock ridge and next to elementary school play fields near Junipero Road are inclined at 2:1, and are interpreted to consist of modern compacted engineered fill.

4.3 Subsurface Conditions

4.3.1 As-Built Pavement Structural Sections

Pavement sections were field measured to the nearest quarter-inch at 4 soil borings and at the 4 auxiliary core locations. Tabulated results and notes are as shown:

Location	AC Thickness (in)	Base or Subbase Thickness (in)	Notes
B-1	5¾	0	Patch pavement circa 2005, but in original road grade.
B-4	1¾	~1½	Fargo Street ROW south of concrete drive apron. Minimal gravelly aggregate base layer.
B-5	5½	5½	New pavement circa 2005, built concurrently with elementary school to the north. Not in present-day regular traffic lane. No surface distress.
B-6	5½	6	"Original" McCall alignment pavement, 2-lift AC section. Subbase is select DG soil fill and not graded aggregate base.
AC-1	5¼	0?	"Original" McCall alignment pavement. Rocky subbase was impenetrable but likely composed of ordinary fill.
AC-2	6¼	4¾+	Tract pavement circa 2005. Not in present-day regular traffic lane. Pavement exhibits some longitudinal thermal cracks.
AC-3	5	18	"Original" McCall alignment pavement. Subbase is granular silty sand fill and not aggregate. Subbase may be bottomed on natural topsoil.
AC-4	5¼	6¾	"Original" McCall alignment pavement, 2-lift AC section. Subbase is select DG soil fill and not graded aggregate base.

4.3.2 Soil and Bedrock

Man-made Fill. Riverside County's original road construction project pushed bedrock-cut spoils westward and eastward into two principal alignment fills. The cut generated sand, gravel, cobble, and boulder-size rock fragments. The deepest parts of the rock cut required blasting. Soil boring B-1 documented a typical pattern of cobble-boulder fill from the deepest and hardest portions of the cut atop much sandier and less-rocky fill derived from the crumbly surficial weathered-rock horizons. Several auger refusals were met before one trial succeeded in fully penetrating the 17 feet of embankment fill at boring B-1. Most of the County fill classified as medium dense, with SPT N-values of 13 to 30. Although the rockiest materials had loose sandy matrix soils and lacked textural clues for organized compaction, deeper fill zones consisted of well-graded sand-gravel mixes with some evidence of placements in lifts. The County fill is considered an undocumented fill.

Other presumed undocumented fills were built later to connect McCall Boulevard with private driveways and several rural-residential streets east of the bedrock ridge. AGI's soil boring B-4 at Fargo Street penetrated almost 15 feet of loose to medium dense silty sand fill derived from weathered granite. Little if any modification to these street intersections appears to be proposed in the April 2022 alignment exhibit, however.

Widened street rights-of-way next to a residential tract and a school development were assumed to be products of modern engineered grading. The fills predate City incorporation [2008], but the relevant engineered grading reports are probably in Riverside County archives. Soil boring B-5 passed through about 8 feet of medium dense "decomposed granite" silty sand fill atop intact granite bedrock. Recovered samples exceeded contemporary standards of at least 90 percent relative compaction of a laboratory-determined maximum dry density for engineered grading. Slightly farther west, similar engineered fill was interpreted to be up to 20 to 22 feet deep at the proposed paved-surface limits.

Native Alluvial Soils. Three soil borings encountered medium dense to very dense silty sand and gravelly sand alluvium. Detrital soils west of the topographic ridge were composed mostly of weathered schist fragments embedded in heavily mottled non-porous fine-grained silty sand. These deposits were only 7 feet thick in soil boring B-1. East of the topographic ridge, grossly similar yellowish brown to strong brown (oxidized) alluvium underlies the street alignment from about Junipero Road to the endpoint at Menifee Road. The alluvium is probably several tens of feet deep at Menifee Road. Clay and silica hardpans occur in the alluvial stack. The deposits are typically consolidated, cohesive, and cemented below only 2½ feet or so. In geological terms the alluvium is very old, with an interpreted age range of middle to early Pleistocene.

Crystalline Bedrock. The existing roadcut through the ridge near the project midpoint exposes a remarkable intrusive contact between light-colored and homogenous igneous granitic rock and an older, dark-colored and layered metasedimentary unit composed of phyllite, fine-grained schist, and quartzite. Both units feature hard and strong rock near the street grade. Weathered rock composes a "rind" of softer and weaker material parallel to the original ground surface and commonly around 15 to 20 feet thick. Both units have close to wide-spaced joints and joint sets, while the metasedimentary unit exhibits a foliation fabric with a steep northeast-directed dip.

Multiple trials with the hollow-stem auger failed to penetrate more than 30 inches or so into either unit near the boulevard's high point. Blasting was used in 1966 to excavated the deepest parts of the cut, and the limited penetration depths appeared to comprise only blasting overbreak. Section 5.2 (Local Geologic Conditions) and the drill logs in Appendix A contain considerable additional descriptions and interpretations of soil and bedrock conditions in the project area.

4.3.3 Laboratory Test Results

Natural dry densities obtained from recovered barrel samples of man-made fill and alluvial soil ranged from approximately 103 to 133 pounds per cubic foot (pcf). Based on laboratory maximum density determinations specific to this investigation, calculated *in situ* relative compaction values were over 90 percent for presumed engineered fill placed in 2005 just west of Junipero Road. The pavement subgrade had a relative compaction of 97 percent at soil boring B-5. Weakly compacted native-soil subgrade below “original” McCall Boulevard pavement at boring B-6 had a relative compaction of 90 percent. AGI’s estimated mean *in situ* relative compaction for old non-engineered road fills such as the boring B-1 and B-4 localities would be under 90 percent.

Laboratory water content determinations collectively ranged from 2.3 to 17.2 percent by weight. Soil moisture in fill was most often under 7 percent, though. Menifee is typified by semi-arid conditions and normally dry or only damp soils except in winter. At the time of AGI’s subsurface investigation, soil moisture within fill or near-surface native deposits was generally well under the optimum water contents for compaction.

Laboratory direct shear tests were assigned for 2 representative “undisturbed” alluvium samples for purposes of trench stability characterization, RCP pipe design, and capacity determinations for drilled-shaft foundations. Peak-strength results of 30 to 33 degrees were obtained for the Mohr-Coulomb friction-angle envelope; cohesion intercepts of 325 to 350 pounds per square foot (psf) were derived. A remolded sample of typical angular “decomposed granite” sand was also tested to obtain data for slope stability analyses. Peak strengths for all local soils were marked by dilative behavior in shear (volumetric expansion).

4.3.4 Groundwater

Free groundwater was not observed in any of the exploratory borings. With the exception of soil boring B-1 at depths below 17 feet, recovered soil samples also lacked visible oxidation mottling or streaking we would associate with episodic groundwater saturation. The deeper parts of boring B-1 intercepted heavily mottled very old alluvium and crumbly, weathered metasedimentary bedrock. Our interpretation is that the top of bedrock plus a residual clay soil horizon are aquitards, and that the mottled older alluvium is an artifact of Ice Age pluvial periods when regional conditions were much wetter.

Upgradient areas from the western project alignment are now parts of a large housing tract. We suspect that wet rainfall years, combined with irrigation practices that invariably add several times normal rainfall amounts to landscaped yards and common-area greenbelts, will renew chances for short-lived perched-water layers to form near fill-native and native soil-bedrock interfaces. New roadway fills and fill keyways will require drainage provisions. Depending upon seasonal conditions, we think slope construction could theoretically intercept a temporary water-bearing horizon; however, seepage volumes would be small and probably easy to handle.

Data concerning depths to the permanent phreatic surface in the project area are sparse, and based on widely scattered points in a region with significant basement-rock relief. Past AGI investigations conducted north of McCall Boulevard included data for a well in the Shadow Ridge residential project. Depths to water of 65 feet to 81 feet were reported in years 2000-2001 by Eastern Municipal Water District (EMWD) officials and the former landowner for State Well No. 5S/3W 23C2, formerly located just north of today’s elementary school along Junipero Road. There were no records or recollections of water depths shallower than 50 feet in the well. We believe static groundwater depths in the vicinity of Menifee Road should be roughly the same as recorded at Well 5S/3W 23C2, or more than 50 feet. Other areas will have greater separation to saturated ground.

Future fluctuations in static water elevations should not be ruled out due to variations in precipitation, temperature, consumptive uses, and other factors (including further urbanization and development) which were not present at the time our observations were made. Nevertheless, outside of the noted potentials for seasonal perched water occurrences within basal old fill or very old alluvium close to soil boring B-1, our findings indicate that natural groundwater occurrences should not materially affect either design or construction of the widened street.

5.0 ENGINEERING GEOLOGIC ANALYSES

5.1 Regional Geologic Setting

All of western Riverside County lies within the Peninsular Ranges Physiographic Province, one of 11 continental provinces recognized in California. The physiographic provinces are topographic-geologic groupings of convenience based primarily on landforms, characteristic lithologies, and late Cenozoic structural and geomorphic history. The Peninsular Ranges encompass southwestern California west of the Imperial-Coachella Valley trough and south of the elevated escarpments of the San Gabriel and San Bernardino Mountains. Most of the province lies outside of California, where it comprises much of the Baja California Peninsula. The province is characterized by youthful, steeply sloped, northwest-trending elongated ranges and intervening valleys. Approaching the northern edge of the province, however, several anomalously flat and low basins stretch from the San Gorgonio Pass region to western Los Angeles as a result of fault junctures and tectonic interaction with the adjacent Transverse Ranges.

Structurally, the Peninsular Ranges province in California is composed of several relatively coherent, elongated crustal blocks bounded by active faults of the San Andreas transform system. The project is located in the central portion of the Perris tectonic block, the longest sides of which are bounded by the San Jacinto fault zone to the northeast and the Whittier-Elsinore and Chino fault systems to the southwest. Peninsular Ranges mountain blocks are dominated by crystalline intrusive rocks and a wide variety of pre-batholithic metamorphic units derived mostly from sedimentary parent rocks.

5.2 Local Geologic Conditions

Consistent with findings throughout most of the inland valleys of Riverside County, the majority of the project alignment not located in deep cut lies atop bedrock pediments: Nearly flat, planed-off rock surfaces that are the hallmarks of ancient erosional surfaces. Pediments extend westward beyond the project limits at Oakhurst Avenue, and east of the ridge cut to an undefined point between Junipero Road and Menifee Road. In offsite properties bordering the road, the pediments have thin covers of residual soil, colluvium, and alluvium, punctuated by a few small bedrock outcroppings in the otherwise featureless flat areas. The pediments appear to be approximately equally well-developed across different bedrock units found in the area, if off-site observations are included (e.g., in Medall, Aragon Geotechnical, 2001a and 2001b).

Regional experience has shown that pediment surfaces are usually characterized by relatively deep weathering of the bedrock units. Weathering profiles normally exhibit a progression from soft, highly or extremely weathered rock near the surface through moderately weathered intermediate zones before slightly weathered or fresh parent materials are encountered at depth. However, textural and mineralogical variations commonly impart some added resistance to weathering in localized areas, resulting in shallow zones of harder rock that can manifest themselves as ridges, knobs, and outcroppings as softer materials are eroded away.

As previously mentioned, geologic mapping and subsurface explorations indicated that the alignment crosses two different crystalline basement rock units and very old alluvial deposits (Plate No. 1). Bedrock nomenclature was derived from Morton (2006). From oldest to youngest, they are briefly considered as follows:

Metasedimentary rocks (Ƭml): Fine to medium-grained, thinly layered and foliated quartzite, schists, and phyllite of low metamorphic grade. This unit constitutes the oldest host rocks for younger intrusive granitic bodies in the area. North of McCall Boulevard the unit is sandwiched between two granite intrusions of disparate age and mineralogy. Layering and foliation within the unit generally strike northwest, with steep northeast dips. Near the surface the metamorphic unit is mostly moderately weathered and moderately hard to hard, but with abundant close fractures. At roadcut depths the unit is hard, strong to locally very strong, and with lower fracture frequency. The unit tends to weather into relatively smooth slopes littered with hard angular fist-sized rocks. However, the crest of the ridge transected by the roadway is defined by one or more resistant layered quartzite subunits that form prominent fin-like outcrops. Age: Triassic, per Morton (2006).

Domenigoni Valley granodiorite (Kgd): Medium to locally coarse-grained light gray, non-foliated, massive granitic rock, found east of the alignment highpoint and a very sharp and conspicuous contact with the dark metasedimentary rocks. This unit tends to form bold, rocky outcroppings in steeper terrain. Fracture spacing appears to be on the order of several feet. The outcroppings are relatively fresh, hard, and strong, and in some areas are considered to be continuous hard-rock masses without concealed weathered zones. Although most pediment areas appear to contain much highly weathered material, hard bouldery outcroppings were also noted south of McCall Boulevard and east of Summit Street in several private lots. Age: Cretaceous.

Very old fan alluvium (Qvof): Geologic interpretations indicate the massive older alluvium probably originated as axial fan deposits that have been moderately dissected by later erosion. Morton (2006) assigns middle to early Pleistocene ages for very old alluvium crossed by the street alignment. Cementation and clay enrichment are pervasive. In our experience the unit is often overconsolidated. AGI did not find evidence in boreholes for slightly younger fan sediments that in some regional maps are depicted near Menifee Road. The flat surface topography is considered part of the "Paloma" erosional surface of Woodford et al. (1971), typified by fairly strongly developed illuvial clay and calcic horizons atop the older parent materials. The older alluvium conceals significant bedrock relief. Water well data indicate top-of-bedrock elevations range from hundreds to locally more than a thousand feet below the Paloma surface in parts of the Romoland – Perris Valley area. The maximum depth of the unit in the study area is not known, but can be reasonably inferred to be in excess of 50 feet near Menifee Road based on regional experience.

5.3 Paleontological Sensitivity

The inland valleys have historically been discovery sites for vertebrate megafauna fossil remains, perhaps most notably during construction of the West Dam of nearby Diamond Valley Lake. This and other sites such as the old Bernasconi Hot Springs have produced assemblages partly matching the famous mix of extinct species (mammoth, camel, horse, giant sloth, saber-tooth cat, etc.) found at the Rancho La Brea "tar pits" of Los Angeles. All "Paloma" surfaces underlain by pre-Holocene sediments have potential for harboring fossil resources.

Riverside County categorizes igneous granitic rocks (Domenigoni Valley granodiorite) along the alignment as "Low Potential" for paleontological resources. The real-world potential is zero. Excavations in this unit will not require monitoring.

The County classifies layered metasediments, unit ₸ml, as “Undefined Potential” for vertebrate fossil remains. Geologic literature checks indicate that only poorly preserved invertebrate fossils (pelecypods and crinoids) have been found in a single recrystallized marble body in the Sun City area. The McCall roadcuts lack marble and are highly deformed into very tight isoclinal folds. It is our judgment that chances for any significant paleontological resources, much less vertebrate remains, in the unit are nil. Monitoring should be waived.

Older fan sediments west of the deepest bedrock cut, and east of approximately Packard Street to the terminus at Menifee Road, are classified as “High Sensitivity High B” by Riverside County. The classification infers that significant non-renewable paleontologic resources may be present starting approximately 4 feet below grade. AGI judgments concerning sediment age and fossil-bearing potential, based on pedogenesis, depositional environment, regional findings, and geomorphology considerations would support the County’s classification. Paleontological monitoring will probably be required by City authorities acting as lead agency, wherever proposed excavations match or exceed 4 feet deep in the Qvof geological unit shown on AGI’s Plate No. 1 exhibit.

5.4 Faulting and Regional Seismicity

The project is situated in a region of active and potentially active faults, as is all of metropolitan Southern California. Active faults present a variety of potential risks to the built environment, the most common of which are strong ground shaking, liquefaction, settlement, mass wasting, and surface rupture along active fault traces. Generally speaking, the following four factors are the principal determinants of seismic risk at a given location:

- Distance to seismogenically capable faults.
- The maximum or "characteristic" magnitude earthquake for a capable fault.
- Seismic recurrence interval, in turn related to tectonic slip rates.
- Nature of earth materials underlying the site.

5.4.1 Fault Rupture Potential

Surface rupture represents a primary or direct potential hazard to linear infrastructure that crosses an active fault zone. The widening project is a minimum of 9.76 miles from the closest known active fault (Casa Loma strand of the greater San Jacinto fault zone), located northeast of the project near the town of San Jacinto. The San Jacinto fault has had recurrent surface rupture within the last 11,700 years based on trenching studies by geological investigators. Hazard maps show the project does not cross or enter official State of California Earthquake Fault Zones (“Alquist-Priolo” zones), or Riverside County Fault Hazard Management Zones for ground rupture hazards. AGI’s findings indicate the potential for direct surface fault rupture affecting public infrastructure along the alignment should be extremely low.

5.4.2 Strong Motion Potential

Intense ground shaking has a high probability to occur within the design lifetime of the proposed improvements, considering historical seismic experience and proximity to multiple regional faults capable of generating large earthquakes. Menifee was spared major damage in 1992 and again in 1999 from events in the high desert north of Coachella Valley. Although magnitudes for these events (Landers and Hector Mine earthquakes) were M_W7.3 and M_W7.1, respectively, epicenters more than 55 miles away and north-directed ruptures along the causative faults helped minimize effects in the Inland Empire.

Site-to-source distance, fault length, and recurrent Holocene rupture history identify the San Jacinto fault zone as the most critical source for strong ground motion in Menifee. The fault zone constitutes a set of *en-échelon* or right- and left-stepping fault segments stretching from near Cajon Pass to the Imperial Valley region. The primary sense of slip along the zone is right-lateral, although many individual fault segments show evidence of at least several thousand feet of vertical displacement. The San Jacinto fault zone has been very active, producing possibly eight historical earthquakes of local magnitude 6.0 or greater. The communities of Hemet and San Jacinto were heavily damaged in 1918 and again in 1923 from events on the San Jacinto Fault. Pre-instrumental interpreted magnitudes for these events were M_L6.8 and M_L6.3, respectively.

Other, more-distant regional faults are unlikely to cause shaking as intense as that caused by rupture of the San Jacinto system. Strong ground motion *per se* is not a design variable for road surfacing design, but is important for retaining walls and for assessing threats from permanent ground deformation hazards such as liquefaction, landsliding and rockfall, lateral spread, and settlement.

Earthquake shaking hazards are quantified by deterministic calculation (specified source, specified magnitude, and a distance attenuation function), or probabilistic analysis (chance of intensity exceedance considering all sources and all potential magnitudes for a specified exposure period). With certain special exceptions, today's engineering codes and practice generally utilize (time-independent) probabilistic hazard analysis. Prescribed parameter values were calculated from the latest 2014 U.S. national hazard model applied to coordinates near the project's geographic midpoint (coincident with the tallest cut slopes while also applicable to proposed alignment walls). Slope and wall locations were assumed Site Class C. Estimated shaking intensities include a 10 percent probability in 50 years of peak ground accelerations (PGA) exceeding approximately 0.38g, and 2 percent chance in 50-year exposure period of exceeding 0.66g (U.S. Geological Survey, 2022). The reported peak ground accelerations were linearly interpolated from 0.01-degree gridded data and include soil correction for the noted site class suited to weathered rock. Predicted peak or spectral acceleration values should never be construed as representing exact predictions of site response, however. *Actual* shaking intensities from any seismic source may be substantially higher or lower than estimated for a given earthquake event, due to complex and unpredictable effects from variables such as:

- Near-source directivity of horizontal shaking components
- Rupture propagation direction, length, and mode (strike-slip, normal, reverse)
- Depth and consistency of unconsolidated sediments or fill
- Surface topography
- Geologic structure underlying the site
- Seismic wave reflection, refraction, and interference (basin effects)

5.5 Liquefaction and Dynamic Soil Densification.

Liquefaction is the transformation of a granular material from a solid state into a semi-fluid state as a consequence of increased pore-water pressure. Certain soil materials subjected to ground vibrations will tend to compact and decrease in volume. If the materials are saturated and drainage is unable to occur, the tendency to decrease in volume will result in an increase in pore-water pressure. Intergranular pressures may build up to a point where they equal the overburden stress and the effective stress becomes zero, whereupon the soil loses strength and may become capable of flowing as a viscous fluid. Liquefaction risks are highest in seismic regions where loose sand or non-plastic silt occur below groundwater.

Data and observations from this investigation indicate near-zero risks from liquefaction, but localized chances for minor seismically induced volumetric strain settlements in fill are interpreted along the alignment. Shallow groundwater is absent, thus eliminating liquefaction opportunity. Also, native alluvial materials that will support embankment fill or pavement sections are characterized by significant geologic age and high relative densities. These characteristics limit liquefaction and dry-soil settlement susceptibilities. We suspect that small settlements could occur in the 1966 road fills, but surface manifestations would arguably be tolerable for paving (the fills are at most 17 to 20 feet deep). Reconstruction of old fill is preliminarily viewed as impractical and overly conservative except as might be specially required for proposed walls. Other ground deformation phenomena such as lateral spreading, ground fissuring, and ground loss from “sand boils” originating in shallow liquefied layers are also ruled out as hazards to the proposed project.

5.6 Slope Stability

Most of the street widening project will be within low-relief terrain marked by gentle topographic gradients. The project is not expected to fundamentally alter two major bedrock cut slopes next to the roadway high point. New fill slopes are proposed in three locations. Fill slope vertical heights will in one case approach 35 feet. AGI’s investigation considered global and surficial stability of model fill slopes and existing bedrock cut slopes under static and earthquake conditions inclusive of rockslide or rockfall threats.

Bedrock Cut Slopes. The existing roadcuts are up to approximately 65 feet high and have relatively smooth inclinations of 1:1. Brow ditches and mid-slope benches are absent. Westbound McCall Boulevard slopes are composed of both granitic rock and a layered, foliated metasedimentary unit. The lower-height and solitary eastbound McCall Boulevard cut slope is composed only of granitic rock. Slope exposures of both rock types include zones of highly weathered material that freshens with increasing depth below original grade. Moderately to only slightly weathered rock occurs near the road elevation. The southern or eastbound slope is much more weathered and tends to have only isolated hard residual corestones up to a few feet across surrounded by softer material. Harder bedrock exhibits multiple joint sets, while fractures in weathered materials are almost impossible to discern. Weathered-rock slopes still show tool marks from the time of original construction, indicating that ordinary bulldozers of the time could handle excavation duties in the weathered zones.

We would judge current cut slopes to be globally stable indefinitely under both static and seismic conditions. The rock slopes are composed of very strong materials from a soil mechanics viewpoint. Stereographic analyses of joint planes and foliation partings indicate minimal risks from wedge popouts, topples, or block glides. The slopes lack evidence of rockslides and seem to have performed very well over a span of 56 years. Nonetheless, weathering, blasting overbreak, and a lack of slope maintenance have created some unstable fragments and pockets of accumulated loose rock perched across the slope faces. Future construction specifications should include a provision for slope scaling to remove all slough and detached rocks.

Proposed Fill Slopes. Section 6.8 includes guidelines for preliminary slope designs that in our opinion should meet future qualitative and quantitative assessments of acceptable factors of safety. For planning purposes, however, AGI evaluated a model 35-foot-high reinforced fill slope for gross and surficial stability, assuming construction utilizing site-derived or equivalent import decomposed granite sand and a design inclination of 1.5:1. Widening of the existing road will result in new-fill prisms "buttressing" the old embankment fills. The model assumed the old fill stays in place except for lateral benching. The software code STABL with STEDwin utility was used for gross stability analysis according to the Modified Bishops method for circular slip surfaces. Calculated factors of safety of 1.52 and 1.14 were obtained for static and pseudostatic (seismic) cases, respectively, higher than the minimum needed factors of 1.5 and 1.1. Sample sections and the software output are included in Appendix C.

Surficial stability was calculated according to Riverside County Transportation and Land Management guidelines for a 4-foot-thick saturated zone parallel to slope surfaces at inclinations of 2:1 and 1.5:1. The analyses confirm that ordinary 2:1 slopes without mechanical stabilization are feasible, but an unreinforced 1.5:1 slope fails to meet required factors of safety. A surficial stability factor of safety of 1.52 was obtained for a hypothetical 1.5:1 *reinforced* fill slope, also exceeding the minimum required factor of safety of 1.5. Reinforcement options are outlined in Section 6.8.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based on the results of the field exploration program, laboratory testing, and professional experience, it is our opinion that alignment soils and geologic conditions do not impose major limitations for the proposed new construction. *Bedrock* is the recommended competent material under proposed new embankment fills and fill slopes, except where new fill will onlap existing road fill. The latter may remain in place. Construction excavations should be far above permanent groundwater levels, and have only slight chances of finding seasonal wet zones near soil-rock contacts. The street infrastructure should also be free from landsliding or liquefaction-related hazards requiring avoidance or unexpected stabilization measures.

In no particular order, a synopsis of key findings and recommendations are presented in the following bulleted list. Multiple specification options appear to be feasible for many aspects of the construction. The prime design consultant and City engineers should endeavor to examine options with respect to constructibility, cost (value engineering), community disruptions, and even aesthetics.

- Reinforced 1.5:1 fill slopes composed of select fill soil are deemed feasible. Widening will probably require import soil, however. Import soil is currently very hard to obtain and very expensive. Volumes could be reduced with special engineering to bring slopes to a 1:1 condition or even steeper. See Section 6.8.
- Existing road fill is not recommended for support of the mid-slope retaining wall depicted on plans east of Oakhurst Avenue. See below, and Section 6.9.
- All new embankment fills will need to be keyed into competent bottom materials and benched laterally into the (saved) existing road fills. See Section 6.7.
- Proposed toe walls at the base of rock cuts may safely ignore any surcharge loads from the ascending 1:1 slopes behind the walls. See Section 6.9.
- Existing McCall Boulevard pavements dating from 1966, and some later patches, have inferior structural sections for an arterial road. AGI recommends full-depth removal and reconstruction of the inferior sections. See Section 6.11.
- Pavements built circa 2005 appear to have structural sections compatible with a 6-lane boulevard. They are in good condition and could be saved. The newer pavements are found in the eastbound side between Oakhurst Avenue and a point roughly 1575 feet east of the intersection, and the westbound side proceeding west of Junipero Road for roughly 620 feet. See Section 6.11 for suggested surface rehabilitation recommendations to correct minor cracking and extend pavement lifetimes.

The mid-slope wall is the most problematic part of the project. The existing road fills are extremely rocky at the surface and deemed compressible. Reconstruction of the old fill within the bearing zone of the wall (nominally 1:1 subsurface projections from the lower edges of foundations) is one option. This may be hard to accomplish, however, due to fill depths and interfering utilities in the traffic lane. An alternative would move the wall to the ROW line. Wall height would determine whether an intermediate (temporary) fill slope rising to street elevations or a level backfill suited to a future No. 3 lane would be possible. A third option for the depicted 2-lane enhancement would be an engineered reinforced slope at an inclination that removes the need for a wall.

6.2 Excavatability

Professional experience with the crystalline basement rocks of Southern California has shown that "rippable" material usually falls below an upper-bound seismic wave velocity of around 6,000 feet per second (fps), considerably less than published Caterpillar Tractor Company cutoff values of 7,300 to 8,400 fps for harder metamorphic and granitic rock types when utilizing newer-model D9-class heavy bulldozers in a single-shank configuration. Most contractors report that the massive nature of many local bedrock units reduces the ability to maintain ripper penetration.

Rippability along the proposed toe walls in the deepest rock cut was assessed with two short seismic refraction line spreads. Measured seismic velocities within both the layered metamorphic rock and Domenigoni Valley granodiorite were very similar at ~4,850 to ~5,600 fps. Given the slightly weathered conditions of the bedrock, we interpreted that rock masses near the toe have undergone some dilation from construction blasting. Presumed fresh rock with seismic wave velocities >15,000 fps was detected at depths of around 17 to 20 feet below the road surface.

AGI does not view a simple dozer "notch" placed at the tallest cut slope toe as feasible. The rocks are too hard for blade excavation with small dozers. Available real estate may be too narrow for large bulldozer passage. Analyses point to these general conclusions:

- Drill-and-blast excavation is not desirable due to risks to the surrounding built environment.
- Most slightly weathered bedrock located at the toe wall alignment will be non-rippable with ordinary excavator buckets.
- Weathered "decomposed granite" east of the (paper-street) Paloma Road intersection should be rippable with small dozers and tracked excavators.
- Non-rippable bedrock will not disintegrate into small particles, but will produce oversize hard-rock fragments that may need to be hauled from the project.

We expect that harder bedrock should be breakable with "medium" hydraulic hammers used on excavators. Some quartzite in the metasedimentary unit, however, may be especially tenacious and exhibit higher impact strength than familiar Riverside County granitic rocks at the same degree of weathering. The work may be slow. Rock strength is probably anisotropic, with lower strength expected in the foliation-parallel direction.

AGI research into hammer attachments, considering multiple factors, would suggest that a 3,000 to 5,000 foot-pound (ft.-lb.) rated breaker might be the optimal tool for this project. Breakers in this energy rating range are easily fitted to Caterpillar 225-class excavators. Smaller hammers will be slower, adding to total construction duration and predicted vibration *cycles*. Bigger hammers will impart stronger ground vibrations that are more likely to exceed allowable threshold velocity *amplitudes* for structures, and they need heavier carriers. Also, many medium-size hammers come in what's called a "boxed silenced" design which encloses the primary hydraulic case in a second layer of steel. These are much quieter than standard breakers. We do think that contractor trials at the start of hard-rock excavation should be considered. Because of nearby residences, it would be prudent to identify the lowest-energy hammer that will accomplish the task while minimizing total energy delivery to the rock. We recommend consultations with excavation contractors in advance of placing any type of equipment specification onto plans. Contractor inquiries to this office are welcomed.

If regular construction practices cannot be used for the rock cut, AGI can recommend the option of non-explosive chemical cracking agents (Betonamit brand).

6.3 Vibration Monitoring

Consideration is advised to implement a vibration monitoring program. Residential lots are as close as 110 feet to future rock excavations and proposed fills. Dump trucks, earthmovers, compactors, excavators, and hydraulic hammer attachments will produce some level of vibration. Nearby buildings will respond to the vibrations with varying results: No perceptible effects at the lowest levels; felt vibrations at moderate levels; and possible damage at the highest levels. Threshold values for *peak particle velocity, PPV*, in units of inches per second (in/sec) are the criteria customarily used to limit damage risks. AGI believes that rock breaking poses the highest chances for causing resident complaints or physical damage. The following discussion is primarily for information purposes. The City or project construction manager should consult with a vibration monitoring expert to assess risk and finalize the project maximum-allowable PPV and detector placements.

Blasting vibrations are better studied than construction vibrations. Vibration sources, velocities, and operations are far more variable in construction vibration than in blasting vibration. Researchers confirm, though, that threshold velocities for a specific probability of damage (or some other similar measure of damage likelihood) should be considerably lower than those for blasting (i.e., the damage probability is higher for a given ground vibration PPV associated with construction work than for the same velocity associated with blasting). Intuitively, this can be seen to be a function of vibration duration or number of cycles which are usually far greater in construction work. Vibration damage tends to be cumulative. Thus, construction vibration standards set lower "acceptable" vibration PPVs for homes than those set for blasting.

Empirical data for PPV as a function of distance from hydraulic hammers on excavators are notably absent. However, Caltrans (2020) suggested the following equation to derive estimated values, based on documented pile driving effects:

$$PPV_{Hydraulic\ Breaker} = PPV_{Ref}\ (25/D)^{n}\ x\ (E_{equip}/E_{Ref})^{0.5}\ (in/sec)$$

Where

PPV_{Ref} = 0.24 in/sec for a reference hammer at 25 feet distance.
D = distance from the hydraulic hammer to the receiver, in feet.
n = attenuation coefficient; recommended value of 1.0 for hard rock condition.
E_{equip} = rated energy of selected hammer, ft.-lbs.
E_{Ref} = 5,000 ft.-lb. (rated energy of reference hammer)

Pre-construction surveys of nearby existing structures and outdoor hardscape improvements are recommended to establish individual baselines of distress in advance of construction. It is often difficult to differentiate between pre-existing damage and damage caused as a result of construction since few structures are completely free of defects or signs of distress. This assessment should include a thorough site walkthrough, complete documentation (notes, photographs, videos) of existing distress, and measurements of pre-existing cracks in slabs and hardscape. Solid documentation of existing conditions and attention to detail is critical. Even a hairline crack in a building or driveway slab can lead to alleged vibration damage of the improvement.

It is a good practice to install vibration monitors some weeks prior to the start of construction activity to establish baseline ground motions caused by passing cars, trucks, and other non-construction vibration sources near the structures of concern.

Following completion of construction, a post-condition report should document any changes to the existing condition of monitored structures. This report would include all data collected from the pre-construction assessment, the levels of vibrations maintained during the actual vibration monitoring, and field observations of construction activities. Before and after records would be compared to support or disprove a claim.

6.4 Temporary Construction Excavations

It is unknown at this time whether utility relocations or new electrical, irrigation, or drainage infrastructure will be required. Trenching could theoretically encounter old roadway fill, newer engineered fill, native alluvium, or bedrock.

Protective trench shoring should be specified for all vertical cuts deeper than 5 feet along McCall Boulevard, including bedrock cuts where some amount of blasting overbreak is assumed. Passive trench wall shields (i.e., trench boxes with gaps to the excavation sidewalls) are preliminarily judged *suitable* for personnel protection in bedrock slots and any deep excavations in old non-engineered fill. Passive shielding protects workers but allows sidewall movement. Tight sheeting could be required to prevent ground loss if unstable conditions are observed in old fill. Active shielding should also be employed where new trenches are proximal to the backfill prisms in parallel utility trenches. Protective systems design by a registered professional engineer is encouraged for excavations shallower than 20 feet, and is required for deeper cuts.

We recommend the use of a rectangular distribution of earth pressure for temporary braced shoring. For an unsaturated level grade supported by the shoring, the maximum earth pressure would be 40H (in psf) where H is the height (in feet) of the shored cut face. Excessive raveling or caving tendencies are not anticipated in older alluvium or bedrock, but are judged possible in old roadway fill. Large rocks may be encountered in any original-fill excavation.

AGI recommends that temporary slopes in engineered fill or native older alluvial deposits be inclined no steeper than 1:1 per Cal-OSHA allowable limits for Type B soils. Surcharge loading should be avoided within 4 feet of the top of all unbraced vertical or sloped cuts, or inside of a line drawn at 45 degrees from the toe of the excavation, whichever is greater. Consideration should be made for secondary support of exposed active utility laterals spanning trenches or excavations more than a few feet in width.

The design of shoring should also include surcharge loading effects of existing structures and anticipated traffic, including periodic semi-trucks and construction equipment, when loading is within a distance from the shoring equal to the depth of excavation. A recommended minimum uniform lateral pressure of 100 pounds per square foot in the upper ten feet of retained trench walls should be incorporated in the design when normal traffic is permitted within 10 feet of the shoring.

6.5 Utility Trench Bedding and Backfill

RCP Storm Drains. It is possible the project could incorporate local drainage improvements. Whether City-owned and maintained, or ultimately dedicated to the Riverside County Flood Control District, we would recommend bedding material and placement dimensions for RCP conform to the requirements of the District's standard drawing Number M815. Foundation zone preparation for RCP drain pipes should include a minimum of 4 inches of mechanically densified sand bedding material in the normal, unsaturated native-ground condition. AGI recommends at least 12 inches of filter bedding where RCP would bear on bedrock. AGI understands that District practice for pipe bedding above invert has now been standardized to the sole selection of controlled low strength material (CLSM) placed as slurry. For CLSM there are particular procedures for backfill staging in order to prevent pipe flotation, which should be addressed by the project civil engineer in the specification. At least 12 inches of CLSM would be recommended laterally at pipe springline elevations.

Other Utilities. Granular sand bedding may be shaded around mains, conduits, or live domestic utility laterals in accordance with each purveyor's particular standards and requirements. Findings from this study indicate the native alluvial soils and old embankment fill along the alignment fail to meet Greenbook specifications for use as pipe bedding. Decomposed granite sand normally does meet public works specifications, but should be confirmed as suitable by sand equivalent testing before use.

Properly compacted granular alignment soils should be acceptable backfill for all drain or utility trenches. Native-soil trench backfill has advantages: It is available on-site without trucking and materials costs, produces a subgrade with a similar elastic modulus as the neighboring portions of the street, and would be less susceptible to transmitting water vertically or laterally. Local soils and granular import soils free of large rocks and any deleterious organic wastes should be geotechnically suitable for compacted regular backfill. Latest-edition "Greenbook" specifications in §306-1.3 would be recommended for classification of oversize particles. Rocks up to 6 inches in diameter will be acceptable in regular trench backfill as long as they are fully surrounded by compacted soil matrix. AGI would recommend rocks larger than 2½ inches in any dimension be excluded from fill placed within one foot of pavement subgrades, however. Cobble- and boulder-size broken rock fragments excavated from old roadway fills will be too coarse to be useful for backfills.

6.6 RCP Design Variables

Flood District standard designs are based on the Marston-Spangler indirect method of RCP specification. Based on a District-standard specification for CLSM bedding, a minimum load factor of 2.4 is considered appropriate and conservative in the "D-load" calculation.

Soil unit weight γ_w in the compacted backfill prism over a prospective drain line may be set at 145 pounds per cubic foot (pcf) for all street alignment soils and crushed-rock aggregate base materials in pavement subgrades. Imposed soil loads on buried pipe will be reduced by soil arching and sidewall friction of the backfill in trenches. The dimensionless product of the Rankine ratio and sidewall friction coefficient ($K\mu'$) may be taken as 0.150 for all local soil type blends (expected classification SM) inclusive of existing fills and very old alluvium. Vertical live loads from vehicles (AASHTO HS20 minimum) should be added to the imposed soil loads for burial depths of less than 8 feet to pipe crown.

6.7 Site Earthwork

AGI recommends the general guidelines presented below be included in the project construction specifications to provide a basis for geotechnical quality control during grading and backfill. It is recommended that all compacted mass fills be placed under continuous engineering monitoring and in accordance with the following:

- Demolition and removal of abandoned and co-located improvements including slabs, pipes, cables, curbs, and sidewalks is recommended. Existing CIDH foundations (signals) can be cut off 24 inches below soil subgrade and the cavity backfilled with ordinary soil. The ends of any defunct wet utility lines intercepted by the new construction should be capped, or plugged at both upstream and downstream ends with at least 5 feet of concrete or flowable fill. Representatives of AGI should perform periodic observations of any project demolition work and pipeline sealing to document the nature and depths of buried improvements.

- Boulders, shot-rock fragments, waste concrete, and any interfering vegetation should be cleared from existing slope faces and toe areas where new embankment fills are proposed. These materials will need to be spoiled from the job. The excavation contractor should be prepared to supply personnel to pick unacceptable inert or organic debris from fill soils during placement.

- Proposed fill slopes will require shear keys at toes bottomed at least 2 feet below the top of competent bedrock. Per soil boring B-1, this recommendation means that toe excavations could approach 10 feet deep along the north side of the street ROW west of the bedrock ridge. It will be critical to remove residual clay soil detected in the boring. Toe excavations could be close to the minimum depth northeast of Vine Street, and should be under 5 feet deep between Durant Street and Junipero Road. Keyways should be at least 15 feet wide and canted at least 1.0 foot lower in the heel than the toe. A "burrito" type drain with tight-line outlet pipe directed beyond the as-built toe should be supplied. Backdrains will not be required. Keys ascending natural slopes lateral to a swale or channel should be stair-stepped wherever native slopes are 5:1 or steeper. Grading exhibits depicting typical dimensions are included in Appendix C. All dimensions and slope features may be subject to field adjustments and revised recommendations during grading as determined by the Geotechnical Engineer.

- At least 1.0 foot of stripping is preliminary recommended for fill placed atop older alluvial areas east of Junipero Road. Stripped areas should be scarified an additional 8 to 10 inches, processed, and compacted before beginning new fill. Where cuts are deeper than 1.0 foot, processing-in-place may be adequate to create dense and unyielding soil subgrades.

- Observation and acceptance of all keyways and stripped areas by the Geotechnical Engineer and/or Engineering Geologist and/or their designated representative shall be done prior to placing fill.

- Shallow processing of 3 to 6 inches of exposed bedrock bottoms is recommended. Scarified materials should be throughly watered, mixed and compacted before initiating engineered fill lifts.

- Fill soils should be uniformly moisture-conditioned by adding water, drying back, mixing, and blending as needed to uniformly achieve the laboratory-determined optimum water content or higher, and placed in loose lifts having thicknesses commensurate with the type of compaction equipment used, but generally no greater than 6 to 8 inches. *Fill water contents below the recommended minimum water content shall constitute a basis for non-acceptance of the fill irrespective of measured relative compaction, and at the discretion of the Geotechnical Engineer, may require the fill be reworked to produce uniform water contents at or over the desired 100% of optimum moisture.*

- Benching is recommended wherever new fill will rise against existing 1960's-era fill or inclined native ground sloped steeper than 5:1. *The existing fills will be very rocky*. Loosened boulder-size fragments should be removed and level benches extended farther into the slope until a stable backcut can be obtained. Bench face heights should strive to maintain the 4-foot dimension shown on AGI grading exhibits.

- Minimum acceptable relative compaction will vary with depth and added mechanical stabilization features. Ordinary unreinforced fill shall meet a minimum bulk dry density of at least **90 percent** of the laboratory maximum dry density, determined according to the ASTM D1557-12 standard. Uniform compaction of at least **95 percent** of the laboratory maximum dry density is recommended for all sloped fills steeper than 2:1, and any granular soil backfill containing geogrid, wire mat, or other MSE elements placed behind slopes and walls. Soils within 1.0 feet of finish street subgrades shall also be compacted to 95 percent relative compaction. Compaction may be supplied by tracked or rubber-tire equipment, dynamic padfoot compaction rollers, compaction wheels mounted on excavators, and other tools at the discretion of the contractor.

- Field testing should be performed to verify that the recommended compaction and soil water contents are being achieved. Where compaction of less than the specified relative compaction is determined, additional compaction effort, with adjustment of the water content as necessary, should be made until the minimum specified compaction is obtained. Density tests in trench backfill should be performed at no less than Riverside County Transportation Department standard minimum frequencies of at least one test per 250 lineal feet of pipe (or a negotiated longer segment not to exceed 300 lineal feet), per 2-foot backfill depth increment, and should be randomly staggered as the backfill rises toward finish surfaces. Field density tests should conform with the ASTM D6938-08 (nuclear gauge) test standard, or alternatively with the ASTM D1556-07 standard (sand cone method).

Import soils will probably be required for the three principal embankment fills shown on Plate No. 1. Because the nature and type of MSE embankment or (optional) retention structures is unknown at this time, specific recommendations for soil gradation and internal shear strength should be provided in later plan reviews. As a general rule, however, reinforced slopes and wall backfill should classify as USCS granular soils with a maximum 30 percent fines and no more than 10 percent clay, and friction angle phi $\geq$30 degrees. Local silty and gravelly sand would meet these targets. Import soils should also have low or negligible expansion potential and be free of deleterious organic matter and large rocks. Off-site borrow areas must be reviewed, and representative samples qualified by appropriate sieve and shear strength tests accepted by the Geotechnical Engineer prior to use.

6.8 Slopes

The project will include manufactured fill slopes, and may optionally include new bedrock cuts slopes. This investigation has confirmed the general feasibility for new fill slopes up to 35 feet in height in the project alignment. However, it is recommended and required by City ordinance that all planned slopes in excess of 30 feet in height be independently evaluated for global stability based on construction plans. Grading activities to construct slopes at the site should be performed in accordance with the following:

- Plain unreinforced fill slopes should be constructed at maximum slope inclinations of 2:1 (horizontal:vertical). Slopes more than 30 feet tall should include a mid-slope bench and non-erosive terrace drain at least 6 feet wide. Slope height for fills that include a toe wall need not include the wall height in the overall slope height calculation.

- Reinforced-soil slopes containing synthetic geogrid as the reinforcing elements will have an allowable maximum inclination of 1.5:1. Slopes more than 30 feet tall should include a mid-slope bench and non-erosive terrace drain at least 6 feet wide. Slope height for fills that include a toe wall need not include the wall height in the overall slope height calculation. Geogrid tensile strength shall meet or exceed the minimum-recommended value of 300 pounds (2% strain limit) per foot parallel to the slope face. The minimum panel length from tail to slope face shall be 15 feet at the minimum strength. Vertical separations between successive course of geogrid reinforcement should be 18 inches or less. A sample product meeting these minimum specifications would be Tensar biaxial geogrid BXSQ100.

- Wire-reinforced slopes, gabion-faced slopes, and other proprietary approaches are feasible for McCall Boulevard and can be considered for face inclinations of 1:1 and sometimes even steeper. Construction approaches typically mirror those for MSE walls, with manufacturer modifications and special soil requirements. We recommend AGI consultations with the civil designer to address specifications if these options are desired.

- Cut slopes should also in general be constructed no steeper than as-built 1:1 inclinations. Consideration may be given to deleting a proposed toe wall in massive weathered granodiorite bedrock on the north side McCall Boulevard and simply moving the slope face north approximately 4 feet. The suitable segment would be from (paper street) Paloma Road eastward to daylight. This option would be faster and likely far more economical than a wall. It is recommended that any planned cut slopes steeper than 2:1 be individually evaluated and accepted prior to grading by the Engineering Geologist based on site-specific conditions.

- The surfaces of all fill slopes should be compacted as generally recommended under Site Grading, and should be free of slough or loose soils. Plain fill slopes may be constructed by overfilling and cutting back to expose fully compacted soil (the preferred method), vertical track-walking with heavy dozers or other suitable tracked equipment, or backrolling with heavy rollers. Reinforced slopes will require careful compaction of each lift to almost match the design slope face, with finishing done by grid rollers to avoid snagging the reinforcing elements. The desired result should be 90 percent or 95 percent relative compaction to the slope face for the plain and reinforced conditions, respectively.

- Embankment fills outboard of existing old fill slopes and any skin fills should maintain a minimum horizontal thickness of 15 feet or one-half the fill slope height (whichever is greater), and be adequately benched into adjacent materials. A bottom bench or keyway must be constructed at the base of the fill per the grading notes in Section 6.7.

- Fill slopes that in the opinion of the Engineering Geologist are founded on impermeable materials (bedrock) should be provided with a heel drain. Generally, a “burrito-style” drain consisting of perforated plastic pipe encased in free-draining aggregate and surrounded by appropriate geotextile filter fabric would be sufficient for this condition. Specific design details should be developed according to the actual needs assumed from future recommended grading plan reviews and/or bottom conditions observed during construction.

- Periodic inspections by the Engineering Geologist of all new bedrock cut slopes are recommended during construction. The inspections should be performed at vertical intervals of not more than about 8 feet. Observations would verify that slopes are free of loose rocks or unfavorably oriented discontinuities, and would include collection of geologic data to produce an acceptable “as-built” geologic map of the alignment at the conclusion of grading.

- The ground surface adjacent to the tops of fill slopes should designed and graded to drain water away from the slopes. Where or if needed, brow ditches should be constructed at the top of (new) cut slopes to intercept normal runoff from adjoining higher ground.

- Erosion control measures should be implemented for all slopes as soon as practicable after slope completion, per applicable City of Menifee ordinances.

AGI recommends scaling of the existing 1:1 roadcuts prior to toe wall construction and street surfacing. The slopes appear to be accessible to a manned bucket lift. Soil and loosened/detached rock particles should be dislodged with rakes, hoes, probes or pry bars until a stable face is achieved. Vegetation is considered beneficial and should be saved. Periodic slope evaluation by the Engineering Geologist during scaling is recommended. Once mechanically scaled, a final wash using a high-pressure stream of water is advised. Because the slopes will continue to shed small rocks indefinitely, toe walls should incorporate barriers to intercept tumbling stones before they land on sidewalks or bounce into traffic lanes.

6.9 Retaining Wall Structural Bearing and Lateral Earth Pressures

Walls founded on bedrock: A maximum allowable bearing value of 3,000 pounds per square foot (psf) may be assumed for slope toe-wall foundations embedded a minimum of 1.0 foot into intact bedrock. Foundation excavations should be cleaned of all loosened overbreak materials and any deeper voids filled with lean 3-sack slurry mix or extra structural concrete. Lateral capacity calculations may utilize an assumed friction coefficient of 0.38 for concrete poured neat atop these materials. A passive pressure of 250 psf per foot of depth is recommended. Bearing values may be increased by one-third when accounting for short-duration seismic loads.

Walls founded on engineered fill: A maximum allowable bearing value of 2,000 pounds per square foot (psf) may be assumed for wall foundations embedded a minimum of 1.5 feet into compacted engineered fill. Lateral capacity calculations should assume a friction coefficient of 0.38 for concrete poured neat atop these materials. Passive pressure resistance may be assumed at 250 psf per foot of depth. These values shall be applicable to alignment-derived soils or select granular import materials with the same or better soil properties. Bearing values may be increased by one-third when accounting for short-duration seismic loads.

<u>*Walls founded on "original" McCall Boulevard fill:*</u> Not recommended. Alternatives include (1) Overexcavate undocumented fill in the wall bearing zone to competent soil or bedrock and restore the excavation to foundation grade as engineered fill; or (2) Move the wall to a location with minimal or no old fill, and perform ordinary remedial grading "removals" to create suitable bearing; or (3) Design a pile and grade beam support system for the wall that does not rely on the old fill for load capacity; or (4) Substitute a steepened MSE slope and delete the wall entirely.

We are proponents of "ductile" reinforced earth systems that allow small soil movements without compromising the integrity of the overall retention system. Foundations are typically only compacted crushed-rock pads. Remedial grading for global stability needs will still be required. Examples would be:

- Rock-filled gabion walls, either vertical, battered, or stepped.
- Wire-reinforced walls.
- Wire-reinforced slopes. Manufacturers can supply wire mats pre-bent to 1:1 face inclinations. Wire-reinforced slopes can receive plant sets or be hydroseeded to blend with local terrain.
- Gravity or reinforced-earth precast modular "big block" walls (e.g., Redi Rock™ and others). Offered in many sizes, colors, and textures. Can accommodate curves (walls don't have to be straight lines). Fast to build.

Project-specific recommended active earth pressure equivalents are shown below. Applicable traffic surcharges should be added to the listed earth loads.

Table 6.9-1
Retaining Wall Active Pressures

Surface slope of retained material	**Equivalent Fluid Weight (lb./ft.^{3})**	
	Braced Walls retaining either native alluvium or compacted fill $\varphi \geq 30°$	Unbraced Walls retaining either native alluvium or compacted fill $\varphi \geq 30°$
Level	64	43
2:1	78	70
1.5:1	106	98

Designers may ignore surcharge loads from bedrock slopes above toe walls. Level backfill behind these walls should be supplied with a concrete V-gutter for drainage and rock catchment. AGI recommends either additional screen wall height over the nominal retained-earth height, or an alternate barrier to stop rocks from entering traffic lanes. Perforated metal mesh or heavy-gauge woven mesh might be options atop a solid wall. The effective opening size should be under one inch, in our opinion. The screen height may proportionally vary with the height of the cut slope, but preliminarily need not exceed 4 feet from the tops of walls. Quantitative rockfall modeling to set the design screen height is recommended once the final wall alignment and gap to the slope face are known.

Wall pressures from seismic inertial loads must also be included for tall walls. Seismic loads may be based on a site-modified peak ground acceleration PGA_M of 0.660g and MCE event magnitude M_w8.1. Other expected site conditions such as drained, granular backfill soils appear to be consistent with the assumptions of the widely used Mononobe-Okabe method or similar later variations of rigid plastic methods for finding force magnitudes on the wall. Standard reduction factors for PGA (e.g., 0.5 for M-O method) may thus be implemented.

6.10 CIDH Foundation Excavations

Although information regarding anticipated foundation loads was not available for this report, lighting or signal poles normally have fairly low axial loads and overturning moments. Foundation plans, once they become available, should be evaluated by AGI for compatibility with the preliminary recommendations presented below. It is anticipated that a standard reference design (Caltrans, FHWA, or AASHTO) will be selected for shaft foundation dimensions and reinforcement. Sheet ES-7F and ES-7N from the 2018 Caltrans Standard Plans were referenced for parameters needed to perform a pile footing analysis.

Cast-In-Drilled-Hole (CIDH) pile footings constructed entirely within undisturbed native material appear feasible for relocated signalized intersection footings at McCall Boulevard and Menifee Road. Axial, lateral and moment loads of 10 kips were assumed for preliminary CIDH analyses. A referenced pile diameter of 3.5 feet and pile length of 13.0 feet were obtained from sheet ES-7F.

Geotechnical parameters for drilled shaft foundations were estimated for use in site-specific computer-aided design (e.g., L-PILE) based on soil boring B-7 data. Lateral and axial capacity of soils within the upper 2 feet should be ignored due to soil disturbance and interfering utilities. Coefficients of subgrade reaction are ultimate values, and appropriate factors of safety should be applied during design checks. Development of vertical load capacity by skin friction only is preferred, unless a clean-out bucket specification is explicitly written into construction plan notes that accompany end-bearing shaft designs. Pile analysis parameters are presented below on the following page and pile analysis results are presented in Appendix C. If design loading conditions for the signal lights differ from the assumptions used in this report, AGI should be notified and provide an updated pile analysis report.

Pile Analysis Parameters

Soil Layer	Depth Interval (ft.)	Effective Unit Weight (pcf)	Soil Type (L-PILE)	Friction Angle (degrees)	Coefficient of Static Subgrade Reaction K_s (pci)
Stratum 1	2 - 5	130.0	Sand/Gravel	30.0	300.0
Stratum 2	5 - 20	130.0	Sand/Gravel	33.0	500.0

6.11 <u>2019 California Building Code Seismic Criteria</u>

Prescriptive mitigation for the hazard of strong ground motion affecting structures is nominally provided by structural design adherence to local adopted building codes. The 2019 CBC, based on the 2018 *International Building Code*, maintains a “look-up” code convention for seismic engineering, using as primary inputs the site’s location and the assigned site class. The latter is a measure of shallow-earth elastic resistance determined by borehole tests, depth to bedrock, and/or geophysical methods. The in-force 2019 code quantifies seismic risk based on the newer probabilistic 2014 National Seismic Hazard model. Design coefficients are ultimately functions of distance to active faults, fault activity, and measured or correlated mean shear wave velocity within 30 meters (~100 feet) of the ground surface. If required for McCall Boulevard improvements, the tabulated recommended criteria presented below were derived in accordance with the rules of Section 1613 of the 2019 CBC and ASCE/SEI Standard 7-16, and the closest project-to-source distance.

Table 6.11-1
2019 CBC Seismic Design Factors and Coefficients
(Lat. 33.721590°N, Long. 117.154228°W)

2019 CBC Section #	Seismic Parameter	Indicated Value or Classification
1613.2.1	Mapped Acceleration MCE_R S_s	1.408g (Note 1)
	Mapped Acceleration MCE_R S_1	0.524g (Note 1)
1613.2.2	Site Class	C (Note 2)
1613.2.3	Site Coefficient F_a	1.2
	Site Coefficient F_v	1.689
1613.2.3	Adjusted MCE_R Spectral Response S_{MS}	0.773g
	Adjusted MCE_R Spectral Response S_{M1}	1.006g
1613.2.4	Design Spectral Response S_{DS}	1.126g (Note 3)
	Design Spectral Response S_{D1}	0.516g (Note 3)

Notes

(1) Interpolated from 0.01-degree gridded data in the probabilistic 2014 National Seismic Hazard Model (SEAOC, 2022), 2% in 50-year exceedance probability.

(2) Based on proposed/predicted fill <30 feet deep at any alignment site, borehole SPT data, depths to bedrock, and estimated minimum V_{s30} ≈760 m/sec.

(3) Defined by 2019 CBC §1613.1 and ASCE/SEI 7-16 §11.4.5. A *site-specific* MCE_R response spectral acceleration at any period shall be taken as the lesser of the probabilistic or deterministic spectral response accelerations, with the latter subject to lower-limit values. The design spectral response accelerations are calculated as ⅔ of the MCE_R value.

Based on ASCE 7-16 and CBC §1613.2.5, a Seismic Design Category of **D** for risk category I-III buildings/structures is assigned for localities where $S_{D1} > 0.20g$ and $S_1 < 0.75g$. The option for alternative seismic design category determination based on a structure's fundamental period and CBC Table 1613.2.5(1) alone is allowed. The site-modified zero-period MCE_G ground motion estimate PGA_M is 0.600g. Seismic response coefficients determined by the SEAOC seismic design tool applied to Figures 22-18A and 22-19A of ASCE 7-16 would be:

$C_{RS} = 0.933$ $C_{R1} = 0.916$

The current code edition will expire December 31, 2022. Entitlement or civil design delays that push project approvals past this date may result in the project being subject to the next code iteration. Seismic design values are expected to change. Geotechnical and geologic reviews of prescriptive design coefficients for any street improvements subject to the code are recommended if delays occur.

6.12 Pavements

Representative R-value determinations were performed in accordance with Caltrans Test Method 301. Collected R-value test samples included existing modern "decomposed granite" fill and natural silty sand with traces of clay, the latter typical of the McCall alignment east of Junipero Road. The obtained R-values were 63 and 58, respectively.

The City of Menifee's minimum-required structural section for a 6-lane urban arterial street (traffic index = 10.0) is 6 inches of hot mix asphalt over 10 inches of aggregate base per Standard Plan No. 96. This standard section was compared to the measured as-built street sections found in soil boring and auxiliary pavement core sites, as well as the laboratory-determined R-value data. It is anticipated that all widened-street embankment fills and any trench backfills that will support asphalt pavements will first be compacted as recommended in Sections 6.5 and 6.7.

When correlated to logged soil classifications and other attributes, we assume that all future as-built subgrades will meet or exceed an R-value of 50. The City standard is confirmed as suitable for an R-value of 42 or greater and is preliminarily recommended for the McCall Boulevard project. However, additional R-value tests should be performed on as-built *new* embankment fill subgrades, as they are predicted to require substantial volumes of import material. Generally speaking, any import soil with an R-value exceeding 42 should be acceptable and will not need pavement section modifications. Qualification of import soil by R-value alone is not advised, however, inasmuch as close-by but lower-quality import may still be more economical than better materials that have to be trucked a long distance.

AGI recommends that all pre-2006 pavements associated with the original 2-lane highway be removed and replaced. Patch pavements in the old alignment are also substandard.

Per the table on page 8, newer pavements have as-built structural sections that approach but do not quite meet the City standard. These sections are at roughly the 80 percent threshold of an (assumed) design life of 20 years. A mill-and-fill option could be contemplated to remove painted traffic markings, provide a uniform final surface appearance, correct block cracking, add more years to the expected serviceability lifetime, and (in the case of the Oakhurst Avenue vicinity) conceal particles causing “rusty” stains that are aesthetically undesirable and can bleed through painted lines. The stains result from decomposition of a component of asphalt aggregate used by a single hot mix supplier, according to AGI Inland Empire experience. The phenomenon is slowed but not halted by slurry seal coatings.

The uppermost 12 inches of soil materials below pavement subgrades shall be processed and compacted to a minimum of 95 percent of the laboratory maximum dry density determined by ASTM D1557-12, except where more-stringent trench backfill specifications shall take precedence. Caltrans Class 2 aggregate base material should be compacted to a minimum of 95 percent of maximum dry density. Street-edge concrete curbs, gutters, and ribbon gutters at flowlines crossing intersecting rural streets are recommended to promote pavement longevity.

Owners, designers, and general contractors should be aware that Class 2 base material may be composed of virgin natural stone (“crushed aggregate base” or CAB), or *reclaimed materials* such as crushed concrete and pulverized asphalt (crushed miscellaneous base, CMB). Reclaimed base has been the source of unsatisfactory pavement performance at multiple Southern California projects due to unintended contamination with reactive aluminum metal fragments. Surface distress manifests as permanent raised pavement “bumps” or “pimples”. It is not clear at this time that the problem is limited to only certain suppliers, or whether local suppliers can provide warranties for delivered product. We recommend a warranty. The most conservative option is to specify only "CAB" for flexible pavement base courses, in our opinion.

6.13 Soil Corrosivity

Chemical analyses were performed to provide a general evaluation of the corrosivity of native soils along the project alignment. Tests included soluble sulfate and chloride ion contents, pH, and saturated minimum resistivity. Findings indicate the project soils should not be highly aggressive to concrete. Reported soluble sulfate and chloride contents for native alluvium were 0.0016 weight percent and 7.8 parts per million respectively, with a mildly acidic pH value of 6.0 and a saturated minimum resistivity value of 8,710 ohm-cm. Local fills derived from bedrock cut would ordinarily be more benign than the very old alluvium. The resistivity value correspond to conditions of "moderate" corrosion potential for unprotected mild steel. Standard Specifications for Public Works Construction "Greenbook" guidelines or a qualified corrosion engineer should be consulted for mitigation of corrosion effects on buried ferrous objects, and for any special corrosion protection design details that may be required.

The measured insignificant sulfate concentration indicates that normal Type I-II cement should be suitable for concrete mix designs utilized for this project, based on American Concrete Institute (ACI) 318 Table 4.3.1. AGI recommends that all concrete which will contact on-site soil materials be selected, batched, and placed in accordance with the latest and ACI technical recommendations.

6.14 Construction Observations

The preliminary geotechnical recommendations presented in this report are based on the assumption that street improvements will be placed on competent engineered materials approved by this office. It is recommended that mass grading, trench backfill, and street restoration operations be performed with observation and quality control testing by AGI personnel. Engineered fill shall constitute any load-bearing soil placements, irrespective of yardage quantity or depth. Continuous observation is a 2019 CBC requirement for engineered fill. Continuous or periodic fill observation and testing may be suitable for trench backfills depending mostly on trench depth and contractor production. Wall footing and CIDH shaft foundation excavations should be observed prior to placing reinforcing steel to verify that foundations are embedded within satisfactory materials and that excavations are free of loose or disturbed soils and made to the recommended depths. AGI services would exclude direction of the contractor's means and methods, field supervision of the contractor's workmen, or assumption of any responsibility for jobsite safety.

6.15 Investigation Limitations

The findings in this preliminary report may require modification as a result of later plan reviews, field explorations, or observations made during construction. Conditions could vary significantly from those assumed or analyzed by this report. Revised geotechnical recommendations may be needed from AGI. We invite iterative plan reviews to hone the final project drawings, particularly with respect to walls. Future reviews must at a minimum quantitatively confirm global stability for all new slopes steeper than 2:1 or higher than 30 feet, and MSE walls/slopes. Rockfall protection screens should have final heights determined by rockfall trajectories estimated by dynamic rebound modeling.

Our conclusions have been based on the results of the field exploration combined with interpolations of soil conditions between a limited number of subsurface explorations. The nature and extent of variations beyond or between the explorations may not become evident until construction. This report and any subsequent plan review reports should be referenced on construction drawings as a part of the overall project specification. Lastly, a pre-construction meeting with project managers, the excavation contractor, and City Engineer is strongly encouraged to present, explain, and clarify geotechnical concerns, uncertainties, and recommendations for the improvements.

7.0 CLOSURE

This report was prepared for the use of KOA Corporation as agent for the City of Menifee in cooperation with this office. All professional services provided in connection with the preceding report were conducted in accordance with generally accepted professional engineering principles and local practice in the fields of soil mechanics, foundation engineering, engineering geology, as well as the general requirements of the City of Menifee in effect at the time of report issuance. We make no other warranty, either expressed or implied.

We appreciate the opportunity to team with KOA Corporation and help expedite the design and eventual construction of the McCall Boulevard improvements. If you have any questions, please contact the undersigned at our Riverside office at (951) 776-0345.

Respectfully submitted,
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Attachments: Appendices A and B

Distribution: (6) Addressee

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AERIAL PHOTOGRAPHS

U.C. Santa Barbara Aerial Image Collections

Date Flown	Flight Number	Scale	Frame Numbers
6-21-38	AXM-1938	1:20,000	Line 54, #61
8-28-53	AXM-1953b	1:20,000	Line 2K, #71
8-18-61	AXM-1961	1:20,000	Line 7BB, #38
5-15-67	AXM-1967	1:20,000	Line 2HH #188
10-27-76	AMI-RIV-76A	1:36,000	#8321

APPENDIX A

A P P E N D I X A

GEOTECHNICAL MAP & SUBSURFACE EXPLORATION LOGS

A modified alignment plan sheet supplied by KOA Corporation (Plate No. 1 in the pocket at the back of this report) was prepared based upon information supplied by the client, or others, along with Aragón Geotechnical's field measurements and observations. Field exploration locations illustrated on the drawing were derived from scaled and taped distances to existing improvements, and should be considered approximate.

The Field Boring Logs on the following pages schematically depict and describe the subsurface (soil and groundwater) conditions encountered at the specific exploration locations on the dates that the explorations were performed. Unit descriptions reflect predominant soil types; actual variability may be much greater. Unit boundaries may be approximate or gradational. Subsurface logs incorporate the field investigator's interpretations of soil conditions between recovered samples, and deductions of material origin and diagenesis. Therefore, the logs contain both factual and interpretive information. Subsurface conditions may differ between exploration locations along the alignment. The subsurface conditions may also change at the exploration locations over the passage of time.

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) standard D420 entitled "Standard Guide for Sampling Soil and Rock" and/or other relevant specifications. Soil samples were preserved and transported to AGI's Riverside laboratory in general accordance with the procedures recommended by ASTM standard D4220 entitled "Standard Practice for Preserving and Transporting Soil Samples". Brief descriptions of the sampling and testing procedures are presented below.

Ring-Lined Barrel Sampling – ASTM D3550-01
In this procedure, a thick-walled barrel sampler constructed to receive a stack of 1-inch-high brass rings is used to collect "undisturbed" soil samples for classification and laboratory testing. Samples were collected at incremental depths variously spaced 2 feet to 5 feet apart. The drilling rig was equipped with a 140-pound mechanically actuated automatic driving hammer operated to fall 30 inches, acting on rods. An 18-inch-long sample barrel fitted with 2.50-inch-diameter rings and a waste barrel extension was subsequently driven a distance of 18 inches or to practical refusal (considered to be ≥50 blows for 6 inches). The raw blow counts for each 6-inch increment of penetration (or fraction thereof) were recorded and are shown on the Field Boring Logs. An asterisk (*) marks refusal within the initial 6-inch seating interval. The hammer weight of 140 pounds and fall of 30 inches allow rough correlations to be made (via conversion factors that

normally range from 0.60 to 0.65 in Southern California practice) to uncorrected Standard Penetration Test N-values, and thus approximate descriptions of consistency or relative density could be derived. Hammer efficiency for most automatically actuated systems is approximately 80 percent. The method provides relatively undisturbed samples that fit directly into laboratory test instruments without additional handling and disturbance.

Standard Penetration Tests – ASTM D1586-11

In rocky fill encountered at the Boring B-1 site, Standard Penetration Tests were performed to recover disturbed samples suitable for classification, and to provide baseline data for settlement susceptibility analysis. A split-barrel sampler with a 2.0-inch outside diameter is driven by successive blows of a 140-pound hammer with a vertical fall of 30 inches, for a distance of 18 inches at the desired depth. The drill rig used for this investigation was equipped with an automatic trip hammer acting on drilling rods. The total number of blows required to drive the sampler the last 12 inches of the 18-inch sample interval is defined as the Standard Penetration Resistance, or "N-value". Penetration resistance counts for each 6-inch interval and the raw, uncorrected N-value for each test are shown on the Field Boring Logs. Drive efficiencies for automatic hammers are higher than older rope-and-cathead systems, which are disappearing from practice. Where practical refusal was encountered within a 6-inch interval, defined as penetration resistance ≥50 blows per 6 inches, the raw blow count was recorded for the noted fractional interval; an asterisk (*) marks refusal within the initial 6-inch seating interval. The N-value represents an index of the relative density for granular soils or comparative consistency for cohesive soils.

Bulk Sample

A relatively large volume of soil is collected with a shovel or trowel. The sample is transported to the materials laboratory in a sealed plastic bag or bucket.

Classification of Samples

Excavated soils and discrete soil samples were visually-manually classified, based on texture and plasticity, utilizing the procedures outlined in the ASTM D2487-93 standard. The assignment of a group name to each of the collected samples was performed according to the Unified Soil Classification System (ASTM D2488-93). The plasticity reported on field logs refers to soil behavior at field water contents unless noted otherwise. Classifications based on laboratory tests superceded and replaced visual-manual classifications if a discrepancy was found. The classifications are reported on the Field Boring Logs.

FIELD LOG OF BORING B - 1

Project: ***McCALL BOULEVARD WIDENING PROJECT***

Location: ***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**26.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1573 Ft. AMSL per street plan**

Comments: W/B McCall paved shoulder, 15' southwest of downslope drain gutter. Deep fill site.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	OTHER TESTS
0				Asphaltic Concrete Pavement: 5¾" thick, no ABM.			BULK: MAX, SIEVE, SE
		Bulk	GM	Cobbles & Boulders: Variegated light brown and yellowish gray; medium dense; slightly moist. Composed of more than 50% angular hard rock fragments, most larger than 6 inches and sized up to 18"+, in stony silty sand matrix. Derived from schist cut farther east. [Fill]			
	1570						
5		SPT 8 8 5 N=13	GM	← Relatively poorly packed mix of rock fragments and yellowish gray silty sand matrix. Sample driving hints at fragment displacement.			
	1565						
			SM	Gravelly Sand: Yellowish brown; medium dense to dense; moist; generally well-graded mixes of sand and fine to coarse gravel-size rock fragments. Coarse particles commonly highly weathered and retained in sampler. Occasional brief layers of granitic-derived sand (D.G.). Drills easily. [Fill]			
10		SPT 22 14 11 N=25	SM				
	1560						
				← Hard rock fragment.			
15							

Continued on next sheet.

FIELD LOG OF BORING B - 1

Project: ***McCALL BOULEVARD WIDENING PROJECT***

Location: ***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK / DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
15		Drive	SPT 11 13 17 N=30		SM	Gravelly Sand: Yellowish brown; dense; moist. Sample at 15' features weathered schist fragments to ~4" in silty sand matrix. Stony at base, with abrupt contact. [Fill]				
	1555				SM	Gravelly Sand with Silt: Mottled yellowish brown, oraNge, and gray; medium dense; moist; fine to coarse grained sand with highly weathered and stained schist fragments to 3" across; estimated 25% fines with trace of clay. [Very old fan alluvium]				
20		Drive	SPT 9 10 11 N=21		SM	← Gravelly sand, as above, with heavy FeO mottles. Not visibly porous.				
	1550									
					CL	Sandy Clay: Brown; stiff; moist; plastic with added water; around 30% sand; not visibly porous. [Residual soil]				
25		Drive	SPT 5 50/6"		ROCK	Schist: Strong brown; fine-grained; foliated; highly weathered and partly friable. Pervasively stained. [Metamorphic bedrock]				

Bottom of boring at 26.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings and rapid-set grout.

FIELD LOG OF BORING B - 2

Sheet 1 of 1

Project:	*McCALL BOULEVARD WIDENING PROJECT*
Location:	*CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.*

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**1.5 Ft. max achieved.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1587.3 Ft. AMSL per street plan**

Comments: W/B McCall unpaved shoulder, between 4" PE gas line and telephone utility. Deep as-built cut in schist.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS BULK	SAMPLE INTERVALS DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0						GM	Gravel and Cobbles: Disturbed rocky fill.				
						ROCK	Schist: Dark grayish brown; fine-grained; foliated; slightly weathered, hard, and generally strong. Does not readily cut with auger. Adjacent slope features thinly layered and almost rhythmic schist-quarzite banding and planar joints spaced ~24-30" with some blasting overbreak.[Metamorphic bedrock]				

Refusal encountered at 1.5 feet (multiple trials).
No groundwater encountered.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 3

Sheet 1 of 1

Project:	*McCALL BOULEVARD WIDENING PROJECT*
Location:	*CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.*

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**2.5 Ft. max achieved.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1585.5 Ft. AMSL per street plan**

Comments: W/B McCall unpaved shoulder, between 4" PE gas line and telephone utility. As-built cut in granodiorite.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS BULK	SAMPLE INTERVALS DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	1585					GM	Sandy Gravel: Disturbed rocky fill; dry; stemming plug and blasting wire noted in loose cuttings.				
						ROCK	Granodiorite: Speckled white and dark brown; medium-grained; equigranular; slightly weathered; hard; strong to very strong. Does not cut with auger. Adjacent slope features nearly unweathered masses up to >8' across, with some blasting overbreak. [Granitic bedrock]				

Refusal encountered at 2.5 feet (multiple trials).
No groundwater encountered.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 4

Sheet 1 of 1

Project:	***McCALL BOULEVARD WIDENING PROJECT***
Location:	***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**16.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1530.5 Ft. AMSL per street plan**

Comments: N. end Fargo Street at McCall, paved through lane. Embankment fill site.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	OTHER TESTS
0	1530		SM	Asphaltic Concrete Pavement: 1¾" thick, suspect 1½" ABM.			
		RING 6 19 25 (44)		Silty Sand: Brown and light yellowish brown; medium dense; dry to slightly moist; fine to coarse grained with typical trace proportions of fine rock fragments. Apparently massive and relatively uniform. Derived from weathered-granite cut farther west. [Fill]	127.2	3.4	BULK: MAX, SHEAR, SIEVE
5	1525	RING 20 23 19 (42)	SM	← Silty sand, as above, massive, not visibly porous.	125.7	3.5	
				← Hard rock fragment.			
		RING 4 6 7 (13)	SM	← Silty sand, becomes loose, slightly moist, not visibly porous and lacks gravel.	110.6	3.3	
10	1520	RING 4 4 7 (11)	SM	← Silty sand, as at 8 feet.	103.1	3.8	
		RING 8 11 12 (23)	SM	← Silty sand, becomes medium dense but continues similar to above.	107.7	5.7	
		RING 18 50/6"	SM	← Silty sand. Clean contact.	117.9	2.3	
15	1515		ROCK	Granodiorite: Speckled light brown and black; medium grained; equigranular; highly weathered; very weak; crumbly and friable; appears massive. [Granitic bedrock]			

Abrupt refusal encountered at 16.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings and rapid-set grout.

FIELD LOG OF BORING B - 5

Sheet 1 of 1

Project:	***McCALL BOULEVARD WIDENING PROJECT***
Location:	***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**13.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1517.0 Ft. AMSL per street plan**

Comments: W/B McCall paved shoulder, opposite intersection with Durant Street. Embankment fill site.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0							Asphaltic Concrete Pavement: 5½" thick, later-generation pavement.				
							Aggregate Base Material: 5½" thick, subjectively meets Caltrans Class 2 sand-gravel mix (non-crushed).				
	1515			RING 8 14 22 (36)		SM		132.9	2.5		BULK: MAX, R-VALUE
							Silty Sand: Grayish brown; medium dense; moist; fine to coarse grained with typical trace proportions of fine rock fragments. Apparently massive and relatively uniform; easily drilled. Derived from weathered-granite source (school site). [Fill - Engineered grading]				
5				RING 13 13 17 (30)		SM	← Silty sand, brown, siltier than above (~25% fines), not visibly porous.	121.5	7.1		
	1510										
						ROCK	Granodiorite: Speckled light brown and black; medium grained; equigranular; highly weathered; very weak; crumbly and friable; appears massive. Drills easily and smoothly. [Granitic bedrock]				
10				RING *50/6"				109.6	3.2		
	1505										

Abrupt refusal encountered at 13.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings and rapid-set grout.

FIELD LOG OF BORING B - 6

Project: ***McCALL BOULEVARD WIDENING PROJECT***

Location: ***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**6.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1477.5 Ft. AMSL per street plan**

Comments: W/B McCall No. 2 lane, opposite and east of intersection with Dales Street. Original road location.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0				Asphaltic Concrete Pavement: 5½" thick, original-generation pavement plus cap lift.				
		RING 7 8 8 (16)	SM	Aggregate Base Material: About 6" thick, not strictly ABM but select coarse D.G. soil fill.	119.4	9.7		
	1475	RING 6 13 42 (55)	SM	Silty Sand: Brown; loose to medium dense; moist; fine to medium grained; about 30% silty fines; uncemented. Genetically considered to be native "topsoil'. Not visibly porous in samples. [Very old fan alluvium]	113.6	9.8		
5		RING 27 50/6"		Gravelly Sand: Yellowish brown; very dense; moist; well-graded mix of fine to coarse sand and angular schist fragments up to ~2" across, latter mostly highly weathered. Cohesive and moderately cemented. [Very old fan alluvium]	111.8	16.7		

Bottom of boring at 6.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings and rapid-set grout.

FIELD LOG OF BORING B - 7

Project: ***McCALL BOULEVARD WIDENING PROJECT***

Location: ***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**21.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1467.0 Ft. AMSL per street plan**

Comments: NWC McCall at Menifee Road, near proposed signal relocation.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	OTHER TESTS
0			SM	Silty Sand: Yellowish brown; medium dense; slightly moist; fine to coarse grained with ~15-20% fine angular schist gravel; uncemented near surface. [Very old fan alluvium]			BULK: MAX, SE, R-VALUE, SULFATE, CHLORIDE, pH, RESISTIVITY
	1465	RING 13 13 26 (39)	SM/SC	← Silty sand with some clay, cohesive, not visibly porous.	122.3	5.6	
5		RING 29 50/5"	SM	← Silty sand with gravel, latter composed of weathered marble-sized schist fragments, cemented, cohesive.	116.3	9.2	SHEAR
	1460	RING 14 30 50/5"	SM	← Silty sand, less cemented than above and fewer gravel clasts, becomes moist, not visibly porous.	129.6	7.9	SHEAR
10		RING *50/6"	SM	← Silty sand, cemented.	110.6	8.0	
	1455						
15							

Continued on next sheet.

FIELD LOG OF BORING B - 7

Sheet 2 of 2

Project:	***McCALL BOULEVARD WIDENING PROJECT***
Location:	***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
15				RING 26 50/5"		SM	Silty Sand: Yellowish brown; very dense; moist; fine to coarse grained sand with traces of fine angular schist gravel; not visibly porous in recovered samples. [Very old fan alluvium]	118.3	6.3		
	1450										
20				RING 19 50/6"		SM/ML	← Very silty sand (50:50 sand and fines), heavy MnO films and veils, and few carbonate clots, massive.	111.1	17.2		

Bottom of boring at 21.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings.

FIELD LOG OF BORING B - 8

Sheet 1 of 1

Project:	***McCALL BOULEVARD WIDENING PROJECT***
Location:	***CITY OF MENIFEE, RIVERSIDE COUNTY, CALIF.***

Date(s) Drilled:	**6/7/22**	Logged By:	**M. Doerschlag**
Drilled By:	**GP Drilling**	Total Depth:	**5.0 Ft.**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic trip**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./30 In.**
Hole Diameter:	**8 In.**	Surface Elevation:	**± 1581.0 Ft. AMSL per street plan**

Comments: E/B McCall paved shoulder, later-generation pavement. As-built cut in granodiorite.

DEPTH (ft.)	ELEVATION (MSL DATUM)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	GRAPHIC LOG	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	WELL COMPLETION	OTHER TESTS
0							Asphaltic Concrete Pavement: 6¾" thick.				
	1580						Aggregate Base Material: 13½" thick, subjectively consistent with Caltrans Class 2 natural sand-gravel base (non-crushed).				
						ROCK	Granodiorite: Light brown; fine-grained; soil-like or saprolitic mass of highly to extremely weathered intrusive rock with clay alteration; soft; extremely weak; friable. Easily drilled. Adjacent slope has small but hard residual corestones; cut does not appear to have required blasting. [Granitic bedrock]				
5											

Bottom of boring at 5.0 feet.
No groundwater encountered.
Boring backfilled with compacted soil cuttings and rapid-set grout.

APPENDIX B

A P P E N D I X   B

LABORATORY TESTING

Water Content - Dry Density Determinations – ASTM D2216-10

The dry unit weight and field moisture content were determined for each of the recovered barrel samples. The moisture-density information provides a gross indication of soil consistency and can assist in delineating local variations. The information can also be used to correlate soils and define units between individual exploration locations on the project site, as well as with units found on other sites in the general area.

Measured dry densities ranged from approximately 103.1 to 132.9 pounds per cubic foot. Water contents in ring samples ranged from 2.3 to 17.2 percent of dry unit weight. Sample locations and the corresponding test results are illustrated on the Field Boring Logs.

Modified Effort Compaction Tests – ASTM D1557-12

Four representative bulk soil samples were selected: (1) Original stony road embankment fill west of the largest cut; (2) Older but post-1966 sandy fill at the intersection of Fargo Street and McCall Boulevard; (3) Modern engineered sandy DG fill dating from ~2005; and (4) Native very old alluvial fan sediments close to Menifee Road. Samples were tested to determine their maximum dry densities and optimum water contents per the Method B procedure in the ASTM standard. The method uses 25 blows of a 10-pound hammer falling 18 inches on each of 5 soil layers in a 1/30 cubic foot cylinder. Soil samples are prepared at varying moisture levels to create a curve illustrating achieved dry density as a function of water content. The test results are listed below and shown graphically on pages B-5 through B-8. Tabulated results include rock correction coefficients.

Modified Effort Compaction Test Results

Soil Description	Location	Maximum Dry Density (pcf)	Optimum Water Content (%)
Light brown Gravelly Sand (Original embankment fill)	B - 1 @ 0.5 - 3 ft.	142.0	5.0
Brown Silty Sand (Post-1966 rural road fill)	B - 4 @ 0.2 - 5 ft.	134.0	6.5
Grayish Brown Silty Sand (Engineered DG fill, W/B lanes)	B - 5 @ 0.9 - 5 ft.	137.0	6.0
Yellowish Brown Silty Sand (Very old native deposits, Qvof)	B - 7 @ 0 - 4 ft.	132.5	7.5

Shear Strength Tests – ASTM D3080-11
Direct shear tests were performed on "undisturbed" samples representative of in situ conditions, and remolded samples intended to represent potential fill or backfill materials. Test samples retained within standard one-inch-high brass rings were pressed with minimal disturbance into the shear box. The samples were saturated, consolidated, and tested in the drained condition using a direct shear machine of the strain control type. Samples were tested at increasing normal loads to determine the Mohr-Coulomb shear strength parameters illustrated on pages B-9 through B-11. Reported strength values are shown for both peak and ultimate strengths based on minimum displacements of 12.5 millimeters in the shear box.

Sand Equivalent Tests – ASTM D2419-09
Two samples were selected to represent "fill" from the western end of the job and native "alluvium" from the eastern end of the improvements. The samples were evaluated for relative measures of silt and clay content. The suitability of materials for use as select bedding material around wet or dry utilities is commonly based on meeting a minimum sand equivalent value. Soil samples were placed in graduated cylinders with a volume of flocculating solution. Agitation and irrigation through a siphon device force the soil fines into suspension. After a prescribed sedimentation period, the heights of flocculated fines and sand are determined. The following table summarizes results.

Sand Equivalent Test Results

Soil Description	Location	Sand Equivalent
Light brown Gravelly Sand (Original embankment fill)	B - 1 @ 0.5 - 3 ft.	19
Yellowish Brown Silty Sand (Very old native deposits, Qvof)	B - 7 @ 0 - 4 ft.	13

R-Value Determinations – ASTM D2844-13
The potential strength of pavement subgrade soils was evaluated by stabilometer and expansion pressure devices by the company LaBelle Marvin, Inc. of Santa Ana, California. Soil samples at varying water contents are kneaded and compacted into cylindrical molds for the latter test; the samples are subsequently pressed from the molds into the stabilometer, laterally stressed to a fixed limit under a 1000-pound normal load, and the stabilometer R-value calculated for an exudation pressure of 300 psi. LaBelle Marvin's test report is attached.

R-Value Test Results

Soil Description	Location	R-Value
Grayish Brown Silty Sand (Engineered DG fill, W/B lanes)	B - 5 @ 0.9 - 5 ft.	63
Yellowish Brown Silty Sand (Very old native deposits, Qvof)	B - 7 @ 0 - 4 ft.	58

Particle Size (Gradation) Analyses – ASTM D422-63

Limited quantitative determinations were made of the distribution of coarse-grained particle sizes and fines proportions in aliquots of bulk samples composed of existing original fill and native very old alluvium. Hollow-stem augers rapidly grind gravel-size and larger particles to sand, especially when clasts are weathered. We judge that achieved gradation results from the tested samples underrepresent gravel proportions in the field condition. Gradation analyses help verify preliminary field classifications of total fines content and materials suitability for certain engineering purposes. Mechanically actuated sieves were utilized for separating the various classes of coarse-grained (sand) particles. Percent passing and percent retained for the sieve analysis are illustrated on the accompanying charts on pages B-12 and B-13.

Soil Corrosivity

A soil sample representative of possibly very slightly saline native alluvium in contact with concrete or ferrous metals was tested in the laboratories of Project X Corrosion Engineers, Murrieta, California, to determine the tabulated data on the next page. The submitted soil sample was tested in general accordance with ASTM and Caltrans Standard Methods listed at the top of the table. Soluble-species quantitative determinations were based on 1:3 water-to-soil extracts.

ARAGÓN GEOTECHNICAL, INC.

16801 Van Buren Blvd.
Riverside, California 92504
(951) 776-0345

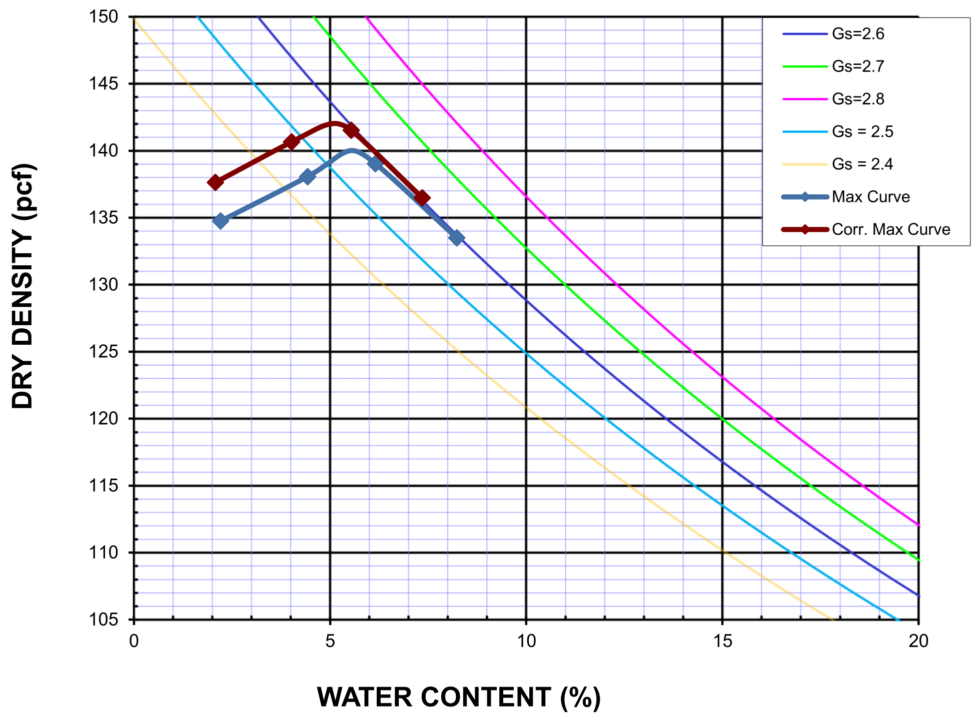
Maximum Density Test

Client:	KOA Cprporation 2141 W. Orangewood Orange, CA. 92868	**Project Name:**	McCall Blvd. Widening Project Menifee, California
Project No.:	4811-SFPV	**Report Date:**	August 4, 2022
Sampled By:	Mark Doerschlag	**Lab ID No.:**	22-1592
Date of Sampling:	June 7, 2022		

Information provided by Technician

X	Performed at Laboratory	X	Moist Preparation
	Performed at Jobsite		Dry Preparation

Tested By:	Cesar Lopez	**Date Tested:**	June 8, 2022
Sample Location:	B-1	**Source:**	-
Depth/Elev:	0.5 - 3 ft		
Sample Description:	Light brown gravilly sand (SM), minus cobble-boulder fragments [Fill]		



B	METHOD USED (A,B or C)
3/8-inch	SIEVE NUMBER
Mechanical	TYPE OF RAMMER
4.2%	AS REC'D MOISTURE
12.4%	PERCENT RETAINED
2.60	OVEN DRY (C127)
140.0	**MAXIMUM DENSITY [PCF]**
5.5	**OPTIMUM MOISTURE [%]**
142.0	**CORRECTED MAXIMUM DENSITY [PCF]**
5.0	**CORRECTED OPTIMUM MOISTURE [%]**

Remarks: No modifications made to test method, followed exact test procedure.

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AASHTO/ASTM/CTM Standards Used:	Unless noted, material was sampled in accordance with AASHTO T2/ASTM D75/CTM 125. Sample tested in accordance with ASTM D2216, D1557 & D4718.

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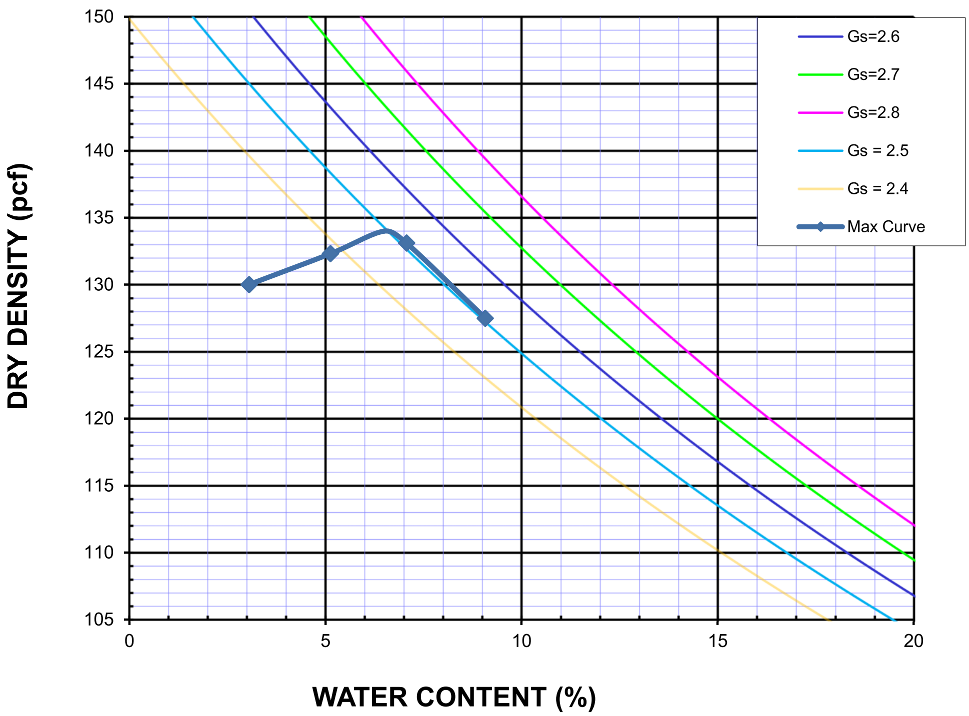
Maximum Density Test

Client:	KOA Cprporation 2141 W. Orangewood Orange, CA. 92868	**Project Name:**	McCall Blvd. Widening Project Menifee, California
Project No.:	4811-SFPV	**Report Date:**	August 4, 2022
Sampled By:	Mark Doerschlag	**Lab ID No.:**	22-1593
Date of Sampling:	June 7, 2022		

Information provided by Technician

X	Performed at Laboratory	X	Moist Preparation
	Performed at Jobsite		Dry Preparation

Tested By:	Cesar Lopez	**Date Tested:**	June 8, 2022
Sample Location:	B-4	**Source:**	-
Sample Description:	Brown silty sand (SM), fine to coarse grained. [Fill]	**Depth/Elev:**	0.2 - 5 ft



B	METHOD USED (A,B or C)
3/8-inch	SIEVE NUMBER
Mechanical	TYPE OF RAMMER
3.1%	AS REC'D MOISTURE
0.5%	PERCENT RETAINED
2.50	OVEN DRY (C127)
134.0	**MAXIMUM DENSITY [PCF]**
6.5	**OPTIMUM MOISTURE [%]**
-	**CORRECTED MAXIMUM DENSITY [PCF]**
-	**CORRECTED OPTIMUM MOISTURE [%]**

Remarks: No modifications made to test method, followed exact test procedure.

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AASHTO/ASTM/CTM Standards Used:	Unless noted, material was sampled in accordance with AASHTO T2/ASTM D75/CTM 125. Sample tested in accordance with ASTM D2216, D1557 & D4718.

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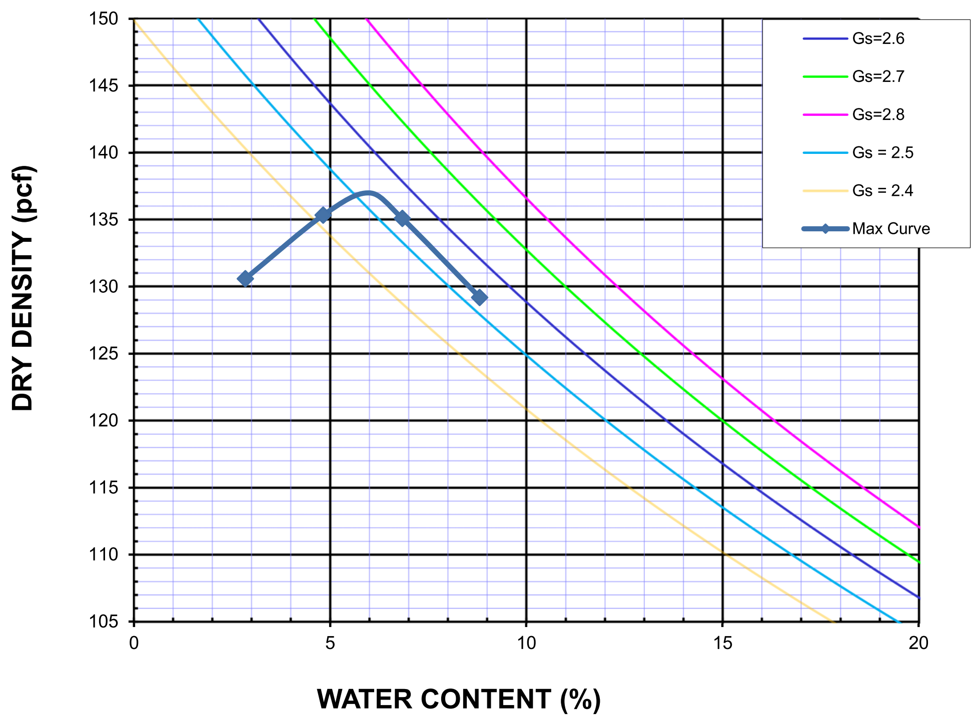
Maximum Density Test

Client:	KOA Cprporation 2141 W. Orangewood Orange, CA. 92868	**Project Name:**	McCall Blvd. Widening Project Menifee, California
Project No.:	4811-SFPV	**Report Date:**	August 4, 2022
Sampled By:	Mark Doerschlag	**Lab ID No.:**	22-1600
Date of Sampling:	June 7, 2022		
Information provided by Technician	X Performed at Laboratory Performed at Jobsite	X Moist Preparation Dry Preparation	
Tested By:	Cesar Lopez	**Date Tested:**	June 8, 2022
Sample Location:	B-5	**Source:** -	**Depth/Elev:** 1 - 5 ft
Sample Description:	Grayish brown silty sand (SM), fine to coarse grained. [Fill]		



B	METHOD USED (A,B or C)
3/8-inch	SIEVE NUMBER
Mechanical	TYPE OF RAMMER
3.1%	AS REC'D MOISTURE
1.9%	PERCENT RETAINED
2.55	OVEN DRY (C127)
137.0	**MAXIMUM DENSITY [PCF]**
6.0	**OPTIMUM MOISTURE [%]**
-	**CORRECTED MAXIMUM DENSITY [PCF]**
-	**CORRECTED OPTIMUM MOISTURE [%]**

Remarks: No modifications made to test method, followed exact test procedure.

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AASHTO/ASTM/CTM Standards Used:	Unless noted, material was sampled in accordance with AASHTO T2/ASTM D75/CTM 125. Sample tested in accordance with ASTM D2216, D1557 & D4718.

ARAGÓN GEOTECHNICAL, INC.

16801 Van Buren Blvd.
Riverside, California 92504
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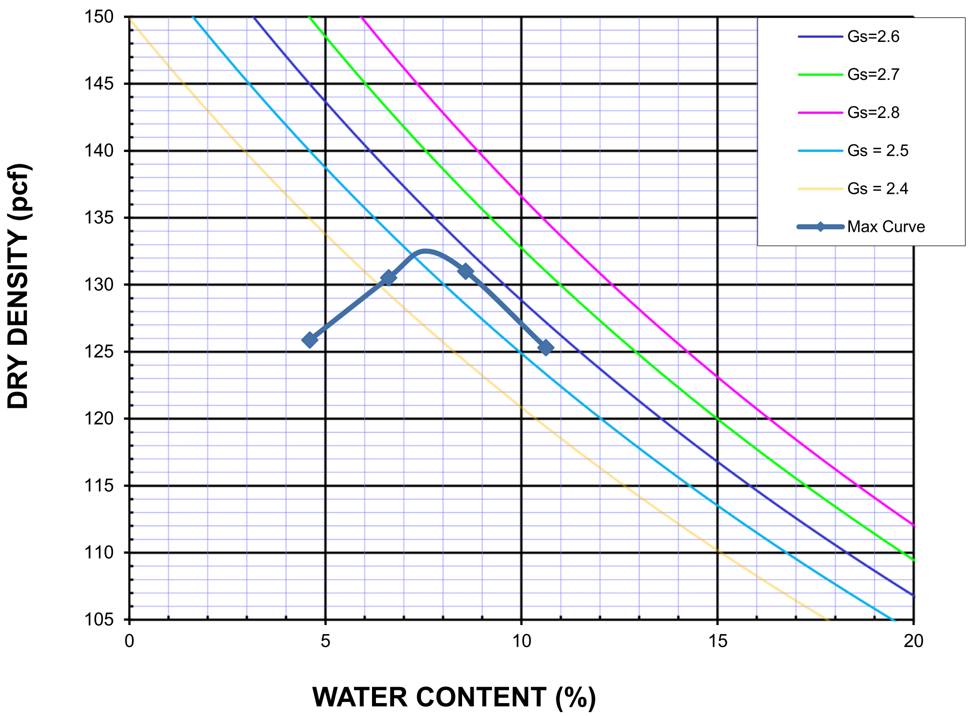
Maximum Density Test

Client:	KOA Cprporation 2141 W. Orangewood Orange, CA. 92868	**Project Name:**	McCall Blvd. Widening Project Menifee, California
Project No.:	4811-SFPV	**Report Date:**	August 4, 2022
Sampled By:	Mark Doerschlag	**Lab ID No.:**	22-1607
Date of Sampling:	June 7, 2022		
Information provided by Technician	X Performed at Laboratory Performed at Jobsite	X Moist Preparation Dry Preparation	
Tested By:	Cesar Lopez	**Date Tested:**	June 8, 2022
Sample Location:	B-7	**Source:** -	**Depth/Elev:** 0 - 4 ft
Sample Description:	Yellowish brown silty sand (SM), some gravel. [Very old alluvium]		



B	METHOD USED (A,B or C)
3/8-inch	SIEVE NUMBER
Mechanical	TYPE OF RAMMER
4.6%	AS REC'D MOISTURE
1.0%	PERCENT RETAINED
2.60	OVEN DRY (C127)
132.5	**MAXIMUM DENSITY [PCF]**
7.5	**OPTIMUM MOISTURE [%]**
-	**CORRECTED MAXIMUM DENSITY [PCF]**
-	**CORRECTED OPTIMUM MOISTURE [%]**

Remarks: No modifications made to test method, followed exact test procedure.

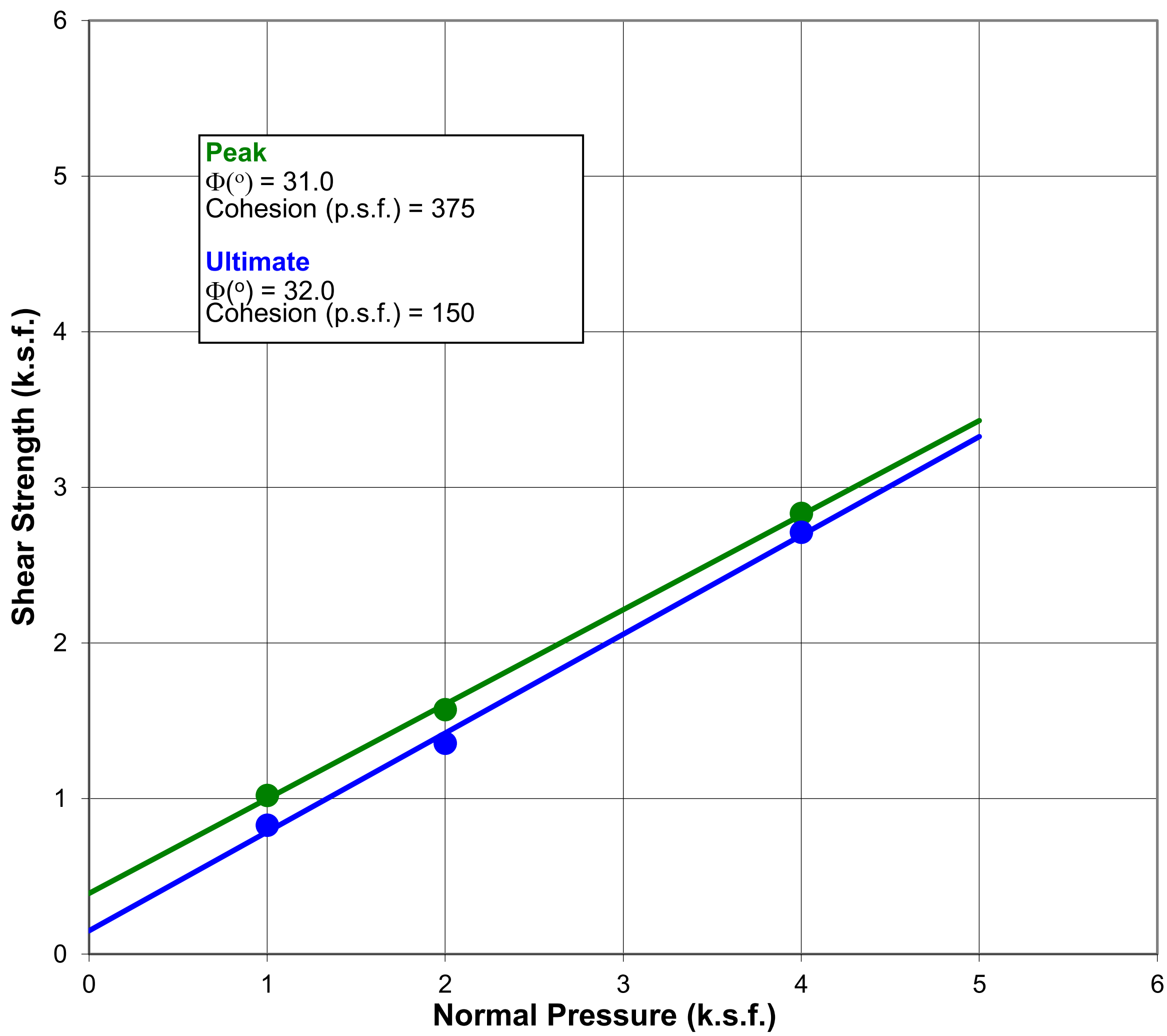
.

AASHTO/ASTM/CTM Standards Used:	Unless noted, material was sampled in accordance with AASHTO T2/ASTM D75/CTM 125. Sample tested in accordance with ASTM D2216, D1557 & D4718.

ARAGÓN GEOTECHNICAL, INC.
16801 Van Buren Blvd., Bldg. B
Riverside, California 92504
951-776-0345

Direct Shear Test Diagram

Project Name:	McCall Blvd. Widening Project		
Project Number:	4811-SFPV	Tested by:	Cesar Lopez
Sample Location:	B-4	Date Tested:	June 17, 2022
Sampled by:	Mark Doerschlag	Depth (ft):	0.2 - 5
Date Sampled:	June 7, 2022	Lab I.D. No.:	22-1593
Test Condition:	Remolded, Consolidated, Drained.		
Sample Description:	Brown silty sand (SM), fine to coarse grained. [Fill]		



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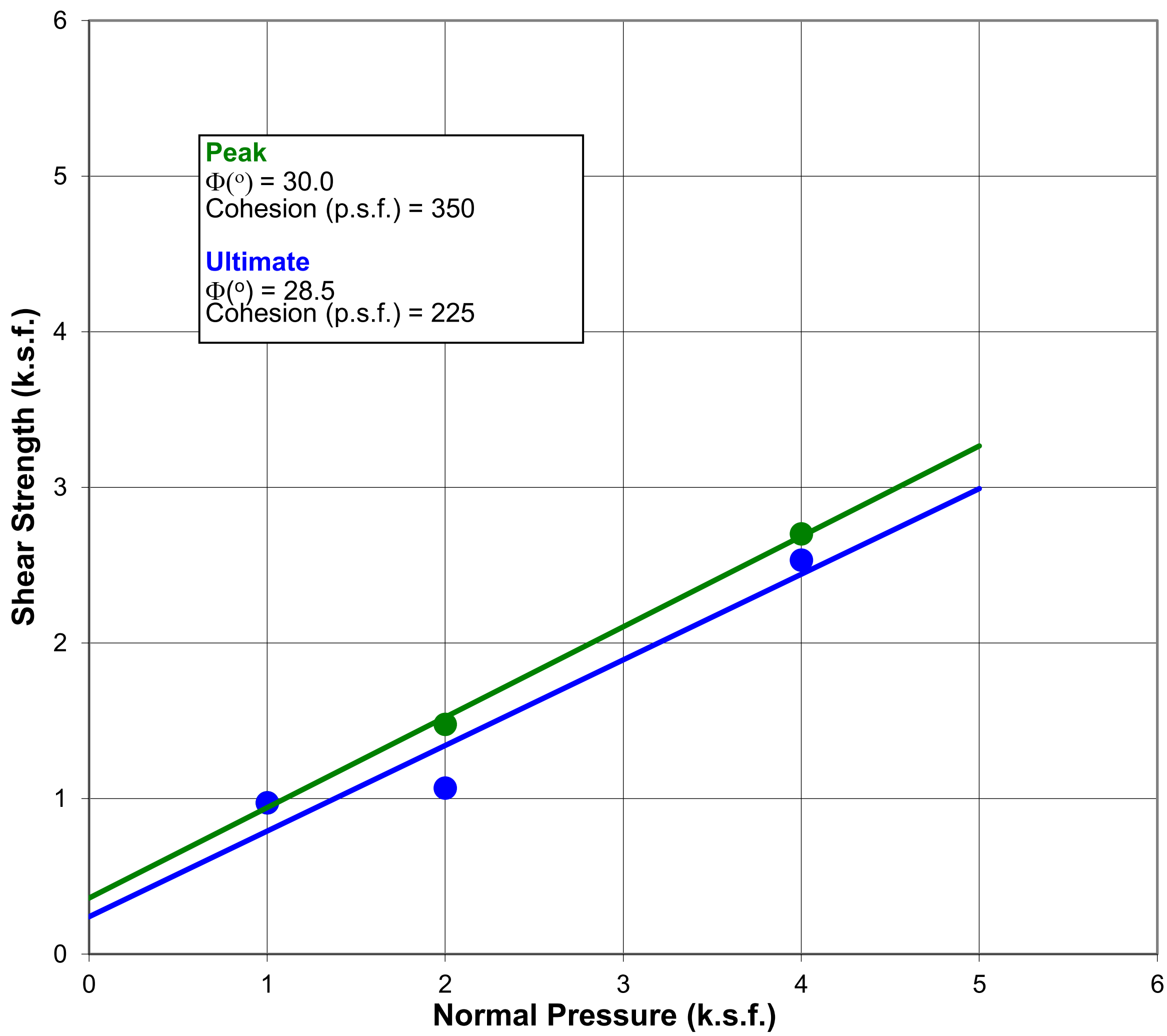
Riverside, California 92504
951-776-0345

Direct Shear Test Diagram

Project Name:	McCall Blvd. Widening Project		
Project Number:	4811-SFPV	Tested by:	Cesar Lopez
Sample Location:	B-7	Date Tested:	June 17, 2022
Sampled by:	Mark Doerschlag	Depth (ft):	4
Date Sampled:	June 7, 2022	Lab I.D. No.:	22-1609
Test Condition:	Undisturbed, Consolidated, Drained.		
Sample Description:	Silty sand with weathered gravel (SM), cohesive. [Very old alluvium]		



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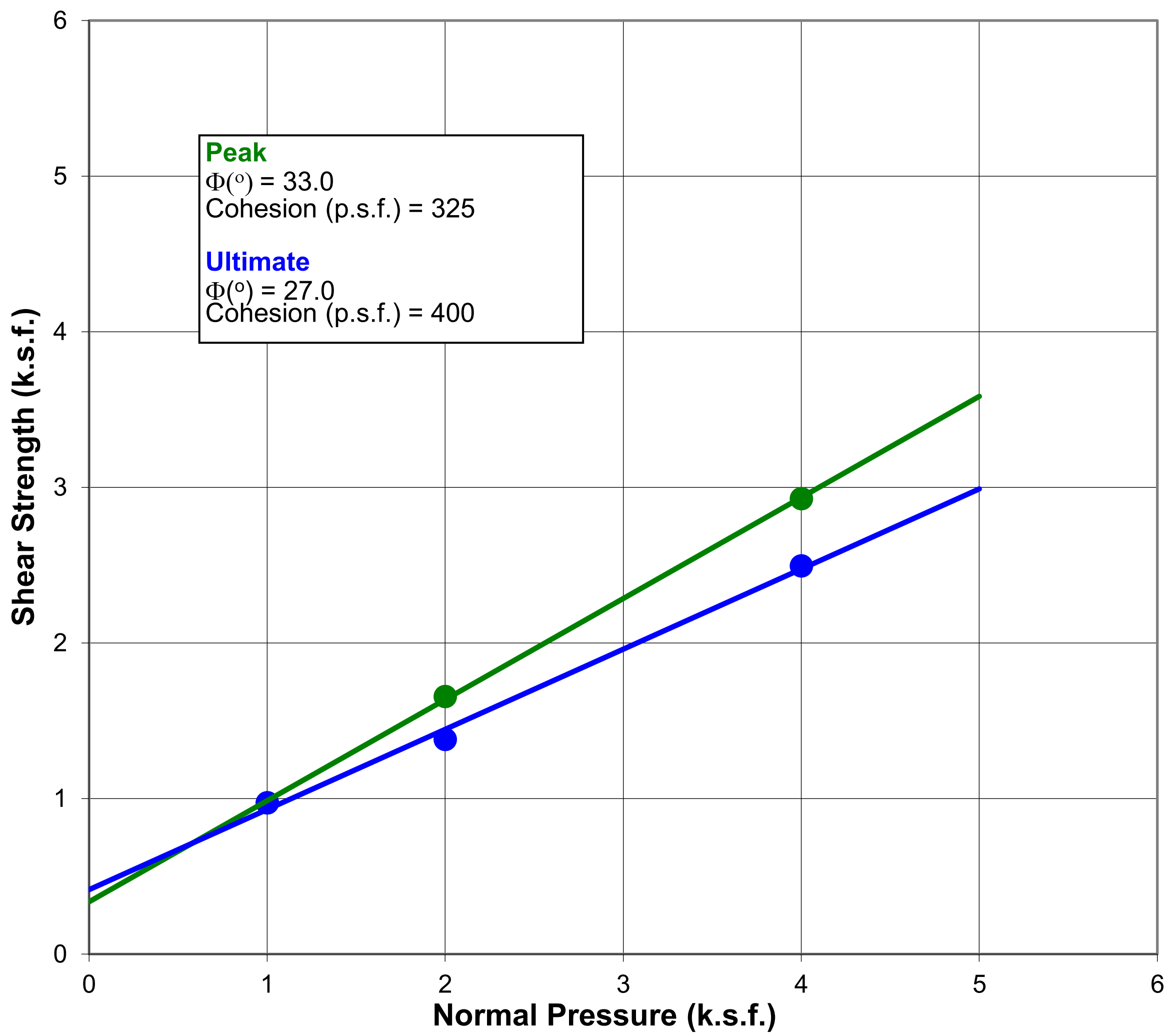
Riverside, California 92504
951-776-0345

Direct Shear Test Diagram

Project Name:	McCall Blvd. Widening Project		
Project Number:	4811-SFPV	Tested by:	Cesar Lopez
Sample Location:	B-7	Date Tested:	June 17, 2022
Sampled by:	Mark Doerschlag	Depth (ft):	6
Date Sampled:	June 7, 2022	Lab I.D. No.:	22-1610
Test Condition:	Undisturbed, Consolidated, Drained.		
Sample Description:	Silty sand (SM), trace fine gravel. [Very old alluvium]		



• ANALYSIS
• DESIGN

LaBelle • Marvin

• SOILS, ASPHALT TECHNOLOGY

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

June 14, 2022

Mr. Carlos Fernando Aragon, P.E.
Aragon Geotechnical, Inc.
16801 Van Buren Boulevard
Riverside, California 92504

Project No. 48345

Dear Mr. Aragon:
Laboratory testing of the bulk soil samples delivered to our laboratory on 6/10/2022 has been completed.

P.N.:	4811-SFPW
Project:	KOA
Samples:	Lab # 22-1600, B-5 @ 1'-5'
	Lab # 22-1607, B-7 @ 0'-4'

A tabulation of test results is transmitted herewith. Any untested portion of the sample will be retained for a period of 60 days prior to disposal. The opportunity to be of service is appreciated and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 48345

DATE: 6/14/2022

BORING NO. Lab # 22-1600, B-5 @ 1'-5'

KOA

P.N. 4811-SFPW

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	40	57	46
Initial Test Water, %	7.5	9.1	8.1
Compact Gage Pressure,psi	350	180	350
Exudation Pressure, psi	484	153	294
Height Sample, Inches	2.48	2.52	2.48
Gross Weight Mold, grams	3114	3124	2935
Tare Weight Mold, grams	1950	1946	1770
Sample Wet Weight, grams	1164	1178	1165
Expansion, Inches x 10exp-4	11	6	10
Stability 2,000 lbs (160psi)	16 / 27	29 / 55	20 / 38
Turns Displacement	4.54	4.98	4.85
R-Value Uncorrected	73	49	62
R-Value Corrected	73	49	62
Dry Density, pcf	132.3	129.8	131.7

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.28	0.52	0.39
G. E. by Expansion	0.37	0.20	0.33

Equilibrium R-Value	63 by EXUDATION	Examined & Checked: 6 /14/ 22
REMARKS:	Gf = 1.25 1.4% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 48345

DATE: 6 /14/ 2022

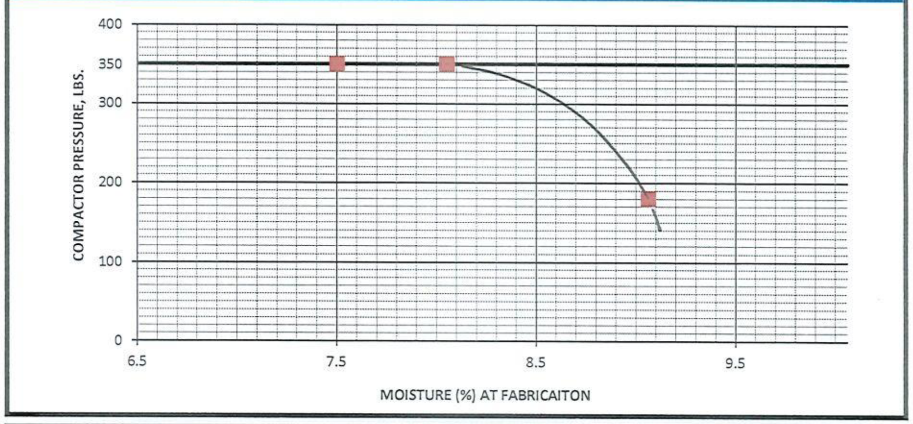
REMARKS:

BORING NO. Lab # 22-1600, B-5 @ 1'-5'

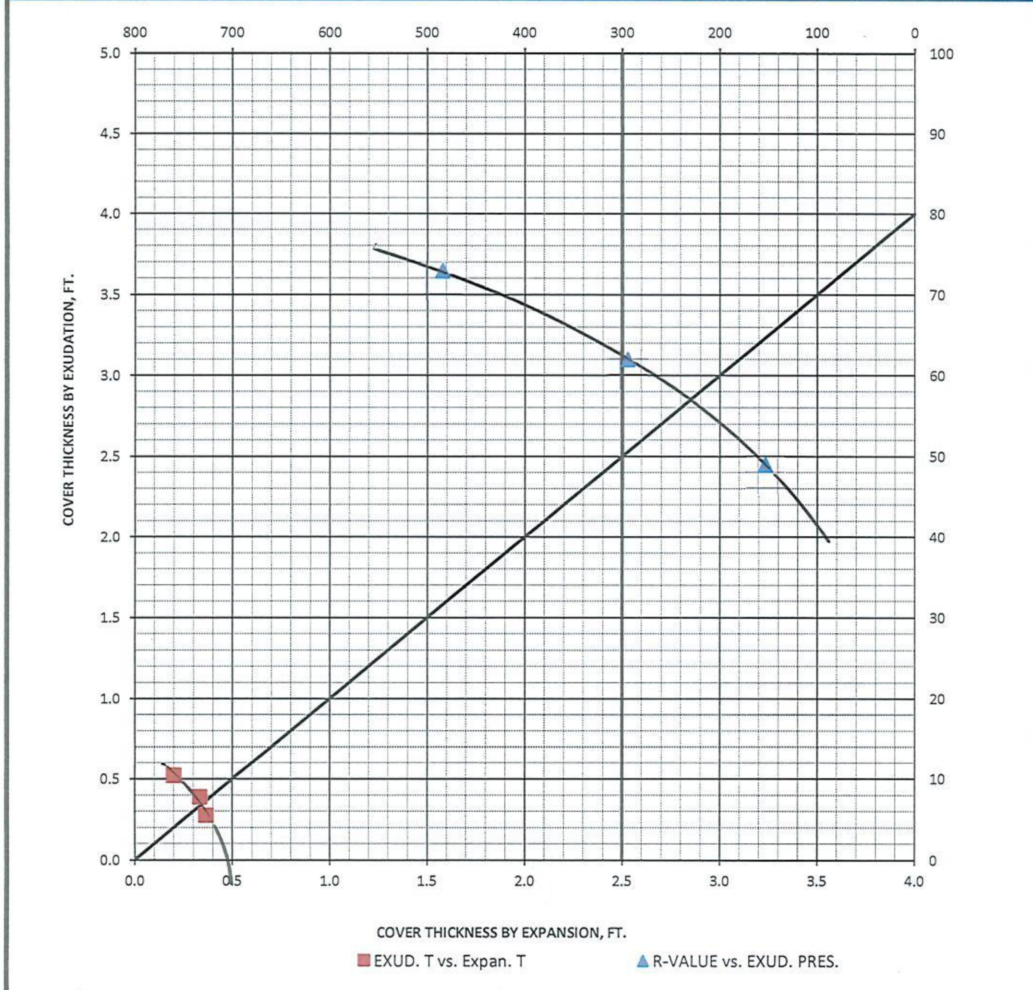
KOA

P.N. 4811-SFPW

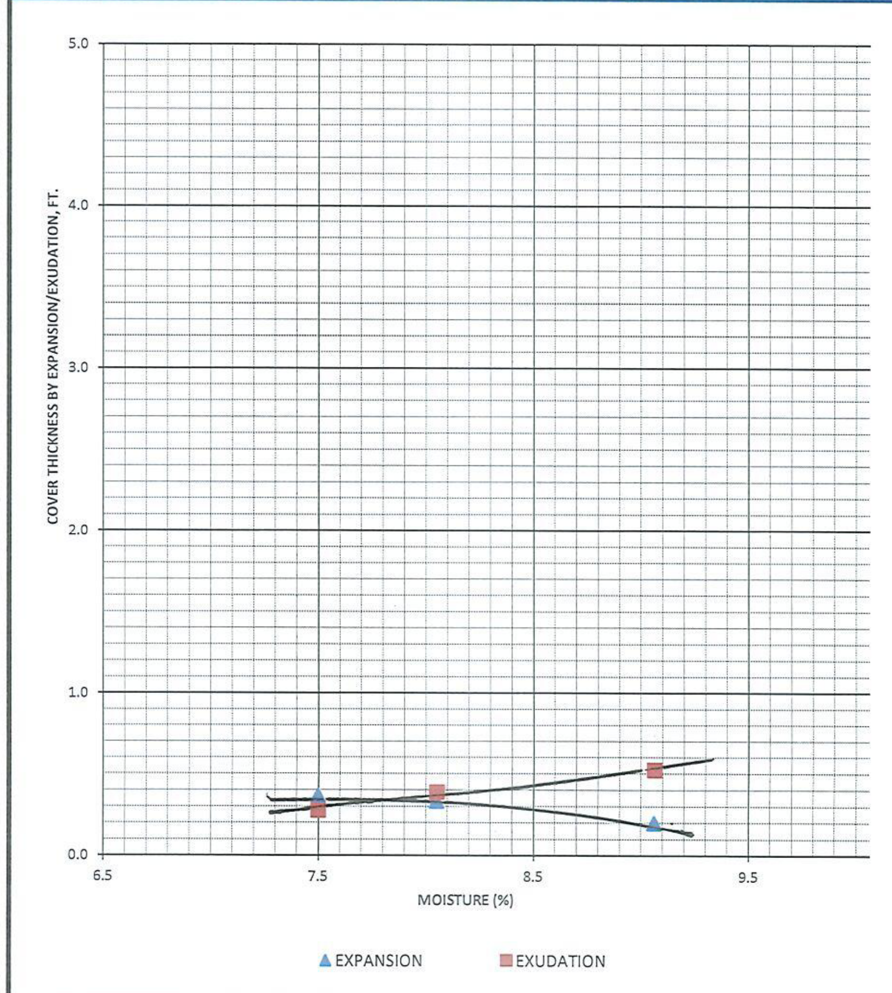
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 48345

DATE: 6/14/2022

BORING NO. Lab # 22-1607, B-7 @ 0'-4'

KOA

P.N. 4811-SFPW

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	4	5	6
Water added, grams	65	54	48
Initial Test Water, %	10.5	9.5	8.9
Compact Gage Pressure,psi	150	325	350
Exudation Pressure, psi	228	334	479
Height Sample, Inches	2.58	2.55	2.50
Gross Weight Mold, grams	3117	3097	3099
Tare Weight Mold, grams	1954	1941	1952
Sample Wet Weight, grams	1163	1156	1147
Expansion, Inches x 10exp-4	3	11	29
Stability 2,000 lbs (160psi)	31 / 61	21 / 40	17 / 30
Turns Displacement	5.42	4.63	4.33
R-Value Uncorrected	43	62	71
R-Value Corrected	45	62	71
Dry Density, pcf	123.6	125.4	127.6

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.56	0.39	0.30
G. E. by Expansion	0.10	0.37	0.97

Equilibrium R-Value	**58** by EXUDATION	Examined & Checked: 6 /14/ 22
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 48345

DATE: 6 /14/ 2022

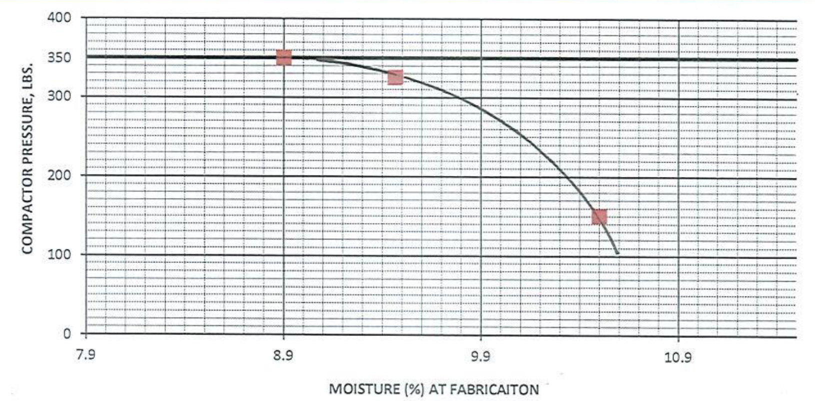
REMARKS:

BORING NO. Lab # 22-1607, B-7 @ 0'-4'

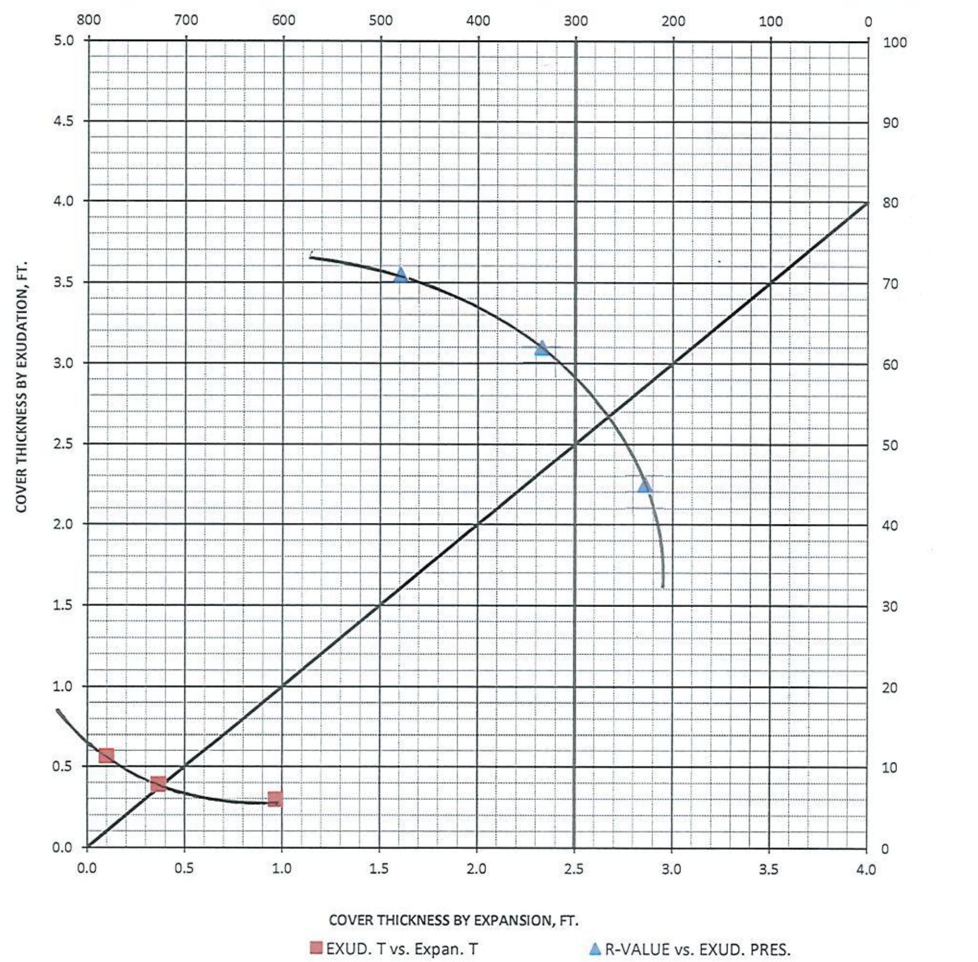
KOA

P.N. 4811-SFPW

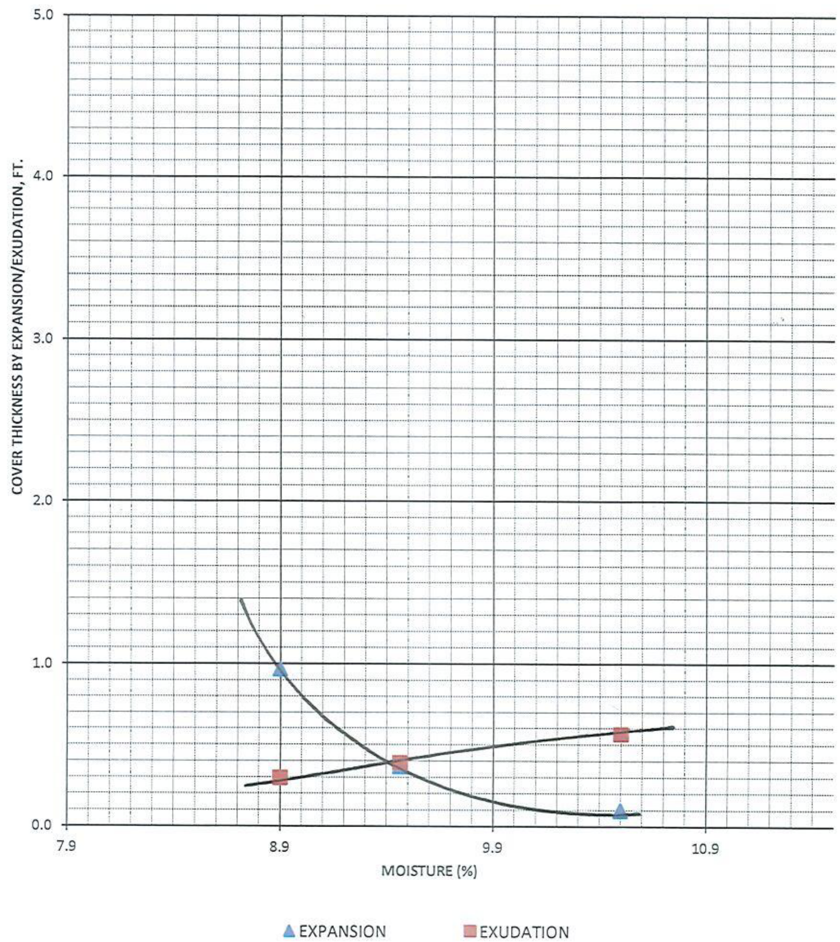
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION

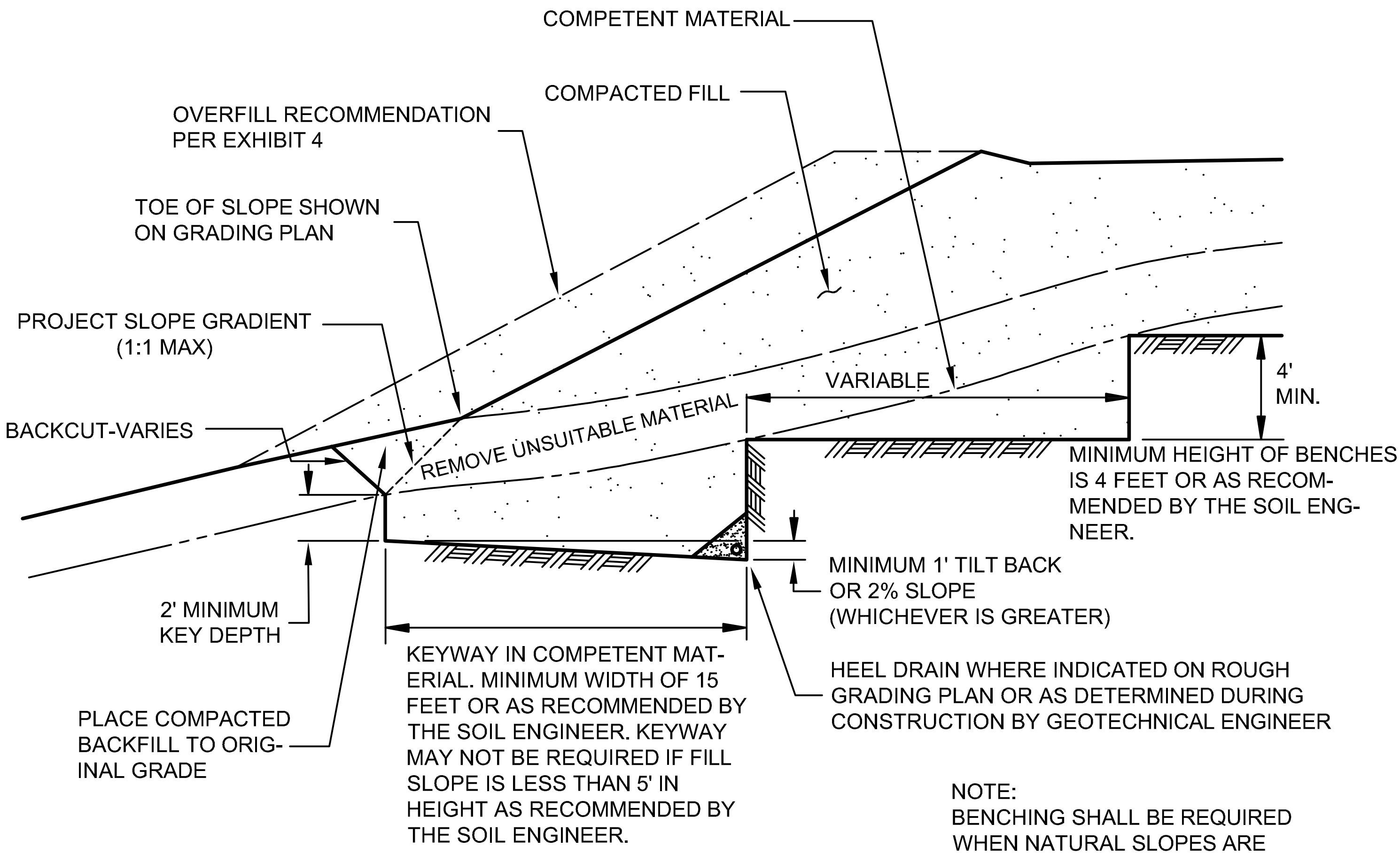


COVER THICKNESS vs MOISTURE %



APPENDIX C

TYPICAL FILL OVER NATURAL SLOPE



NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER.

Exhibit 1

Aragón Geotechnical, Inc.

TYPICAL FILL-OVER-CUT SLOPE

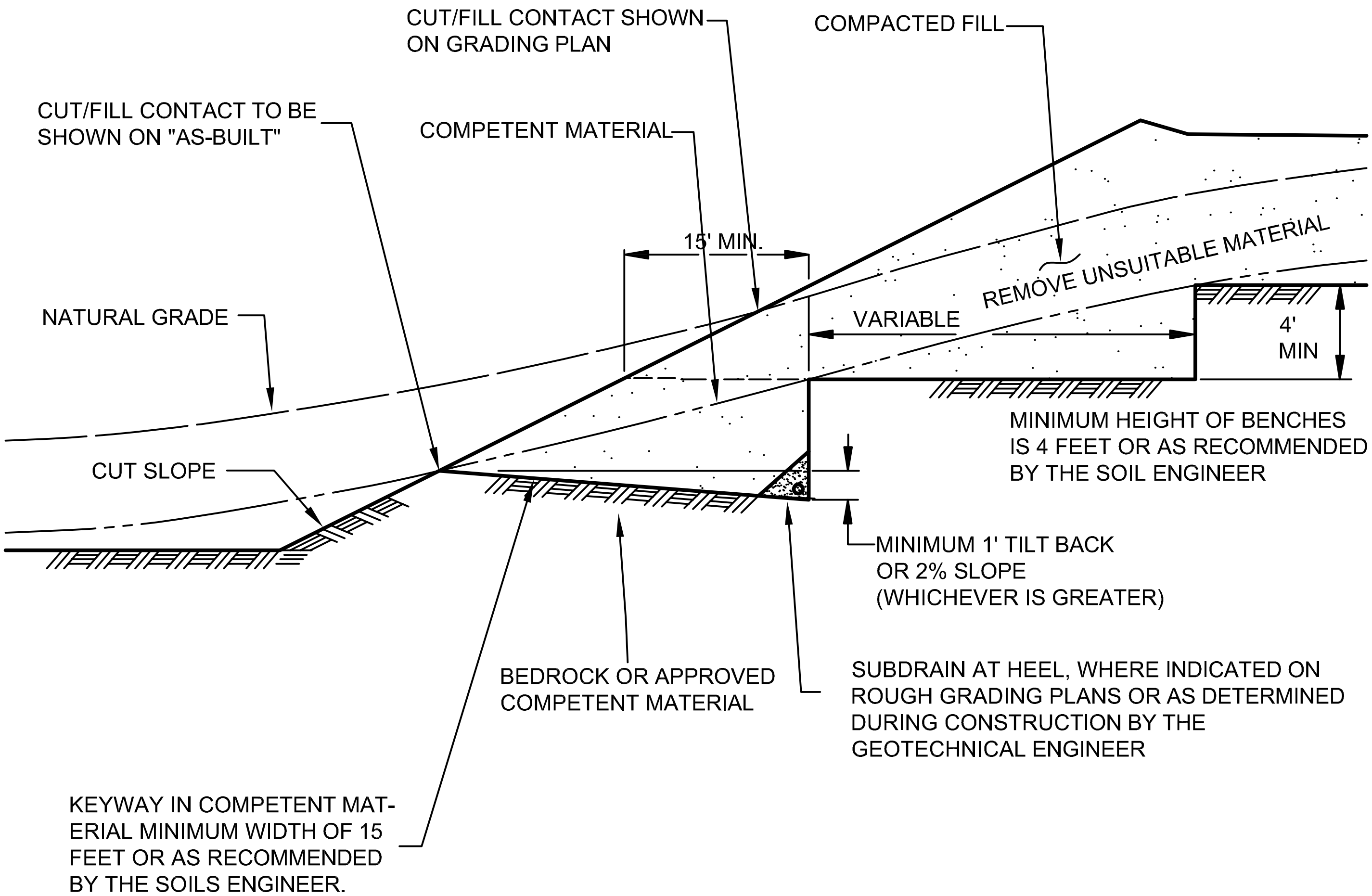
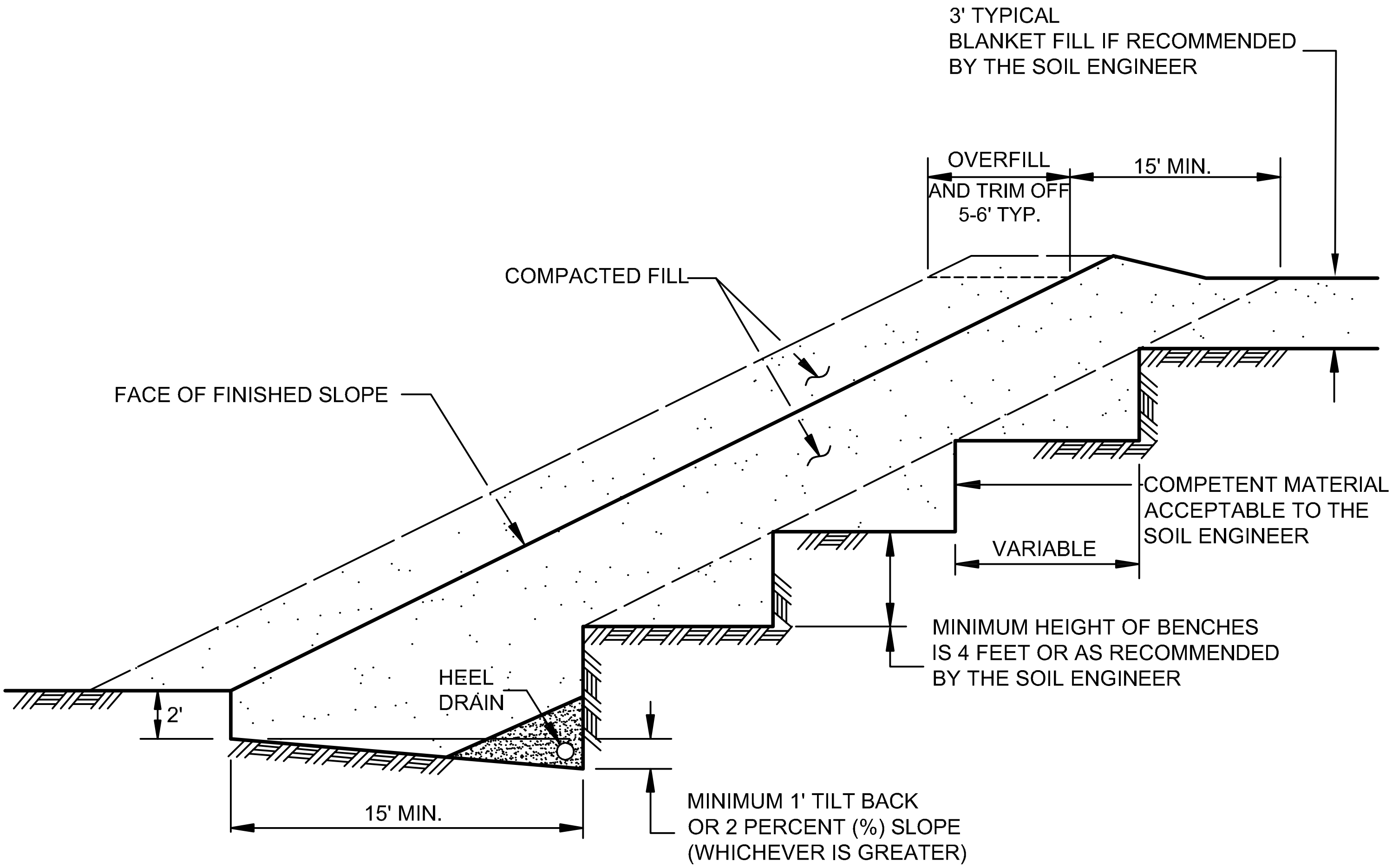


Exhibit 2

TYPICAL STABILIZATION FILL



HEEL DRAIN DETAILS:

1) MINIMUM 4-INCH-DIAMETER PVC SCH. 40 OR SDR 35 WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE, INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. LONGITUDINAL SLOPE AT ≥ 2 PERCENT TO OUTLET.

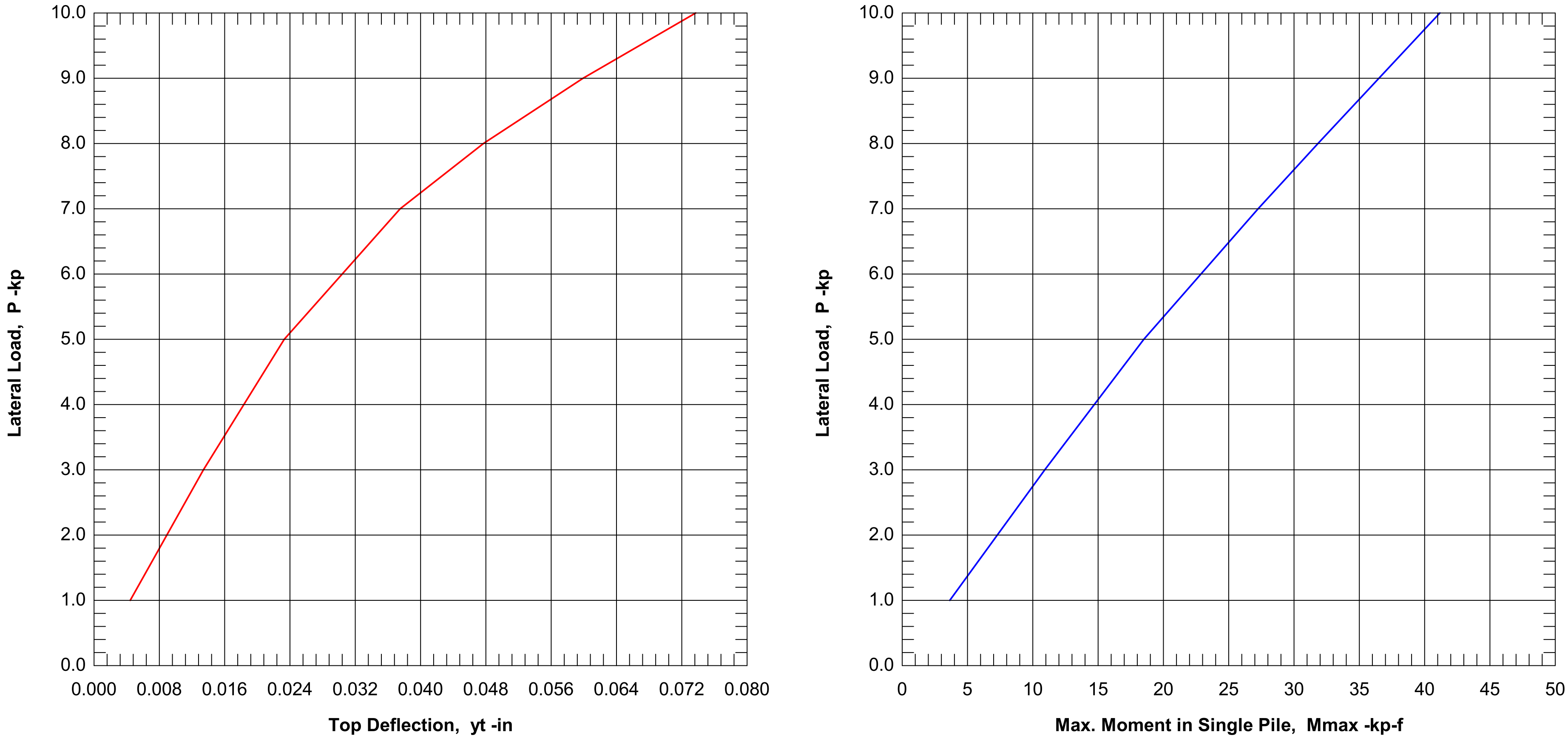
2) BEDDING TO CONSIST OF FIVE CUBIC FEET OF ASTM CLASS I CRUSHED ROCK ($\frac{3}{8}$" - $\frac{3}{4}$" MAX) PER FOOT OF PIPE, ENCASED IN FILTER FABRIC.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT ACCEPTABLE TO ENGINEER. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

*

Exhibit 3

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



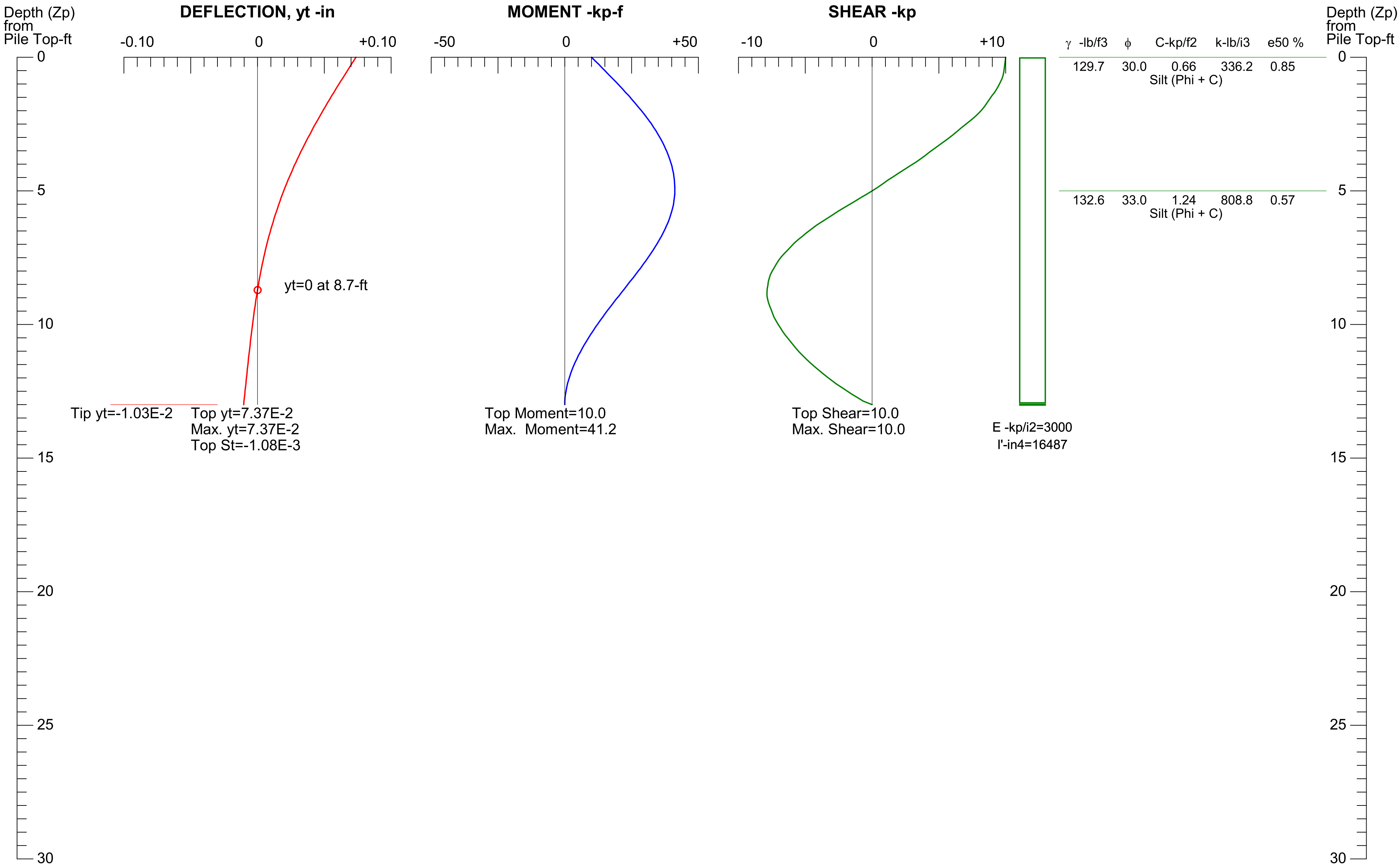
CivilTech Software

McCall Blvd. Widening Project, CIP 22-03
City of Menifee

Figure 1

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=1, Kbc=1

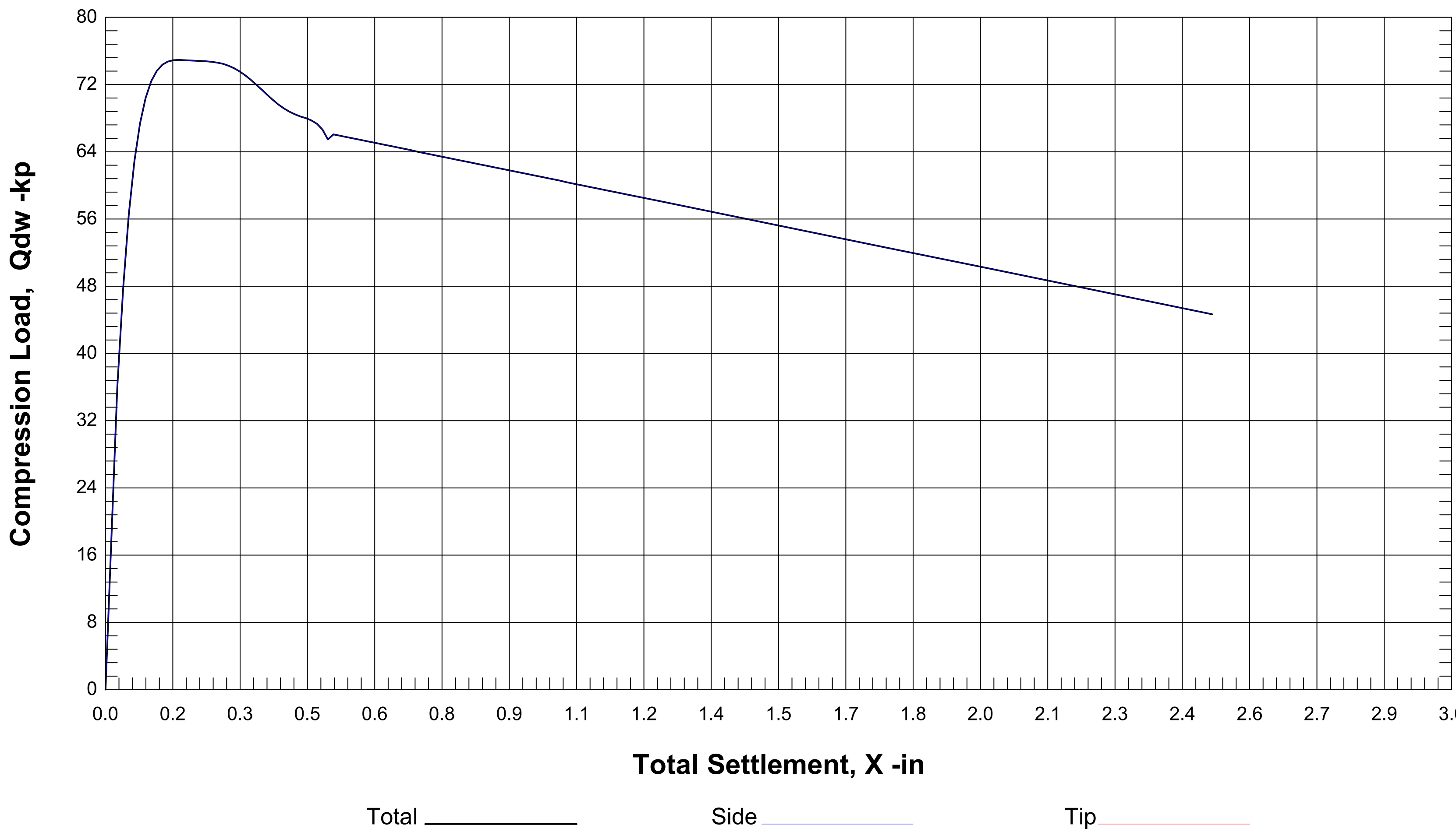


CivilTech Software

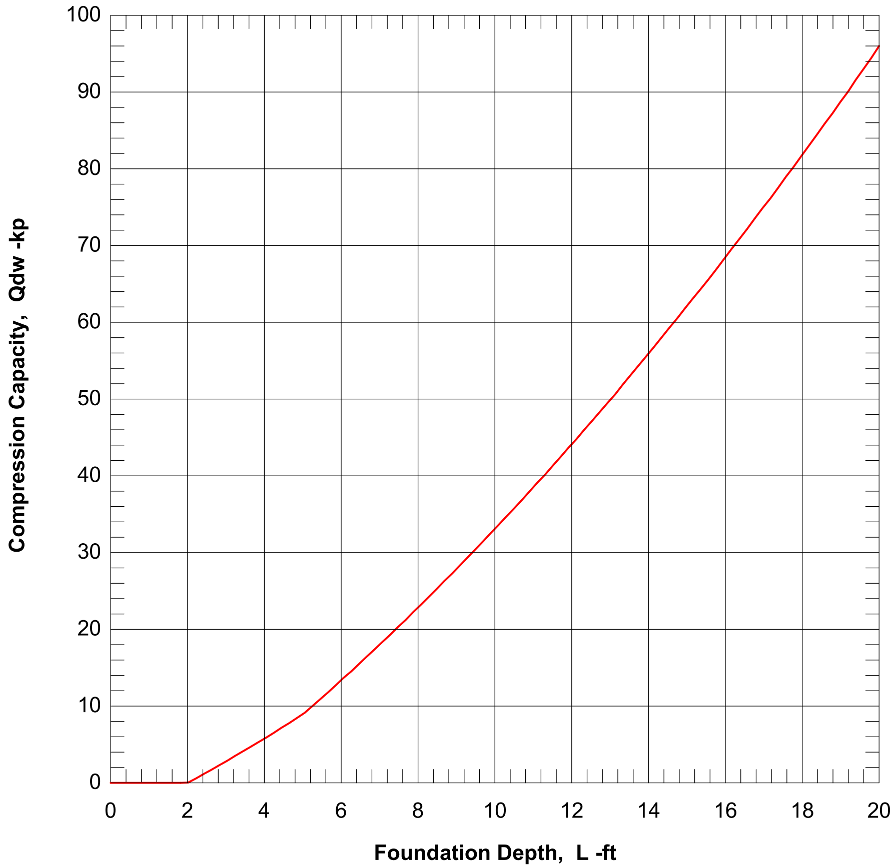
McCall Blvd. Widening Project, CIP 22-03
City of Menifee

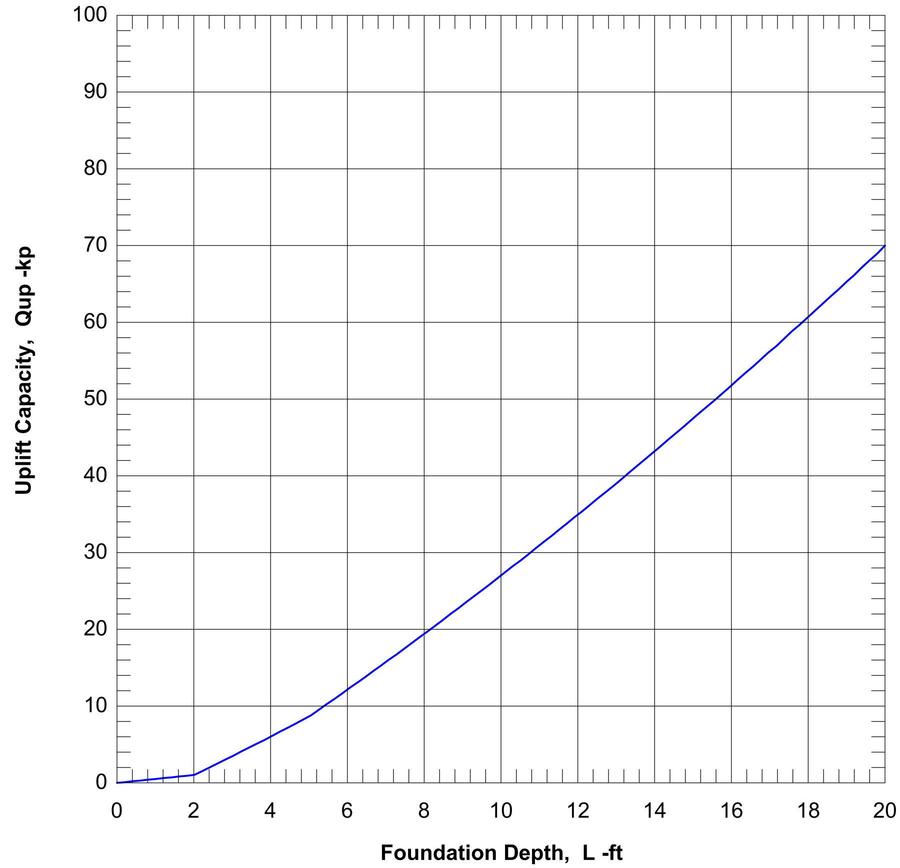
Figure 2

Vertical Load vs. Total Settlement



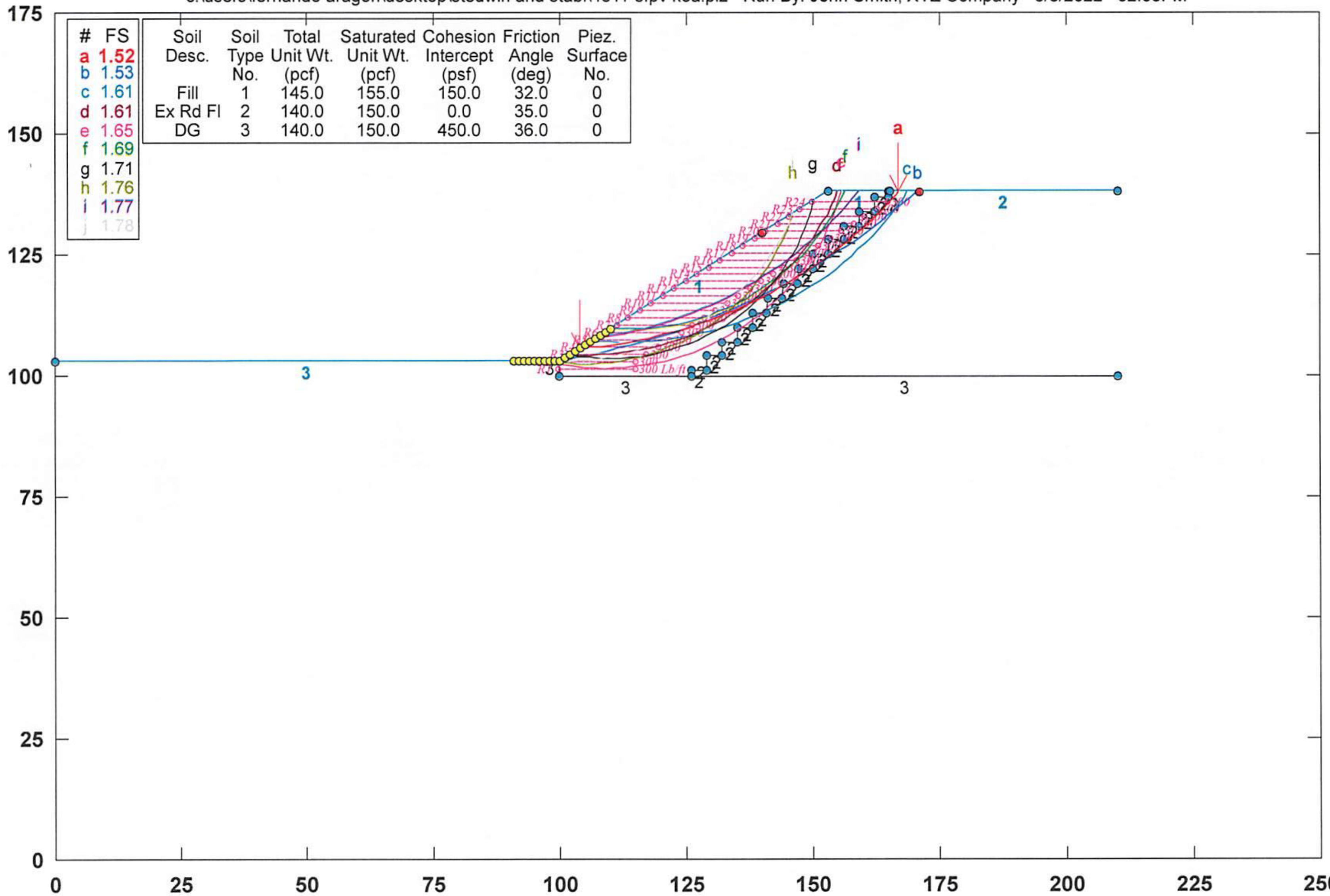
ALLOWABLE CAPACITY vs FOUNDATION DEPTH





McCall Widening Project For KOA City of Menifee

c:\users\fernando aragon\desktop\stedwin and stabl\4811-sfpv koa.pl2 Run By: John Smith, XYZ Company 8/8/2022 02:08PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	145.0	155.0	150.0	32.0	0
Ex Rd Fl	2	140.0	150.0	0.0	35.0	0
DG	3	140.0	150.0	450.0	36.0	0

STABL6H FSmin=1.52

Safety Factors Are Calculated By The Modified Bishop Method

**** STABL6H ****

by
Purdue University
--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 8/8/2022
Time of Run: 02:08PM
Run By: John Smith, XYZ Company
Input Data Filename: C:4811-sfpv koa.in
Output Filename: C:4811-sfpv koa.OUT
Plotted Output Filename: C:4811-sfpv koa.PLT
PROBLEM DESCRIPTION McCall Widening Project For KOA
City of Menifee

BOUNDARY COORDINATES
4 Top Boundaries
34 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	103.00	100.00	103.00	3
2	100.00	103.00	153.00	138.00	1
3	153.00	138.00	165.00	138.00	1
4	165.00	138.00	210.00	138.00	2
5	126.00	100.00	126.10	101.00	2
6	126.10	101.00	129.00	101.00	2
7	129.00	101.00	129.10	104.00	2
8	129.10	104.00	132.00	104.00	2
9	132.00	104.00	132.10	107.00	2
10	132.10	107.00	135.00	107.00	2
11	135.00	107.00	135.10	110.00	2
12	135.10	110.00	138.00	110.00	2
13	138.00	110.00	138.10	113.00	2
14	138.10	113.00	141.00	113.00	2
15	141.00	113.00	141.10	116.00	2
16	141.10	116.00	144.00	116.00	2
17	144.00	116.00	144.10	119.00	2
18	144.10	119.00	147.00	119.00	2
19	147.00	119.00	147.10	122.00	2
20	147.10	122.00	150.00	122.00	2
21	150.00	122.00	150.10	125.00	2
22	150.10	125.00	153.00	125.00	2
23	153.00	125.00	153.10	128.00	2
24	153.10	128.00	156.00	128.00	2
25	156.00	128.00	156.10	131.00	2
26	156.10	131.00	159.00	131.00	2
27	159.00	131.00	159.10	134.00	2
28	159.10	134.00	162.00	134.00	2
29	162.00	134.00	162.10	137.00	2
30	162.10	137.00	165.00	137.00	2
31	165.00	137.00	165.10	138.00	2
32	100.00	103.00	100.00	100.00	3
33	100.00	100.00	126.00	100.00	3
34	126.00	100.00	210.00	100.00	3

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	155.0	150.0	32.0	0.00	0.0	0
2	140.0	150.0	0.0	35.0	0.00	0.0	0
3	140.0	150.0	450.0	36.0	0.00	0.0	0

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40
Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	0.00

```
    2        210.00        0.00
 WATER SURFACE DATA HAS BEEN SUPPRESSED
Searching Routine Will Be Limited To An Area Defined By  1 Boundaries
 Of Which The First  1 Boundaries Will Deflect Surfaces Upward
 Boundary     X-Left     Y-Left    X-Right    Y-Right
    No.        (ft)       (ft)       (ft)       (ft)
    1          0.00       0.00       0.00       0.00
 SEARCH LIMIT BOUNDARY DATA HAS BEEN SUPPRESSED
 A Horizontal Earthquake Loading Coefficient
 Of0.150 Has Been Assigned
 A Vertical Earthquake Loading Coefficient
 Of0.000 Has Been Assigned
 Cavitation Pressure =    0.0 psf
 A Horizontal Earthquake Loading Coefficient
 Of0.000 Has Been Assigned
 A Vertical Earthquake Loading Coefficient
 Of0.000 Has Been Assigned
 Cavitation Pressure =    0.0 psf
REINFORCING LAYER(S)
    24 REINFORCING LAYER(S) SPECIFIED
 REINFORCING LAYER NO.   1
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        100.00    101.50   300.00     1.000
        2        115.00    101.50   300.00     1.000
 REINFORCING LAYER NO.   2
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        100.00    103.00   300.00     1.000
        2        115.00    103.00   300.00     1.000
 REINFORCING LAYER NO.   3
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        102.27    104.50   300.00     1.000
        2        117.27    104.50   300.00     1.000
 REINFORCING LAYER NO.   4
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        104.54    106.00   300.00     1.000
        2        119.54    106.00   300.00     1.000
 REINFORCING LAYER NO.   5
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        106.81    107.50   300.00     1.000
        2        121.81    107.50   300.00     1.000
 REINFORCING LAYER NO.   6
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        109.09    109.00   300.00     1.000
        2        124.09    109.00   300.00     1.000
 REINFORCING LAYER NO.   7
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        111.36    110.50   300.00     1.000
        2        126.36    110.50   300.00     1.000
 REINFORCING LAYER NO.   8
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        113.63    112.00   300.00     1.000
```

```
      2        128.63    112.00    300.00     1.000
REINFORCING LAYER NO.   9
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        115.90    113.50    300.00     1.000
      2        130.90    113.50    300.00     1.000
REINFORCING LAYER NO.  10
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        118.17    115.00    300.00     1.000
      2        133.17    115.00    300.00     1.000
REINFORCING LAYER NO.  11
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        120.44    116.50    300.00     1.000
      2        135.44    116.50    300.00     1.000
REINFORCING LAYER NO.  12
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        122.71    118.00    300.00     1.000
      2        137.71    118.00    300.00     1.000
REINFORCING LAYER NO.  13
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        124.99    119.50    300.00     1.000
      2        139.99    119.50    300.00     1.000
REINFORCING LAYER NO.  14
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        127.26    121.00    300.00     1.000
      2        142.26    121.00    300.00     1.000
REINFORCING LAYER NO.  15
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        129.53    122.50    300.00     1.000
      2        144.53    122.50    300.00     1.000
REINFORCING LAYER NO.  16
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        131.80    124.00    300.00     1.000
      2        146.80    124.00    300.00     1.000
REINFORCING LAYER NO.  17
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        134.07    125.50    300.00     1.000
      2        149.07    125.50    300.00     1.000
REINFORCING LAYER NO.  18
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        136.34    127.00    300.00     1.000
      2        151.34    127.00    300.00     1.000
REINFORCING LAYER NO.  19
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                     FACTOR
      1        138.61    128.50    300.00     1.000
      2        153.61    128.50    300.00     1.000
```

REINFORCING LAYER NO. 20
2 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	140.89	130.00	300.00	1.000
2	155.89	130.00	300.00	1.000

REINFORCING LAYER NO. 21
2 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	143.16	131.50	300.00	1.000
2	158.16	131.50	300.00	1.000

REINFORCING LAYER NO. 22
2 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	145.43	133.00	300.00	1.000
2	160.43	133.00	300.00	1.000

REINFORCING LAYER NO. 23
2 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	147.70	134.50	300.00	1.000
2	162.70	134.50	300.00	1.000

REINFORCING LAYER NO. 24
2 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	149.97	136.00	300.00	1.000
2	164.97	136.00	300.00	1.000

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
40 Trial Surfaces Have Been Generated.
2 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 91.00 ft.
and X = 110.00 ft.
Each Surface Terminates Between X = 140.00 ft.
and X = 171.00 ft.
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00 ft.
2.00 ft. Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Failure Surface Specified By 38 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	104.00	105.64
2	105.99	105.83
3	107.98	106.06
4	109.96	106.34
5	111.93	106.66
6	113.90	107.02
7	115.86	107.42
8	117.81	107.86
9	119.75	108.35
10	121.68	108.87
11	123.60	109.44
12	125.50	110.05
13	127.40	110.70
14	129.27	111.39
15	131.13	112.12
16	132.98	112.89
17	134.81	113.69
18	136.62	114.54
19	138.41	115.43

20	140.19	116.35
21	141.94	117.31
22	143.67	118.31
23	145.38	119.35
24	147.07	120.42
25	148.74	121.53
26	150.38	122.67
27	151.99	123.85
28	153.58	125.07
29	155.15	126.31
30	156.69	127.59
31	158.19	128.90
32	159.67	130.25
33	161.13	131.63
34	162.55	133.03
35	163.94	134.47
36	165.30	135.93
37	166.63	137.43
38	167.11	138.00

Circle Center At X = 96.1 ; Y = 198.7 and Radius, 93.4
*** 1.517 ***
Failure Surface Specified By 41 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	103.00
2	101.97	103.35
3	103.93	103.73
4	105.89	104.14
5	107.84	104.58
6	109.79	105.05
7	111.72	105.54
8	113.65	106.07
9	115.58	106.62
10	117.49	107.20
11	119.40	107.81
12	121.29	108.44
13	123.18	109.10
14	125.06	109.79
15	126.92	110.51
16	128.78	111.26
17	130.63	112.03
18	132.46	112.83
19	134.28	113.65
20	136.09	114.50
21	137.89	115.38
22	139.67	116.28
23	141.44	117.21
24	143.20	118.17
25	144.94	119.15
26	146.67	120.16
27	148.38	121.19
28	150.08	122.24
29	151.77	123.32
30	153.43	124.43
31	155.08	125.56
32	156.72	126.71
33	158.33	127.89
34	159.93	129.09
35	161.52	130.31
36	163.08	131.56
37	164.63	132.83
38	166.15	134.12
39	167.66	135.44
40	169.15	136.77
41	170.48	138.00

Circle Center At X = 76.8 ; Y = 238.2 and Radius, 137.2
*** 1.532 ***

Failure Surface Specified By 38 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	106.00	106.96
2	107.99	106.81
3	109.99	106.72
4	111.99	106.69
5	113.99	106.72
6	115.99	106.80
7	117.98	106.95
8	119.97	107.16
9	121.96	107.43
10	123.93	107.75
11	125.89	108.14
12	127.84	108.58
13	129.78	109.09
14	131.70	109.65
15	133.60	110.26
16	135.48	110.94
17	137.35	111.67
18	139.18	112.45
19	141.00	113.29
20	142.79	114.19
21	144.55	115.13
22	146.28	116.13
23	147.98	117.18
24	149.65	118.29
25	151.29	119.44
26	152.89	120.64
27	154.45	121.88
28	155.98	123.18
29	157.46	124.52
30	158.91	125.90
31	160.31	127.32
32	161.67	128.79
33	162.99	130.30
34	164.26	131.84
35	165.48	133.42
36	166.66	135.04
37	167.78	136.69
38	168.62	138.00

Circle Center At X = 112.0 ; Y = 173.5 and Radius, 66.8

*** 1.608 ***

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	102.00	104.32
2	104.00	104.24
3	106.00	104.23
4	108.00	104.30
5	109.99	104.44
6	111.98	104.66
7	113.96	104.95
8	115.93	105.31
9	117.88	105.75
10	119.81	106.26
11	121.73	106.84
12	123.62	107.50
13	125.48	108.22
14	127.32	109.01
15	129.12	109.87
16	130.89	110.80
17	132.63	111.79
18	134.33	112.85
19	135.99	113.97
20	137.60	115.15
21	139.17	116.39

22	140.69	117.68
23	142.17	119.04
24	143.59	120.44
25	144.95	121.90
26	146.27	123.41
27	147.52	124.97
28	148.72	126.57
29	149.85	128.22
30	150.93	129.91
31	151.93	131.63
32	152.88	133.40
33	153.76	135.19
34	154.57	137.02
35	154.96	138.00

Circle Center At X = 105.2 ; Y = 157.8 and Radius, 53.6

*** 1.612 ***

Failure Surface Specified By 39 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	97.00	103.00
2	98.95	102.55
3	100.92	102.18
4	102.89	101.90
5	104.88	101.69
6	106.88	101.57
7	108.88	101.53
8	110.88	101.58
9	112.88	101.71
10	114.86	101.91
11	116.84	102.21
12	118.81	102.58
13	120.76	103.03
14	122.68	103.57
15	124.59	104.18
16	126.46	104.87
17	128.31	105.64
18	130.12	106.49
19	131.90	107.41
20	133.64	108.40
21	135.33	109.46
22	136.98	110.59
23	138.58	111.79
24	140.13	113.06
25	141.63	114.38
26	143.07	115.77
27	144.45	117.22
28	145.77	118.72
29	147.02	120.28
30	148.22	121.88
31	149.34	123.54
32	150.39	125.24
33	151.38	126.98
34	152.29	128.76
35	153.12	130.57
36	153.88	132.42
37	154.57	134.30
38	155.17	136.21
39	155.66	138.00

Circle Center At X = 108.8 ; Y = 149.9 and Radius, 48.3

*** 1.648 ***

Failure Surface Specified By 30 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	110.00	109.60
2	112.00	109.50
3	114.00	109.47
4	116.00	109.54

5	117.99	109.69
6	119.98	109.93
7	121.95	110.25
8	123.91	110.66
9	125.85	111.15
10	127.76	111.73
11	129.65	112.38
12	131.51	113.12
13	133.33	113.94
14	135.12	114.84
15	136.87	115.81
16	138.57	116.86
17	140.23	117.98
18	141.84	119.17
19	143.39	120.43
20	144.89	121.75
21	146.33	123.14
22	147.70	124.59
23	149.02	126.10
24	150.27	127.67
25	151.44	129.28
26	152.55	130.95
27	153.59	132.66
28	154.54	134.41
29	155.43	136.21
30	156.21	138.00

Circle Center At X = 113.5 ; Y = 155.7 and Radius, 46.2
*** 1.685 ***
Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	102.00	104.32
2	103.97	103.97
3	105.95	103.72
4	107.95	103.56
5	109.95	103.50
6	111.95	103.55
7	113.94	103.68
8	115.93	103.92
9	117.90	104.26
10	119.85	104.69
11	121.78	105.21
12	123.68	105.84
13	125.55	106.55
14	127.38	107.35
15	129.17	108.25
16	130.91	109.23
17	132.61	110.29
18	134.25	111.44
19	135.82	112.67
20	137.34	113.97
21	138.79	115.35
22	140.18	116.79
23	141.49	118.30
24	142.72	119.88
25	143.88	121.51
26	144.95	123.20
27	145.94	124.94
28	146.84	126.72
29	147.65	128.55
30	148.38	130.41
31	149.01	132.31
32	149.54	134.24
33	149.93	135.98

Circle Center At X = 110.1 ; Y = 144.2 and Radius, 40.7
*** 1.714 ***
Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	98.00	103.00
2	99.98	102.73
3	101.97	102.54
4	103.97	102.46
5	105.97	102.46
6	107.97	102.56
7	109.96	102.75
8	111.94	103.03
9	113.90	103.41
10	115.85	103.88
11	117.77	104.44
12	119.66	105.08
13	121.52	105.82
14	123.35	106.64
15	125.13	107.54
16	126.87	108.53
17	128.56	109.60
18	130.20	110.74
19	131.78	111.96
20	133.31	113.26
21	134.77	114.62
22	136.17	116.05
23	137.50	117.54
24	138.76	119.10
25	139.95	120.71
26	141.05	122.37
27	142.08	124.09
28	143.03	125.85
29	143.90	127.65
30	144.68	129.49
31	145.37	131.37
32	145.98	133.28
33	146.00	133.38

Circle Center At X = 104.9 ; Y = 145.3 and Radius, 42.8

*** 1.758 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	107.00	107.62
2	108.98	107.94
3	110.94	108.30
4	112.90	108.71
5	114.85	109.17
6	116.78	109.68
7	118.70	110.23
8	120.61	110.84
9	122.50	111.49
10	124.38	112.18
11	126.23	112.92
12	128.07	113.71
13	129.89	114.54
14	131.69	115.42
15	133.46	116.34
16	135.21	117.31
17	136.94	118.32
18	138.64	119.37
19	140.32	120.46
20	141.97	121.59
21	143.59	122.77
22	145.18	123.98
23	146.74	125.23
24	148.26	126.52
25	149.76	127.85
26	151.22	129.21
27	152.65	130.61

28	154.04	132.05
29	155.40	133.52
30	156.72	135.02
31	158.00	136.55
32	159.16	138.00

Circle Center At X = 95.3 ; Y = 187.7 and Radius, 81.0

*** 1.774 ***

Failure Surface Specified By 31 Coordinate Points

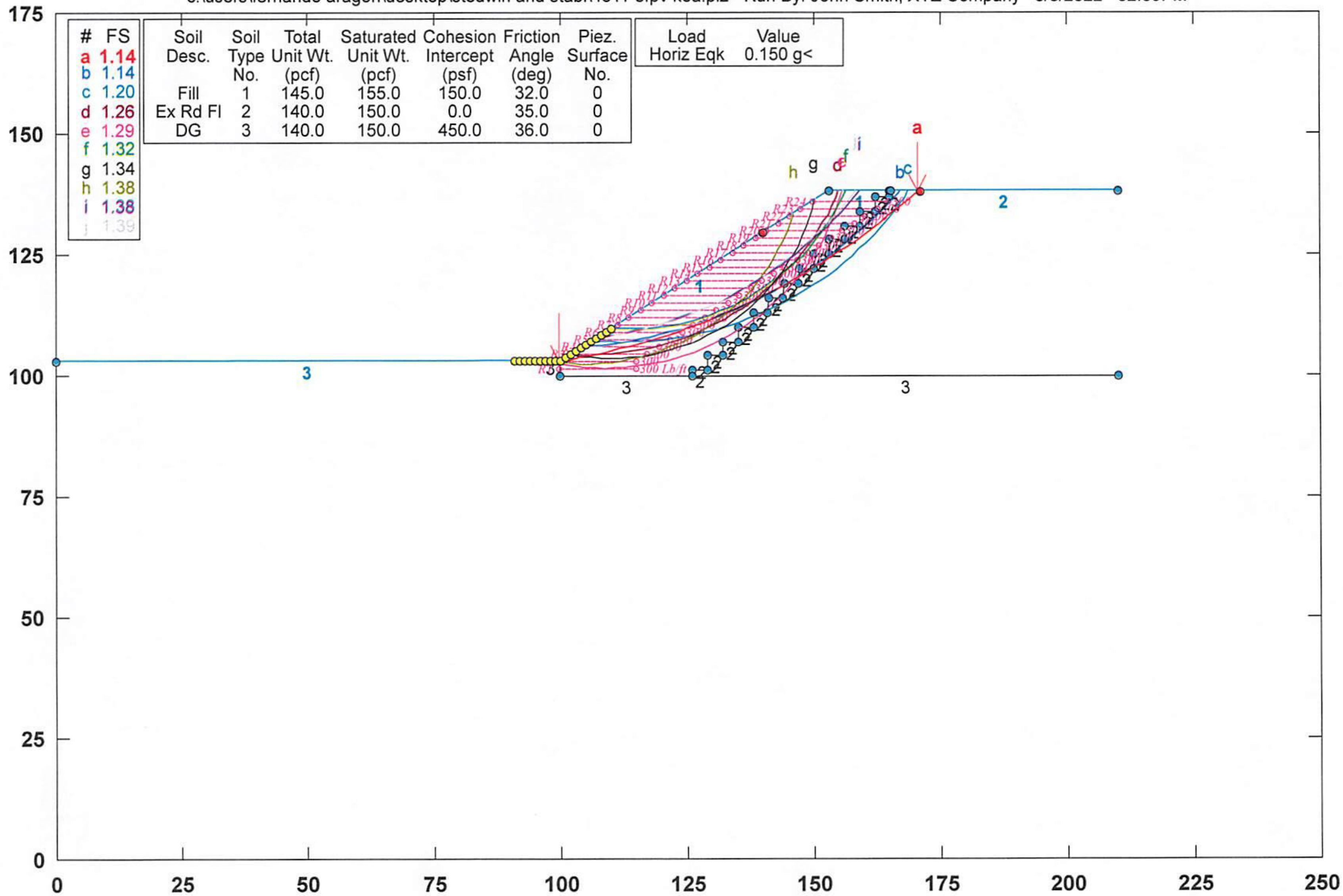
Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	103.00
2	102.00	103.01
3	104.00	103.11
4	105.99	103.28
5	107.97	103.54
6	109.95	103.88
7	111.90	104.29
8	113.84	104.79
9	115.75	105.36
10	117.65	106.02
11	119.51	106.74
12	121.34	107.55
13	123.14	108.42
14	124.90	109.37
15	126.62	110.39
16	128.30	111.48
17	129.93	112.64
18	131.51	113.86
19	133.04	115.14
20	134.52	116.49
21	135.95	117.90
22	137.31	119.36
23	138.62	120.87
24	139.86	122.44
25	141.03	124.06
26	142.14	125.72
27	143.19	127.43
28	144.16	129.18
29	145.06	130.96
30	145.88	132.79
31	146.17	133.49

Circle Center At X = 100.7 ; Y = 152.2 and Radius, 49.2

*** 1.779 ***

McCall Widening Project For KOA City of Menifee

c:\users\fernando aragon\desktop\stedwin and stabl\4811-sfpv koa.pl2 Run By: John Smith, XYZ Company 8/8/2022 02:09PM



STABL6H FSmin=1.14

Safety Factors Are Calculated By The Modified Bishop Method

**** STABL6H ****
by
Purdue University
--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 8/8/2022
Time of Run: 02:09PM
Run By: John Smith, XYZ Company
Input Data Filename: C:4811-sfpv koa.in
Output Filename: C:4811-sfpv koa.OUT
Plotted Output Filename: C:4811-sfpv koa.PLT
PROBLEM DESCRIPTION McCall Widening Project For KOA
City of Menifee

BOUNDARY COORDINATES
4 Top Boundaries
34 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	103.00	100.00	103.00	3
2	100.00	103.00	153.00	138.00	1
3	153.00	138.00	165.00	138.00	1
4	165.00	138.00	210.00	138.00	2
5	126.00	100.00	126.10	101.00	2
6	126.10	101.00	129.00	101.00	2
7	129.00	101.00	129.10	104.00	2
8	129.10	104.00	132.00	104.00	2
9	132.00	104.00	132.10	107.00	2
10	132.10	107.00	135.00	107.00	2
11	135.00	107.00	135.10	110.00	2
12	135.10	110.00	138.00	110.00	2
13	138.00	110.00	138.10	113.00	2
14	138.10	113.00	141.00	113.00	2
15	141.00	113.00	141.10	116.00	2
16	141.10	116.00	144.00	116.00	2
17	144.00	116.00	144.10	119.00	2
18	144.10	119.00	147.00	119.00	2
19	147.00	119.00	147.10	122.00	2
20	147.10	122.00	150.00	122.00	2
21	150.00	122.00	150.10	125.00	2
22	150.10	125.00	153.00	125.00	2
23	153.00	125.00	153.10	128.00	2
24	153.10	128.00	156.00	128.00	2
25	156.00	128.00	156.10	131.00	2
26	156.10	131.00	159.00	131.00	2
27	159.00	131.00	159.10	134.00	2
28	159.10	134.00	162.00	134.00	2
29	162.00	134.00	162.10	137.00	2
30	162.10	137.00	165.00	137.00	2
31	165.00	137.00	165.10	138.00	2
32	100.00	103.00	100.00	100.00	3
33	100.00	100.00	126.00	100.00	3
34	126.00	100.00	210.00	100.00	3

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	155.0	150.0	32.0	0.00	0.0	0
2	140.0	150.0	0.0	35.0	0.00	0.0	0
3	140.0	150.0	450.0	36.0	0.00	0.0	0

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40
Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	0.00

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    2        210.00        0.00
 WATER SURFACE DATA HAS BEEN SUPPRESSED
Searching Routine Will Be Limited To An Area Defined By  1 Boundaries
 Of Which The First  1 Boundaries Will Deflect Surfaces Upward
 Boundary     X-Left     Y-Left    X-Right    Y-Right
    No.        (ft)       (ft)       (ft)       (ft)
     1         0.00       0.00       0.00       0.00
 SEARCH LIMIT BOUNDARY DATA HAS BEEN SUPPRESSED
 A Horizontal Earthquake Loading Coefficient
 Of0.150 Has Been Assigned
 A Vertical Earthquake Loading Coefficient
 Of0.000 Has Been Assigned
 Cavitation Pressure =    0.0 psf
REINFORCING LAYER(S)
    24 REINFORCING LAYER(S) SPECIFIED
 REINFORCING LAYER NO.   1
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        100.00    101.50    300.00     1.000
        2        115.00    101.50    300.00     1.000
 REINFORCING LAYER NO.   2
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        100.00    103.00    300.00     1.000
        2        115.00    103.00    300.00     1.000
 REINFORCING LAYER NO.   3
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        102.27    104.50    300.00     1.000
        2        117.27    104.50    300.00     1.000
 REINFORCING LAYER NO.   4
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        104.54    106.00    300.00     1.000
        2        119.54    106.00    300.00     1.000
 REINFORCING LAYER NO.   5
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        106.81    107.50    300.00     1.000
        2        121.81    107.50    300.00     1.000
 REINFORCING LAYER NO.   6
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        109.09    109.00    300.00     1.000
        2        124.09    109.00    300.00     1.000
 REINFORCING LAYER NO.   7
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        111.36    110.50    300.00     1.000
        2        126.36    110.50    300.00     1.000
 REINFORCING LAYER NO.   8
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
        1        113.63    112.00    300.00     1.000
        2        128.63    112.00    300.00     1.000
 REINFORCING LAYER NO.   9
   2 POINTS DEFINE THIS LAYER
      POINT     X-COORD   Y-COORD   FORCE   INCLINATION
       NO.                                    FACTOR
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        1        115.90     113.50     300.00      1.000
        2        130.90     113.50     300.00      1.000
REINFORCING LAYER NO.  10
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        118.17     115.00     300.00      1.000
        2        133.17     115.00     300.00      1.000
REINFORCING LAYER NO.  11
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        120.44     116.50     300.00      1.000
        2        135.44     116.50     300.00      1.000
REINFORCING LAYER NO.  12
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        122.71     118.00     300.00      1.000
        2        137.71     118.00     300.00      1.000
REINFORCING LAYER NO.  13
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        124.99     119.50     300.00      1.000
        2        139.99     119.50     300.00      1.000
REINFORCING LAYER NO.  14
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        127.26     121.00     300.00      1.000
        2        142.26     121.00     300.00      1.000
REINFORCING LAYER NO.  15
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        129.53     122.50     300.00      1.000
        2        144.53     122.50     300.00      1.000
REINFORCING LAYER NO.  16
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        131.80     124.00     300.00      1.000
        2        146.80     124.00     300.00      1.000
REINFORCING LAYER NO.  17
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        134.07     125.50     300.00      1.000
        2        149.07     125.50     300.00      1.000
REINFORCING LAYER NO.  18
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        136.34     127.00     300.00      1.000
        2        151.34     127.00     300.00      1.000
REINFORCING LAYER NO.  19
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        138.61     128.50     300.00      1.000
        2        153.61     128.50     300.00      1.000
REINFORCING LAYER NO.  20
  2 POINTS DEFINE THIS LAYER
     POINT      X-COORD    Y-COORD    FORCE    INCLINATION
      NO.                                          FACTOR
        1        140.89     130.00     300.00      1.000
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       2       155.89    130.00    300.00     1.000
REINFORCING LAYER NO.  21
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                    FACTOR
       1       143.16    131.50    300.00     1.000
       2       158.16    131.50    300.00     1.000
REINFORCING LAYER NO.  22
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                    FACTOR
       1       145.43    133.00    300.00     1.000
       2       160.43    133.00    300.00     1.000
REINFORCING LAYER NO.  23
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                    FACTOR
       1       147.70    134.50    300.00     1.000
       2       162.70    134.50    300.00     1.000
REINFORCING LAYER NO.  24
  2 POINTS DEFINE THIS LAYER
     POINT     X-COORD   Y-COORD   FORCE   INCLINATION
      NO.                                    FACTOR
       1       149.97    136.00    300.00     1.000
       2       164.97    136.00    300.00     1.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 40 Trial Surfaces Have Been Generated.
  2 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between  X =  91.00 ft.
                              and  X = 110.00 ft.
Each Surface Terminates Between   X = 140.00 ft.
                             and   X = 171.00 ft.
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =  0.00 ft.
 2.00 ft. Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Examined.  They Are Ordered - Most Critical
      First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
      Failure Surface Specified By 41 Coordinate Points
        Point      X-Surf      Y-Surf
         No.        (ft)        (ft)
          1        100.00      103.00
          2        101.97      103.35
          3        103.93      103.73
          4        105.89      104.14
          5        107.84      104.58
          6        109.79      105.05
          7        111.72      105.54
          8        113.65      106.07
          9        115.58      106.62
         10        117.49      107.20
         11        119.40      107.81
         12        121.29      108.44
         13        123.18      109.10
         14        125.06      109.79
         15        126.92      110.51
         16        128.78      111.26
         17        130.63      112.03
         18        132.46      112.83
         19        134.28      113.65
         20        136.09      114.50
         21        137.89      115.38
         22        139.67      116.28
         23        141.44      117.21
         24        143.20      118.17
```

25	144.94	119.15
26	146.67	120.16
27	148.38	121.19
28	150.08	122.24
29	151.77	123.32
30	153.43	124.43
31	155.08	125.56
32	156.72	126.71
33	158.33	127.89
34	159.93	129.09
35	161.52	130.31
36	163.08	131.56
37	164.63	132.83
38	166.15	134.12
39	167.66	135.44
40	169.15	136.77
41	170.48	138.00

Circle Center At X = 76.8 ; Y = 238.2 and Radius, 137.2
*** 1.135 ***
Failure Surface Specified By 38 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	104.00	105.64
2	105.99	105.83
3	107.98	106.06
4	109.96	106.34
5	111.93	106.66
6	113.90	107.02
7	115.86	107.42
8	117.81	107.86
9	119.75	108.35
10	121.68	108.87
11	123.60	109.44
12	125.50	110.05
13	127.40	110.70
14	129.27	111.39
15	131.13	112.12
16	132.98	112.89
17	134.81	113.69
18	136.62	114.54
19	138.41	115.43
20	140.19	116.35
21	141.94	117.31
22	143.67	118.31
23	145.38	119.35
24	147.07	120.42
25	148.74	121.53
26	150.38	122.67
27	151.99	123.85
28	153.58	125.07
29	155.15	126.31
30	156.69	127.59
31	158.19	128.90
32	159.67	130.25
33	161.13	131.63
34	162.55	133.03
35	163.94	134.47
36	165.30	135.93
37	166.63	137.43
38	167.11	138.00

Circle Center At X = 96.1 ; Y = 198.7 and Radius, 93.4
*** 1.136 ***
Failure Surface Specified By 38 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	106.00	106.96
2	107.99	106.81

3	109.99	106.72
4	111.99	106.69
5	113.99	106.72
6	115.99	106.80
7	117.98	106.95
8	119.97	107.16
9	121.96	107.43
10	123.93	107.75
11	125.89	108.14
12	127.84	108.58
13	129.78	109.09
14	131.70	109.65
15	133.60	110.26
16	135.48	110.94
17	137.35	111.67
18	139.18	112.45
19	141.00	113.29
20	142.79	114.19
21	144.55	115.13
22	146.28	116.13
23	147.98	117.18
24	149.65	118.29
25	151.29	119.44
26	152.89	120.64
27	154.45	121.88
28	155.98	123.18
29	157.46	124.52
30	158.91	125.90
31	160.31	127.32
32	161.67	128.79
33	162.99	130.30
34	164.26	131.84
35	165.48	133.42
36	166.66	135.04
37	167.78	136.69
38	168.62	138.00

Circle Center At X = 112.0 ; Y = 173.5 and Radius, 66.8

*** 1.202 ***

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	102.00	104.32
2	104.00	104.24
3	106.00	104.23
4	108.00	104.30
5	109.99	104.44
6	111.98	104.66
7	113.96	104.95
8	115.93	105.31
9	117.88	105.75
10	119.81	106.26
11	121.73	106.84
12	123.62	107.50
13	125.48	108.22
14	127.32	109.01
15	129.12	109.87
16	130.89	110.80
17	132.63	111.79
18	134.33	112.85
19	135.99	113.97
20	137.60	115.15
21	139.17	116.39
22	140.69	117.68
23	142.17	119.04
24	143.59	120.44
25	144.95	121.90
26	146.27	123.41

27	147.52	124.97
28	148.72	126.57
29	149.85	128.22
30	150.93	129.91
31	151.93	131.63
32	152.88	133.40
33	153.76	135.19
34	154.57	137.02
35	154.96	138.00

Circle Center At X = 105.2 ; Y = 157.8 and Radius, 53.6
*** 1.256 ***
Failure Surface Specified By 39 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	97.00	103.00
2	98.95	102.55
3	100.92	102.18
4	102.89	101.90
5	104.88	101.69
6	106.88	101.57
7	108.88	101.53
8	110.88	101.58
9	112.88	101.71
10	114.86	101.91
11	116.84	102.21
12	118.81	102.58
13	120.76	103.03
14	122.68	103.57
15	124.59	104.18
16	126.46	104.87
17	128.31	105.64
18	130.12	106.49
19	131.90	107.41
20	133.64	108.40
21	135.33	109.46
22	136.98	110.59
23	138.58	111.79
24	140.13	113.06
25	141.63	114.38
26	143.07	115.77
27	144.45	117.22
28	145.77	118.72
29	147.02	120.28
30	148.22	121.88
31	149.34	123.54
32	150.39	125.24
33	151.38	126.98
34	152.29	128.76
35	153.12	130.57
36	153.88	132.42
37	154.57	134.30
38	155.17	136.21
39	155.66	138.00

Circle Center At X = 108.8 ; Y = 149.9 and Radius, 48.3
*** 1.287 ***
Failure Surface Specified By 30 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	110.00	109.60
2	112.00	109.50
3	114.00	109.47
4	116.00	109.54
5	117.99	109.69
6	119.98	109.93
7	121.95	110.25
8	123.91	110.66
9	125.85	111.15

10	127.76	111.73
11	129.65	112.38
12	131.51	113.12
13	133.33	113.94
14	135.12	114.84
15	136.87	115.81
16	138.57	116.86
17	140.23	117.98
18	141.84	119.17
19	143.39	120.43
20	144.89	121.75
21	146.33	123.14
22	147.70	124.59
23	149.02	126.10
24	150.27	127.67
25	151.44	129.28
26	152.55	130.95
27	153.59	132.66
28	154.54	134.41
29	155.43	136.21
30	156.21	138.00

Circle Center At X = 113.5 ; Y = 155.7 and Radius, 46.2

*** 1.316 ***

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	102.00	104.32
2	103.97	103.97
3	105.95	103.72
4	107.95	103.56
5	109.95	103.50
6	111.95	103.55
7	113.94	103.68
8	115.93	103.92
9	117.90	104.26
10	119.85	104.69
11	121.78	105.21
12	123.68	105.84
13	125.55	106.55
14	127.38	107.35
15	129.17	108.25
16	130.91	109.23
17	132.61	110.29
18	134.25	111.44
19	135.82	112.67
20	137.34	113.97
21	138.79	115.35
22	140.18	116.79
23	141.49	118.30
24	142.72	119.88
25	143.88	121.51
26	144.95	123.20
27	145.94	124.94
28	146.84	126.72
29	147.65	128.55
30	148.38	130.41
31	149.01	132.31
32	149.54	134.24
33	149.93	135.98

Circle Center At X = 110.1 ; Y = 144.2 and Radius, 40.7

*** 1.341 ***

Failure Surface Specified By 33 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	98.00	103.00
2	99.98	102.73
3	101.97	102.54

4	103.97	102.46
5	105.97	102.46
6	107.97	102.56
7	109.96	102.75
8	111.94	103.03
9	113.90	103.41
10	115.85	103.88
11	117.77	104.44
12	119.66	105.08
13	121.52	105.82
14	123.35	106.64
15	125.13	107.54
16	126.87	108.53
17	128.56	109.60
18	130.20	110.74
19	131.78	111.96
20	133.31	113.26
21	134.77	114.62
22	136.17	116.05
23	137.50	117.54
24	138.76	119.10
25	139.95	120.71
26	141.05	122.37
27	142.08	124.09
28	143.03	125.85
29	143.90	127.65
30	144.68	129.49
31	145.37	131.37
32	145.98	133.28
33	146.00	133.38

Circle Center At X = 104.9 ; Y = 145.3 and Radius, 42.8

*** 1.378 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	107.00	107.62
2	108.98	107.94
3	110.94	108.30
4	112.90	108.71
5	114.85	109.17
6	116.78	109.68
7	118.70	110.23
8	120.61	110.84
9	122.50	111.49
10	124.38	112.18
11	126.23	112.92
12	128.07	113.71
13	129.89	114.54
14	131.69	115.42
15	133.46	116.34
16	135.21	117.31
17	136.94	118.32
18	138.64	119.37
19	140.32	120.46
20	141.97	121.59
21	143.59	122.77
22	145.18	123.98
23	146.74	125.23
24	148.26	126.52
25	149.76	127.85
26	151.22	129.21
27	152.65	130.61
28	154.04	132.05
29	155.40	133.52
30	156.72	135.02
31	158.00	136.55
32	159.16	138.00

Circle Center At X = 95.3 ; Y = 187.7 and Radius, 81.0

*** 1.379 ***

Failure Surface Specified By 32 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	107.00	107.62
2	108.98	107.92
3	110.95	108.28
4	112.90	108.68
5	114.85	109.14
6	116.79	109.64
7	118.71	110.20
8	120.61	110.80
9	122.50	111.46
10	124.38	112.16
11	126.23	112.92
12	128.06	113.72
13	129.87	114.56
14	131.66	115.46
15	133.43	116.40
16	135.17	117.39
17	136.88	118.42
18	138.57	119.49
19	140.22	120.61
20	141.85	121.78
21	143.45	122.98
22	145.01	124.23
23	146.55	125.51
24	148.04	126.84
25	149.51	128.20
26	150.93	129.60
27	152.32	131.04
28	153.68	132.51
29	154.99	134.02
30	156.26	135.56
31	157.50	137.14
32	158.13	138.00

Circle Center At X = 96.4 ; Y = 183.7 and Radius, 76.8

*** 1.388 ***


GEOTECHNICAL LEGEND
B-8 Approximate location of exploratory boring
AC-4 Location of auxiliary pavement core site
SL-2 Approximate location of seismic refraction line spread
af Man-made fill, various placement dates.
Qvof Very old fan alluvium
Kdvg Domenigoni Valley granodiorite
Trml Layered phyllite, schist, and quartzite
Approximate location of geologic contact
70 Strike and dip of joint
56 Strike and dip of foliation
MAP LEGEND:
TRAFFIC SIGNAL- NO MODIFICATION TS
TRAFFIC SIGNAL- MODIFICATION TS
PROPOSED RIGHT-OF-WAY
EXISTING RIGHT-OF-WAY
PROPOSED RIGHT-OF-WAY ACQUISITION
PROPOSED CURB/ EP
EXISTING CURB/ EP
PROPOSED SIDEWALK
EXISTING SIDEWALK TO REMAIN
PROPOSED ASPHALT
PROPOSED STRIPING
PROPOSED RETAINING WALL
PROPOSED DAYLIGHT LINE
EXISTING POWER POLE TO REMAIN
EXISTING POWER POLE TO BE RELOCATED
50 25 0 50 100 150
SCALE: 1"=50'
Kdvg
af
Kdvg (cut)
Tml
Qvof
B-1
B-2
B-3
B-5
B-6
B-7
AC-1
AC-2
AC-3
AC-4
OAK HURST AVE
WOODSIDE ST
PALOMAR RD
VINE ST
SUMMIT ST
FARGO ST
DURANT ST
JUNIPERO RD
DALES ST
PACKARD ST
MENIFEE RD
McCALL BLVD
ROCKS
GEOTECHNICAL MAP
McCall Blvd. Widening, CIP 22-03, Menifee, CA
PROJECT NO. 4811-SFPV
DATE: 8/03/22
PLATE NO. 1
CADD PROJECT FILE NAME: STRIPING EXHIBIT
TIME: 9:57 AM
DATE: 12/2/2019
MANUEL BARRIOS
RCE 68190
EXP. 9/30/21
DATE
CARLOS E. GERONIMO, RCE 75635
CITY OF MENIFEE
ENGINEERING DEPARTMENT
CIP 22-03
McCALL BLVD WIDENING (HOSPITAL-MENIFEE)
McCALL BLVD ALIGNMENT EXHIBIT
SHEET NO.
1
OF 1
PROJECT NO: CIP 22-03