GEOTECHNICAL EXPLORATION REPORT
PROPOSED ETHANAC ROAD BRIDGE OVER
SAN JACINTO RIVER
PERRIS, CALIFORNIA

Prepared for

RICHLAND COMMUNITIES
1361 Michelson Drive, Suite 425
Irvine, California 92612

Project No. 11127.003

February 23, 2018

Leighton and Associates, Inc.
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Richland Communities
1361 Michelson Drive, Suite 425
Irvine, California 92612

Attention: Mr. Brian Hardy, Vice President - Land Development

Subject: Geotechnical Exploration Report
Proposed Ethanac Road Bridge Over San Jacinto River
Perris, California

In accordance with your request and authorization, we are pleased to present this *Geotechnical Exploration Report* for the proposed Ethanac Road Bridge. This report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects for the proposed bridge. It is our opinion that the proposed improvements are geotechnically feasible provided the recommendations provided herein are incorporated into the design and construction.

We appreciate this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 EXECUTIVE SUMMARY

This *Geotechnical Exploration Report* is provided in support of the design of the *Ethanac Road Bridge Over San Jacinto River, Perris California (see Figure 1)*. Conclusions and recommendations presented herein are based on prevailing subsurface conditions and available information from published and in-house geologic information. Based on this information, our main geotechnical findings and recommendations are as follows:

- **Site Geology**: The proposed bridge is underlain by metamorphic rock with variable thickness of overlying alluvium (up to 18 feet where explored).
- **Active Surface Faulting**: The proposed bridge is not located within currently designated Alquist-Priolo (AP) Special Studies Zones, neither is the site located within a Riverside County designated fault zone. No known faults cross or trend into the planned bridge area.
- **Liquefaction**: Due to the relatively shallow bedrock and anticipated foundation embedment into compacted fill or metamorphic rock, liquefaction is not a constraint for the proposed bridge.
- **Bridge Foundations**: We anticipate that the new bridge will be supported on spread/shallow footings poured against metamorphic rock or properly placed engineer fill. Pile type foundations may also be considered for this bridge, depending upon possible grading restrictions due to protected river bottom habitat. However, based on the preliminary foundation report and telecommunications with the structural engineer, the bridge will likely be founded on spread/shallow footings.

2.0 INTRODUCTION

2.1 Purpose and Scope of Work

The purpose of this *Geotechnical Exploration Report* is to provide relevant geotechnical findings and provide recommendations for design of the bridge foundations. Our scope of work generally included research of existing information relevant to this project, a field exploration involving the excavation of 10 borings, geotechnical analyses, and preparation of this report. Reviewed documents are referenced at the end of this report.

2.2 Site Description Improvements

The site of the proposed bridge is the intersection of the western extension of Ethanac Road with the San Jacinto River located in the City of Perris, California (see *Site Location Map – Figure 1)*. Ethanac Road is aligned in an east-west direction; once completed it will provide access for east-west traffic from I-215 to State Route 74. Ethanac Road currently terminates approximately 400 feet east of the San Jacinto River. The north side of Ethanac Road, east the river is occupied by single-family residences. The south side of Ethanac Road east of the river is currently undeveloped; however, another residential development is located approximately 600 feet south of Ethanac Road. The river/channel itself is largely undeveloped. Ethanac Road resumes at about 1½ miles west of the San Jacinto River.

2.3 Proposed Improvements

Based on available information to date, the proposed bridge will consist of a cast-in-place, pre-stressed concrete box girder bridge with 13 box girder cells providing 3 lanes in each direction over the San Jacinto River. The proposed bridge will have an approximate length of 450feet and an approximate width of 79¾ feet including the proposed median, shoulders and sidewalks. The bridge is proposed as a three-span bridge with abutments on both sides of the San Jacinto River and two piers consisting of three columns each. The abutments and bent are anticipated to be supported on a system of spread footings bearing on metamorphic rock or properly prepared engineered fill. Although details are not completed, we anticipate embankment slopes of inclinations of 1½:1 (horizontal:vertical) and will be protected with riprap.

3.0 PHYSICAL SETTINGS

3.1 Climate

The project area is located in a semi-arid climate, which can be considered as an “Inland Valley” Climatic Region per Topic 615 of Caltrans HDM. The hottest months are July, August, and September when high temperatures average in the mid 90’s (°F) and low temperatures average in the low 60’s (°F). The coolest temperatures occur in the winter months when the average highs are in the low 60’s and average lows are just above freezing (32°F). The extreme high temperatures range from about 85°F to as high as 115°F in July, August, and September. The extreme low temperatures range from approximately 30°F in December and January to the mid 50’s (°F) in the summer months. Freezing occurs occasionally during winter nights when the probability of freezing can be as high as 50 to 60 percent. Annual precipitation is in the 10 to 15 inch range, with most rain (about 80 percent) falling between November and March. This climate does not affect the design of the proposed improvements; however, it should affect the selection of asphalt binder grade.

3.2 Topography and Drainage

The overall site topography, in the vicinity of the proposed bridge slopes gently towards the San Jacinto River, which flows in a south-southwest direction. The existing river embankments vary in steepness but generally at 2:1 to 3:1 (horizontal to vertical) and locally steeper.

3.3 Prior Land Use

The site of the proposed improvements is currently occupied by the unimproved San Jacinto River Channel and undeveloped land. Trails and dirt roads parallel the river and cross the site on both sides of the river.

3.4 Man-Made and Natural Features

No natural features are present that would preclude construction of the proposed improvements. Light to moderate vegetation should be expected outside the river channel with shrubs and trees within the channel alignment.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Our field exploration consisted of the excavation of ten (10) exploratory borings within accessible areas of the site to provide basis for ground preparation and foundation design of the proposed bridge structure. During excavation, in-situ undisturbed (Cal Ring) and disturbed/bulk samples were collected from the exploration borings for further laboratory testing and evaluation. Approximate locations of these exploratory borings are depicted on the Boring Location Map (Figure 4). Sampling was conducted by a staff geologist/engineer from our firm. After logging and sampling, the excavations were loosely backfilled with spoils generated during excavation. The exploration logs are included in Appendix A.

4.2 Laboratory Testing

Laboratory tests were performed on representative bulk samples to provide a basis for development of remedial earthwork and geotechnical design parameters. Selected samples were tested to determine the following parameters: maximum dry density and optimum moisture, direct shear, expansion index, consolidation, in-situ moisture and density, soluble sulfate content, chloride content, minimum resistivity and pH. The results of our laboratory testing are presented in Appendix B-1.

5.0 GEOLOGY

5.1 Regional

The site is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. It is characterized by steep, elongated ranges and valleys that trend northwestward. More specifically, the site is situated within the Perris Block, an eroded mass of Cretaceous and older crystalline rock.

The Perris Block, approximately 20 miles by 50 miles in extent, is bounded by the San Jacinto Fault Zone to the northeast, the Elsinore Fault Zone to the southwest, the Cucamonga Fault Zone to the northwest, and the Temecula Basin to the southeast. The southeast boundary of the Perris block is poorly defined. The Perris Block has had a complex tectonic history, apparently undergoing relative vertical land movements of several thousand feet in response to movement on the Elsinore and San Jacinto Fault Zones. Thin sedimentary and volcanic materials locally mantle the crystalline bedrock. Alluvial and colluvial deposits fill the lower valley areas.

5.2 Site

As indicated on the *Regional Geologic Map*, (Figure 2), the natural geologic units within the site are metamorphic rock overlain by alluvial wash deposits (within the river channel). In addition, fill soil should be anticipated beneath the western terminus of the existing Ethanac Road. Based on our exploration, these different soil units may be further described as follows:

5.2.1 Fill

Although not explored as part of this investigation, fill soil should be anticipated beneath the western terminus of Ethanac Road. The fill soil is likely to be locally derived excavated and recompacted metamorphic rock and alluvium. Documentation of the fill placement and compaction was not available for our review at the time of this report.

5.2.2 Alluvium

Alluvial wash deposits were encountered in all exploratory borings and consist of clay silt (ML) to sandy/silty clay (CL) and interbedded silty to clayey sand (SM/SC) with varying amounts of gravel. The thickness of the encountered alluvium ranged from approximately 2 feet away from the channel (LB-10) to as much as 18 feet closer to the channel banks (LB-3). These alluvial sediments are generally medium stiff to very stiff and possess low to high expansion potential. These materials are generally compressible if subjected to additional loads.

5.2.3 Metamorphic Rock

Cretaceous- and Pre-Cretaceous-aged metamorphic rock is mapped on both sides of the San Jacinto River at the location of the proposed bridge. Rock outcroppings are also visible in the vicinity. The metamorphic rock was encountered in all borings at depths ranging from 2 to 18 feet below existing ground surface (see logs of borings in Appendix). Within the depth explored, the metamorphic rock is weathered and was recovered as clayey-silty sand with gravel. Drill auger advancement refusal occurred within typically the upper 15 feet of bedrock.

5.3 Faulting and Seismicity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along northwest-trending regional fault systems such as the San Andreas, San Jacinto, and Elsinore Fault Zones. Currently, these fault systems accommodate up to approximately 55 millimeters per year (mm/yr) of slip between the plates. The San Jacinto Fault Zone is estimated to accommodate slip of approximately 12 mm/yr (WGCEP, 1995).

Historically, the San Jacinto fault zone has produced earthquakes in the magnitude range of 6.2Mw to 7.2Mw (‘Mw’ is the Moment Magnitude as defined by the USGS). The San Jacinto Fault and the San Andreas Fault are among the most active fault systems in California. As shown on Figure 3, the site is not located within a state or county designated fault zone. A list of major local faults and their seismic characteristics is presented in table below.

TABLE 1. LOCAL ACTIVE FAULTS

Fault Name	Fault Type	Maximum Moment Magnitude (MMax)	Peak Ground Acceleration (g)	Distance from Site (km)
Elsinore, Glen Ivy (Fault ID: 365)	SS	7.7	0.22	12.17
Elsinore, Temecula (Fault ID: 378	SS	7.7	0.26	14.31
San Jacinto, Anza (Fault ID: 362)	SS	7.7	0.22	19.23

*Information above from Caltrans ARS Online tool

Based on a probabilistic spectrum obtained from the Caltrans ARS Online analysis tool (version 2.3.06) for 5% probability of exceedance in 50 years, the peak ground acceleration expected at the site is 0.51g. The design spectrum is based on the larger of the deterministic and probabilistic spectral values. Both the deterministic and probabilistic spectra account for soil effects through incorporation of the parameter Vs30, the average shear wave velocity in the upper 30 meters of the soil profile, which is assumed to be 560 m/sec for this site.

6.0 GEOTECHNICAL CONDITIONS

6.1 Groundwater and Surface Water

According to the California Department of Water Resources, depth of groundwater in the vicinity of this site is reported to be approximately 22 feet below existing ground surface. Groundwater was encountered in 7 of the 10 borings excavated. The encountered groundwater was typically at the alluvium to bedrock contact, or slightly within the weathered bedrock. Below is a table showing depths to groundwater and relative elevations in our borings as encountered at the time of exploration.

TABLE 2. DEPTHS TO GROUNDWATER

Boring ID	Approximate Depth to Groundwater (ft)	Correspondent Groundwater Elevation (MSL)
LB-1	15.0	1396
LB-2	17.0	1394
LB-3	16.0	1396
LB-4	11.0	1402
LB-5	11.0	1402
LB-6	8.2	1402
LB-7	8.5	1401

Groundwater or perched ground conditions are expected to fluctuate due to seasonal variations.

6.1.1 Surface Water

Surface water within the river channel should be anticipated during construction. The project should be planned such that construction will occur during the dry season when river is relatively dry or surface water is restricted to the Low Flow Channel. Surface water (localized ponding) was observed in channel bottom at the time of exploration.

6.1.2 Scour

A scour analysis is being performed by others. Sediment analysis (gradation) of two samples collected in the channel area is included in Appendix B.

6.1.3 Erosion

Onsite soil (silt and sand or fine sandy loam per USDA) are inherently subject to erosion. Provisions for site drainage, slope planting and other measures in

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accordance with Caltrans requirements should be fulfilled to provide adequate protection against short- and long-term erosion.

6.2 Secondary Seismic Hazards

Secondary hazards generally associated with severe ground shaking during an earthquake are ground rupture, tsunamis and seiches, landslides, rockfalls, ground fissuring, liquefaction, and seismic densification. These hazards are discussed below:

- *Seismic Densification:* We anticipate that the near-surface loose/soft alluvial deposits susceptible to such seismically induced settlement will be removed and recompacted during grading.
- *Liquefaction Settlement:* Due to shallow metamorphic rock and proposed remedial grading, it is our opinion that the potential for liquefaction is not a design issue.
- *Tsunamis and Seiches*: Due to the distance to large bodies of water (inland seas, large rivers, and oceans) from the site, the possibility of tsunamis is considered nil. The site is located within the Perris Reservoir Dam inundation zone. Flooding of the site is considered likely in the event of a seiche breaching the Perris Reservoir Dam.
- *Rock Falls*: The potential for rock fall due to either erosion or seismic ground shaking is considered very low or non-existent on this site.
- *Ground Rupture*: As shown on Figure 3, the site is not located with a state or county designated fault zone and therefore the potential for ground rupture is considered very low.

6.3 Slope Stability

Temporary excavations, including temporary shoring may be necessary to construct bents/piers/retaining walls/footings will need to be designed by the contractor for surficial and deep-seated stability, once the means and methods of construction are evaluated.

6.4 Excavation Characteristics

Based on our experience with similar soil, the onsite fill, alluvium and highly weathered bedrock should generally be excavatable with conventional earthmoving equipment. Excavation in the metamorphic rock should be expected to present moderate to very difficult ripping depending on depth of excavation. Oversized materials (i.e. greater than 6 inches) might be generated in deep cuts within the onsite rock.

6.5 Embankments

The proposed embankments will be composed of fill soil and vary up to 15 feet in height with graded side slopes varying from 4:1 (H:V) to as steep as 2:1 (H:V).

6.5.1 Embankment Foundations

All alluvial soils beneath new embankments should be over-excavated prior to placing new fill.

6.5.2 Embankment Soil

Based on our exploration, the surficial materials/alluvium generally consists of clayey soils. Due to very moist conditions and high expansion potential, these soils are considered not suitable for reuse as compacted fill.

6.6 Stability of Embankments and Subgrade Soil

Fill slopes as steep as 2:1 (H:V) are considered stable with respect to deep-seated failure. Slope inclinations up to 1.5:1 (H:V) along channel sides should be further evaluated and may require riprap protections. As such, slopes steeper than 2:1 should be analyzed for stability once slope design configurations are known.

6.7 Other Geologic Hazards

There are no other geologic hazards known at this time.

7.0 RECOMMENDATIONS AND CONCLUSIONS

7.1 Bridge Foundations

We understand that the proposed bridge will be supported on a system of shallow foundations bearing on metamorphic rock and/or properly placed engineered fill. If the planned depth of remedial earthwork is not feasible due to restricted habitat areas, deep foundations, such as driven piles or cast in drilled hole (CIDH) piles may be considered for this project. However, the scope of this report is to provide only design recommendations for conventional shallow foundations.

7.1.1 Response Spectra

A Caltrans design ARS curve was developed following Caltrans *Seismic Design Criteria* (2006b) and *Geotechnical Services Design Manual* (Caltrans, 2009). The ARS curve was generated using Caltrans ARS online program. The ARS curve/digitized values for the site for the 975-year return period are presented in table below.

TABLE 3. RECOMMENDED CALTRANS ARS CURVE/ SPECTRUM

Period (sec)	Spectral Acceleration (g)
0.01	0.51
0.05	0.84
0.1	1.03
0.15	1.13
0.2	1.21
0.25	1.13
0.3	1.08
0.4	0.91
0.5	0.80
0.6	0.72
0.7	0.67
0.85	0.61
1	0.56
1.2	0.47
1.5	0.38
2	0.29
3	0.19
4	0.13
5	0.11

7.1.2 Allowable Bearing Pressure

We understand that the bridge foundations will extend approximately 5 to about 18 feet below existing ground surface (BGS). As such, the footings are expected to be founded on metamorphic rock or on engineered fill compacted to a minimum of 95 percent relative compaction per ASTM 1557. As such, vertical allowable bearing pressures of 5,000 psf may be used for design of spread or continuous footings with a minimum width of 4 feet. The bearing pressure value may be increased by 500 psf for each additional foot of width to a maximum vertical bearing value of 8,000 psf.

7.1.3 Foundation Settlement

The total long-term service settlement for the proposed piers and abutments founded on metamorphic rock or a maximum of 5 feet of compacted fill as described above is estimated to be less than 0.5 inch. Differential settlement between the two piers or 30-foot horizontal distance along abutment is also expected to be less than 0.5 inch.

7.2 Embankments

Where right-of-way allows, embankment side slopes should be constructed at an inclination no steeper than 4:1 in accordance with Caltrans design requirements. However, in areas where space is constrained by limited right of way or other physical constraints, stable slopes are expected to be feasible from a geotechnical perspective with inclinations up to 2:1. Stable slopes protected by riprap are expected to be feasible from a geotechnical perspective with inclinations up to 1.5:1, but may require special handling such as select fill, or slope reinforcement.

The onsite surficial soil/alluvium (CL, CL-ML and SC) are generally very moist and possess high expansion and corrosion potential and as such, these materials are not considered suitable for reuse as compacted/engineered fill. Fill used to construct proposed embankments should conform to Caltrans Structure Backfill requirements found in Section 19-3.02C of the *Caltrans Standard Specifications*. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in thickness. In addition, the upper 2.5 feet of subgrade and base materials should be compacted in compliance with Section 19 of the Standard Specifications and Section 614.6 of HDM.

Imported soil/fill placed within the upper 2.5 feet of finished grade within paving areas should have a minimum R-value of 40 and should be non-corrosive and of low expansion. Other construction materials such as aggregates, asphalt, and Portland cement should be imported from local commercial sources. No potential sources for import soil or materials have been pre-tested for this project. Prior to import, the soil or materials should be tested by the Geotechnical Engineer. Slope stability evaluation should be performed when development plans become available.

In addition, slope faces are inherently subject to erosion, particularly if exposed to rainfall or irrigation. Landscaping and slope maintenance should be conducted as soon as possible in order to increase long-term surficial stability

7.3 Retaining Walls

7.3.1 General

If applicable to this project, there are two types of retaining walls that can be implemented per Caltrans Standard Drawings:

- **Type 1 Caltrans Reinforced Concrete**: For modest heights, particularly for fill ≤10 feet high
- **Mechanically Stabilized Earth (MSE)**: For fill zones with heights >12 feet

We recommend that Type 1 and MSE retaining walls be backfilled with non-corrosive and non-expansive silty sand soils, or imported sandy soils, and constructed with proper drainage. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on retaining walls, and are not recommended for MSE walls.

7.3.2 Retaining Wall Lateral Earth Pressures

Based on the recommendations above, the following geotechnical parameters may be used for preliminary design of retaining walls to the extent required for preliminary cost estimates, based on an ultimate shear strength friction-angle of 32 degrees:

TABLE 4. LATERAL EARTH PRESSURES

Drained Earth Pressure Conditions	Static Equivalent Fluid Pressure (pounds-per-cubic-foot)	
	Level Backfill	2:1 (horizontal:vertical) Sloped Backfill
Active (cantilever)	36	55
At-Rest (braced)	55	75
Passive	250 (allowable) (Maximum of 4,000 psf)	95 (allowable downslope direction)

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using active earth pressures. Rigid walls and walls braced at the top should be designed using at-rest earth pressures. Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.40 may be used at the concrete and soil interface for concrete poured/cast on undisturbed metamorphic rock, native sands, and properly compacted Caltrans Structure Backfill. Lateral passive resistance should be taken into account only if the soil providing passive resistance, against embedded shallow foundation elements, will remain intact with time (not erodible). These above values have already been reduced by a factor-of-safety of 1.5.

Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces. If hydrostatic conditions are anticipated, Leighton should be contacted to provide additional recommendations. MSE walls should be avoided in areas subject to flooding.

7.3.3 Retaining Wall Surcharges

In addition to the above lateral earth forces, surcharge due to improvements, such as an adjacent structure, and/or traffic loading should be considered in design of retaining walls. Loads applied within a 1:1 (horizontal:vertical) projection down from the surcharging structure on the stem of the wall should be considered in wall design. A third of uniform vertical surcharge-loads should be applied as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. For sliding and overturning analyses, soil unit weight of 120 pounds-per-cubic-foot (pcf) may be assumed for calculating density of properly compacted fill soil over wall footings.

At the discretion of the project Structural Engineer (SE), incremental seismic earth pressures of 23H pounds-per-cubic-foot (pcf), where H is the retaining wall stem height in feet, may be used in addition to earth and surcharge pressure presented above. Traditionally, this incremental seismic earth pressure has been applied as an inverted triangle (inverted equivalent fluid pressure), with the largest earth pressure occurring at the top of the wall. Resultant seismic earth pressure force has traditionally been applied at approximately 0.5H from the bottom of the wall, where H is the wall (stem) height. However, recent studies (Sitar, et. al.) suggest a uniform pressure distribution is likely closer to actual lateral seismic loads, so a uniform pressure of 11H applied as a uniform/rectangular pressure distribution can also be considered (based on current research and observations).

7.3.4 Retaining Wall Foundations

For retaining walls up to 16 feet tall, founded on compacted fill or metamorphic rock, footings should have a minimum width of 4 feet and a minimum embedment of 2 feet below the lowest adjacent grade. An allowable bearing capacity of 3,000 pounds-per-square-foot (psf) may be used for footing design, based on these minimum footing dimensions. This bearing value may be increased by 500 psf per foot increase in footing width or depth to a maximum allowable bearing pressure of 5,000 psf provided fill thickness below footings do not exceed 5 feet.

7.4 Site Preparation and Over-Excavation

Prior to earthwork, the areas that need to be cut, or receive fill and new pavement and bridge foundations, should be cleared and stripped of debris, deleterious material, organics, and vegetation. Cleared and grubbed material and rubble waste that may be encountered or created, should be removed and appropriately disposed of, in accordance with Sections 17-2 and 19-1 of the *Caltrans Standard Specifications* (Caltrans, 2015). Other material can be removed and delivered to an approved landfill. After clearing and grubbing, areas to receive compacted fill or foundations should be overexcavated to remove all alluvium and upper 1 foot of metamorphic rock. The overexcavation should extend horizontally a minimum distance from edges of new fills or foundations by projecting a 1:1 plane down and away from outer edges of fill/foundation elements to the depth of removal. Actual removal depths should be evaluated in the field by a representative of Leighton.

7.4.1 Approach Fill

Imported soil to be placed within the upper 2.5 feet of the roadway finished grade should have a low expansion potential (EI<51), a minimum R-value of 40, and should be non-corrosive. Class 3 aggregate subbase can be used for import within the upper 4 feet of finished grade.

The abutments should be backfilled in accordance with Caltrans *Standard Specifications*. Abutments should consist of soil relatively free of organic material and construction debris, with SE greater than 20, and grading requirements as presented in Section 19-3.02C of *Caltrans Standard Specifications* (Caltrans, 2015).

The slopes of the existing embankments should be benched into a minimum of 6 feet horizontally as the new fill is brought up in layers. Excavated soil should be recompacted along with the new embankment fill. Fill soil and placement should conform to Sections 19-6 and 19-7 of Caltrans *Standard Specifications*.

Due to the nature of sandy soil, settlement is expected to occur during or within a short period after placement of the embankment/approach fill. Based on our experience with similar soil and assumed new embankment loads we estimate that settlements will be on the order of 1 inch.

7.5 Rippability

Metamorphic rock is expected to be predominantly rippable to the anticipated removal depths. However, some areas of moderately to non-rippable rock may be encountered on the west side of the proposed bridge. Additionally, grading may generate oversize material requiring special handling.

7.6 Other Earthwork Considerations

7.6.1 Import Soil

If import soil is needed to fill below foundations and establish the site design elevations, it should be granular in nature, relatively free of organic material, have an expansion index less than 51 (per ASTM Test Method D4829), and have a low corrosion impact to the proposed improvements. Import soil, if needed, and potential borrow sites should be evaluated by the geotechnical consultant prior to being imported to the site.

7.6.2 Trench Excavation and Backfill

Utility trenches should be backfilled with compacted fill in accordance with the project specifications or Standard Specifications for Public Works Construction, ("Greenbook"), 2015 Edition. Fill soil should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D 1557). The upper 6 inches of backfill in pavement areas should be compacted to at least 95 percent relative compaction. Trench backfill within 150 feet of each bridge abutment should be compacted to at least 95 percent relative compaction in accordance with the Standard Specifications.

7.7 Soil Corrosivity

7.7.1 Concrete Corrosivity:

As a preliminary screening process for sulfate and chloride content in soils, we have performed laboratory tests on two representative surface soil-samples. As summarized in Table below, our laboratory test results indicated relatively high concentration of soluble sulfate and chloride in soils.

TABLE 5. CORROSION RESULTS

Sample Number	Sulfate Content (ppm)	Chloride Content (ppm)	Minimum Resistivity (ohm-cm)	pH
LB-1, B-1	5323	124	135	7.69
LB-6, B-1	2612	1146	440	7.43

Caltrans Corrosion Guidelines Section 6.1 (Caltrans, 2012) states that a site is considered to be corrosive to foundation elements or underground structures if one or more of the following conditions exist for the soil and/or water samples taken at the site:

- Chloride concentration greater than or equal to 500 ppm
- Sulfate concentration greater than or equal to 2,000 ppm
- pH of 5.5 or less

Based on the above, the onsite soils are considered corrosive. Thus, the concrete cover and mix design for bridge foundations and any proposed RCP culverts should follow Caltrans standard requirements for corrosive environment.

7.7.2 Ferrous Corrosivity:

Many factors can affect corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989), the relationship between soil resistivity and soil corrosiveness was developed as tabulated below:

TABLE 6. RELATIONSHIP BETWEEN SOIL RESISTIVITY AND SOIL CORROSIVITY

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Based on minimum-resistivity laboratory test results (Table 5), the onsite soil is considered very severely-corrosive to ferrous metals. Ferrous pipe can be protected by polyethylene bags, tape or coatings, di-electric fittings, concrete encasement or other means to separate the pipe from wet onsite clayey soils. Further testing of import and possibly site soil corrosivity could be performed and specific recommendations for corrosion protection may need to be provided by a qualified corrosion engineer.

8.0 OTHER CONSIDERATIONS

8.1 Temporary Excavations and Shoring

Excavations associated with construction may need shoring. Excavations during construction should be carried out in such a manner that failure and excessive ground movement do not occur. In general, unsupported slopes for temporary construction greater than 5 feet in height should be limited to a gradient of 1:1 (vertical to horizontal), or as field conditions dictate to provide a safe and stable slope. Surcharge loads from vehicles and stockpiled material should be kept away from the top of temporary excavations with a distance equal to at least one half of the excavation depth. Surface drainage should be controlled along the top of the temporary excavations to prevent excessive wetting and erosion of excavation faces. Where there is insufficient space for open excavations, shoring should be used to support the excavation.

Temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications, OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders (see: http://www.dir.ca.gov/title8/sb4a6.html). Contractors should be advised that sandy soil (such as fills generated from onsite alluvium) will primarily be encountered along the alignment, with sections of metamorphic rock. Fill and cohesionless alluvium should be classified as Type C soil.

The contractor must be responsible for providing a "competent person" as defined in Article 6 of the California Construction Safety Orders. During construction, exposed soil conditions should be regularly evaluated to check that conditions are as anticipated. Close coordination between their competent person and the geotechnical engineer of record should be maintained to facilitate construction while providing safe excavations.

Leighton

9.0 GEOTECHNICAL REVIEW

Geotechnical review is of paramount importance in engineering practice. The poor performances of many foundation and earthwork projects have been attributed to inadequate construction review by the geotechnical consultant. We recommend that Leighton be provided the opportunity to review geotechnical aspects of the project including the following:

9.1 Plan Review

Leighton should review the improvement plans prior to release for bidding and construction. Such review is necessary to evaluate whether the geotechnical recommendations have been effectively incorporated into the plans.

9.2 Construction Observation and Testing

It should be anticipated that the substrata exposed during grading may vary from that encountered in the previously excavated borings. Reasonably continuous geotechnical observation and testing during construction allows for evaluation of the actual soil conditions and the ability to provide appropriate revisions during grading, if required.

Site preparation, removal of unsuitable soil, testing of imported soil, fill placement and other site geotechnically-related operations should be observed and tested by the geotechnical consultant.

10.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, histories of occurrences, and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This evaluation was performed with the understanding that the subject site is proposed for bridge construction.

This report was prepared for Richland Communities based on their needs, directions, and requirements at the time of our evaluation. This report is not authorized for use by, and is not to be relied upon by any party except Richland Communities, and its successors and assigns as owner of the property, with whom Leighton and Associates, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton and Associates, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton and Associates, Inc.

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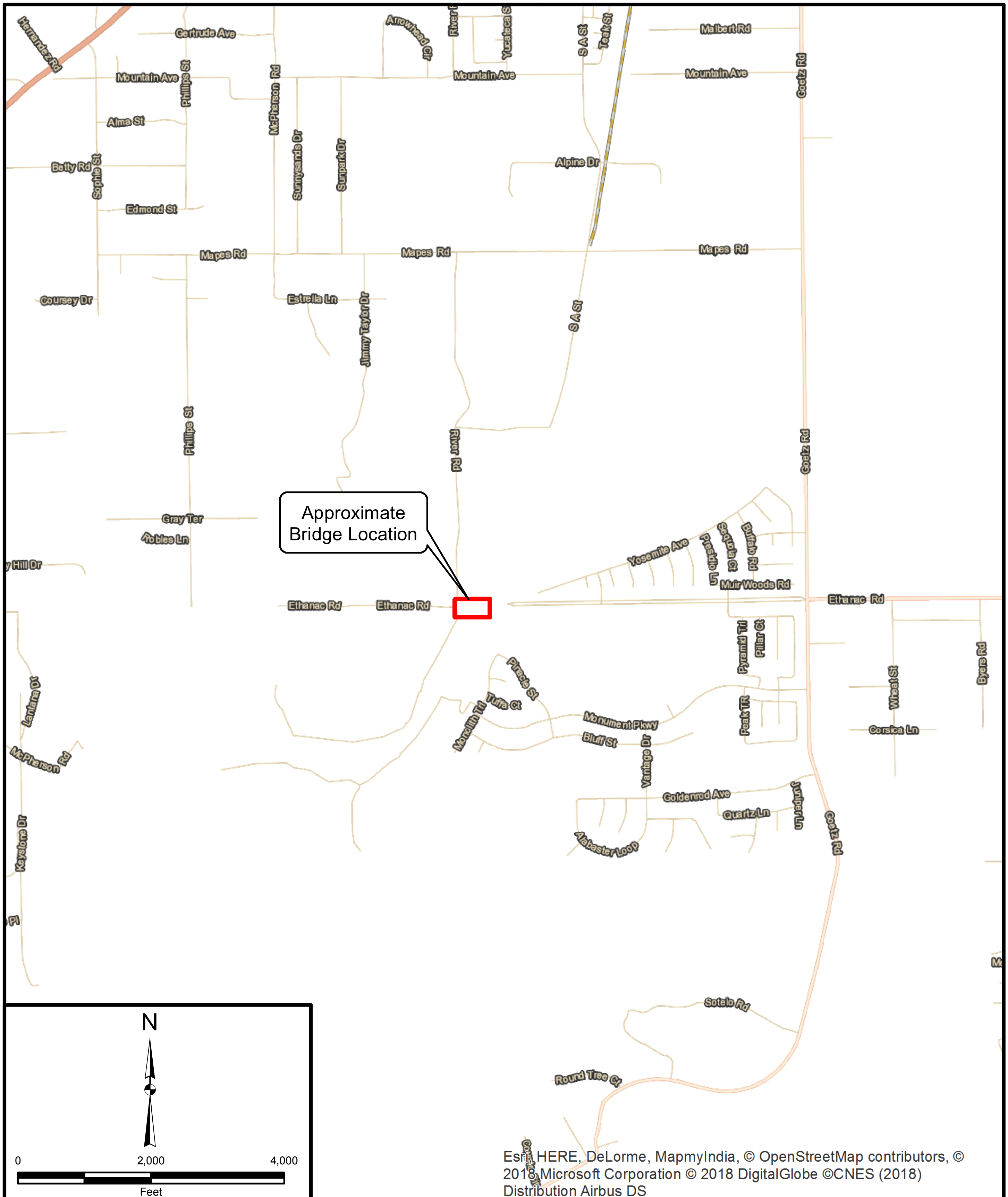


Project: 11127.003	Eng/Geol: SIS/RFR
Scale: 1 " = 2,000 '	Date: February 2018

Base Map: ESRI ArcGIS Online 2018
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

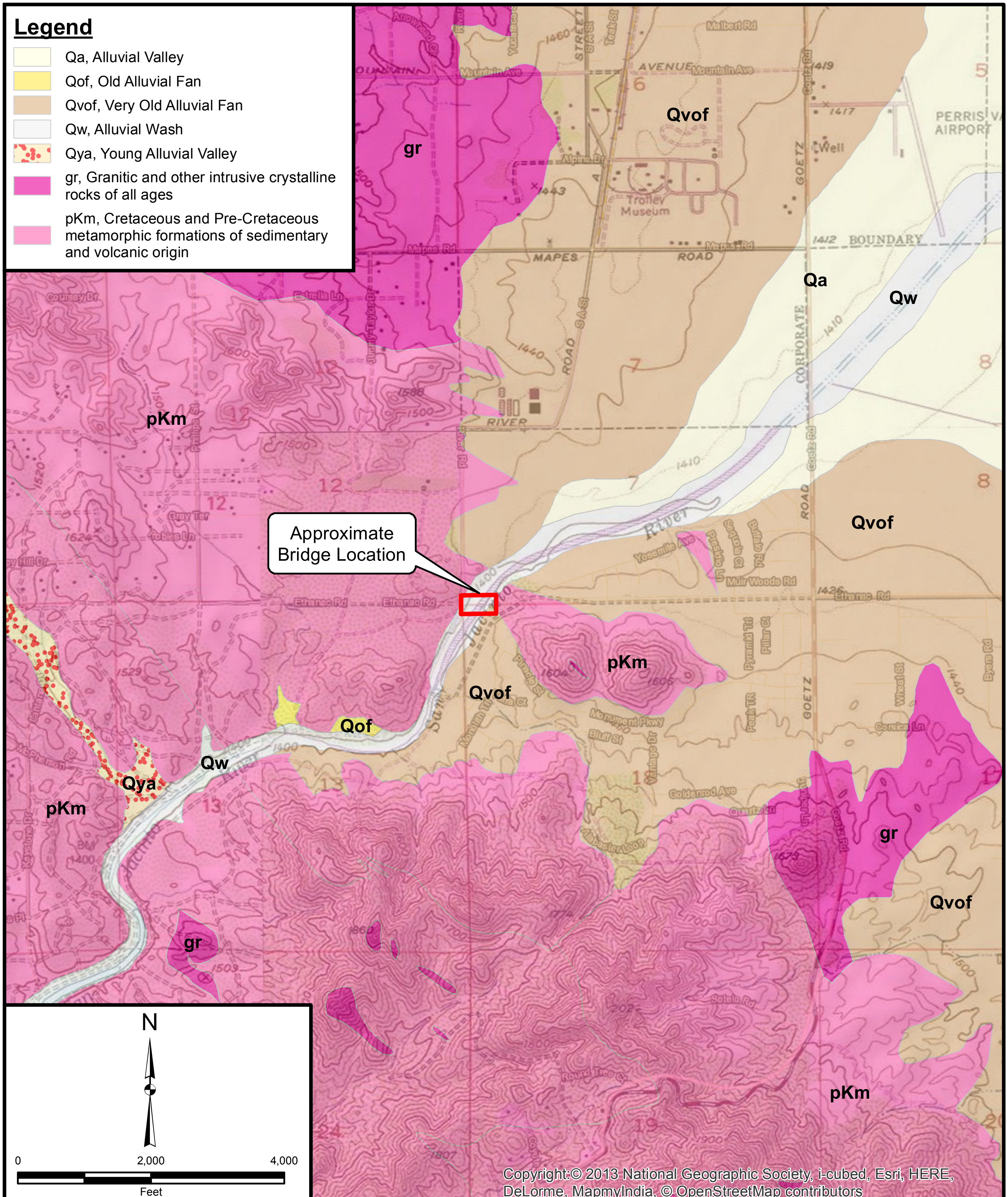
SITE LOCATION MAP

Proposed Ethanac Road Bridge Over San Jacnito River
Perris, California

Figure 1

Leighton

Map Saved as P:\Drafting\11127\003\Maps\11127-003-F01_SLM_2018-01-19.mxd on 1/19/2018 10:09:51 AM



Project: 11127.003	Eng/Geol: SIS/RFR
Scale: 1 " = 2,000 '	Date: February 2018

Base Map: ESRI ArcGIS Online 2018
Thematic Information: Preliminary Geology 2010, Leighton, USGS
Author: Leighton Geomatics (mmurphy)

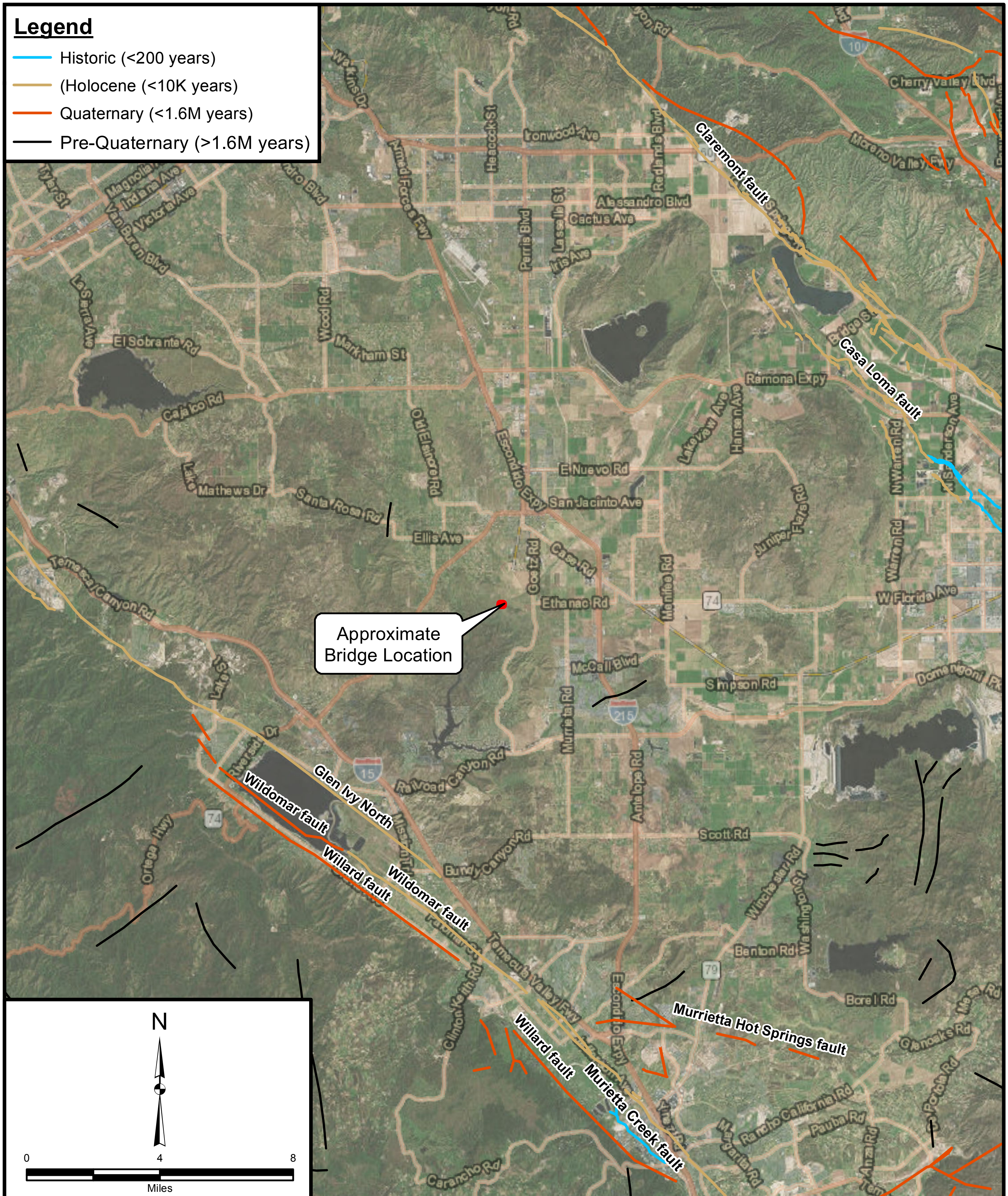
REGIONAL GEOLOGY MAP

Proposed Ethanac Road Bridge Over San Jacnito River
Perris, California

Figure 2

Leighton

Map Saved as P:\Drafting\11127\003\Maps\11227-003_F02_RGM_2018-01-19.mxd on 1/19/2018 10:24:02 AM



Project: 11127.003	Eng/Geol: SIS/RFR
Scale: 1 " = 4 miles	Date: February 2018

Base Map: ESRI ArcGIS Online 2018
Thematic Information: Leighton, CGS, Bryant 2010
Author: Leighton Geomatics (mmurphy)

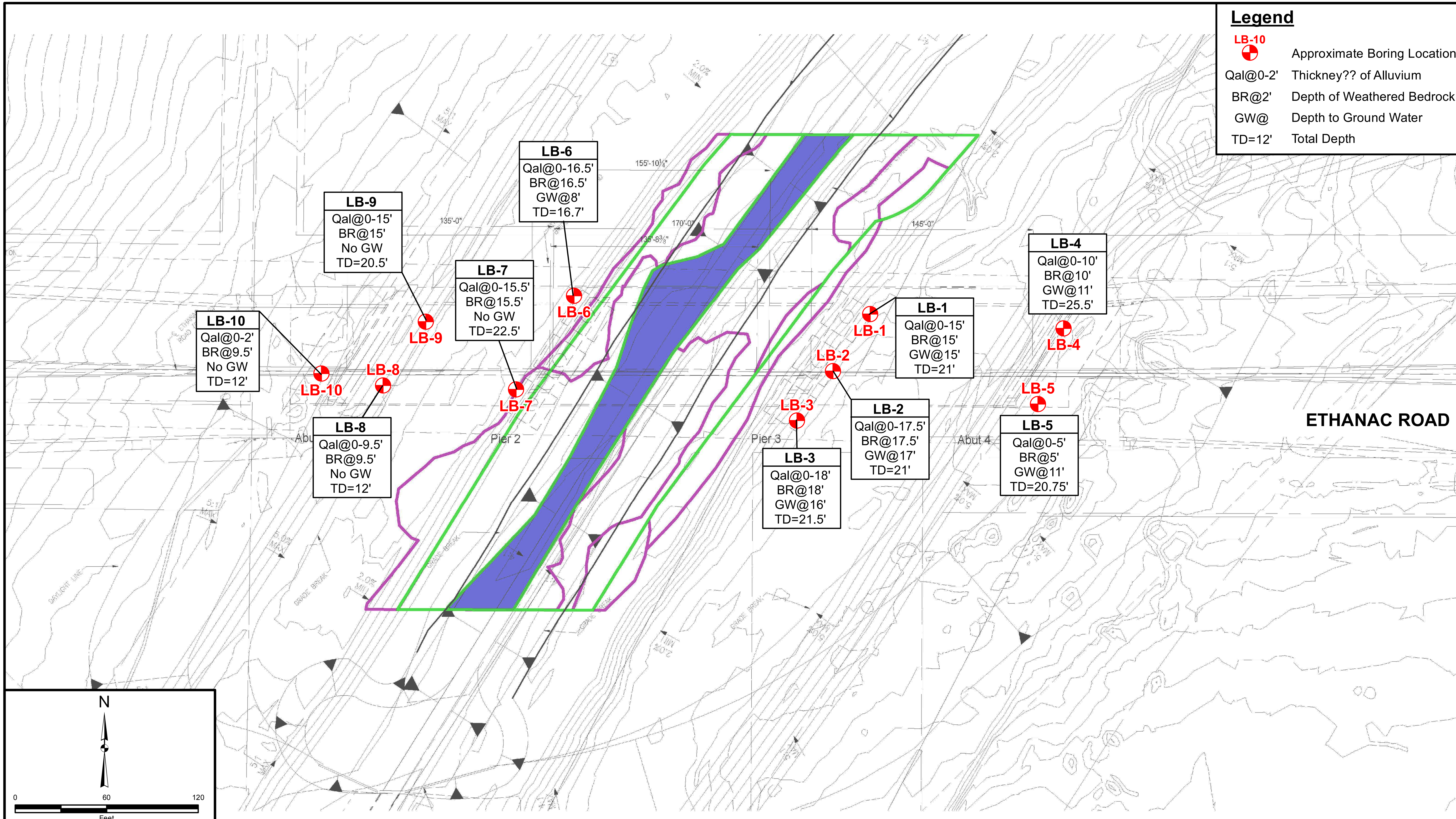
REGIONAL FAULT MAP

Proposed Ethanac Road Bridge Over San Jacnito River
Perris, California

Figure 3

Leighton

Map Saved as P:\Drafting\11127\003\Maps\11127-003_F03_RFM_2018-01-19.mxd on 1/19/2018 10:44:02 AM



Project: 11127.003	Eng/Geol: SIS/RFR
Scale: 1 " = 60 '	Date: February 2018

Base Map: Ethanac Road 450' Bridge Conceptual Layout by Albert A. Webb Associates Enkgineering Consultants.

Author: (mmurphy)

BORING LOCATION MAP

Ethanc Bridge
Menifee, California

Figure 4

Leighton

Map Saved as P:\Drafting\11127\003\Maps\11127-003_F04_BLM_2018-01-19.mxd on 2/5/2018 12:37:20 PM

APPENDIX A

FIELD EXPLORATION – LOGS OF EXPLORATIONS

The exploration logs included within this Appendix and related information depicts subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

GEOTECHNICAL BORING LOG LB-1

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1411'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
1410	0			B-1				CL-ML	**Quaternary Alluvium (Qal)**; Silty CLAY to clayey SILT, light brownish gray, dry to moist, trace fine to coarse grained sand with fine gravel, MD = 113.9 @ 17.8%	MD, CR
1405	5			R-1	3 7 14	92	31		very moist, abundant caliche	
								CL	Lean CLAY with SAND, dark grayish brown, moist, fine to medium grained sand	
1400	10			R-2	4 6 18	116	15	SC-SM	SILTY, CLAYEY SAND with GRAVEL, medium dense, dark gray, moist, fine to coarse grained sand	
1395	15 ▼			R-3	8 26 50/5"			SC-SM	**Bedrock (Mzg)**; Highly Weathered, recovered as: SILTY, CLAYEY SAND, dense, dark graysih brown, moist to wet, fine grained sand	DS
1390	20			S-1	50/3"			SM	Slightly Weathered, recovered as: SILTY SAND with GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel	
									Auger Refusal @ 21' Groundwater at 15' Backfilled with cuttings	
1385	25									
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-2

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1411'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
1410	0							SM	**Quaternary Alluvium (Qal)**; SILTY SAND with GRAVEL, light brownish gray, dry to moist, fine to coarse grianed sand with fine gravel	
								CL	SANDY Lean CLAY, light brownish gray, dry to moist, fine to medium grained sand	
1405	5			R-1	5 6 7	93	13	ML	SANDY SILT, medium stiff, dark grayish brown to brown, moist, fine grained sand	
								CL	SANDY Lean CLAY, dark grayish brown, moist, fine to medium grained sand	
1400	10			R-2	2 5 6	95	25		SANDY Lean CLAY, medium stiff, dark grayish brown, moist, fine to coarse grained sand	DS
1395	15			R-3	4 7 8	92	31		SANDY Lean CLAY, stiff, gray, moist to wet, fine grained sand	
									Bedrock (Mzq)	
1390	20			R-4	37 50/4"			SM	Moderately Weathered, recovered as: SILTY SAND with GRAVEL, dense, dark grayish brown, moist to wet, fine grained sand with fine angular gravel	
									Auger Refusal @ 21' Groundwater at 17' Backfilled with cuttings	
1385	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-3

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1412'
Location	See Boring Location Map	**Sampled By**	JTD

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
1412	0			B-1				SM	**Quaternary Alluvium (Qal)**; SILTY SAND with GRAVEL, light brownish gray, dry to moist, fine to coarse grained sand with gravel to 1.5"	
1410								SC-SM	SILTY, CLAYEY SAND, light gray, dry to moist, fine to medium grained sand	
	5			R-1	7 13 16	109	12		SILTY, CLAYEY SAND with GRAVEL, medium dense, light brown, moist, fine to coarse grained sand with gravel to 1"	
1405								SM	SILTY SAND, light brownish gray, moist, fine to medium grained sand	
	10			R-2	2 2 4	93	25	CL	SANDY Lean CLAY, medium stiff, dark grayish brown, moist, fine to coarse grained sand	
1400	15			R-3	5 7 11	91	31		Lean CLAY with SAND, stiff, dark grayish brown, moist, fine grained sand	
1395				R-4	30 50/4"			SW	Well-graded SAND with GRAVEL, dense, dark grayish brown, wet, fine to coarse grained sand, with gravel to 2"	
				S-1	10 9 11			SM	**Bedrock (Mzq)**; Highly Weathered, recovered as: SILTY SAND with GRAVEL, medium dense, dark grayish brown, moist to wet, fine to coarse grained sand with fine gravel	
	20			R-5	32 50/3"			SC-SM	Moderately Weathered, recovered as: SILTY, CLAYEY SAND with GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel, limited recovery	
1390									Auger Refusal @ 21.5' Groundwater at 16' Backfilled with cuttings	
1385	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-4

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1413'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	**Quaternary Alluvium (Qal)**; SILTY SAND with GRAVEL, dark grayish brown, dry to moist, fine to coarse grained sand with gravel, cobble and boulders to 60"	
1410								CL	SANDY Lean CLAY, dark grayish brown, moist, fine to medium grained sand	
	5			R-1	5 11 14	108	18	CL-ML	SILTY CLAY, stiff, yellowish brown, moist, abundant caliche stringers	
1405								CL	SANDY Lean CLAY, yellowish brown to olive brown, moist, fine to medium grained sand	
	10			R-2 B-1	20 30 31	123	13	SM	**Bedrock (Mzg)**; Highly Weathered, recovered as: SILTY, CLAYEY SAND, dense, dark brown and dark grayish brown, fine to medium grained sand, few fine gravel, MD = 126.3 @ 9.3%	MD
1400										
	15			R-3	30 50/4"			SM	Moderately Weathered, recovered as: SILTY SAND with GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel	
1395										
	20			R-4	50/6"			SW-SM	Slightly Weathered, recovered as: Well-graded SAND with SILT and GRAVEL, dense, dark grayish brown, moist to wet, fine to coarse grained sand with fine gravel	
1390										
	25			S-1	50/5"				As Above	
									Drilled to 25.42' Sampled to 25.42' Groundwater at 11' Backfilled with cuttings	
1385										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-5

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1413'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**Quaternary Alluvium (Qal)**; SILTY SAND with GRAVEL, dark grayish brown, moist, fine to coarse grained sand with gravel to 1"	
1410								SC-SM	SILTY, CLAYEY SAND, dark brown, moist, fine to medium grained sand	
	5			R-1	11 15 26	109	12	SM	**Bedrock (Mzg)**; Highly Weathered, recovered as: SILTY SAND, medium dense, dark grayish brown, moist, fine grained sand	
1405									Recovered as: SILTY SAND, oliver brown, moist, fine to medium grained sand	
	10			R-2	43 50/3"	119	6	SW-SM	Slightly Weathered, recovered as: Well-graded SAND with SILT and GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel	
1400										
	15			R-3	50/6"				as above, wet	
1395										
	20			S-1	44 50/3"				as above	
									Drilled to 20.75' Sampled to 20.75' Groundwater at 11' Backfilled with cuttings	
1390										
	25									
1385										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-6

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1410'
Location	See Boring Location Map	**Sampled By**	JTD

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
1410	0			B-1				SC	**Quaternary Alluvium (Qal)**; CLAYEY SAND, light brownish gray, dry to moist, fine to coarse grained sand	CR
								CL	SANDY Lean CLAY, gray, moist, fine to medium grained sand	
1405	5			R-1	3 6 10	95	29		Lean CLAY, stiff, dark gray, moist	
									Lean CLAY, dark gray, moist	
1400	10			R-2	3 5 6	96	28		Lean CLAY, stiff, grayish brown, moist, manganese oxide and iron oxide staining	
1395	15			R-3	Push Push Push			SW-SC	Well-graded SAND with CLAY (or SILTY CLAY), loose, gray, wet, fine to coarse grained sand	
				S-1	50/2"			GW	**Bedrock (Mzq)**; Slightly Weathered to Fresh, recovered as two small gravel in sampler	
									Auger Refusal @ 16.67' Groundwater at 8.16' Backfilled with cuttings	
1390	20									
1385	25									
1380	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-7

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1409'
Location	See Boring Location Map	**Sampled By**	JTD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	**Quaternary Alluvium (Qal)**; CLAYEY SAND, light brownish gray, fine to coarse grained sand	
1405								CL	SANDY Lean CLAY, dark grayish brown, moist, fine to medium grained sand	
	5			R-1	7 11 11	99	25		Lean CLAY, stiff, dark grayish brown, moist, few caliche stringers	
1400									SANDY Lean CLAY, dark grayish brown, moist, fine to medium grained sand	
	10			R-2	3 5 8	102	23	SC-SM	SILTY, CLAYEY SAND, loose, gray, moist to wet, fine grained sand	
1395	15			R-3	7 50/5"			SP-SM	Poorly graded SAND with SILT, medium dense, light gray, moist to wet, fine grained sand	
								SW-SM	**Bedrock (Mzq)**; Moderately Weathered, recovered as: SILTY SAND with GRAVEL, dense, dark gray, moist to wet, fine to coarse grained sand with fine gravel	
1390	20			R-4	39 50/3"				Slightly Weathered, recovered as: Well-graded SAND with SILT and GRAVEL, dense, very dark gray, moist to wet, fine to coarse grained sand with fine gravel	
1385	25								Auger Refusal @ 22.5' Groundwater at 8.5' Backfilled with cuttings	
1380	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-8

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1409'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**Quaternary Alluvium (Qal)**; Lean CLAY with SAND, dark gray, moist, fine to coarse grained sand, EI = 103	EI
1405									SANDY Lean CLAY, dark grayish brown, moist to wet, fine to medium grained sand	
	5			R-1	5 8 12	98	28		Lean CLAY, stiff, dark grayish brown, moist, abundant caliche stringers	
1400									SANDY Lean CLAY, grayish brown, moist, fine to medium grained sand	
	10			R-2	50/5"	107	10	SC	**Bedrock (Mzg)**; Moderately Weathered, recovered as: CLAYEY SAND with GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel	
1395									Auger Refusal @ 12' Groundwater not encountered Backfilled with cuttings	
	15									
1390										
	20									
1385										
	25									
1380										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-9

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1409'
Location	See Boring Location Map	**Sampled By**	JTD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							CL	**Quaternary Alluvium (Qal)**; SANDY Lean CLAY, dark brown, moist, fine to coarse grained sand	
									SANDY Lean CLAY, gray, moist, fine to medium grained sand	
1405										
	5			R-1	5 11 14	95	23		Lean CLAY, stiff, dark grayish brown, moist, few caliche stringers	
									Lean CLAY, dark gray, moist	
1400										
	10			R-2	3 4 5	93	30		SANDY Lean CLAY, medium stiff, dark gray, moist, fine grained sand	CN
1395										
	15			R-3	50/5"			SM	**Bedrock (Mzg)**; Moderately Weathered, recovered as: SILTY SAND with GRAVEL, dense, dark grayish brown, moist, fine to coarse grained sand with fine gravel	
1390										
	20			S-1	50/4"			GW	Slightly Weathered to Fresh, recovered as: 2-1" gravel	
									Auger Refusal @ 20.5' Groundwater not encountered Backfilled with cuttings	
1385										
	25									
1380										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-10

Project No.	11127.003	**Date Drilled**	1-17-18
Project	Ethanac Road Bridge	**Logged By**	JTD
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1410'
Location	See Boring Location Map	**Sampled By**	JTD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
1410	0							SM	**Quaternary Alluvium (Qal)**; CLAYEY SAND, brown, dry to moist, fine to coarse grained sand	
								SM	**Bedrock (Mzq)**; Highly Weathered, recovered as: SILTY SAND with GRAVEL, light brownish gray, dry to moist, fine to coarse grained sand	
1405	5			R-1	42 50/3"	117	3		Moderately Weathered, recovered as: SILTY SAND with GRAVEL, dense, fine to coarse grained sand	
									As Above	
1400	10			R-2	50/5"				no recovery	
									Auger Refusal @ 12' Groundwater not encountered Backfilled with cuttings	
1395	15									
1390	20									
1385	25									
1380	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

APPENDIX B

RESULTS OF GEOTECHNICAL LABORATORY TESTING

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Leighton

Project Name:	RC Ethanac Rd Bridge	Tested By :	M. Vinet	Date:	02/06/18
Project No. :	11127.003	Data Input By:	M. Vinet	Date:	02/08/18
Boring No.:	N/A	Checked By:	M. Vinet	Date:	02/08/18
Sample No.:	S-1	Depth (ft.) :	0 - 1.0		

Visual Sample Description: Silty, Clay (CL-ML), Light Brown.

				Hygroscopic Moisture Content of Soils Passing #10	Corrected Weight of Air-Dry Soil Passing #10	After Hydrometer & wet sieve ret. on #200 sieve
Liquid Limit:	N/A	LL,PL,PI:	N/A			
Plastic Limit:	N/A	GR:SA:FI:	0:8:92			
Plasticity Index:	N/A	Grp. Symbol:	CL-ML			
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(gm.)		10.00	**	**
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (gm.)		10.00	52.40	720.80
Wt.of Air-Dry Soil + Cont. (gm.)	200.0	Wt. of Container No.___ (gm.)		0.00	**	716.20
Wt. of Container	0.0	Moisture Content (%)		0.0	**	**
Dry Wt. of Soil (gm.)	200.00	Wt. of Dry Soil (gm.)		10.00	52.40	4.60

Coarse Sieve

U.S. Sieve Size	Cumulative Wt.of Dry Soil Retained(gm)	% Passing
3"	0.0	100.0
1½"	0.0	100.0
3/4"	0.0	100.0
3/8"	0.0	100.0
No. 4	0.0	100.0
No. 10	0.0	100.0
Pan		

Sieve after Hydrometer & Wet Sieve

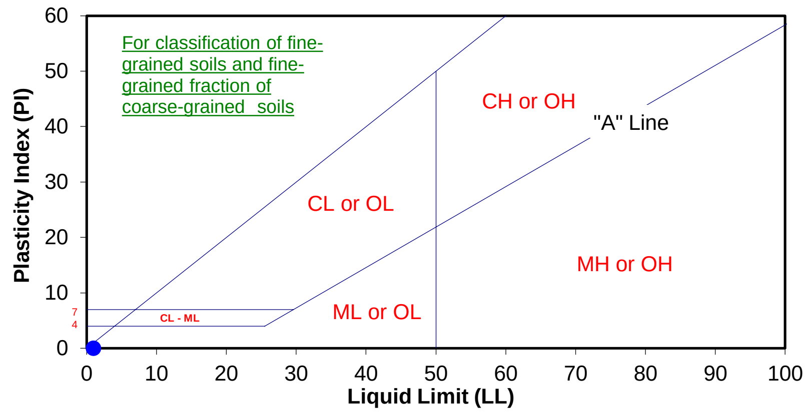
U.S. Sieve Size	Cumulative Wt. of Dry Soil Retained (gm)	% Passing	% Total Sample
No. 10	0.0	100.0	100.0
No. 20	0.2	99.6	99.6
No. 40	0.5	99.0	99.0
No. 60	0.9	98.3	98.3
No. 100	1.9	96.4	96.4
No. 200	4.2	92.0	92.0
Pan			

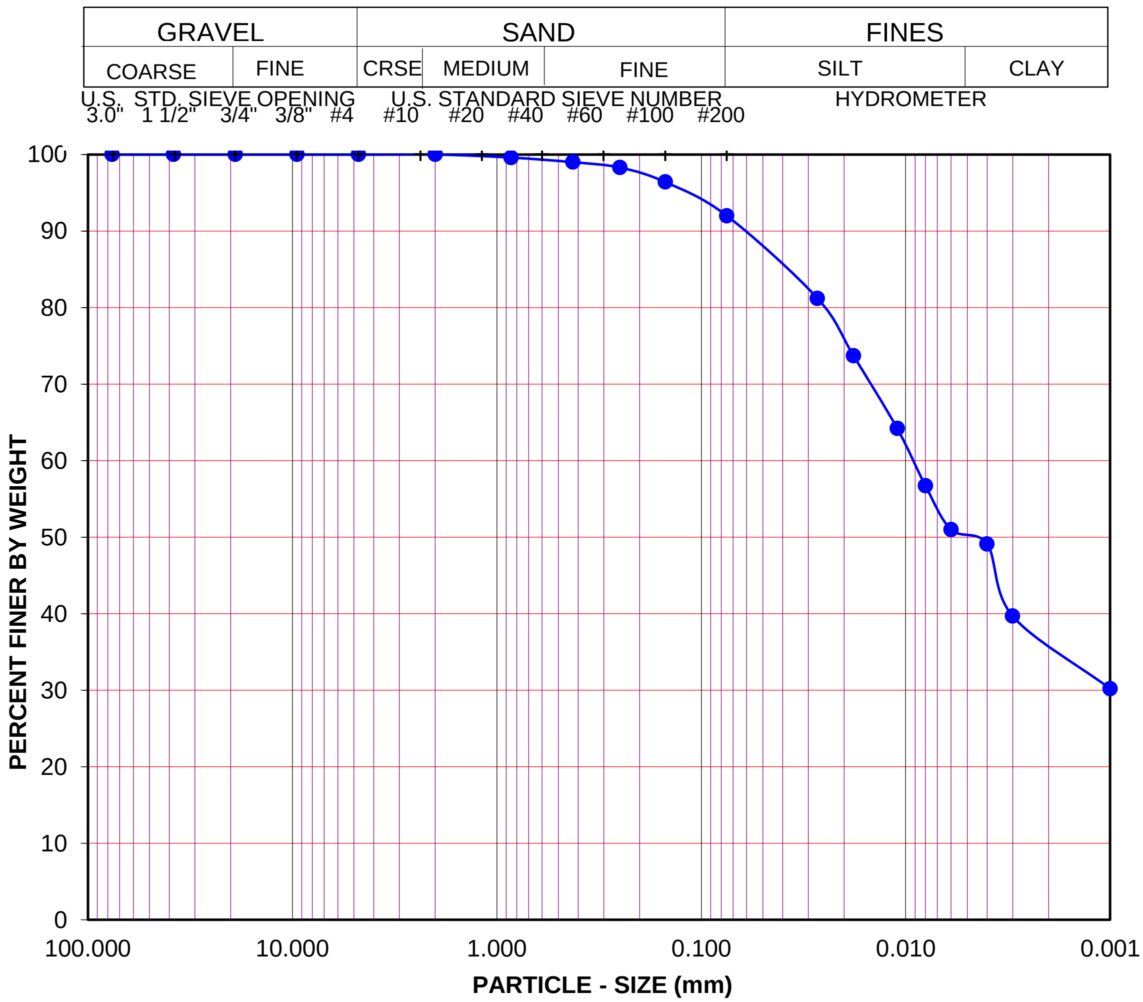
Hydrometer

Wt. of Air-Dry Soil (gm) 52.4 Wt. of Dry Soil (gm) 52.4

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°c)	Composite Correction 152 H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
2/6/18	7:20	0	21	5.0			
	7:22	2	21	5.0	48.0	81.2	0.027
	7:25	5	21	5.0	44.0	73.7	0.018
	7:35	15	21	5.0	39.0	64.2	0.011
	7:50	30	21	5.0	35.0	56.7	0.008
	8:20	60	21	5.0	32.0	51.0	0.006
	9:20	120	21	5.0	31.0	49.1	0.004
	11:30	250	21	5.0	26.0	39.7	0.003
2/7/18	7:20	1440	21	5.0	21.0	30.2	0.001





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
N/A	S-1	0 - 1.0	CL-ML	0:8:92	N/A

Sample Description:

Silty, Clay (CL-ML), Light Brown.

Leighton

Project No.: 11127.003

RC Ethanac Rd Bridge

ATTERBERG LIMITS, PARTICLE - SIZE CURVE

ASTM D 4318, D 422

Rev. 08-04

Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name:	RC Ethanac Rd Bridge	Tested By :	M. Vinet	Date:	02/06/18
Project No. :	11127.003	Data Input By:	M. Vinet	Date:	02/08/18
Boring No.:	N/A	Checked By:	M. Vinet	Date:	02/08/18
Sample No.:	S-2	Depth (ft.) :	0 - 1.0		

Visual Sample Description: Lean Clay (CL), Light Brown.

				Hygroscopic Moisture Content of Soils Passing #10	Corrected Weight of Air-Dry Soil Passing #10	After Hydrometer & wet sieve ret. on #200 sieve
Liquid Limit:	N/A	LL,PL,PI:	N/A			
Plastic Limit:	N/A	GR:SA:FI:	0:8:92			
Plasticity Index:	N/A	Grp. Symbol:	CL			
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(gm.)		10.00	**	**
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (gm.)		10.00	51.40	704.30
Wt.of Air-Dry Soil + Cont. (gm.)	97.4	Wt. of Container No.___ (gm.)		0.00	**	699.90
Wt. of Container	0.0	Moisture Content (%)		0.0	**	**
Dry Wt. of Soil (gm.)	97.40	Wt. of Dry Soil (gm.)		10.00	51.40	4.40

Coarse Sieve

U.S. Sieve Size	Cumulative Wt.of Dry Soil Retained(gm)	% Passing
3"	0.0	100.0
1½"	0.0	100.0
3/4"	0.0	100.0
3/8"	0.0	100.0
No. 4	0.0	100.0
No. 10	0.0	100.0
Pan		

Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. of Dry Soil Retained (gm)	% Passing	% Total Sample
No. 10	0.0	100.0	100.0
No. 20	0.1	99.8	99.8
No. 40	0.4	99.2	99.2
No. 60	0.9	98.2	98.2
No. 100	2.3	95.5	95.5
No. 200	3.9	92.4	92.4
Pan			

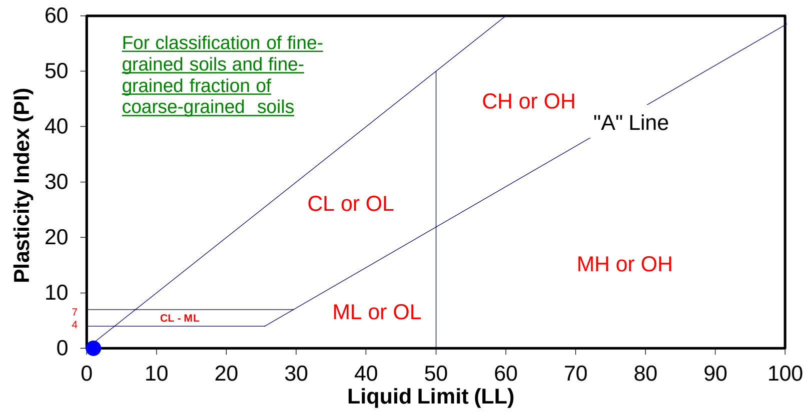
Hydrometer

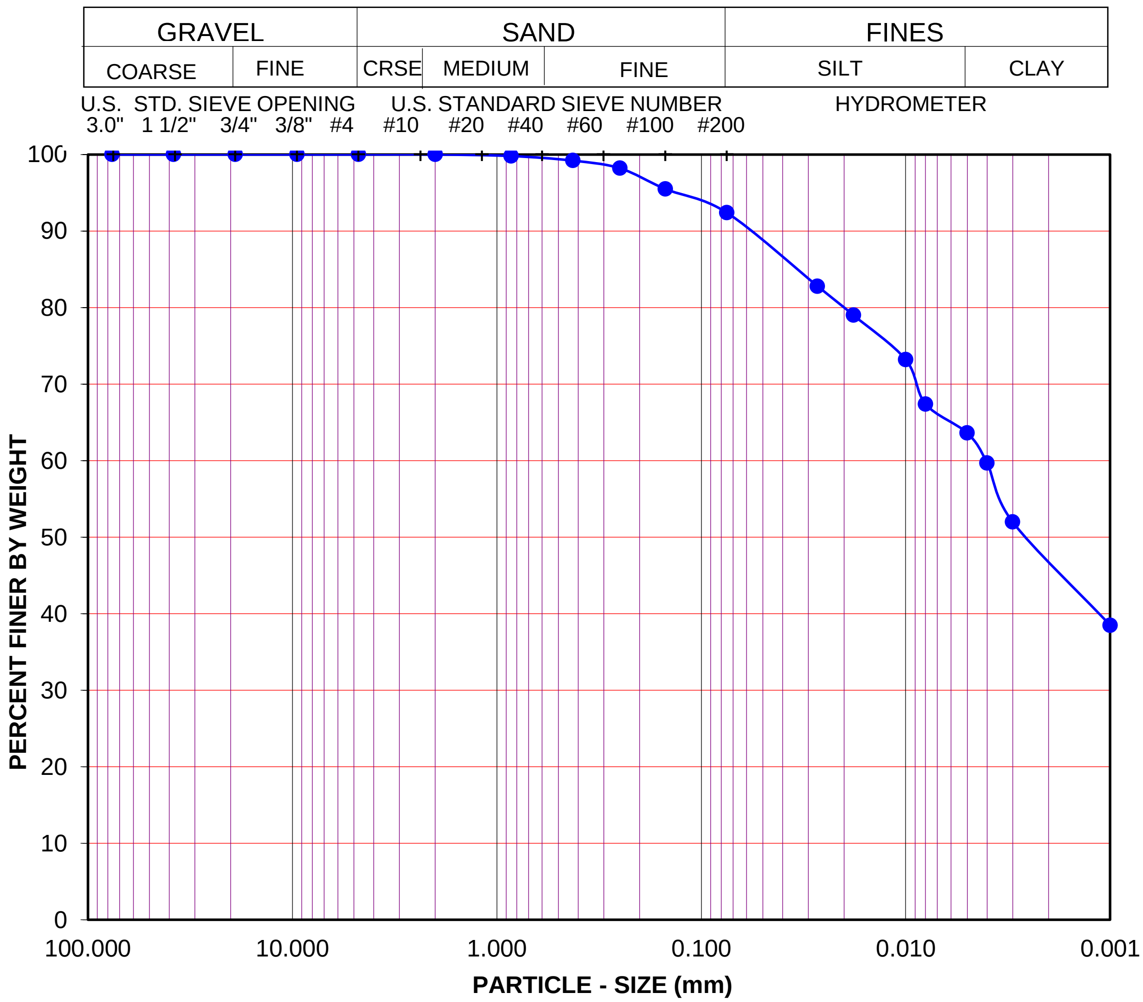
Wt. of Air-Dry Soil (gm) 51.4

Wt. of Dry Soil (gm) 51.4

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°c)	Composite Correction 152 H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
2/6/18	7:27	0	21	5.0			
	7:29	2	21	5.0	48.0	82.8	0.027
	7:32	5	21	5.0	46.0	79.0	0.018
	7:42	15	21	5.0	43.0	73.2	0.010
	7:57	30	21	5.0	40.0	67.4	0.008
	8:27	60	21	5.0	38.0	63.6	0.005
	9:27	120	21	5.0	36.0	59.7	0.004
	11:37	250	21	5.0	32.0	52.0	0.003
2/7/18	7:27	1440	21	5.0	25.0	38.5	0.001





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
N/A	S-2	0 - 1.0	CL	0:8:92	N/A

Sample Description:
Lean Clay (CL), Light Brown.

Project No.: 11127.003
RC Ethanac Rd Bridge

Leighton

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Rev. 08-04

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	RC Ethanac Rd Bridge	Tested By:	F. Mina	Date:	01/30/18
Project No.:	11127.003	Input By:	M. Vinet	Date:	02/05/18
Boring No.:	LB-1	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Clay (CL-ML), Dark Brown.				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5442	5570	5540			
Weight of Mold (g)	3542	3542	3542			
Net Weight of Soil (g)	1900	2028	1998			
Wet Weight of Soil + Cont. (g)	2058.1	2175.2	2148.8			
Dry Weight of Soil + Cont. (g)	1803.1	1872.3	1821.1			
Weight of Container (g)	159.1	152.2	157.8			
Moisture Content (%)	15.5	17.6	19.7			
Wet Density (pcf)	125.4	133.9	131.9			
Dry Density (pcf)	108.6	113.8	110.2			

Maximum Dry Density (pcf) 113.9 **Optimum Moisture Content (%)** 17.8

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

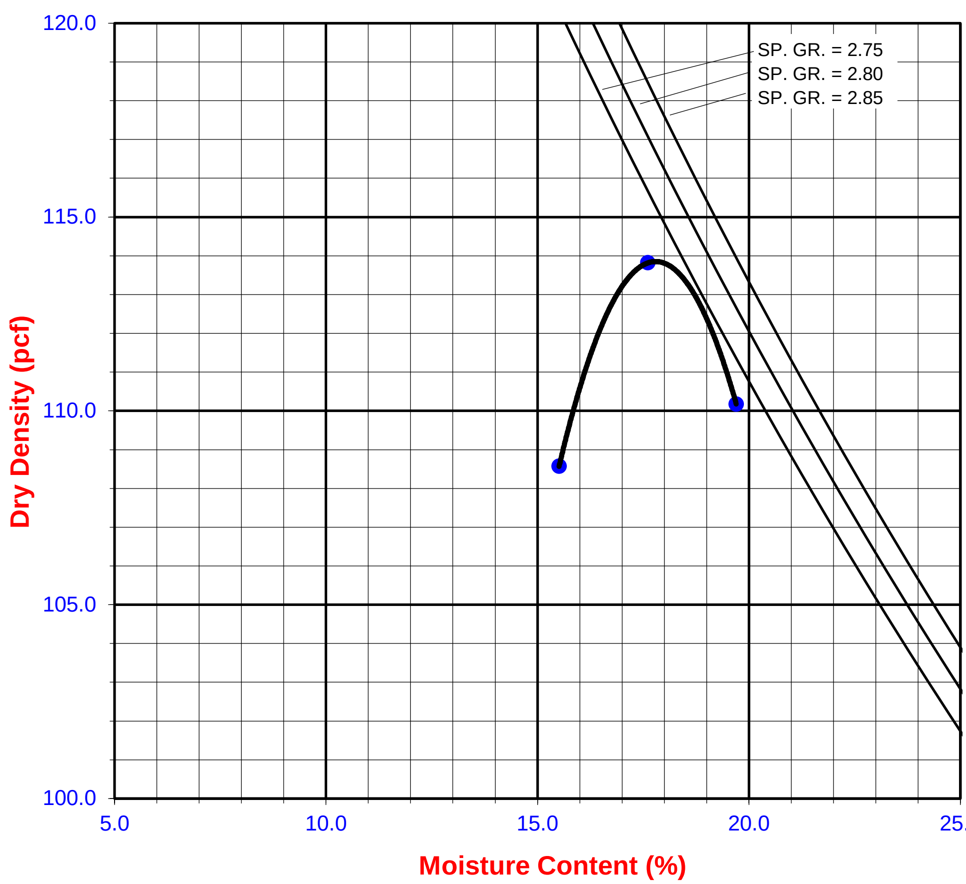
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	RC Ethanac Rd Bridge	Tested By:	F. Mina	Date:	01/30/18
Project No.:	11127.003	Input By:	M. Vinet	Date:	02/05/18
Boring No.:	LB-4	Depth (ft.):	10.0 - 15.0		
Sample No.:	B-1				
Soil Identification:	Silty, Clayey Sand (SC-SM), Dark Yellowish Brown.				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5550	5627	5644	5577		
Weight of Mold (g)	3542	3542	3542	3542		
Net Weight of Soil (g)	2008	2085	2102	2035		
Wet Weight of Soil + Cont. (g)	2452.2	2783.8	2311.5	2235.7		
Dry Weight of Soil + Cont. (g)	2332.0	2611.6	2098.1	1987.8		
Weight of Container (g)	619.8	700.0	215.2	218.1		
Moisture Content (%)	7.0	9.0	11.3	14.0		
Wet Density (pcf)	132.5	137.6	138.7	134.3		
Dry Density (pcf)	123.8	126.2	124.6	117.8		

Maximum Dry Density (pcf) **126.3** **Optimum Moisture Content (%)** **9.3**

PROCEDURE USED

[ ] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[X] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

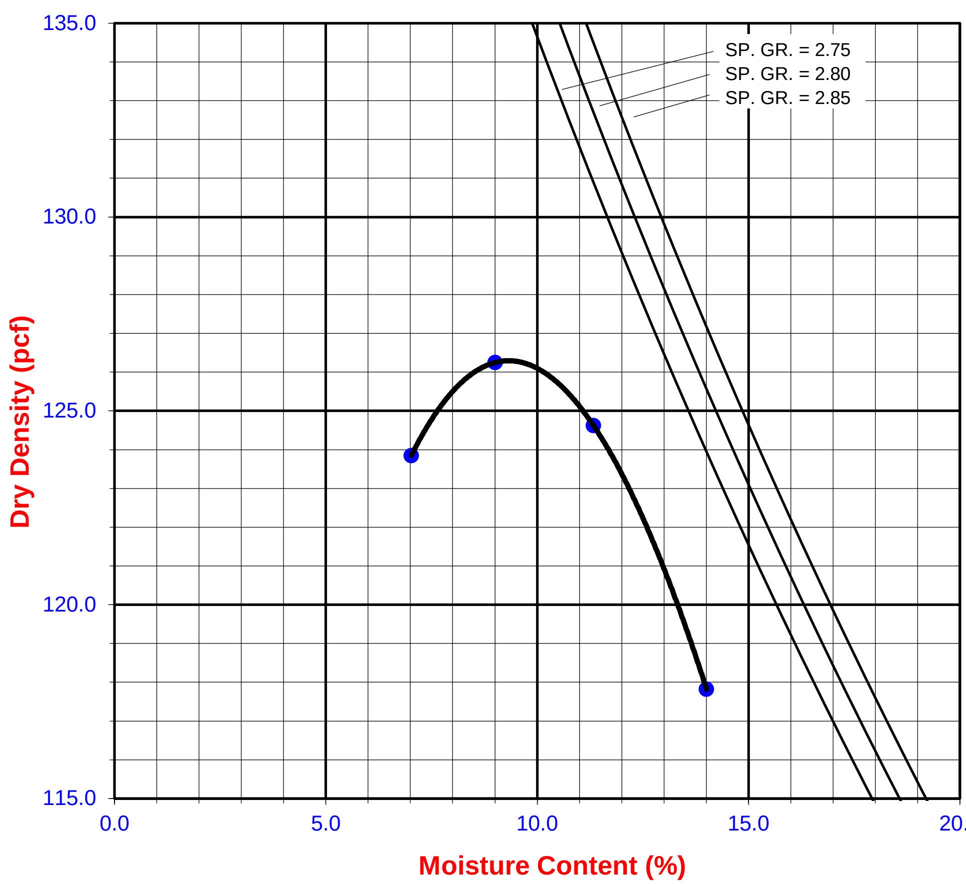
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	RC Ethanac Rd Bridge	Tested By:	F. Mina	Date:	01/30/18
Project No.:	11127.003	Input By:	M. Vinet	Date:	02/05/18
Boring No.:	LB-6	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Lean Clay (CL), Dark Brown.				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5491	5556	5526			
Weight of Mold (g)	3542	3542	3542			
Net Weight of Soil (g)	1949	2014	1984			
Wet Weight of Soil + Cont. (g)	2110.3	2183.6	2121.5			
Dry Weight of Soil + Cont. (g)	1855.0	1890.4	1780.5			
Weight of Container (g)	163.1	171.0	14.3			
Moisture Content (%)	15.1	17.1	19.3			
Wet Density (pcf)	128.6	132.9	131.0			
Dry Density (pcf)	111.8	113.6	109.8			

Maximum Dry Density (pcf) **113.7** **Optimum Moisture Content (%)** **17.0**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

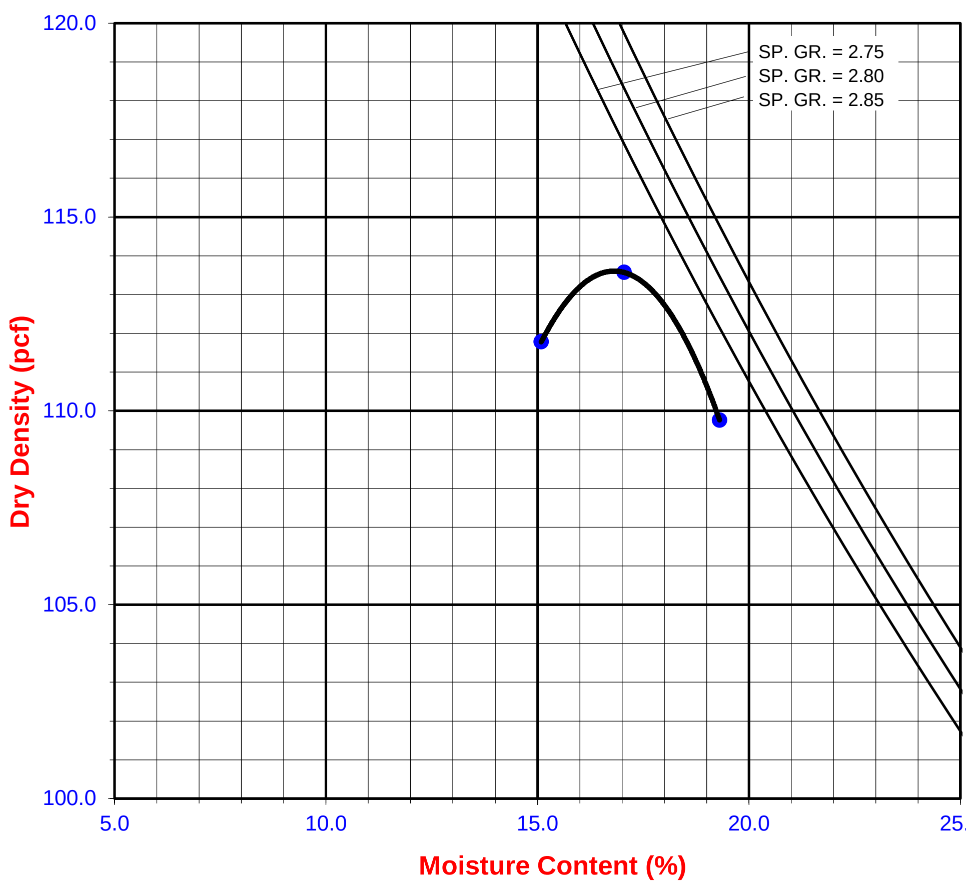
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

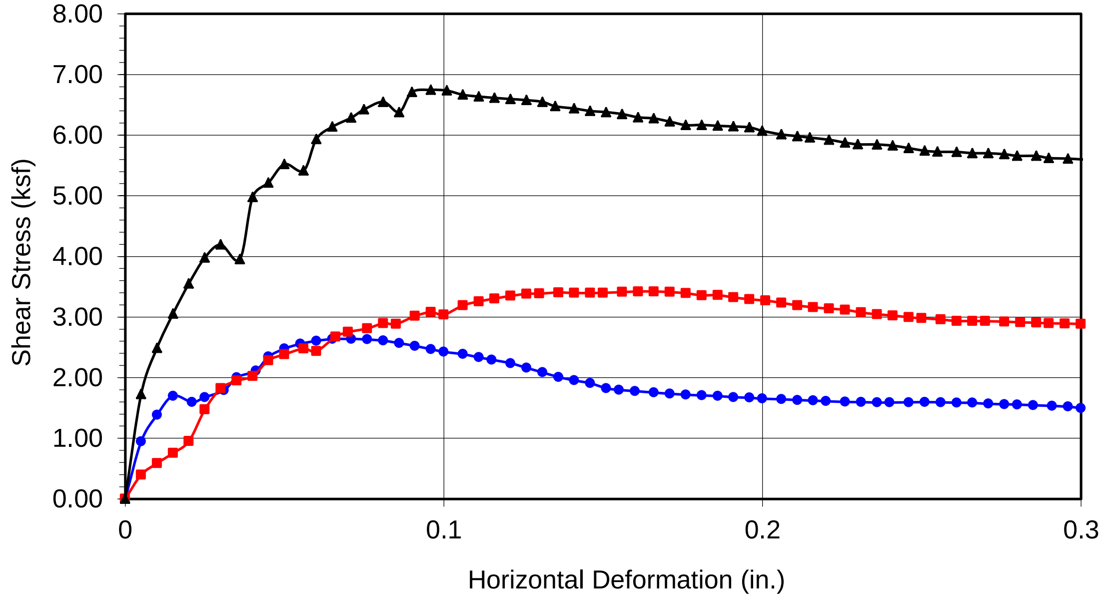
Particle-Size Distribution:

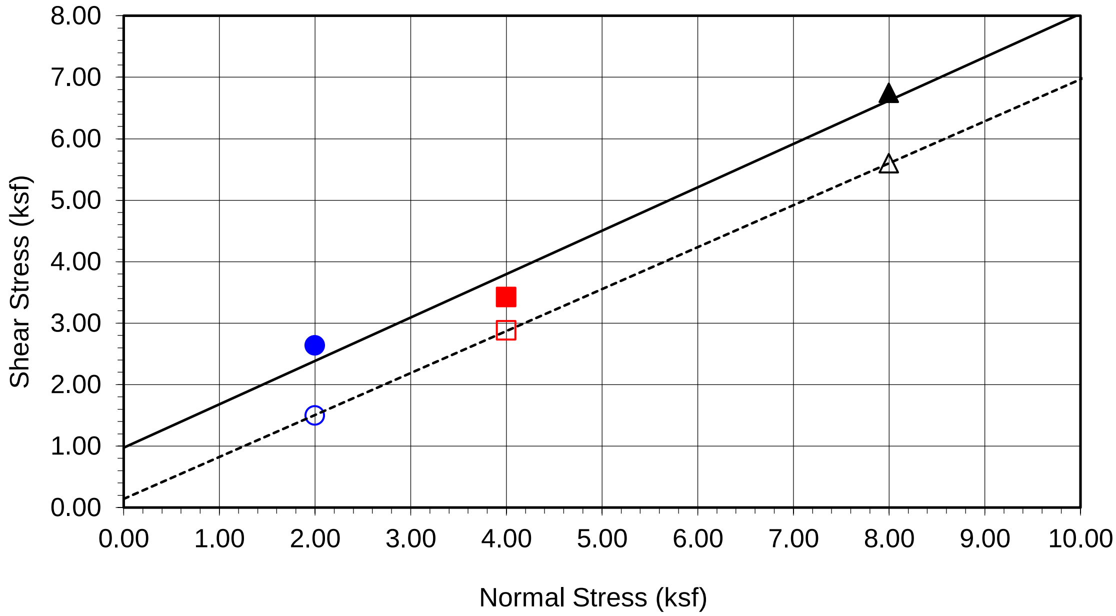
GR:SA:FI

Atterberg Limits:

LL,PL,PI







Boring No.	**LB-1**
Sample No.	**R-3**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:
Silty, Clayey Sand (SC-SM), Dark Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	976	35
Ultimate	141	34

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.639	■ 3.421	▲ 6.747
Shear Stress @ End of Test (ksf)	○ 1.498	□ 2.884	△ 5.598
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	15.29	15.29	15.29
Dry Density (pcf)	116.1	116.7	117.9
Saturation (%)	91.5	92.8	96.0
Soil Height Before Shearing (in.)	0.9863	0.9721	0.9571
Final Moisture Content (%)	19.5	17.7	16.8

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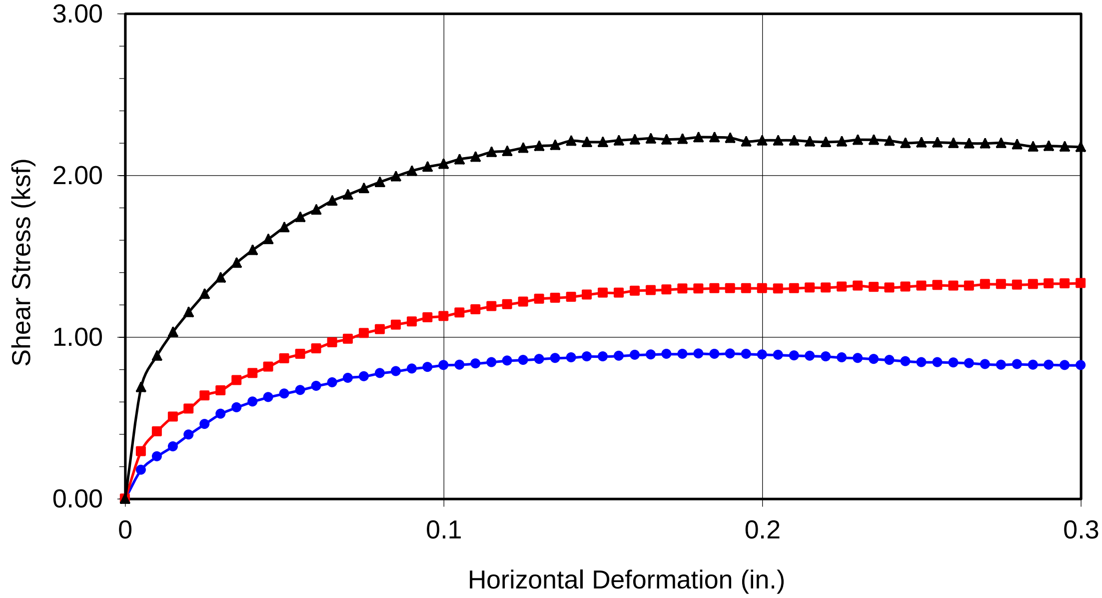
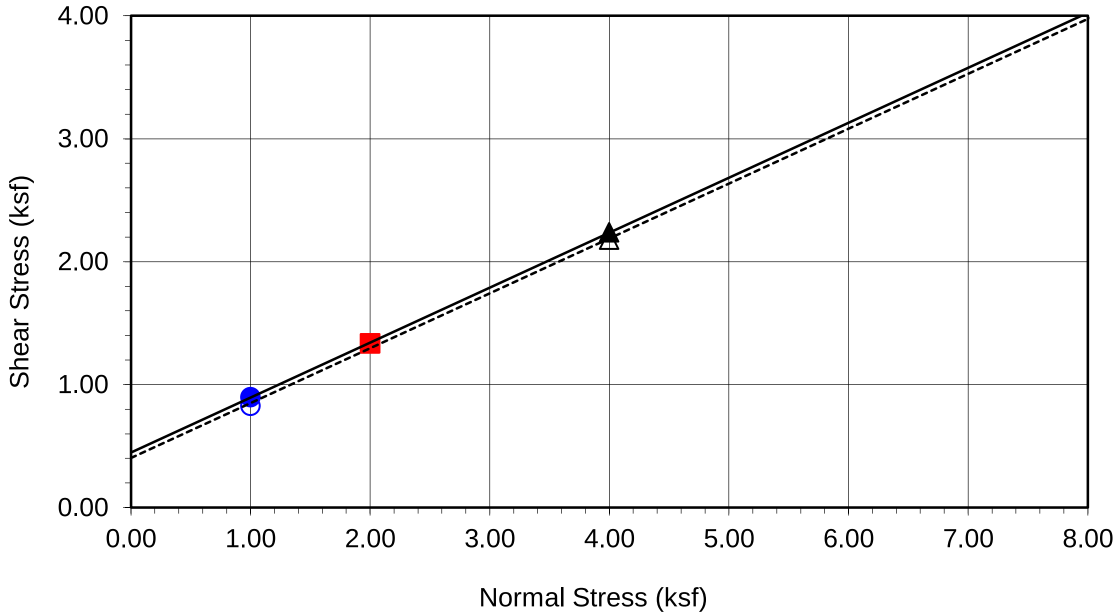
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11127.003

RC Ethanac Rd Bridge

01-18





Boring No.	**LB-2**
Sample No.	**R-2**
Depth (ft)	**10**
Sample Type:	Ring
Soil Identification: Lean Clay (CL), Dark Brown.	

Strength Parameters

	C (psf)	ϕ (°)
Peak	447	24
Ultimate	405	24

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.898	■ 1.334	▲ 2.237
Shear Stress @ End of Test (ksf)	○ 0.826	□ 1.334	△ 2.177
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	24.96	24.96	24.96
Dry Density (pcf)	98.4	93.1	92.1
Saturation (%)	94.4	83.1	81.2
Soil Height Before Shearing (in.)	0.9718	0.9150	0.8868
Final Moisture Content (%)	26.0	25.6	25.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11127.003

RC Ethanac Rd Bridge

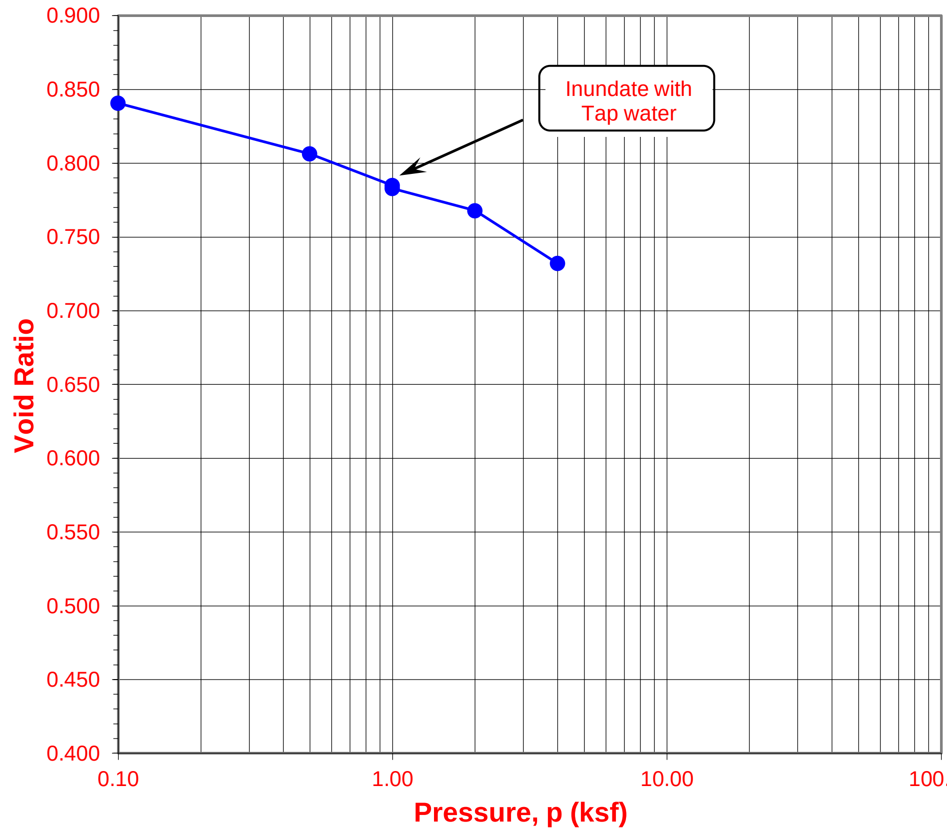
01-18

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	RC Ethanac Rd Bridge	Tested By: M. Vinet	Date: 01/29/18
Project No.:	11127.003	Checked By: M. Vinet	Date: 02/05/18
Boring No.:	LB-9	Depth (ft.): 10.0	
Sample No.:	R-2	Sample Type:	Drive
Soil Identification:	Lean Clay (CL), Brown.		

Sample Diameter (in.):	2.416
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	190.30
Weight of Ring (g):	46.60
Height after consol. (in.):	0.9410
Before Test	
Wt. of Wet Sample+Cont. (g):	355.30
Wt. of Dry Sample+Cont. (g):	284.10
Weight of Container (g):	49.90
Initial Moisture Content (%)	30.4
Initial Dry Density (pcf)	91.6
Initial Saturation (%):	98
Initial Vertical Reading (in.)	0.0000
After Test	
Wt. of Wet Sample+Cont. (g):	231.69
Wt. of Dry Sample+Cont. (g):	200.50
Weight of Container (g):	50.86
Final Moisture Content (%)	30.27
Final Dry Density (pcf):	91.0
Final Saturation (%):	96
Final Vertical Reading (in.)	0.0590
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

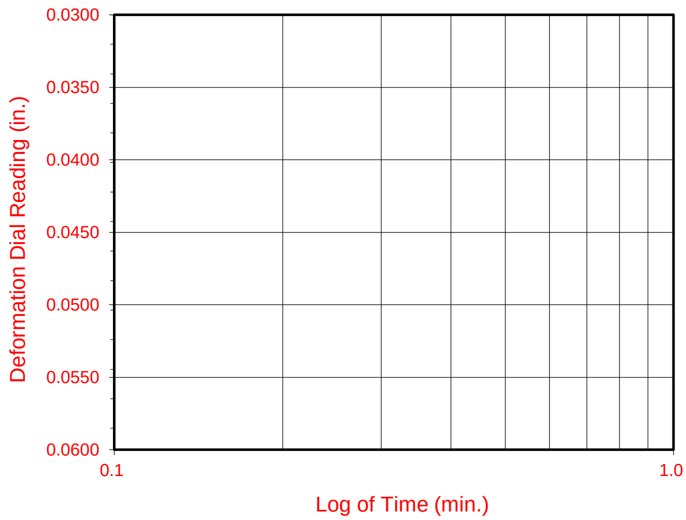


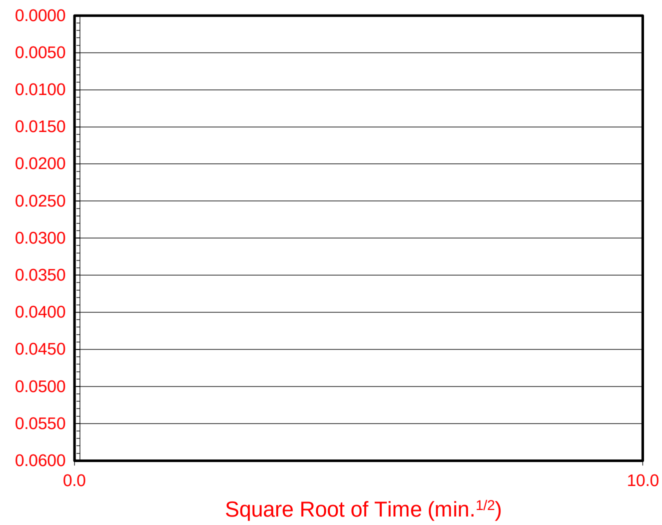
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.0000	1.0000	0.00	0.00	0.841	0.00
0.50	0.0187	0.9813	0.00	1.87	0.806	1.87
1.00	0.0303	0.9697	0.00	3.03	0.785	3.03
1.00	0.0314	0.9686	0.00	3.14	0.783	3.14
2.00	0.0396	0.9604	0.00	3.96	0.768	3.96
4.00	0.0590	0.9410	0.00	5.90	0.732	5.90

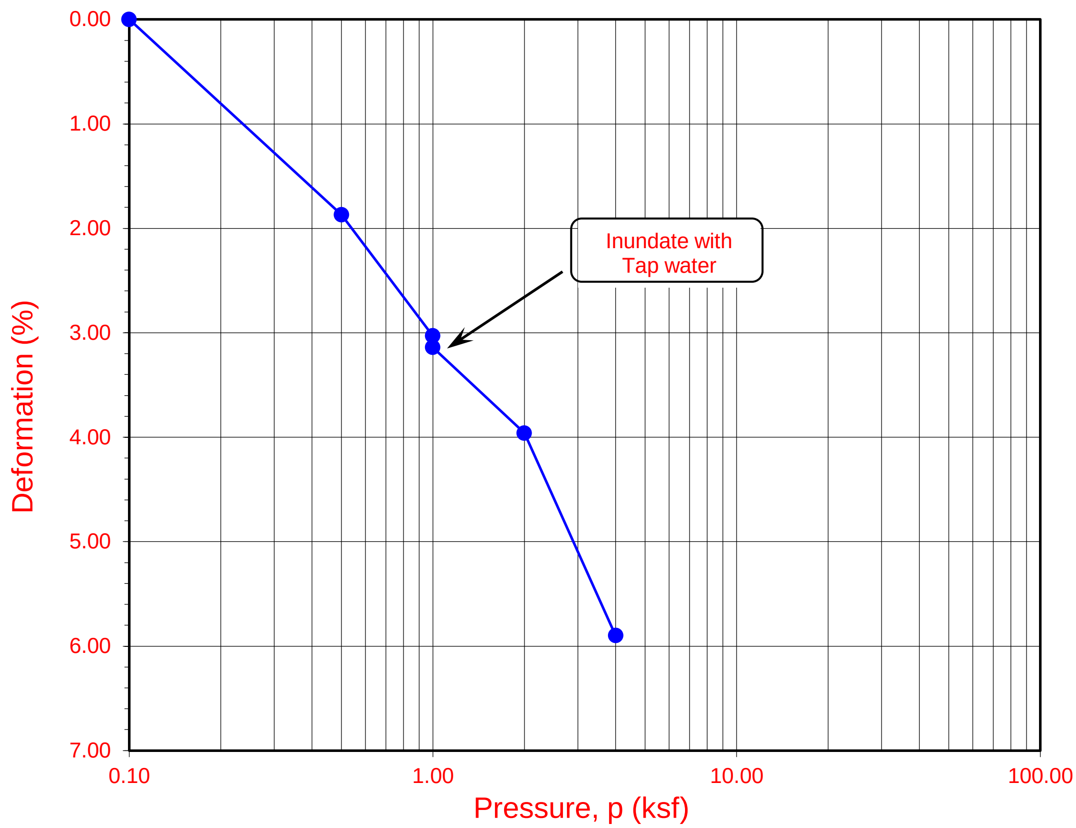
No Time Readings Taken

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

No Time Readings Taken







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-9	**R-2**	**10**	**30.4**	**30.3**	**91.6**	**91.0**	**0.841**	**0.768**	**98**	**96**

Soil Identification: Lean Clay (CL), Brown.

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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11127.003

RC Ethanac Rd Bridge

02-18

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	RC Ethanac Rd Bridge	Tested By: F. Mina	Date: 1/31/18
Project No. :	11127.003	Checked By: M. Vinet	Date: 2/5/18
Boring No.:	LB-8	Depth: 0 - 5.0	
Sample No. :	B-1	Location: N/A	
Sample Description:	Lean Clay (CL), Dark Brown		

Dry Wt. of Soil + Cont. (gm.)	1622.6
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	1622.6
Weight Soil Retained on #4 Sieve	11.3
Percent Passing # 4	99.3

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.1025
Wt. Comp. Soil + Mold (gm.)	558.9	615.5
Wt. of Mold (gm.)	182.7	182.7
Specific Gravity (Assumed)	2.70	2.70
Container No.	7	7
Wet Wt. of Soil + Cont. (gm.)	338.5	615.5
Dry Wt. of Soil + Cont. (gm.)	304.0	332.9
Wt. of Container (gm.)	38.5	182.7
Moisture Content (%)	13.0	30.0
Wet Density (pcf)	113.5	118.4
Dry Density (pcf)	100.4	91.1
Void Ratio	0.679	0.851
Total Porosity	0.404	0.460
Pore Volume (cc)	83.7	104.9
Degree of Saturation (%) [S meas]	**51.7**	**95.2**

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
1/31/18	11:00	1.0	0	0.5000
1/31/18	11:10	1.0	10	0.5000
	Add Distilled Water to the Specimen			
2/1/18	10:00	1.0	1370	0.6025
2/1/18	11:00	1.0	1430	0.6025

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	102.5
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Heigh	**103**

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Ethanac Bridge	Tested By :	G. Berdy	Date:	01/24/18
Project No. :	11127.003	Data Input By:	J. Ward	Date:	02/01/18

Boring No.	LB-1	LB-6		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Olive CL	Olive CL		
Wet Weight of Soil + Container (g)	168.40	124.21		
Dry Weight of Soil + Container (g)	158.05	117.73		
Weight of Container (g)	64.16	58.67		
Moisture Content (%)	11.02	10.97		
Weight of Soaked Soil (g)	100.58	100.00		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	304	308		
Crucible No.	12, 16	9, 17		
Furnace Temperature (°C)	860	860		
Time In / Time Out	9:00/9:45	9:00/9:45		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	47.8970	43.4642		
Wt. of Crucible (g)	47.7819	43.4077		
Wt. of Residue (g) (A)	0.1151	0.0565		
PPM of Sulfate (A) x 41150	4736.36	2324.97		
PPM of Sulfate, Dry Weight Basis	**5323**	**2612**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	5		
ml of AgNO3 Soln. Used in Titration (C)	1.3	1.9		
PPM of Chloride (C -0.2) * 100 * 30 / B	110	1020		
PPM of Chloride, Dry Wt. Basis	**124**	**1146**		

pH TEST, DOT California Test 643

pH Value	**7.69**	**7.43**		
Temperature °C	21.7	21.7		

SOIL RESISTIVITY TEST
DOT CA TEST 643

Leighton

Project Name:	Ethanac Bridge	Tested By :	G. Berdy	Date:	01/30/18
Project No. :	11127.003	Data Input By:	J. Ward	Date:	02/01/18
Boring No.:	LB-1	Depth (ft.) :	0-5		
Sample No. :	B-1				

Soil Identification:* Olive CL

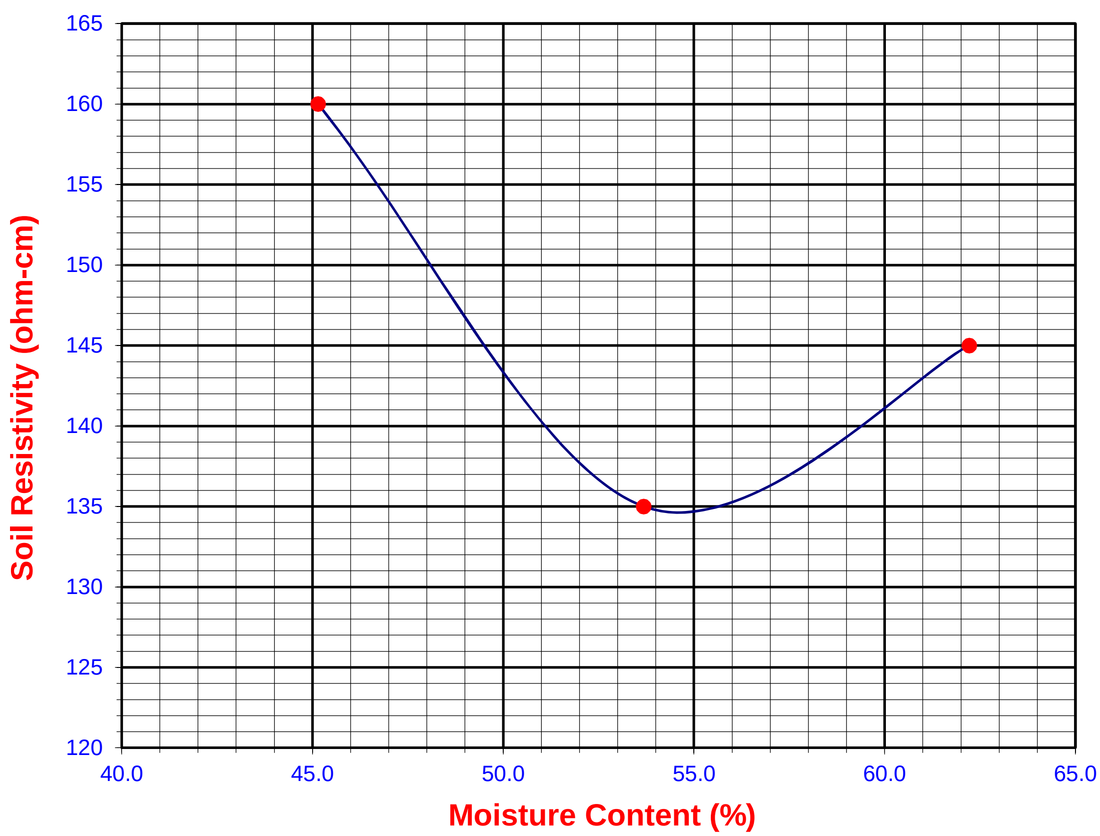
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	40	45.15	160	160
2	50	53.68	135	135
3	60	62.21	145	145
4				
5				

Moisture Content (%) (MCi)	11.02
Wet Wt. of Soil + Cont. (g)	168.40
Dry Wt. of Soil + Cont. (g)	158.05
Wt. of Container (g)	64.16
Container No.	
Initial Soil Wt. (g) (Wt)	130.13
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
135	**54.6**	**5323**	**124**	**7.69**	**21.7**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Ethanac Bridge	Tested By :	G. Berdy Date: 01/30/18
Project No. :	11127.003	Data Input By:	J. Ward Date: 02/01/18
Boring No.:	LB-6	Depth (ft.) :	0-5
Sample No. :	B-1		

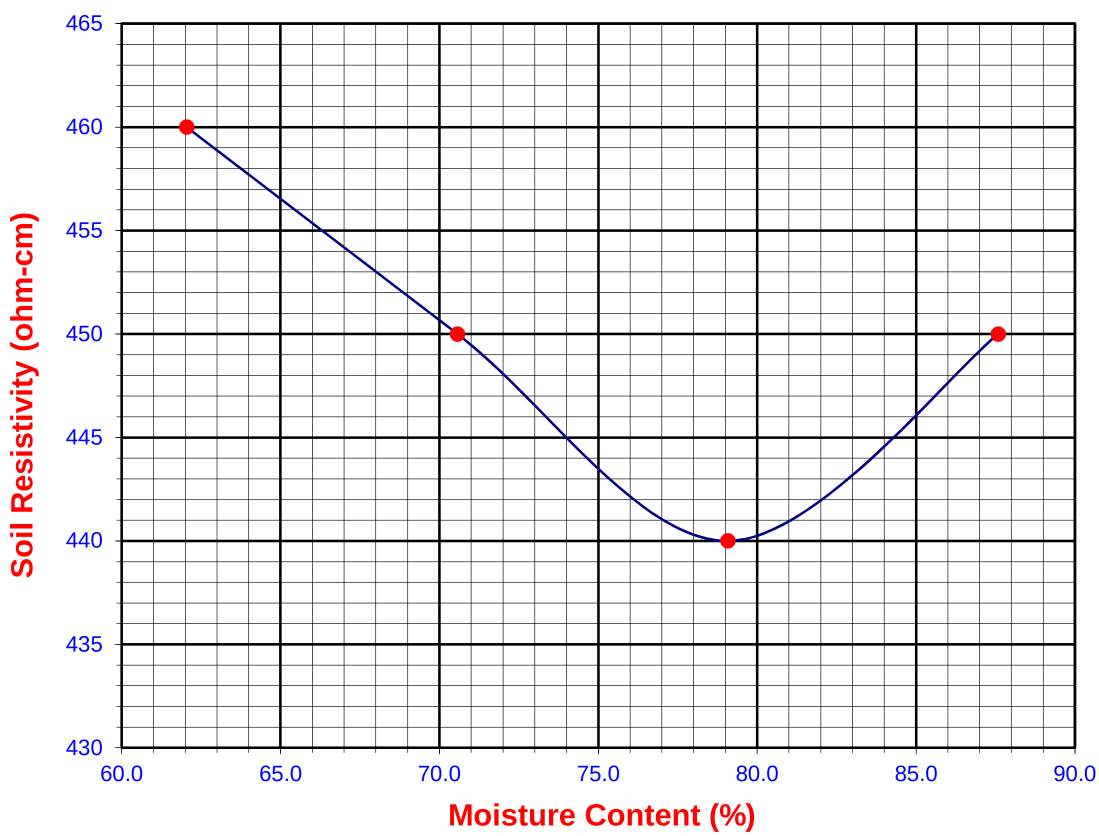
Soil Identification:* Olive CL

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	60	62.05	460	460
2	70	70.57	450	450
3	80	79.08	440	440
4	90	87.59	450	450
5				

Moisture Content (%) (MCi)	10.97
Wet Wt. of Soil + Cont. (g)	124.21
Dry Wt. of Soil + Cont. (g)	117.73
Wt. of Container (g)	58.67
Container No.	
Initial Soil Wt. (g) (Wt)	130.35
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
440	**79.1**	**2612**	**1146**	**7.43**	**21.7**



APPENDIX C

EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX C

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

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Standard Details

A - Keying and Benching ... Rear of Text

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

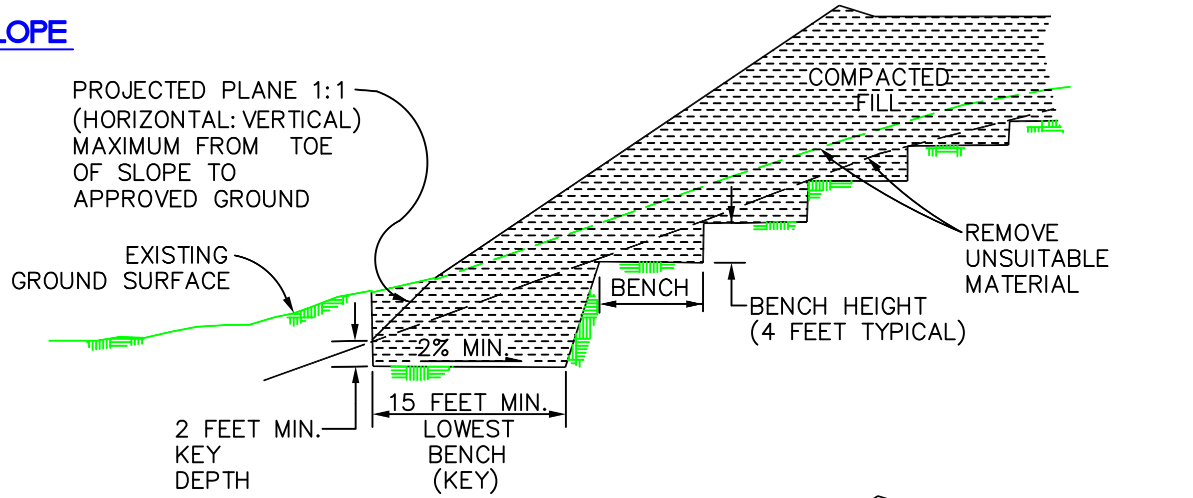
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

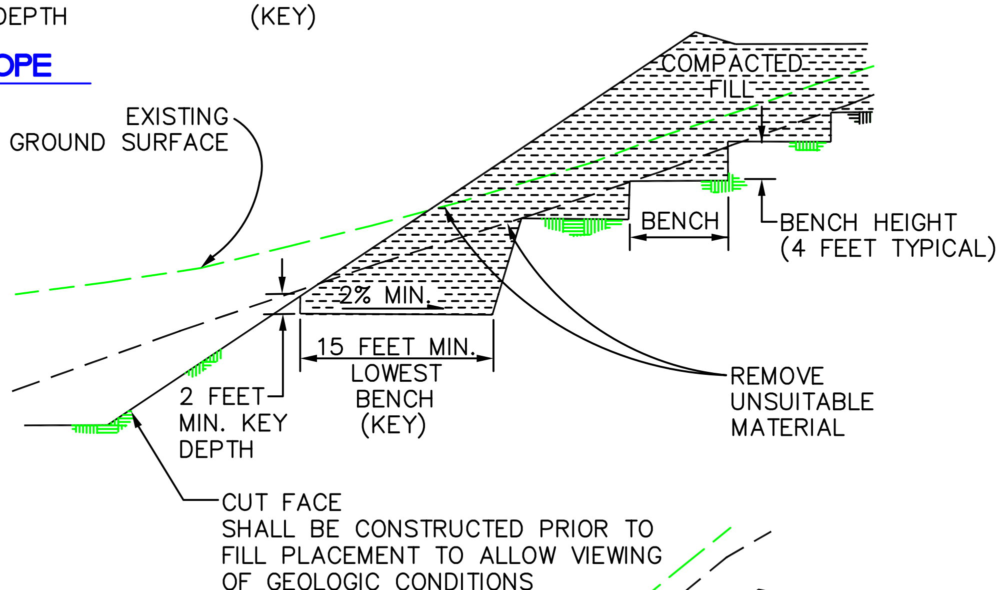
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

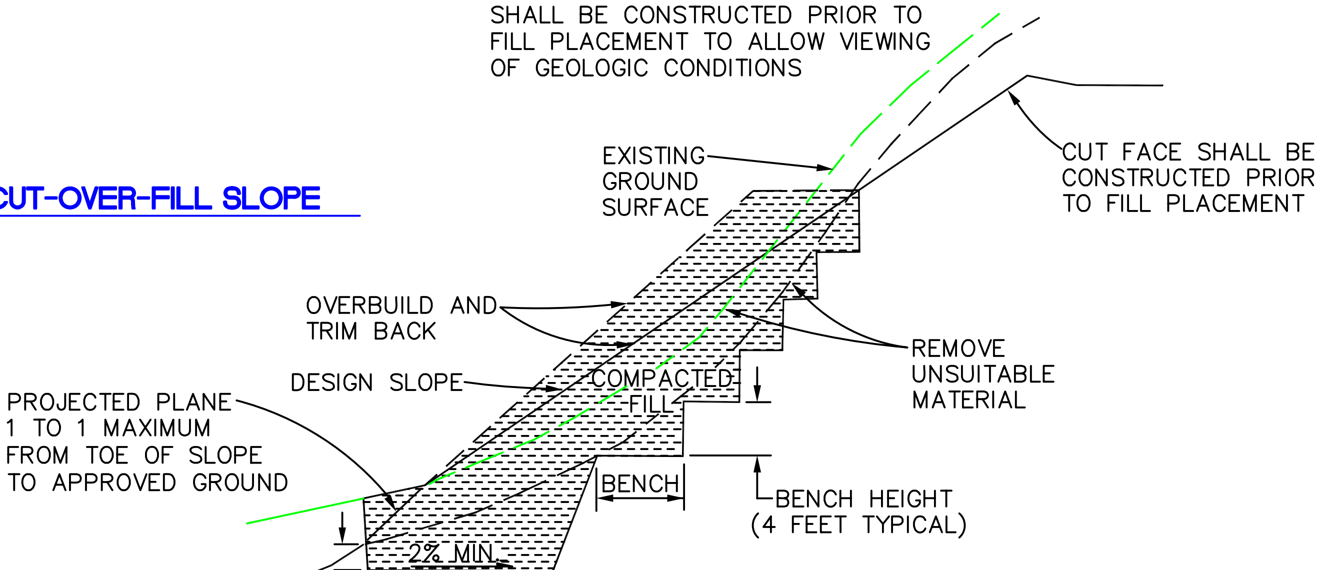
FILL SLOPE



FILL-OVER-CUT SLOPE



CUT-OVER-FILL SLOPE



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

APPENDIX D

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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