Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL DESIGN REPORT (GDR)

MAGNOLIA AVENUE BRIDGE AND ROADWAY WIDENING (BR NO. 56C-0199, PM-40.9)

CITY OF CORONA PROJECT NUMBER 2105-15
FEDERAL AID PROJECT NO. STPL-5104 (046)
CITY OF CORONA, RIVERSIDE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 18-81-147-03

Prepared For:
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December 28, 2020

Converse Consultants
Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

December 28, 2020

Mr. James Lu, PE
Principal Engineer
CNS Engineering, Inc.
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Riverside, CA 92505

Subject: **GEOTECHNICAL DESIGN REPORT (GDR)**
Magnolia Ave. Bridge and Roadway Widening
(BR No. 56C-0199, PM-40.9)
El Camino Avenue to 1,000 Feet East of All-American Way
City of Corona Project Number 2015-15
Federal Aid Project No. STPL-5104 (046)
City of Corona, Riverside County, California
Converse Project No. 18-81-147-03

Dear Mr. Lu:

Converse Consultants (Converse) is pleased to submit this Geotechnical Design Report (GDR) to assist CNS Engineering, Inc in preparing the Project Specifications and Estimation (PS&E) for the proposed Magnolia Avenue Bridge and Roadway Widening project, located in the City of Corona, Riverside County, California. The content of this report follows California Department Transportation (Caltrans) *Geotechnical Design Report Guidelines* (Caltrans, 2020). The recommendations provided in this report are based on site-specific field investigation and subsurface information contained on the Log-of-Test-Borings (LOTBs) sheet included with the as-built plans, provided by Caltrans. This report was prepared in accordance with our revised proposal dated April 5, 2018 and your Subconsultant Professional Service Agreement dated July 29, 2019.

We appreciate the opportunity to be of continued service to CNS Engineering, Inc. Should you have any questions, please contact us at 909-796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, PhD, PE, GE
Principal Engineer

Dist: 3/Addressee
ZA/RG/HSQ/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the individuals whose seals and signatures appear herein.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering, engineering geologic principles, and practice in this area of Southern California. There is no warranty, either expressed or implied.

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1.0 INTRODUCTION

This report presents the geologic and geotechnical information and, design and construction recommendations for the proposed Magnolia Avenue Bridge and Roadway Widening project, located in the City of Corona, Riverside County, California. The interchange location is shown on Figure No. 1, *Approximate Project Location Map*.

The purposes of this report were to document subsurface geotechnical conditions, provide analyses of site conditions, and to recommend design and construction criteria for the project. Our scope of services consisted of review of existing data, a field investigation program, laboratory testing, and preparation of this report. The report provides the following:

- A description of the proposed project including a site vicinity map showing the location of the project limit and the approximate locations of the exploration borings.
- A summary of the field exploration and laboratory testing programs, including a log of test borings.
- A general description of the surface and subsurface materials, including groundwater conditions.
- Recommendations on earthwork and excitability.
- Recommendations on trenchless pipeline construction.
- Comments on percolation rate.
- Comments on disposal of on-site materials unsuitable for construction.
- Comments on local available material sources.
- Comments on the general corrosion potential of on-site soils to buried metal and Concrete.

This Geotechnical Design Report follows the requirements in accordance with California Department of Transportation (Caltrans) *Geotechnical Design Report Guidelines* (Caltrans, 2020).

2.0 PERTINENT REPORTS AND INVESTIGATIONS

A review of readily available publications from various public and private files addressing the surface and subsurface conditions in the project area was conducted. The objective of this task was to develop an initial understanding of the geologic, faulting, hydrogeologic, and geotechnical considerations for the improvements. The list of all documents reviewed is presented in the Section 14.0 *References*.

Approximate Project Location Map

N

Project:	Magnolia Avenue Bridge and Roadway Widening
Location:	El Camino Avenue to 1,000 feet East of All American Way City of Corona, Riverside County, California
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Figure No.
1

3.0 PROJECT DESCRIPTION

Project improvements will occur on Magnolia Avenue between El Camino Avenue to 1,000 feet east of All-American Way, which is close to the intersection of eastbound lane of Leeson Lane. Magnolia Avenue is accessible from the I-15 Freeway. The Temescal Creek Channel, a rectangular concrete channel at this location, crosses under Magnolia Avenue in a north-south direction.

3.1 Project Purpose

The purpose of the Project is to increase existing traffic capacity and improve pedestrian and non-motorized travel on Magnolia Avenue between El Camino Avenue to 1,000 feet east of All-American Way at Leeson Lane. The proposed improvements will accomplish the following in the Project area.

- Provide sidewalks, curbs, gutters, and ADA compliance.
- Provide an additional lane of travel in each direction, per the City's General Plan.
- Widen the bridge over Temescal Creek Channel (Channel) to accommodate the additional travel lanes, sidewalks, curbs, and gutters.
- Provide for ultimate build-out of the roadway per the City's General Plan.

3.2 Project Need

Magnolia Avenue is an east-west divided Major Arterial in the City of Corona, accessible from Interstate 15 (I-15). It is identified as six lanes in the City's General Plan, but it was only striped/constructed to accommodate four lanes. The Project improvements will begin at El Camino Avenue, approximately 600 feet east of the I-15. Land uses along the Project alignment include light industrial to heavy industrial on both sides of the road. The heavy industrial uses include a quarry located south of the Project alignment, accessible on the south side of Magnolia Avenue from Sherborn Street and All-American Way.

Given its proximity to the I-15 and the mix of light and heavy industrial uses, this approximately 2,100 linear foot Project alignment experiences a high volume of heavy truck traffic. Build-out of the roadway to the design as envisioned by the City's General Plan would improve overall circulation in this section.

3.3 Existing Conditions

The proposed Project alignment is located in the City of Corona, along Magnolia Avenue, beginning at approximately the intersection of El Camino Avenue and ending approximately 1,000 feet east of All-American Way at Leeson Lane.

Western Section of Alignment (El Camino Avenue to Temescal Creek Channel Bridge)

The paved travel way in this section is generally approximately 82 feet wide, contains two lanes of travel in each direction, turn lanes, and a striped median to the Temescal Creek Channel Bridge. The right-of-way in this section is approximately 100 feet wide - approximately 40 feet to the north and approximately 60 feet to the south of centerline. Sidewalk, curb and gutter exist on the south side but not on the north side. City-owned streetlights are present on both sides of the street.

The BNSF railroad crossing exists approximately 80 feet east of the intersection with El Camino Avenue.

Sherborn Street intersects on the south side, approximately halfway between El Camino Avenue and the bridge approach.

All electrical and low-voltage (phone, cable) utilities are located underground throughout this section.

Temescal Creek Channel Bridge

The Temescal Creek Channel is an improved, 84-foot-wide by 15-foot-deep rectangular concrete channel. There is a storm drain into the channel, which includes a grated drop inlet at the north side of Magnolia Avenue west of the Channel; a 30-inch storm drain line that ties into the Channel at the northeast, southeast and southwest corners of the bridge. The channel is owned and maintained by the Riverside County Flood Control and Water Conservation District (RCFC &WCD).

The existing bridge over the Channel is 67.5 feet wide providing a travelled way of 64 feet from barrier to barrier. The bridge deck is striped with two lanes in each direction and a painted median. At each approach, the bridge barrier is protected by a standard metal beam guardrail. There are no sidewalks on the bridge. The existing structure was built in 1986. It consists of two spans of cast-in-place reinforced concrete box girder, a pier wall along the centerline of the Channel, and two abutments. The bridge abutments were constructed outside the rectangular concrete channel. The bridge has a high Sufficiency Rating of 95.8 indicating the feasibility of the proposed structure widening with proper rehabilitation, as required.

The City of Corona's 30-inch water line (Cross-Town Transmission Feeder) is attached to the exterior edge of the south side of the bridge, and other utilities (Southern California Edison and cable and phone) are within conduits attached to the bridge exterior along the north side. An electrical/phone overhead line spans over the Channel on the south side of the bridge.

Eastern Section of Alignment (Temescal Creek Bridge to Eastbound Leeson Lane)

The paved travel way in this section is generally approximately 82 feet wide, contains two lanes of travel in each direction with turn lanes. A narrow-raised concrete median is present in this section, from approximately 1475 Magnolia Avenue to the alignment terminus at the eastbound lane of Leeson Lane. The right-of-way in this section is approximately 110 feet wide - approximately 60 feet to the north and approximately 50 feet to the south of centerline.

Sidewalk, curb, and gutter exist on both the north and south sides, but not in front of the Corona Auto Parts Store, located at 1450 Magnolia Avenue, which is on the southeast corner of All-American Way and Magnolia Avenue intersection. City-owned streetlights are present on both sides of the street.

All American Way intersects immediately east and adjacent to the bridge on the south side. Other intersecting streets include Trademark Circle and Leeson Lane on the south side toward the end of the alignment.

Low voltage utilities (i.e., phone and cable) rise approximately 112 feet west of the bridge and are located on poles on the south side of the street, for approximately 679 feet to 1480 Magnolia Avenue. The utilities then transition to underground at this location and remain underground through the end of the Project alignment at the eastbound Leeson Lane.

The photographs below show the overall site condition within the project limit.

Photograph No. 1, Magnolia Avenue, east from El Camino Avenue, railroad crossing in view.

Photograph No. 2, Magnolia Avenue Bridge, facing southwest.

Photograph No. 3, Northwest side of the bridge.

Photograph No. 4, Magnolia Avenue, southwest of the bridge.

3.4 Proposed Improvements

The City of Corona is proposing to widen the Magnolia Avenue Bridge over Temescal Creek Channel and Magnolia Avenue from El Camino Avenue to 1,000 feet east of the All-American Way generally to increase the number of travel lanes per the City's General Plan, and construct sidewalks, curbs, and gutters. Improvements will include restriping for three 12-foot-wide lanes in each direction, a 12-foot-wide median, 5-foot-wide shoulders, and 6-foot-wide sidewalks/curbs and gutters at locations that currently lack sidewalk/curb/gutter. The total roadway width would be increased to approximately 100 feet, curb to curb, throughout the alignment, and right-of-way varies throughout the alignment.

The work will include the following.

- Roadway widening including drainage improvements.
- Modification to street signs, streetlights, and landscaping.
- Pavement rehabilitation where required.
- Modifying the existing roadway striping.
- Installing new curbs and gutters, and sidewalks in the missing sections.
- Re-striping and/or replacing the existing BNSF railroad crossing. The crossing arms and railroad signals may be preserved; however, it is to be further

determined based on the results of the field Railroad Diagnostic Meeting with CPUC and BNSF Railway.

- Widening and rehabilitating the concrete bridge over the Temescal Creek Channel.
- Relocating utilities that conflict with the planned improvements. and
- Providing ADA compliant access ramps at all intersections.

As a part of the bridge construction, the abutment at each end of the bridge would be extended, along with one pier within the Temescal Creek Channel.

3.5 Potential Right-of-Way Requirements and/or Special Considerations

The Project will generally be constructed within the City's rights-of-way (ROW). However, additional ROW or permissions may be required including the following:

- Magnolia Avenue north side, west of Temescal Creek Channel Bridge: Providing the desired roadway section with a sidewalk will result in the need to acquire additional right of way from the limits of BNSF Railroad to the Channel. The right of way acquisition will be limited to the back edge of the sidewalk. The preliminary impact of this right-of-way acquisition is along the frontage of the Clow Valve facility at 1375 Magnolia Avenue. Clow Valve facility fronting Magnolia Avenue is mostly used as a lay-down yard for their product and there is a segment of landscaped parkway fronting an office building.

- Magnolia Avenue, south side, east of Temescal Creek Channel Bridge: Providing the desired roadway section with a sidewalk will result in the need to acquire 6 feet of additional right of way from All American Way to the eastbound lane of Leeson Lane. The right of way acquisition will be limited to the back edge of the sidewalk. The primary impact of this right-of-way acquisition will include:

    - Corona Auto Parts Business, located at 1450 Magnolia Ave., on the southeast corner of All American Way and Magnolia Avenue intersection, immediately east of the Temescal Creek Channel Bridge: There is no sidewalk, and the existing parking lot connects to the edge of the traveled way pavement. There are no defined driveways on this parcel. Under the existing condition, there is just enough clearance between the edge of the roadway and the face of the building for cars to maneuver into parking stalls perpendicular to the front of the building. Constructing curb and gutter, sidewalk and additional travel lane consistent with the City's General Plan will place the curb and gutter approximately 35 feet from the building. Therefore, Project improvements will likely reduce the number of customer parking spaces at the business by six spaces. Design alternatives to the parking lot have been developed to minimize impacts.

  - Existing landscaped buffer areas on the south side of Magnolia Avenue between 1460 Magnolia Avenue (adjacent to the Corona Auto Parts business) and 1560 Magnolia Avenue (at Leeson Lane): In this section, a sidewalk exists in the City's portion of the right-of-way. Within the private property immediately adjacent to the sidewalk exists landscaped buffer areas that separate the sidewalk from the customer parking for the businesses along this section. The landscaped buffer areas range from approximately 11 feet wide at 1480 Magnolia Avenue to approximately 27 feet wide at 1580 Magnolia Avenue. Trees and shrubs in these landscaped areas would be removed, but customer parking would not be impacted.

- Burlington-Northern Santa Fe (BNSF) Railroad: The intersection of El Camino Avenue and Magnolia Avenue is located east and adjacent to a BNSF grade crossing. The proposed roadway improvements may require upgrades to grade crossing equipment and operation, although major improvements are not expected. Close coordination with the California Public Utilities Commission (CPUC) and BNSF railroad will be required to obtain approvals and permits within the Project schedule. Conceptual plans will be drafted indicating proposed improvements and presented to all stakeholders during a railroad diagnostic meeting.

- Temescal Creek Channel: Bridge widening will require an additional 20 feet of right-of-way on both the north and south side of the bridge (for a total of approximately 40 feet) to be acquired from the Riverside County Flood Control and Water Conservation District (RCFC &WCD).

- Utility Relocation: Some streetlights (owned by the City) will need to be temporarily relocated during Project construction to facilitate sidewalk construction. Additionally, all streetlights within the Project limits will be converted to light-emitting diode (LED). The SCE conduits and lower voltage utilities that are attached to the bridge structure on the north side will be relocated to within new cells inside the bridge. The 30-inch water main from the City of Corona, attached to the existing bridge on the south side will also be reattached to the new southern edge of the widened bridge. All pole-mounted utilities located on the south side, between All American Way and 1480 Magnolia Avenue, will be relocated during construction only but remain above ground.

4.0 EXCEPTION TO POLICY

There is no exception that deviates from Caltrans policy related to the preparation of this report.

5.0 SCOPE OF WORK

To prepare this materials report, the following tasks were conducted.

- Discussed the project with the project team.
- Reviewed published maps and literature related to site soil, rock, groundwater and geologic conditions.
- Reviewed published geotechnical data and as-built information for existing structures in the project area.
- Prepared a boring locations map and submitted to CNS for review and approval.
- Conducted a site and alignment reconnaissance and marked the borings at locations approved by CNS.
- Obtained permit from the City of Corona.
- Prepared a traffic control plans.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring location of any conflict with existing underground utilities.
- Engaged a California-licensed driller to drill exploratory borings.

6.0 FIELD INVESTIGATION PROGRAM

Six exploratory borings (A-20-001 through A-20-005 and O-20-001) were drilled to investigate the subsurface conditions for the project. The borings (A-20-001 through A-20-005) were advanced using a standard CME 85 drill rig equipped with 8-inch diameter hollow-stem augers. The hammer energy transfer ratio of the drill rig is 86.2 percent (attached in appendix A-1). Due to the presence of cobbles and boulders, borings at the bottom of the bridge foundation could not be penetrated up to the maximum required depth of 90 feet bgs. Therefore, one additional boring (O-20-001) was drilled using Becker Hammer up to 90 feet bgs. The Becker hammer energy transfer ratio is 86.2 and 83 percent (attached in appendix A-1). A summary of boring information is presented in the following table.

Table No. 1, Summary of Borings

Boring No.	Associated Improvements	Location		Approx. Station	Approx. Ground Surface Elev. (feet, NAVD 88)	Boring Depth (ft, bgs)	Date Completed
		Latitude	Longitude				
A-20-001	Percolation	33.8683N	117.5382W	25+00	645.47	16.5	10/15/2020
A-20-002	Roadway	33.8686N	117.5377W	26+50	646.79	16.5	10/15/2020
*A-20-003	Bridge	33.8697N	117.5358W	33+75	646.84	20.5	10/6/2020
*A-20-004	Bridge	33.8696N	117.5352W	35+20	647.78	32.0	10/6/2020

Table No. 1, Summary of Borings (continued)

Boring No.	Associated Improvements	Location		Approx. Station	Approx. Ground Surface Elev. (feet, NAVD 88)	Boring Depth (ft, bgs)	Date Completed
		Latitude	Longitude				
A-20-005	Roadway	33.8711N	117.5334W	42+80	647.78	11.5	10/7/2020
**O-20-001	Bridge	33.8696N	117.5351W	35+20	644.78	90.0	11/4/2020

Notes:
Stations and ground surface elevations were based on the project plans provided by CNS.
*Borings were terminated due to presence of cobbles and possible boulders.
**Becker Hammer was used to drill.

The approximate boring locations are shown in Figure No. 2, *Approximate Boring and Percolation Test Locations Map*. Detailed description of the field exploration program, a summary table of boring information, and boring records are presented in Appendix A, *Field Exploration.*

The exploration locations and depths were selected by CNS is in consultation with Converse Consultants in accordance with the boring spacing and depth requirements provided in AASHTO LRFD, 2020 and other relevant documents.

7.0 LABORATORY TESTING PROGRAM

The following laboratory soil tests will be performed when a site-specific field investigation is completed after approval of bridge type selection during PS&E phase.

- *In-situ* moisture content and dry densities (ASTM D2216/D2937)
- Expansion Index (ASTM D4829)
- Sand equivalent (ASTM D2419)
- R-value (California Test 301)
- Soil corrosivity (California Tests 643, 422, and 417)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

8.0 GEOLOGIC AND GEOTECHNICAL CONDITIONS

The regional and local geology and subsurface conditions are discussed below.

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Approximate Boring and Percolation Test Locations Map

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Figure No.
2

8.1 Regional Geology

The project site is located in the northwestern portion of the Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges province is characterized by northwest tending valleys and mountain ranges which have formed in response to regional tectonic forces along the boundary between the Pacific and North American tectonic plates. The geologic structure is dominated by northwest trending right-lateral faults, most notable, the San Andreas Fault, San Jacinto Fault, Elsinore Fault, Whittier Fault, and the Newport-Inglewood Fault. The province extends southward from the Transverse Ranges province at the north end of the Los Angeles Basin to the southern tip of the Baja California Peninsula.

Basement rocks in the region are predominantly granitic and metamorphic rocks associated with the Mesozoic-age Southern California Batholith. Erosional remnants of granitic rocks are exposed in isolated hilly outcrops within the northern portions of the Chino Basin. Cenozoic-age sedimentary rocks overly the basement rocks in many areas and are well exposed in the Santa Ana Mountains and the Chino Hills southwest and west of the site.

8.2 Local Geology

The project site is underlain by Holocene and late Pleistocene artificial fill and alluvial deposits. These deposits primarily consist of fine to medium-grained sand with gravel and possible cobbles. (Morton et al, 2002). Descriptions of each unit are provided below.

- **Qaf: Artificial fill (late Holocene)—**Deposits of fill, may exist on the site, resulting from human construction or mining activities; includes numerous noncontiguous areas related to sand and gravel operations and flood control in and adjacent to Temescal Wash and to road grade and ramps along Corona Freeway segment of Interstate 15.
- **Qya: Young alluvial channel deposits (Holocene and late Pleistocene)—**Gray, unconsolidated alluvium. Found chiefly in Temescal Wash and its tributaries, where it consists of medium- to fine-grained sand in lower reaches and coarsens to gravel and cobbles up stream. Also found in Wardlaw Canyon and its tributaries, and in Ladd Canyon in southwestern part of quadrangle.
- **Qyf: Young alluvial fan deposits (Holocene and late Pleistocene)—**Gray-hued gravel and boulder deposits derived largely from volcanic and sedimentary units of Santa Ana Mountains. Fans consisting mainly of gravel emanate and coalesce from Tin Mine, Hagador, Main Street, and Eagle Canyons. Fan emanating from Bedford Canyon is coarser grained, containing a large component of boulders. All fans coarsen toward mountains. Locally, young alluvial fan deposits are divided into subunits based on sequential terrace development and other factors; one such unit is found in quadrangle.

The site and surrounding local geology are shown on Figure No. 3, *Geologic Map* on the following page.

8.3 Subsurface Soil Conditions

According to the Log of Test Borings (LOTB) sheet (attached in Appendix C) included with the as-built plans (Caltrans, 1984), two borings (B-1 and B-2) were drilled in December 1983 and January 1984, near the bridge crossing areas during the field investigation by the Caltrans Bridge Department.

Boring No. B-1, which was located on the northwest side of the bridge, encountered dense to very dense sandy gravel with cobbles from the surface to approximately 20 feet bgs. Dense silty sand and sand was encountered from approximately 20 to 35 feet bgs. Very dense coarse gravel and sand was encountered from 35 feet bgs to the boring termination at approximately 40 feet bgs.

Boring No. B-2, which was located on the southeast side of the bridge, encountered dense sand and gravel with scattered cobbles from the ground surface to approximately 15 feet bgs. Very dense sandy gravel with abundant large cobbles and occasional boulders was encountered from approximately 15 feet bgs to 35 feet bgs. Very dense cobbles and boulders were encountered from approximately 35 feet bgs to 39 feet bgs. Very dense sand and coarse gravel was encountered from approximately 39 feet bgs to the boring termination at approximately 42 feet bgs.

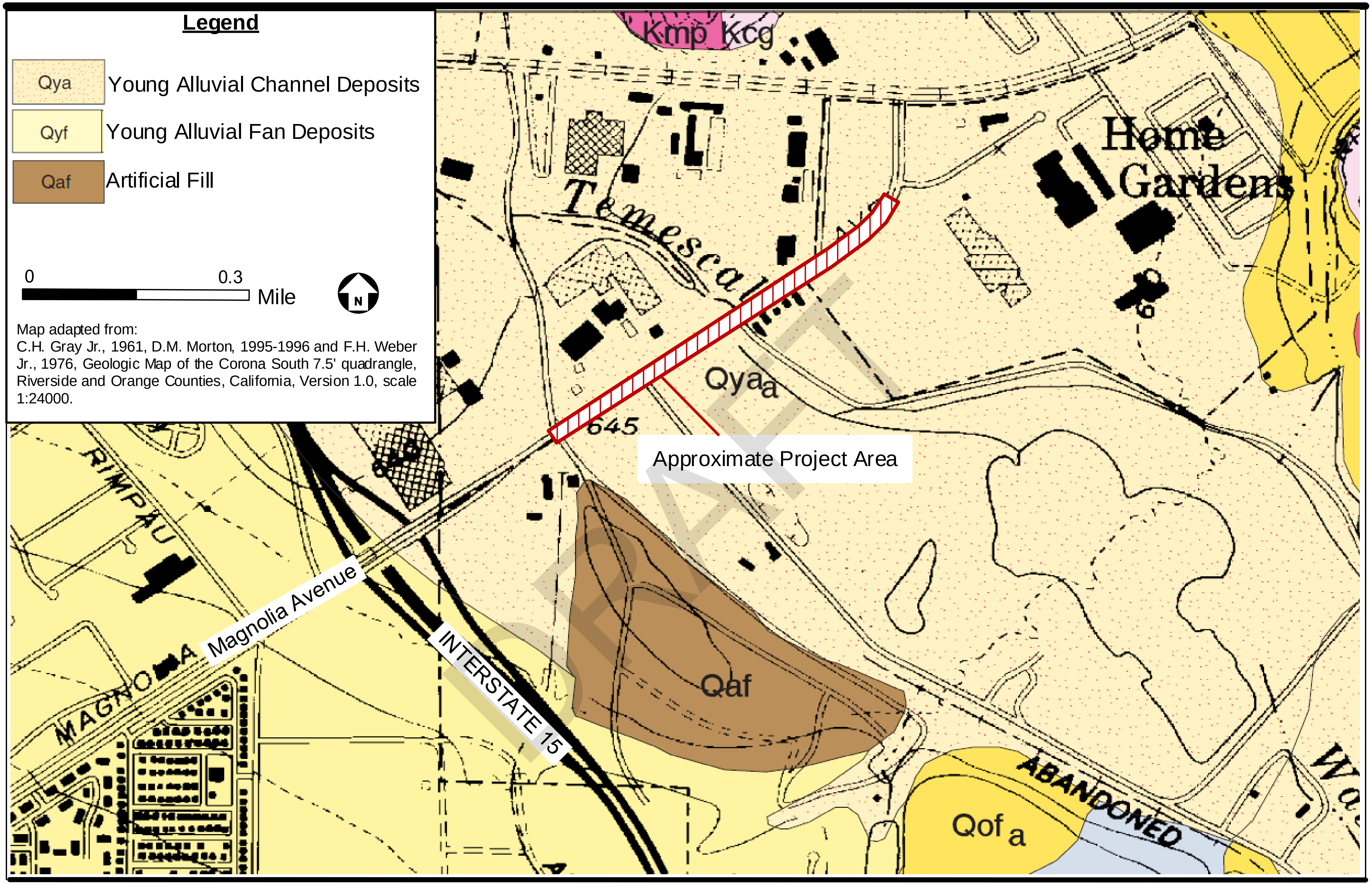
Based on the exploratory borings and laboratory test results (Converse, 2020), the alluvium soils consist primarily of sand, silt, gravel and cobbles. Scattered to some gravel up to 2.5 inches and scattered to few cobbles up to 5 inches in largest dimension were encountered to the maximum explored depth of 90 feet bgs. Possible boulders may present at depth greater than 20 to 31 feet bgs. Two sandy clay layers were encountered at depths between 36.5 and 45.0 feet, and 70.0 and 75.0 feet bgs in boring O-20-001.

For a detailed description of the subsurface materials encountered in the exploratory borings see, *Boring Records*, in Appendix A, Field Exploration.

8.4 Groundwater

At the time of field investigation (1983 and 1984), groundwater was encountered at approximately 12 feet bgs, corresponding to an elevation of 632 feet (assumed NGVD 29).

During this field investigation (2020), groundwater was encountered only in the boring (O-20-001) at depth of approximately 50.0 feet bgs, corresponding to elevation of 596.8 feet (assumed NAVD, 88).



Map adapted from:
C.H. Gray Jr., 1961, D.M. Morton, 1995-1996 and F.H. Weber Jr., 1976, Geologic Map of the Corona South 7.5' quadrangle, Riverside and Orange Counties, California, Version 1.0, scale 1:24000.

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Geologic Map

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FIGURE
3

The GeoTracker database (SWRCB, 2020) was reviewed for groundwater data from sites within close proximity of the project. Two sites were identified within a 1.0-mile radius of the project site that contained groundwater elevation data.

- SHELL MAGNOLIA CORONA (T0606500247), located approximately 3,500 feet southwest of the project site, reported groundwater at depths ranging between 100 and 118 feet bgs between 2005 and 2009.
- SMOG CHECK OF CORONA (T0606500118), located approximately 5,200 feet northeast of the project site, reported groundwater at a depth of 37 feet bgs in 2005.

Data was not found on the National Water Information System (USGS, 2020).

Based on available data, the historical high groundwater level near the site is estimated to be approximately 12 feet. Groundwater is not expected to be encountered during construction of the roadway. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

8.5 Faulting

The site is not located within a recognized State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2019). The nearest mapped fault is the Elsinore Fault, which lies approximately 3.7 miles to the southwest of the project site.

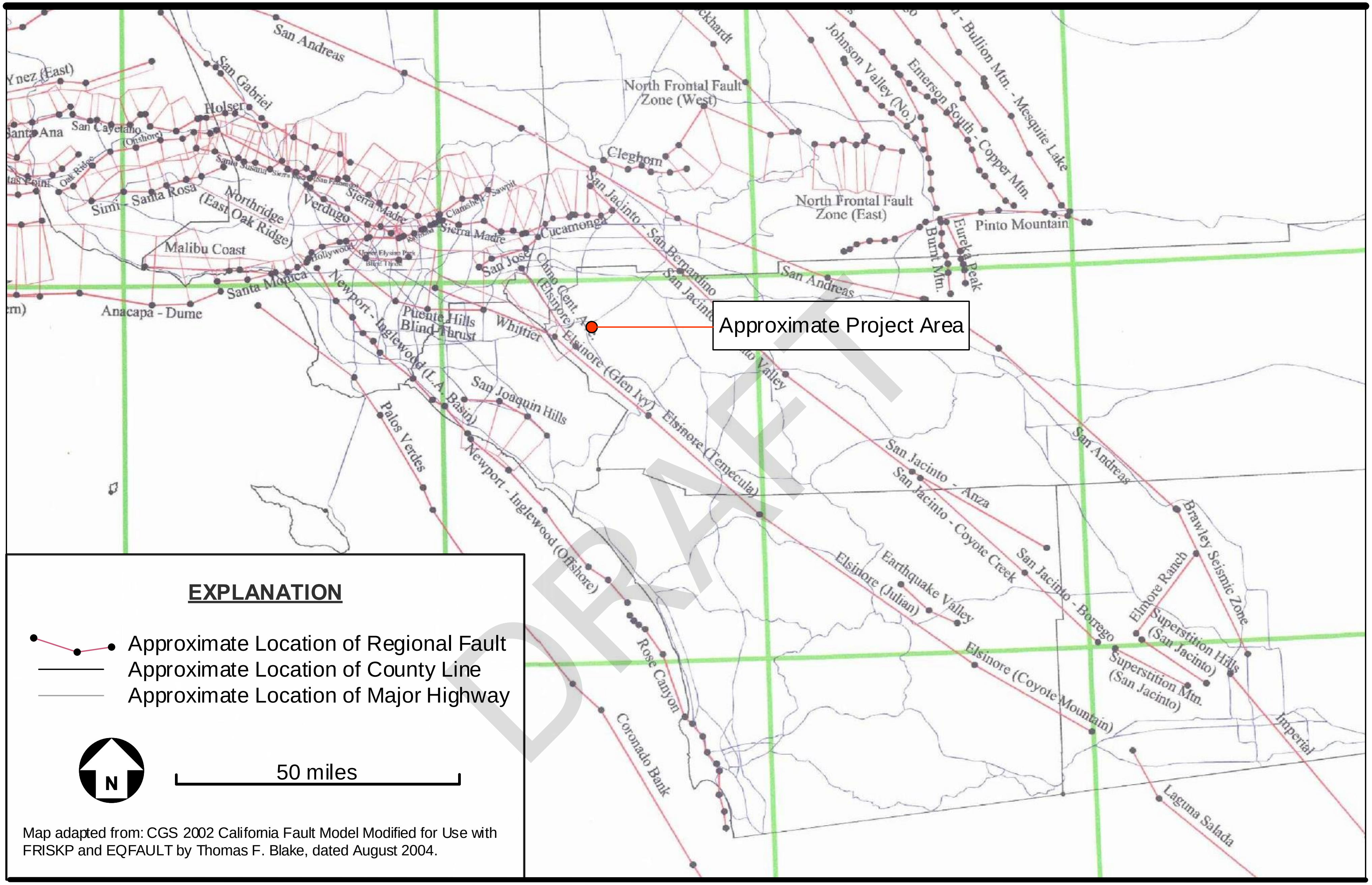
The site location relative to regional faults is shown on Figure No. 4, *Regional Fault Map* on the following page.

8.6 Seismicity

The project area is shown relative to the nearest mapped seismic hazards in Figure No. 5, *Seismic Hazard Zone Map* on the following page. The seismic hazards are described as follows.

8.6.1 Ground Motion

The Caltrans ARS Online tool version 3.0.2 (Caltrans, 2020) was used to develop the ARS seismic design curves using the site Latitude = 33.869643°, Longitude = -117.535671°. This tool complies with Caltrans Seismic Design Criteria (SDC) Version 2.0 (Caltrans SDC, 2019). The following response spectra were considered.



Regional Fault Map

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For: CNS Engineering, Inc.

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Figure No.
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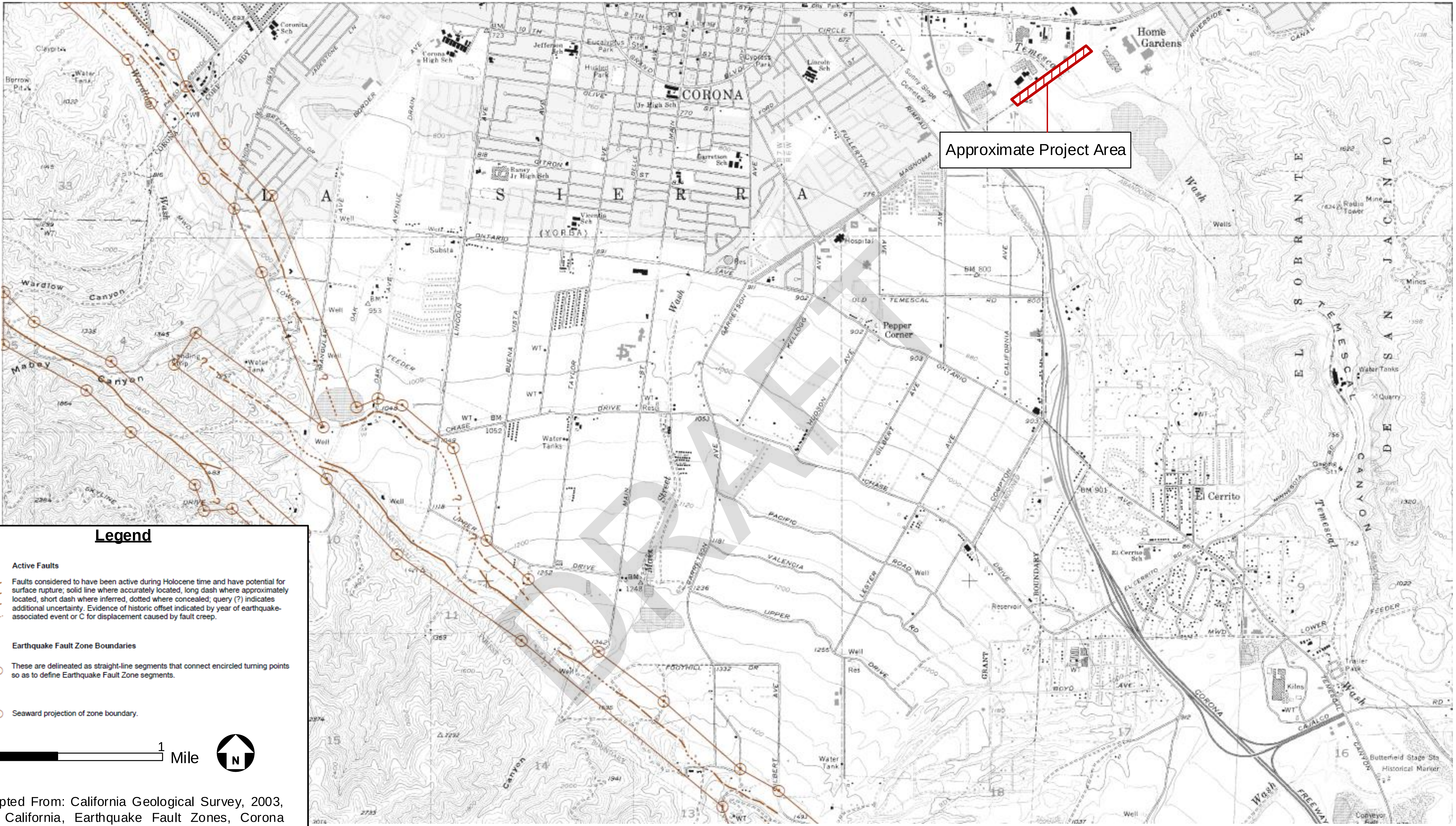

Legend

Active Faults

1906 C

Faults considered to have been active during Holocene time and have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

Earthquake Fault Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define Earthquake Fault Zone segments.

Seaward projection of zone boundary.

0 1 Mile

N

Map Adapted From: California Geological Survey, 2003, State of California, Earthquake Fault Zones, Corona South Quadrangle, scale 1:24,000, dated May 1, 2003.

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Seismic Hazard Zone Map

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Figure No.
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- The Design Response Spectrum is based on the USGS 975-year uniform hazard spectrum (5% in 50 years probability of exceedance). Adjustment factors for near-fault effects and basin amplification are also applied.
- The mean site-source distance based on a hazard deaggregation performed at 1s spectral period is 7.15 miles (11.5 km).

Based on a site-specific evaluation of average shear wave velocity (Vs30), Soil Profile Type D and Vs30 value of 905.5 feet/sec (276 m/sec) was determined and used to generate design spectrum (ARS curve) (attached in appendix F). The recommended design ARS curve is presented in Figure No. 6, *Design ARS Curve*.

Based on the above analysis, the peak ground acceleration (PGA) of the site is 0.7g. The USGS deaggregation shows the magnitude 6.47 event (site to source distance 4.03 miles = 6.48 km) contributes the most to the seismic hazard.

8.6.2 Liquefaction Potential

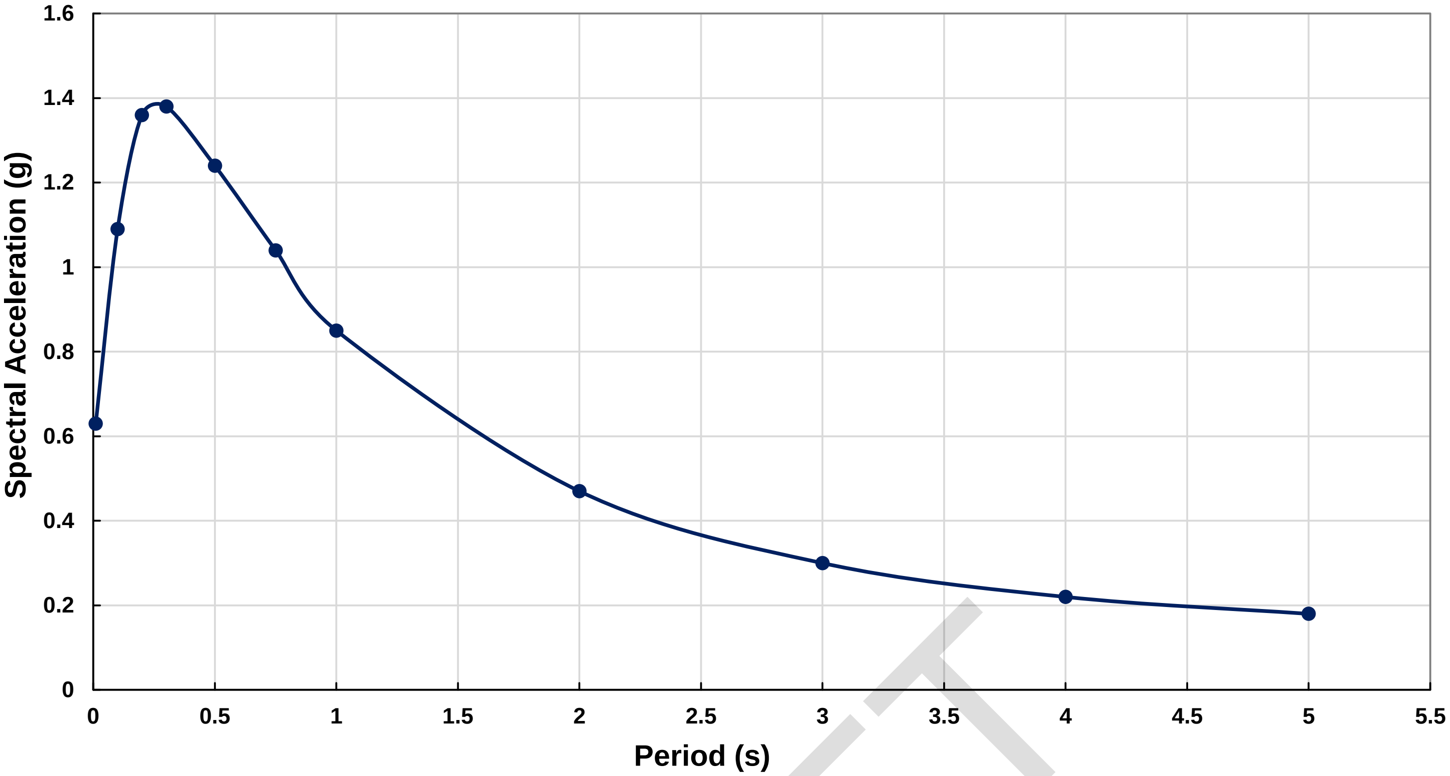
Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper 50 feet of the ground surface suffers a substantial reduction in its shear strength, due to the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows.

- Soils must be submerged.
- Soils must be primarily granular.
- Soils must be loose to medium-dense.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

The project site is located within a Riverside County-designated area of high liquefaction potential (Riverside County, 2019).

Based on a site-specific liquefaction analysis presented in Appendix E, *Liquefaction and Seismic Settlement Analyses*, the project site has negligible potential of liquefaction and dry seismic settlement under current and historic groundwater conditions.



Period (s)	Spectral Acceleration (g)
0.01	0.63
0.1	1.09
0.2	1.36
0.3	1.38
0.5	1.24
0.75	1.04
1	0.85
2	0.47
3	0.3
4	0.22
5	0.18

1. Latitude = 33.869643; Longitude = -117.535671
2. Peak Ground Acceleration (PGA) = 0.7g
3. Shear Wave Velocity (Vs30) = 276 m/s
4. Damping = 5%
5. ARS Online Tool V3.0.2 and Caltrans SDC 2.0.
6. Deaggregation analysis: Mean magnitude = 6.62
   and mean site to source distance 7.15 miles (11.5 km).

Design ARS Curve

Project:	Magnolia Avenue Bridge and Roadway Widening
Location:	El Camino Avenue to 1,000 feet East of All American Way City of Corona, Riverside County, California
For:	CNS Engineering, Inc.

Project No.
18-81-147-03

Converse Consultants

Figure No.
6

8.6.3 Fault Rupture

The site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2019). There are no known active faults projecting toward or extending across the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

The proposed site is situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site.

9.0 MATERIAL SOURCES

Converse has not evaluated any site for use as import borrow. The contractor must make his own arrangements for obtaining materials and is responsible for the grading and quality requirements.

Embankment fill will be required for the widening of the approaches to the bridges and interchanges. Quantities of fill are not known at this time. It is assumed that import material sources will be listed on the current *AB 3098 List* at the time of construction and all materials will be approved prior to importing to the site. On-site soils are expected to be adequate for use as compacted fill.

Commercial suppliers for sand, gravel, aggregate base, and concrete near the project area should be identified during the PS&E phase of the project. Existing pavement (asphalt concrete and Portland cement concrete) can be pulverized and used as aggregate base (AB). Pulverized material should be processed and must meet the requirements specified in Caltrans Standard Specifications (Caltrans, 2018). Caltrans must approve the use of pulverized material for AB. On-site soils can be a source material. However, laboratory testing will be required to conform their suitability as construction materials. Other sites as potential sources of fill or other materials were not assessed in this report.

10.0 MATERIAL DISPOSAL

Debris, topsoil, vegetation, etc., will be present at the site. These materials are unsuitable for use in construction and should be properly disposed of at an approved location or stockpiled and reused for landscaping purposes as suitable within the project. Disposal of spoils from excavated soils is expected during construction. It is the responsibility of the contractor to make arrangements to dispose of such materials and follow guidelines provided in Section 7-1.13 of the Caltrans Standard Specifications (Caltrans, 2018).

11.0 CONCLUSIONS AND RECOMMENDATIONS

Conclusions and recommendations are presented below.

11.1 Earthwork

Earthwork should conform to requirements of the Caltrans Standard Specifications (Caltrans, 2018), Section 19, *Earthwork*. Soil compaction should be accomplished in accordance with Section 19-5, *Compaction* of the Standard Specification. Fill placed during widening of the embankments should be benched into the existing slopes as described in Section 19-6, *Embankment Construction* of the Standard Specifications. Actual depths and extents of toe-of-fill keyways will be determined during site specific geotechnical investigations. All earthwork should be observed by a qualified geotechnical engineer.

In areas where compacted fill will be placed, all debris, deleterious material, and surficial soils including compressible existing topsoil, loose or soft alluvium or fill soil, dry and saturated soil, and otherwise any unsuitable materials should be removed prior to fill placement. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

11.2 Excavatability

The on-site soils should be generally excavatable with conventional heavy-duty earthmoving equipment. Excavation will be difficult due to the presence of gravel, cobbles or possible boulders.

The phrase “conventional heavy-duty excavation equipment” is intended to include commonly used equipment such as excavators, scrapers, and trenching machines. It does not include hydraulic hammers (“breakers”), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment models should be done by an experienced earthwork contractor.

11.3 Soil Expansion Potential

Coarse-grained soils (sandy soils) are generally anticipated to be non-expansive or have a very low expansion potential. Fine-grained soils (silts and clays) may be susceptible to low to high expansion potential. Soil expansion potential should be evaluated during PS&E phase of the project. If the expansion potential is very low (expansion index <20), no mitigation is necessary. If low, medium or high expansion potential is observed, mitigation should be utilized to reduce the potential for uplift and distress due to soil expansion.

Based on the soil types and laboratory test result (EI = 0), the expansion potential within the project limit is very low. So, no mitigation is required.

11.4 Soil Erosion Potential

Since the native soils are anticipated to be predominantly fine- to coarse-grained sand, silty sand and gravel, the soils can suffer moderate to severe erosion. However, the existing Temescal Wash (Channel) is concrete lined at the bottom and both sides. Therefore, the potential for surface soil erosion can be expected to be minimal.

11.5 Scour Potential

The proposed bridge improvements do not cross any unlined channels. A scour analysis is not required.

11.6 Liquefaction Potential and Seismically Induced Settlement

Based on a site-specific liquefaction analysis presented in Appendix E, Liquefaction and Seismic *Settlement Analyses*, the project site has negligible potential of liquefaction and dry seismic settlement. However, we recommend a total of 1-inch total dynamic settlement and 0.5-inch of dynamic differential settlement should be used for the design purpose.

11.7 Static Settlement

Static settlement related to bridge foundation will be presented in a separate Foundation Report.

11.8 Cuts and Excavations

Temporary and permanent cuts and excavations are anticipated for the proposed project. We expect that most of the cut slopes will be stable at slopes of 2H:1V or flatter within native soils and engineered fills unless adverse conditions are encountered, such as weak or adverse bedding planes, clay lenses or existing landslides.

11.9 Embankments and Fills

We do not identify any proposed major embankment and fills within the project limit.

11.10 Pole Foundation

Traffic signal and light post poles will be supported on CIDH Piles and should be selected based on project plans and Caltrans Standard Plans, 2018. Typically, the diameter and length of CIDH piles for signal foundation varies from 2.5 to 4.5 feet and 6 to 15 feet, respectively.

If Caltrans Standards Plans do not apply, poles can be supported on CIDH piles deriving their support primarily through skin friction. The piles may be designed for compression using an allowable skin friction value of 200 psf per foot. This value may be increased by 33 percent for transient wind and seismic forces. For pier design in tension, 50 percent of the recommended allowable skin friction values in compression may be used. For design purpose, the upper 2 feet of the soils should be neglected in determining the skin friction.

11.11 Pipeline Recommendations

An existing 30-in waterline runs over the channel, hanging with the bridge. This pipeline between Station 10+00 and Station 12+82 will be relocated approximately 100 feet southeast from the Magnolia Avenue centerline due to the bridge and roadway widening. Bore and jack (B&J) method between Station 10+64 and Station 11+90 will be used to cross the channel. Based on the encountered materials (sand, gravel, cobbles and possible boulders), B&J method will be difficult. However, appropriate means and methods should be selected by the designer and specialty constructor. Pipe bedding and trench zone backfill should be as per City of Corona Standards. The following recommendations should be considered.

11.11.1 Backfill of Boring/Jacking and Receiving Pits

The bore-and-jack crossing will require jacking and receiving pits. We understand that the depths of the boring/jacking and receiving pits will be approximately 24 feet below the existing grade. The size of boring/jacking and receiving pits are approximately 15’x40’ and 15’x15’, respectively. The pits should be backfilled following construction of the pipe crossings.

The pit bottoms should be free of trash, debris or other unsatisfactory materials at the time of backfill placement. The bottoms of the excavations should be scarified to a minimum depth of 12 inches below subgrade, moisture conditioned to within 3 percent of optimum moisture content, and recompacted to at least 90 percent of the laboratory maximum dry density.

The backfill soils should be well-blended, and moisture conditioned to within 3 percent of optimum moisture content. Particles larger than 6 inches should not be used as backfill materials. The backfill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 90 percent of the laboratory maximum dry density per ASTM Standard D1557. If the ground surface is to be paved, the backfill within 12 inches of the pavement subgrade should be compacted to at least 95 percent of the laboratory maximum dry density. Shoring should be removed gradually while backfilling to prevent side soils from caving.

The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, existing facilities, utilities, or completed work.

11.11.2 Jacking Force

The pipe jacking force is function of soil conditions, over burden pressure, pipe weight, size, annular space between pipe and soil, lubricant of the pipe, and installation time. The jacking force is equal to penetration resistance plus frictional resistance. Proper assessment of jacking force is required to design and select jacking pipes and thrust block.

The penetration resistance varies along the bore-and-jack depending on soil type and shape and steering action of the boring head.

Presence of concentrated gravel, cobbles and boulders in the path of bore-and-jack operation can bring a sudden increase in the jacking force. Therefore, installation of pressure relief valves at the pit and indicators on the control panel is desirable to ensure that the allowable jacking force is not exceeded. Based on the information from Erik Howard with ERSC, if any refusal occurs during angering, contractor will pull the auger and remove the obstruction manually. Once cleared, the auger would be reinserted to continue the B&J operation.

Design parameters presented Table No. 2, *Jacking System Design Parameters*, may be used to design jacking force system.

Table No. 2, Jacking System Design Parameters

Locations	Parameter	Value
Temescal Wash (Channel)	Bearing Pressure (psf)	2,500
	At-rest Lateral Earth Pressure (psf)	58
	Passive Earth Pressure (psf)	250
	Soil Unit weight (pcf)	120
	Friction, between soil and steel	0.25

Note:
No borings were drilled for the purpose of B&J. However, borings A-20-004 and O-20-001 are located very close to the proposed receiving pit. For boring/jacking pit, as-built LOTB can be used.

We recommend that the ultimate compressive strength of the pipe should be at least 2.5 times the design jacking loads of the pipe.

The pipe designer should determine an appropriate factor of safety to be incorporated into the design of thrust block. The bore-and-jack contractor is responsible for selection of jacking force system and the final design of thrust blocks.

The jacking operations should always be controlled to minimize loss of ground. Steel casing sections should be jacked forward concurrently with the boring operation to provide continuous ground support.

A welded steel pipe casing is required to be installed at the crossing location. The annulus should be injected with cellular concrete or grout to fill any possible voids created by the crossing operation.

11.12 Slope Stability

The existing channel bottom and sides are concrete lined. After bridge and roadway widening, it will be lined again with concrete. Therefore, we do not anticipate any issues with the stability of the channel sides.

11.13 Infiltration Rate

One percolation test was performed on October 15, 2020 at boring A-20-001 in accordance with the Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011). The percolation test result is tabulated in the following table.

Table No. 3, Estimated Infiltration Rate

Boring No.	Depth of Boring (feet)	Predominant Soil Types (USCS)	Average Infiltration Rate (inches/hour)
A-20-001	15	Silty Sand (SM)	2.55

A combined safety factor of 3.44, provided to us by Ceazar Aguilar with Aguilar Consulting, Inc. was applied to the measured infiltration rates to account suitability assessment and design factors. Details of the percolation tests are presented in Appendix D, Percolation Testing. The designer should determine whether additional design-related safety factors are required and for design of the proposed infiltration system.

11.14 Soil Corrosivity

Typically, fine-grained soils (silts and clays) increase site corrosive conditions, whereas coarse-grained soils (sand) tend to be non-corrosive. According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater. A minimum resistivity value less than 1,100 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion.

Corrosion test results, presented in Table B-4 in *Appendix B*, indicate the soils are non-corrosive based on Caltrans Corrosion Guidelines. Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

12.0 CONSTRUCTION CONSIDERATIONS

Considerations for the proposed improvements are presented below.

12.1 General

Prior to the start of construction, all existing underground utilities should be located along the pipeline alignment. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Vertical braced excavations are feasible along the pipeline alignment. Sloped excavations may not be feasible in locations adjacent to existing utilities (if any).

Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the owner’s representative and the competent person employed by the contractor in accordance with regulations. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

12.2 Pile foundation Construction

Bridge pile foundation construction recommendations will be provided in a separate Foundation Report.

12.3 Temporary Sloped Excavations

Temporary open-cut trenches may be constructed in areas not adjacent to existing underground utilities improvements with side slopes as recommended in the table below. Temporary cuts encountering soft and wet fine-grained soils, dry loose, cohesionless soils, or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 4, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand (SM), Sand with Silt (SP-SM) and Sand (SP)	C	0-10	1.5:1
		10-20	2:1

[1] Slope ratio is assumed to be constant from top to toe of slope, with level adjacent ground.

For shallow excavations up to 4 feet bgs, slope can be vertical. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trench shields should be provided by the contractor as necessary to protect the workers in the excavation.

Surfaces exposed in sloped excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within 5 feet of the unsupported slope edge. Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

12.4 Shoring Design

Temporary shoring will be required where open sloped excavations will not be feasible due to unstable soils or due to nearby existing structures or facilities. Temporary shoring may consist of conventional soldier piles and lagging or sheet piles or any piles selected by contractor. The shoring for the pipe excavations may be laterally supported by walers and cross bracing or may be cantilevered. Drilled excavations for soldier piles will require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation.

The active earth pressure behind any shoring depends primarily on the allowable movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures.

The lateral earth pressures to be used in the design of shoring is presented in the following table.

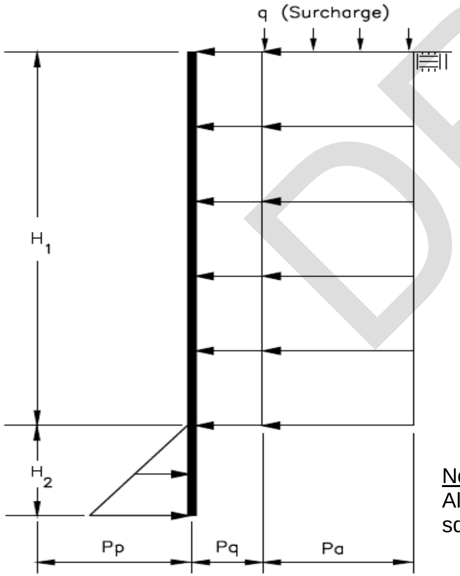
Table No. 5, Lateral Earth Pressures for Temporary Shoring

Lateral Resistance Soil Parameters*	Value
Active Earth Pressure (Braced Shoring) (psf) (A)	23
Active Earth Pressure (Cantilever Shoring) (psf) (B)	38
At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)	58
Passive earth pressure (psf per foot of depth) (D)	250
Maximum allowable bearing pressure against native soils (psf) (E)	2,500
Coefficient of friction between sheet pile and native soils, fs (F)	0.25

* Parameters A through F are used in Figures No. 3 and 4 below.

Restrained (braced) shoring systems should be designed based on Figure No. 7, *Lateral Earth Pressures for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

Figure No. 7, Lateral Earth Pressures for Temporary Braced Excavation



Total Earth Pressure, P

P = Pq + Pa

$P_q = 0.5q$ - incremental surcharge pressure

$P_a = (A)H_1$ - active earth pressure (Braced walls)

Lateral Pressure Resistance

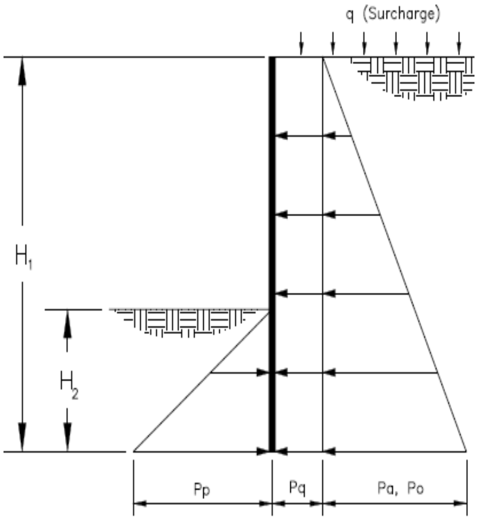
$P_p = (D)\ H_2 \leq (E)$ psf - passive earth pressure (on native soils)

$\mu = (F)$ - ultimate friction coefficient between steel sheet piles and soil

Note:
All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).

Unrestrained (cantilever) design of cantilever shoring consisting of soldier piles spaced at least two diameters on-center or sheet piles, can be based on Figure No. 8, *Lateral Earth Pressures on Temporary Cantilever Wall*.

Figure No. 8, Lateral Earth Pressures on Temporary Cantilever Wall



Total Earth Pressure, P

P = Pq + Pa, Po

Pq = 0.5q - incremental surcharge pressure

Pa = (B)H_1 - active earth pressure (Un-restrained)

Po = (C)H_1 - at rest earth pressure (Restrained)

Lateral Pressure Resistance

Pp = (D) $H_2 \leq$ (E) psf - passive earth pressure (on native soils)

μ = (F) - ultimate friction coefficient between steel sheet piles and soil

Note:
All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).

The provided pressures assume no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below the ground-water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

Passive resistance includes a safety factor of 1.5. The upper 1 foot for passive resistance should be ignored unless the surface is confined by a pavement or slab.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. As previously mentioned, all shoring should be designed and installed in accordance with state and federal safety regulations.

The contractor should have provisions for soldier pile and sheet pile removal. All voids resulting from removal of shoring should be filled. The method for filling voids should be selected by the contractor, depending on construction conditions, void dimensions and available materials. The acceptable materials, in general, should be non-deleterious, and able to flow into the voids created by shoring removal (e.g., concrete slurry, “pea” gravel, etc.).

Excavations for the proposed pipeline should not extend below a 1:1 horizontal:vertical (H:V) plane extending from the bottom of any existing structures, utility lines or streets. Any proposed excavation should not cause loss of bearing and/or lateral supports of the existing utilities or streets.

If the excavation extends below a 1:1 (H:V) plane extending from the bottom of the existing structures, utility lines or streets, a maximum of 10 feet of slope face parallel to the existing improvement should be exposed at a time to reduce the potential for instability. Backfill should be accomplished in the shortest period of time and in alternating sections.

12.5 Trenchless Pipe Crossing Recommendations

Trenchless pipe crossing recommendations are presented in the following subsections.

12.5.1 Ground Classification for Trenchless Pipe Crossing

The Tunnelman's Ground Classification (USDOT, 2009) categorizes predictive soil behaviors for saturated and unsaturated conditions as presented in the Table No. 6, *Tunnelman's Ground Classification for Soils*.

Table No. 6, Tunnelman's Ground Classification for Soils

Ground Classification	Ground Behavior	Typical Soil Types
Hard	Tunnel heading may be advanced without roof support.	Cemented sand and gravel and over-consolidated clay above the ground water table.
Firm	Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water tale, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing	Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.

Table No. 6, Tunnelman's Ground Classification for Soils (continued)

Ground Classification	Ground Behavior	Typical Soil Types
Swelling	Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.
Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (approx. 30º -35º). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry angular materials.
Cohesive Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (approx. 30º -35º). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs.
Flowing	A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.

It is our opinion that trenchless construction at the proposed location can be accomplished by an experienced contractor using bore and jack equipment. Provisions for controlling raveling and running sandy soils should be provided during the trenchless operation to minimize ground loss and ground subsidence.

It is the contractor's responsibility to design and select the appropriate bore and jack construction method, support system and to follow the requirements of the health and safety rules of the State of California pertaining to tunnel construction and permit requirements of the Riverside County, and other local agencies, if applicable.

12.5.2 Bore and Jack Construction Recommendations

Bore-and-jack is a trenchless construction method for installing pipes where open-cut technique is not feasible. This is a multi-stage process of construction which includes a temporary horizontal jacking platform and a starting alignment track in an entrance pit at a desired elevation. Manual control is used to jack the pipe at the starting point of the alignment with simultaneous excavation of the soil being accomplished by a rotating cutting head in the leading edge of the pipe's annular space.

The selection of trenchless pipe crossing methods and equipment depends on pipe material, length of crossing, and anticipated ground conditions, and should be made by the contractor. Bore-and-jack pipe construction operations involve the initial construction of a jacking/tunneling pit and a receiving pit at each end of the pipe segment to be jacked. Site-specific ground conditions and soil classifications pertaining to this project are presented in the following table.

Table No. 7, Site-Specific Ground Classifications

Crossing Location	Boring No.	Approximate Depth (Feet)*	Soil Types	Ground Classification
Temescal Wash (Channel)	A-20-004/O-20-001	30	SM, SP-SM, SP, GP-GM with gravel, cobbles and boulders	Running and Raveling
	B-2 (As-Built)	30	SP, GP with gravel, cobbles and boulders	
Note: *Depth presented up to 30 feet bgs due the proposed depth of pits is approximately 24 feet bgs.				

The working/access shafts are utilized to remove the spoil and to transport the construction materials and personnel for a bore-and-jack project. The vertical face of the working shaft may be shored with sheet piles and/or soldier piles and lagging. The face of the shaft also can be supported by ribs and laggings. The design of sheet piling, soldier beam and lagging system may be designed according to the recommendations provided in Section 12.3, *Shoring Design*. Frequent contact grouting may be necessary to reinforce the support during construction.

The total load that can be developed in the jacking plate would depend on the depth and area of the plate. The jacking equipment should not impose a reaction of more than the allowable net bearing pressure summarized in Table No. 2, *Jacking System Design Parameters* on the stabilized soils within the boring/jacking pit.

Grouting through the pipe casing after jacking is recommended to fill any possible voids created by the jacking operation. Jacking operations should be performed in accordance with the Standard Specifications for Public Works Construction, Sections 306-2 and 306-3 (Public Works Standards, 2018). Contractor should maintain standard grouting method so that no heave occurs.

Excavation procedures and shoring systems should be properly designed and implemented/installed to minimize the effect of settlement during construction. The contractor is responsible for minimizing impacts of crossing operations. Ground distress potential along a crossing alignment depends on a number of factors, including type of soils, type of face support, internal pressure maintained to support the face, length of unlined zone, if any, and the amount of gap between the shield and the surrounding soils. The potential of any significant ground distress at the surface can be minimized by selecting the proper equipment and construction method.

The zone of influence of properly performed pipe crossing should be limited to a distance of about 2D above the crown of the shield, where D is the diameter of the shield. When the depth of crown cover is about 2D or more, maximum ground surface settlement, if any, can be expected to be less than the thickness of the gap around the pipe. Higher ground settlement may occur for less depth of cover and inadequately supported pits can induce significant ground movement or even collapse.

It is the contractor's responsibility to document the existing pre-construction conditions of streets and any facilities, and monitor deformations during construction. We recommend that the ground surface above crossing operations be continuously monitored during construction using a surface settlement monument to make sure any vertical and horizontal movements are within allowable limits. Corrective action will be required by the contractor if deformations exceed the allowable limits.

12.6 Construction Monitoring

The project geotechnical consultant should review plans and specifications as the project design progresses. Such review is necessary to identify design elements, assumptions, or new conditions which require revisions or additions to our geotechnical recommendations.

The project geotechnical consultant should be present to observe conditions during construction. Testing should be performed to determine density and moisture of the during construction as needed to verify compliance with project specifications. Additional geotechnical recommendations may be required based on subsurface conditions encountered during construction.

13.0 LIMITATIONS

Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that the recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

14.0 REFERENCES

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Appendix A

Field Exploration

APPENDIX A

FIELD EXPLORATION

Our field investigation included site reconnaissance and a subsurface exploration program consisting of drilling soil borings. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by CNS. The boring locations should be considered accurate only to the degree implied by the method used to mark them in the field.

Six exploratory borings (A-20-001 through A-20-005 and O-20-001) were drilled to investigate the subsurface conditions for the project. The borings (A-20-001 through A-20-005) were advanced using a standard CME 85 drill rig equipped with 8-inch diameter hollow-stem augers. The hammer energy transfer ratio of the drill rig is 86.2 percent (attached in appendix A-1). Due to the presence of cobbles and boulders, borings at the bottom of the bridge foundation could not be penetrated up to the maximum required depth of 90 feet bgs. Therefore, one additional boring (O-20-001) was drilled using Becker Hammer up to 90 feet bgs. The Becker hammer energy transfer ratio is 86.2 and 83 percent (attached in appendix A-1). A summary of boring information is presented in the following table.

Table No. A-1, Summary of Borings

Boring No.	Associated Improvements	Location		Approx. Station	Approx. Ground Surface Elev. (feet, NAVD 88)	Boring Depth (ft, bgs)	Date Completed
		Latitude	Longitude				
A-20-001	Percolation	33.8683N	117.5382W	25+00	645.47	16.5	10/15/2020
A-20-002	Roadway	33.8686N	117.5377W	26+50	646.79	16.5	10/15/2020
*A-20-003	Bridge	33.8697N	117.5358W	33+75	646.84	20.5	10/6/2020
*A-20-004	Bridge	33.8696N	117.5352W	35+20	647.78	32.0	10/6/2020
A-20-005	Roadway	33.8711N	117.5334W	42+80	647.78	11.5	10/7/2020
**O-20-001	Bridge	33.8696N	117.5351W	35+20	644.78	90.0	11/4/2020

Notes:
Stations and ground surface elevations were based on the project plans provided by CNS.
*Borings were terminated due to presence of cobbles and possible boulders.
**Becker Hammer was used to drill.

Encountered earth materials were continuously logged and visually classified in the field using Unified Soil Classification System by a Converse staff engineer. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3 inches outside diameter) lined with thin sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches. Blow counts at each sample interval are presented on the boring logs. Samples were retained in brass rings (2.4-inches inside diameter and 1 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of representative soil types were also collected.

Standard Penetration Testing (SPT) was also performed in borings A-20-002, A-20-003 and A-20-004 in accordance with the ASTM Standard D1586 test method using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Boring Records.

The Becker Hammer Drill is capable to penetrate through cobbles and boulders. The discharged material is accumulated in suitable containers as it emerges from the cyclone, and drive samples are taken at specified intervals for analysis of the materials drilled.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between driven samples are indicated in the log at the top of the next drive sample.

Following the completion of logging and sampling, the borings (A-20-002 through A-20-005) performed with drill rig were backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight, and, where applicable, the surface was patched with cold asphalt concrete. The boring (O-20-001) performed with Becker Hammer was backfilled with mix of soil cuttings and cement and compacted by pushing down with augers using drill rig weight. After completion of percolation test in boring (A-20-001), the pipe was cut below the asphalt surface, backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight and surface patched with cold asphalt concrete.

If construction is delayed, the surface may settle over time. We recommend the contractor of record monitor the boring locations and backfill any depressions that might occur or provide protection around the area of the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring records, refer to, *Key to Boring Records.* Logs of the exploratory borings are presented in, *Boring Records*.

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Name
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND and SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Name
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; CTM 417-06; CTM 422-06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeablility (CTM 220-05)
PA	Particle Size Analysis (ASTM D422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301-00)
SE	Sand Equivalent (CTM 217-99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Mud Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- ▽ First Water Level Reading (during drilling)
- ▼ Static Water Level Reading (short-term)
- ▼ Static Water Level Reading (long-term)

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REPORT TITLE	HOLE ID
BORING RECORD	**Key**

DIST	COUNTY	ROUTE	POSTMILE	EA
8	**Riverside**		**40.9**	

PROJECT OR BRIDGE NAME
Magnolia Avenue Bridge and Roadway Widening

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
BR NO.56C-0199	**Mahmoud Suliman**	**12/14/2020**	**1 of 2**

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	<0.25	<0.25	<0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	>4.0	>4.0	>2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	>50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OF PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

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REPORT TITLE **BORING RECORD**				HOLE ID **Key**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **2 of 2**	

LOGGED BY **Mahmoud Suliman** BEGIN DATE **10/15/2020** COMPLETION DATE **10/15/2020**	BOREHOLE LOCATION (Lat/Long) **33.86832° N, 117.53815° W**	HOLE ID **A-20-001**
DRILLING CONTRACTOR **2 R drilling**	BOREHOLE LOCATION (Station, Offset, Line) **25+00**	SURFACE ELEVATION **644.8 ft**
DRILLING METHOD **Hollow Stem Auger**	DRILL RIG **CME 75**	BOREHOLE DIAMETER **8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **86.2 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings**	GROUNDWATER READING DURING DRILLING **Not Encountered** AFTER DRILLING	TOTAL DEPTH OF BORING **16.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	0 – 0.8	■	10" ASPHALT CONCRETE/ NO BASE El. 644.0'												
	0.8	▨	ALLUVIUM: Well-graded SAND with SILT and GRAVEL (SP-SM): , yellowish brown to brown, fine to coarse-grained, little gravel up to 2.5" and few cobbles up to 4.5" in largest dimension.	▨											
				⧗	B1 D2	11 20 27	57	100		3	124.0				PA
				▨											
				⧗	D3	11 22 34	56	NR							
				⧗	D4	14 19 32	51	100		2	117.7				
				⧗	D5	12 19 30	49	100		1	101.7				
				▨	B6										
629.8	15.0	▨	El. 629.8' SILTY SAND (SM): , brown, fine to coarse-grained.	⧗	D7	4 5 8	13	100		4	116.8				
628.3	16.5		El. 628.3'												

Bottom of Borehole at 16.5 feet bgs.
End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole prepared for percolation test on 10/15/2020.
After completion the test, pipe was cut below the asphalt surface, Borehole backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight and surface patched with cold asphalt concrete on 10/15/2020.

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REPORT TITLE **BORING RECORD (PERCOLATION_)**				HOLE ID **A-20-001**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **1 of 1**	

LOGGED BY **Mahmoud Suliman**	BEGIN DATE **10/15/2020**	COMPLETION DATE **10/15/2020**
BOREHOLE LOCATION (Lat/Long) **33.86859° N, 117.53767° W**	HOLE ID **A-20-002**	
DRILLING CONTRACTOR **2 R drilling**	BOREHOLE LOCATION (Station, Offset, Line) **26+50**	SURFACE ELEVATION **645.5 ft**
DRILLING METHOD **Hollow Stem Auger**	DRILL RIG **CME 75**	BOREHOLE DIAMETER **8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **86.2 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings**	GROUNDWATER READING DURING DRILLING AFTER DRILLING **Not Encountered**	TOTAL DEPTH OF BORING **16.5 ft**

DEPTH (ft)	DESCRIPTION	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
0.7	7.5" ASPHALT CONCRETE/ NO BASE El. 644.8'											
	ALLUVIUM: Poorly-graded SAND WITH SILT and GRAVEL (SP-SM): brown, fine to coarse-grained, some gravel up to 2" in largest dimension.	B1										R
		D2	11 13 13	26	100		2	109.3				
		D3	11 14 16	30	100		3	98.2				
		B4										
		D5	10 17 21	38	100		2	109.9				
		D6	11 15 15	30	100		1	106.2				
15.0	El. 630.5'											
16.5	SILTY SAND (SM): brown, fine to coarse-grained, trace clay. El. 629.0'	D7	4 5 8	13	100		18	104.8				

Bottom of Borehole at 16.5 feet bgs.
End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight and surface patched with cold asphalt concrete on 10/15/2020.

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REPORT TITLE **BORING RECORD (ROADWAY)**				HOLE ID **A-20-002**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **1 of 1**	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long)	HOLE ID
Mahmoud Suliman	**10/6/2020**	**10/6/2020**	**33.86973° N, 117.53582° W**	**A-20-003**

DRILLING CONTRACTOR
2 R drilling

BOREHOLE LOCATION (Station, Offset, Line)
33+75

SURFACE ELEVATION
646.8 ft

DRILLING METHOD
Hollow Stem Auger

DRILL RIG
CME 75

BOREHOLE DIAMETER
8" in

SAMPLER TYPE(S) AND SIZE(S) (ID)
SPT (1.4"), Modcal (2.4")

SPT HAMMER TYPE
Automatic, Weight = 140 lbs/ Drop = 30"

HAMMER EFFICIENCY, ERi
86.2 %

BOREHOLE BACKFILL AND COMPLETION
Soil Cuttings

GROUNDWATER READING DURING DRILLING AFTER DRILLING
Not Encountered

TOTAL DEPTH OF BORING
20.5 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
646	0 1 2		ALLUVIUM SILTY SAND with Gravel (SM): brown, fine to coarse-grained, little gravel up to 1" in largest dimension.		B1										
644	3 4				D2	40 40 42	82	100		5	121.3				PA
642	5		5.0 El. 641.8'												
	6		Poorly-graded SAND with SILT and GRAVEL (SP-SM): yellowish brown to brown, fine to coarse-grained, some gravel up to 1.5" in largest dimension.		D3	15 19 15	34	100		2	108.5				
640	7				B4										CP SE
638	8 9				D5	8.18 22	40	100		1	112.0				
636	10 11				D6	8 11 20	31	100		1	105.7				
634	12 13 14														
632	15		15.0 El. 631.8'												
	16		Poorly-graded GRAVEL with SAND and SILT (GP-GM): yellowish brown to brown, fine to coarse-grained sand, some gravel up to 2" and scattered cobbles up to 4" in the largest dimension.		D7	20 50-6"	70+	NR							
630 628	17 18 19														
	20		20.5 El. 626.3'		S8	50-4"	100+	NR							
626 624 622 620 618	21 22 23 24 25 26 27 28 29 30		Bottom of Borehole at 20.5 feet bgs. Boring terminated at 20.5 feet bgs due to refusal on cobbles and possible boulders. No groundwater encountered. Borehole backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight on 10/6/2020												

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REPORT TITLE
BORING RECORD (BRIDGE)

HOLE ID
A-20-003

DIST	COUNTY	ROUTE	POSTMILE	EA
8	**Riverside**		**40.9**	

PROJECT OR BRIDGE NAME
Magnolia Avenue Bridge and Roadway Widening

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
BR NO.56C-0199	**Mahmoud Suliman**	**12/14/2020**	**1 of 1**

LOGGED BY **Mahmoud Suliman**	BEGIN DATE **10/6/2020**	COMPLETION DATE **10/6/2020**	BOREHOLE LOCATION (Lat/Long) **33.86957° N, 117.53516° W**	HOLE ID **A-20-004**
DRILLING CONTRACTOR **2 R drilling**			BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Hollow Stem Auger**			DRILL RIG **CME 75**	BOREHOLE DIAMETER **8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**			SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **86.2 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings**			GROUNDWATER READING DURING DRILLING AFTER DRILLING **Not Encountered**	TOTAL DEPTH OF BORING **31.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	0		ALLUVIUM Silty SAND with GRAVEL (SM): brown, fine to coarse-grained, little gravel up to 1.5" in largest dimension.												
646	1														
	2				B1										
644	3				D2	33 44 37	81	100		4	107.7				SE
	4														
642	5		5.0 El. 641.8' Poorly-graded SAND with SILT and GRAVEL (SP-SM): brown, fine to coarse-grained, some gravel up to 3" and scattered cobbles up to 4.5" in largest dimension.												
	6				D3	17 24 34	58	100		3	112				
640	7														
	8				D4	50-5"	100+	NR							
638	9														
	10				D5	50-1"	100+	NR							
636	11														
	12														
634	13														
	14														
632	15				D6	13 50-4"	63+	70		2	95.5				
	16														
630	17														
	18														
628	19														
	20														
626	21				S7	17 16 30	43	100							
	22														
624	23														
	24														
622	25		25.0 El. 621.8' Poorly-graded GRAVEL with SAND and SILT (GP-GM): grayish brown, fine to coarse-grained, some gravel up to 3" and scattered cobbles up to 5" in largest dimension.		D8	50-0"	100+	NR							
	26														
620	27														
	28														
618	29														
	30														

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REPORT TITLE **BORING RECORD (BRIDGE)**				HOLE ID **A-20-004**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**		DATE **12/14/2020**	SHEET **1 of 2**

LOGGED BY **Mahmoud Suliman**	BEGIN DATE **10/6/2020**	COMPLETION DATE **10/6/2020**	BOREHOLE LOCATION (Lat/Long) **33.86957° N, 117.53516° W**
HOLE ID **A-20-004**	DRILLING CONTRACTOR **2 R drilling**	BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Hollow Stem Auger**	DRILL RIG **CME 75**	BOREHOLE DIAMETER **8" in**	
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **86.2 %**	
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings**	GROUNDWATER READING DURING DRILLING AFTER DRILLING **Not Encountered**	TOTAL DEPTH OF BORING **31.5 ft**	

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	30														
616	31		ALLUVIUM Poorly-graded GRAVEL with SAND and SILT (GP-GM): grayish brown, fine to coarse-grained, some gravel up to 3" and scattered cobbles up to 5" in largest dimension.	☒	S9	7 12 21	33	100							
	31.5		El. 615.3' Bottom of Borehole at 31.5 feet bgs. Boring terminated at 31.5 feet bgs due to refusal on cobbles and possible boulders. No groundwater encountered. Borehole backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight on 10/6/2020.												
	32														
614	33														
	34														
612	35														
	36														
610	37														
	38														
608	39														
	40														
606	41														
	42														
604	43														
	44														
602	45														
	46														
600	47														
	48														
598	49														
	50														
596	51														
	52														
594	53														
	54														
592	55														
	56														
590	57														
	58														
588	59														
	60														

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REPORT TITLE **BORING RECORD (BRIDGE)**			HOLE ID **A-20-004**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**
EA			
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**			
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **2 of 2**

LOGGED BY **Mahmoud Suliman** BEGIN DATE **10/7/2020** COMPLETION DATE **10/7/2020**	BOREHOLE LOCATION (Lat/Long) **33.87105° N, 117.53342° W**	HOLE ID **A-20-005**
DRILLING CONTRACTOR **2 R drilling**	BOREHOLE LOCATION (Station, Offset, Line) **42+80**	SURFACE ELEVATION **647.8 ft**
DRILLING METHOD **Hollow Stem Auger**	DRILL RIG **CME 75**	BOREHOLE DIAMETER **8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **86.2 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings**	GROUNDWATER READING DURING DRILLING AFTER DRILLING **Not Encountered**	TOTAL DEPTH OF BORING **11.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	0 1	■	6.5" ASPHALT CONCRETE/ 11" AGGREGATE BASE 1.5 El. 646.3'												
646	2	⋯	Poorly-graded SAND with GRAVEL (SP): brown, fine to coarse-grained, little gravel up to 1" in largest dimension.	▨											
644	3 4	⋯		⧗	B1 D2	8 9 12	21	100		1	111.5				R
	4 5	⋯		▨											
642	5 6	⋯		⧗	D3	9 15 20	35	100		2	112.1				
	7	⋯		▨	B4										
640	8 9	⋯		⧗	D5	9 18 19	37	100		2	108.9				PA
638	10	⋯		▨											
	11	⋯	11.0 El. 636.8'	⧗	D6	36 50-5"	86+	67		2	113.8				
636 634 632 630 628 626 624 622 620 618	12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30		Bottom of Borehole at 11.0 feet bgs. End of boring at 11.0 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings, compacted by pushing down with augers using drill rig weight and surface patched with cold asphalt concrete on 10/7/2020.												

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REPORT TITLE **BORING RECORD (ROADWAY)**				HOLE ID **A-20-005**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**		DATE **12/14/2020**	SHEET **1 of 1**

LOGGED BY **Mahmoud Suliman** BEGIN DATE **11/4/2020** COMPLETION DATE **11/4/2020**	BOREHOLE LOCATION (Lat/Long) **33.86956° N, 117.53514° W**	HOLE ID **O-20-001**
DRILLING CONTRACTOR **Great West Drilling**	BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Becker Hammer**	DRILL RIG	BOREHOLE DIAMETER **6 5/8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **83 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings with Cement Mix**	GROUNDWATER READING DURING DRILLING AFTER DRILLING **50.0 ft**	TOTAL DEPTH OF BORING **90.1 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
646 644	0 1 2 3 4		ALLUVIUM: Silty SAND with GRAVEL (SM): brown, fine to coarse-grained, little gravel up to 1.5" and scattered cobbles up to 4" in largest dimension.												
642	5		5.0 El. 641.8'												
640 638	6 7 8 9		Well-graded SAND with SILT and GRAVEL (SP-SM): brown, fine to coarse-grained, little gravel up to 2" and scattered cobbles up to 4" in largest dimension.		B1										
636	10 11				D3	40 41 45	86	100		5	108.7				
634 632 630 628	12 13 14 15 16 17 18 19				B2										CR EI PA
	20		20.0 El. 626.8'												
626	21		Poorly-graded SAND with GRAVEL (SP): grayish brown, fine to coarse-grained, some gravel up to 3", scattered cobbles up to 4.5" in largest dimension.		S5	13 22 22	44	100		6.5	110.6				
624	22 23 24				B4										CR CP PA DS
622	25		25.0 El. 621.8'												
620 618	26 27 28 29		Poorly-graded GRAVEL with SAND and SILT (GP-GM): Very Dense, grayish brown, moist, fine to coarse-grained, little gravel up to 3", few cobbles up to 4.5" in largest dimension, possible boulders.												
	30		30.0 El. 616.8'												

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REPORT TITLE **BORING RECORD (BRIDGE)**				HOLE ID **O-20-001**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**		SHEET **1 of 4**

LOGGED BY **Mahmoud Suliman**	BEGIN DATE **11/4/2020**	COMPLETION DATE **11/4/2020**	BOREHOLE LOCATION (Lat/Long) **33.86956° N, 117.53514° W**	HOLE ID **O-20-001**
DRILLING CONTRACTOR **Great West Drilling**			BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Becker Hammer**			DRILL RIG	BOREHOLE DIAMETER **6 5/8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**			SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **83 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings with Cement Mix**			GROUNDWATER READING — DURING DRILLING — AFTER DRILLING **50.0 ft**	TOTAL DEPTH OF BORING **90.1 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	30		ALLUVIUM: Well-graded SAND with SILT and GRAVEL (SP-SM): brown, fine to coarse-grained, some gravel up to 2" and scattered cobbles up to 4" in largest dimension.		D6	50-3"	100+	NR							
616 614 612	31 32 33 34 35 36				B7										CR PA
	36.5		El. 610.3'												
610 608	37 38 39 40		SANDY CLAY (CL): olive brown, fine to medium-grained, scattered gravel up to 2.5" in largest dimension.												
606	41				S8	4 17 30	47	100							
604	42 43 44				B9										CR PA
602	45.0		El. 601.8'												
600 598	45 46 47 48 49 50 ▼		Poorly-graded SAND with GRAVEL (SP): grayish brown, fine to coarse-grained, little gravel up to 3"and scattered cobbles up to 4.5" in largest dimension.												
596	51				D10	25 32 39	71	100		17.9	101.0				DS
594 592	52 53 54 55				B11										
590 588	56 57 58 59														
	60.0		El. 586.8'												

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REPORT TITLE **BORING RECORD (BRIDGE)**				HOLE ID **O-20-001**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **2 of 4**	

LOGGED BY **Mahmoud Suliman**	BEGIN DATE **11/4/2020**	COMPLETION DATE **11/4/2020**	BOREHOLE LOCATION (Lat/Long) **33.86956° N, 117.53514° W**	HOLE ID **O-20-001**
DRILLING CONTRACTOR **Great West Drilling**			BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Becker Hammer**			DRILL RIG	BOREHOLE DIAMETER **6 5/8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**			SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **83 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings with Cement Mix**			GROUNDWATER READING — DURING DRILLING — AFTER DRILLING **50.0 ft**	TOTAL DEPTH OF BORING **90.1 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	60	⁘	ALLUVIUM: Poorly-graded SAND with GRAVEL (SP): grayish brown, fine to coarse-grained, little gravel up to 3"and scattered cobbles up to 4.5" in largest dimension.	▨											
586	61	⁘		▨											
	62	⁘		▨	B12										
584	63	⁘		▨											
	64	⁘		▨											
582	65	⁘													
	66	⁘													
580	67	⁘													
	68	⁘													
578	69	⁘	70.0 El. 576.8'												
	70	▧	SANDY CLAY (CL): olive brown, fine to medium-grained, scattered gravel up to 2.5" in largest dimension	▨											
576	71	▧		▨											
	72	▧		▨	B13										
574	73	▧		▨											
	74	▧	75.0 El. 571.8'	▨											
572	75	⁘	Poorly-graded SAND with GRAVEL (SP): grayish brown, fine to coarse-grained, little gravel up to 3"and scattered cobbles up to 4.5" in largest dimension.												
	76	⁘													
570	77	⁘													
	78	⁘													
568	79	⁘													
	80	⁘		▨											
566	81	⁘		▨											
	82	⁘		▨	B14										
564	83	⁘		▨											
	84	⁘		▨											
562	85	⁘		▨											
	86	⁘		▨											
560	87	⁘		▨	B15										
	88	⁘		▨											
558	89	⁘	90.0 El. 556.8'	▨											
	90														

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REPORT TITLE **BORING RECORD (BRIDGE)**				HOLE ID **O-20-001**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **3 of 4**	

LOGGED BY **Mahmoud Suliman** BEGIN DATE **11/4/2020** COMPLETION DATE **11/4/2020**	BOREHOLE LOCATION (Lat/Long) **33.86956° N, 117.53514° W**	HOLE ID **O-20-001**
DRILLING CONTRACTOR **Great West Drilling**	BOREHOLE LOCATION (Station, Offset, Line) **35+20**	SURFACE ELEVATION **646.8 ft**
DRILLING METHOD **Becker Hammer**	DRILL RIG	BOREHOLE DIAMETER **6 5/8" in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Modcal (2.4")**	SPT HAMMER TYPE **Automatic, Weight = 140 lbs/ Drop = 30"**	HAMMER EFFICIENCY, ERi **83 %**
BOREHOLE BACKFILL AND COMPLETION **Soil Cuttings with Cement Mix**	GROUNDWATER READING DURING DRILLING AFTER DRILLING **50.0 ft**	TOTAL DEPTH OF BORING **90.1 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Type	Sample ID	Blows per 6 in.	Blows per foot	Recovery %	RQD %	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks (Other Tests)
	90		Bottom of Borehole at 90.1 feet bgs. End of boring at 90.0 feet bgs. Groundwater encountered at 50.0 feet bgs. Borehole backfilled with soil cuttings mixed with Cement quick-mix, compacted by pushing down with augers using drill rig weight on 11/4/2020.												
556	91														
	92														
554	93														
	94														
552	95														
	96														
550	97														
	98														
548	99														
	100														
546	101														
	102														
544	103														
	104														
542	105														
	106														
540	107														
	108														
538	109														
	110														
536	111														
	112														
534	113														
	114														
532	115														
	116														
530	117														
	118														
528	119														
	120														

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REPORT TITLE **BORING RECORD (BRIDGE)**				HOLE ID **O-20-001**
DIST **8**	COUNTY **Riverside**	ROUTE	POSTMILE **40.9**	EA
PROJECT OR BRIDGE NAME **Magnolia Avenue Bridge and Roadway Widening**				
BRIDGE NUMBER **BR NO.56C-0199**	PREPARED BY **Mahmoud Suliman**	DATE **12/14/2020**	SHEET **4 of 4**	

Appendix A-1

Hammer Calibration Record

SPT CAL

SPT HAMMER ENERGY MEASUREMENTS

Prepared by;

SPT CAL
5512 Belem Dr
Chino Hills, CA 91709

909-730-2161
bc@sptcal.com

Prepared for;

2R Drilling, Inc
6939 Schaefer Ave Ste D-304
Chino, CA 91710-9100

909-490-0530

Date: 03/12/20

Project Title: 2R Rig 6
Project Description: Rig 6 Ontario

Energy Transfer Ratio = 86.2% at 56.9 blows per minute

Testing was performed on March 12, 2020 in Ontario, California

Hammer Energy Measurements performed in accordance to ASTM D4633 using an approved and calibrated SPT Analyzer from Pile Dynamics, Inc.

PRESENTATION OF SPT ANALYZER TEST DATA

1. Introduction

This report presents the results of SPT Hammer Energy Measurements recorded with an SPT Analyzer from Pile Dynamics carried out on March 12, 2020 in Ontario, California.

2. Field Equipment and Procedures

The drill used is a CME 75. It is referred to at at 2R Drilling as Rig 6 or 2R6. It has an attached CME SPT Automatic Hammer.

This CME SPT Automatic Hammer uses a 140 lb. weight dropped 30” on to an anvil above the bore hole. The drill rod connects the anvil to a split spoon type soil sampler inside an 8” o.d. hollow stem auger at the designated sample depth. After a seeding blow the sampler is driven 18”. The number of blows required to penetrate the last 12" is referred to as the “N value”, which is related to soil strength.

The first recording was taken at 5’ below ground surface and then every 5' to final recording at 30’.

3. Instrumentation

An SPT Analyzer from Pile Dynamics was used to record and the process the data. The raw data was stored directly in the SPT Analyzer computer with subsequent analysis in the office with PDA-W and PDIPlot software. The measurements and analysis were conducted in general accordance with ASTM D4945 and ASTM D6066 test standards.

The SPT Analyzer is fully compliant with the minimum digital sampling frequency requirements of ASTM D4633-05 (50 kHz) and EN ISO 22476-3:2005 (100 kHz), as well as with the low pass filter, (cutoff frequency of 5000 Hz instead of 3000 Hz) requirements of ASTM D4633-05. All equipment and analysis also conform to ASTM D6066.

A 2' instrumented section of AWJ rod, with two sets of accelerometers and strain transducers mounted on opposite sides of the drill rod, was placed below the anvil. It measured strain and acceleration of every hammer blow. The SPT Analyzer then calculates the amount of energy transferred to the rod by force and velocity measurements.

4. Observations

The drill rig motor is diesel fueled. The drill and sample equipment looked to be well operated and maintained.

5. Results

Results from the SPT Hammer Energy Measurements are summarized below. It shows the Energy Transfer Ratio (ETR) at each sampling depth. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of the fall. 140 lb x 30” = 4200 lb-in = 0.350 kip-ft.

Energy Transfer Ratio = 86.2% at 56.9 blows per minute

N60=(ETR/60)N

Depth	ETR%	BPM
5	85.8	58.2
*10	70.5	56.6
15	84.9	56.8
20	86.8	56.2
25	87.7	55.8
30	85.8	58.0
Average	**86.2**	**56.9**

*** The sample at 10’ had blow counts too low to be included in the average above. The N value at 10’ was less 5. Anything less than 10 is considered too low for an accurate measurement of hammer energy transferred.**

If you have any questions please do not hesitate to call or email.Thank you,

Brian Serl
Calibration Engineer
SPT CAL
909-730-2161
bc@sptcal.com

Job No. 148146-1

Report on: Energy Measurement for Dynamic
Penetrometers
Standard Penetration Tests (SPT)
California

Prepared for Great West Drilling, Inc.
By Camilo Alvarez, MSCE, P.E. and
Anna M. Klesney, MSCE, E.I.T.

December 11, 2014

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Dynamic Measurements and Analyses
engineers, inc.

www.GRLengineers.com info@GRLengineers.com

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engineers, inc.

December 11, 2014

Jim Benson
Great West Drilling, Inc.
9431 Resenda Avenue
Fontana, California 92335

Re: Energy Measurement for Dynamic Penetrometers
Standard Penetration Tests (SPT)
California
GRL Job No. 148146-1

Dear Mr. Jim Benson:

This report transmits our findings from energy measurements and related data analysis conducted by GRL Engineers, Inc. for your drill rig and the TH14 Phase 5B project. One automatic hammer and penetrometer system was monitored during Standard Penetration Tests for two boring locations. Dynamic testing summarized in this report was conducted on November 17, 2014.

A Pile Driving Analyzer® Model PAX recorded, processed and displayed the dynamic data to meet the objectives of the hammer system calibration. Discussions on the test methods, limitations and implementation are provided in Appendix A. The energy measurement results are summarized in Tables 1A and 1B, with the average and standard deviation provided in Appendix B, and representative plots of force and normalized velocity are in Appendix C.

EQUIPMENT

Hammer and Penetrometer System

Energy measurements were recorded during standard penetration tests conducted for one automatic hammer and the following drill rig type and name.

Drill Rig Type	Drill Rig Name
DEEPROCK	GW

Measurements were recorded for two boring locations for the one drill rig. Great West Drilling, Inc. advanced the penetrometer to a minimum depth of 15.0 feet prior to energy measurements. The instrumented subassembly was connected to the top of the drill rod string and measurements recorded at 5 foot intervals for three depths of data at each boring location.

Measurements were recorded for every blow required to advance the sampler 18 inches or terminated upon encountering refusal conditions. Results are provided for the final 12 inches of the sampler advancement alone (i.e., excluding the initial 6 inches of advancement). ASTM Standard D4633 states that tests for energy evaluation should be limited to SPT N-values between 10 and 50. All energy measurements are included in the averages reported herein.

The following drill rod dimensions, of rod size AWJ or AW, were employed during testing.

Drill Rod Area sq. inch		Outside Diameter Inch		Inside Diameter inch	
A	B	A	B	A	B
1.19		1.75		1.25	
Depth of Penetrometer * feet		Drill Rod Section Lengths * feet		Transducer to Penetrometer Length * feet	
A	B	A	B	A	B
15.0	25.0	20.0	30.0	20.0	30.0
20.0	30.0	25.0	35.0	25.0	35.0
25.0	35.0	30.0	40.0	30.0	40.0

* A (Boring Location B1); B (Boring Location B17).

Instrumentation

A Pile Driving Analyzer was employed for recording, processing, and displaying the dynamic data. An instrumented subassembly, inserted at the top of the drill rod string below the hammer and anvil system and above the drill rods to record force and acceleration data. The subassembly was instrumented with two foil strain gages in a full bridge circuit and two piezoresistive accelerometers attached on diametrically opposite sides of the subassembly. Data sampling frequency was 50.0 kHz.

The PAX utilizes a digital system, and with the employed sampling frequency of 50.0 kHz, the signal conditioning conforms to ASTM D4633. Results for the maximum hammer operating rate, rod top force and velocity, and transferred energy are provided in Appendix B and summarized in the Tables 1. Discussions on the test method and its limitations can be found in Appendix A.

MEASUREMENTS AND CALCULATIONS

The primary objective of testing was the measurement of the energy transmitted from the hammer impact through the anvil into the instrumented subassembly and drill rods. Strain transducers and accelerometers were employed for the calculation of the transferred energy using force, F(t) and velocity v(t), records as follows:

$$\mathrm{EMX} = \int_{b}^{a} F(t)v(t)dt$$

where time "b" is to the beginning of the energy transfer and time "a" is to the time at which the energy transfer reaches a maximum. Force is calculated as the product of the measured strain, elastic modulus and cross-sectional area, and measured acceleration is integrated to velocity.

Integrated over the complete impact event and calculated from measured force and velocity, the energy transferred to the top of the drill rod was calculated as a function of time. The maximum transferred energy (i.e., EMX or also referred to as EFV) is used as an indicator of the energy content of the event. The described method is the only theoretically correct method of measuring energy transfer and automatically corrects for rod non-uniformities such as connector masses or loose joints. The EF2 method results included in Appendix B are inherently incorrect and included in Appendix B for reference alone.

TEST RESULTS

Result Discussion

Dynamic data was evaluated for the hammer operating rate, rod top force and velocity, and transferred energy. Appendix B provides the evaluated quantities for blows making up the SPT N-value, with their averages and standard deviation, plotted and printed as a function of depth for the monitored sequences of the standard penetration tests.

The plots in Appendix B include:

- FMX – the maximum measured rod top force
- VMX – the maximum measured rod top velocity
- BPM – the hammer operating rate in blows per minute
- EMX – the maximum calculated energy (EFV) transferred to the rod top
- EF2 – the maximum of the integral of the square of force, theoretically incorrect energy transfer calculation

Corresponding tables also include:

- ETR – ratio of transferred energy (EFV) to the maximum theoretical potential energy
- CSX – the maximum measured rod top compressive stress, averaged over the cross-sectional area

The maximum theoretical potential energy is the product of the standard 140 lb hammer impact mass dropped the standard 30 inches.

TABLE 1A: Summary of Field Results

Energy Measurement for Dynamic Penetrometers

Location	Depth(s)	Uncorrected N value (1)	Corrected N value (2)	Hammer Operating Rate (BPM)	Average Transferred Energy (EMX) (3)	Energy Transfer Ratio (ETR) (3)	Maximum Compressive Measured Top Stress (CSX) (4)	Maximum Compressive Impact Top Force (FMX) (5)
	ft	blows	N_{60}	bpm	ft-lbs	%	ksi	kips
B1								
	15.0 - 16.5	27	38	23	299	85	31	37
	20.0 - 21.4	82 for 11"	- - -	26	280	80	24	29
	25.0 - 26.5	24	33	18	291	83	28	33
	Overall System Performance			22	290	83	28	33

Notes

1. Uncorrected N-value, number of hammer blows required to advance sampler the final 12 inches, unless noted otherwise.
2. Corrected N-value, number of hammer blows required to advance sampler the final 12 inches, corrected for calculated energy transfer ratio (ETR).
3. Average transferred energy at transducer location; ratio of transferred energy to theoretical potential energy of hammer.
4. Average, measured Compressive driving Stress averaged over the drill rod cross section at transducer location.
5. Average, measured Compressive driving Force at transducer location.

Appendix B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the Boring Records, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

In-Situ Moisture Content and Dry Density

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance with the ASTM Standard D2216 and ASTM D2937 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the subsurface soils. For test results, see the Boring Records in Appendix A, *Field Exploration.*

Expansion Index

One representative bulk soil sample was tested in accordance with ASTM Standard D4829 test method to evaluate the expansion potential of materials encountered at the site. The test result is presented in the following table.

Table No. B-1, Summary of Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
O-20-001	10-20	Well-graded Silty Sand with Gravel (SM)	0	Very Low

Sand Equivalent

Two representative soil samples were tested in accordance with the ASTM Standard D2419 test method to determine the sand equivalent. The test results are presented in the following table.

Table No. B-2, Sand Equivalent Test Results

Boring No.	Depth (feet)	Soil Description	Sand Equivalent
A-20-003	5-10	Poorly graded Sand with Silt and Gravel (SP-SM)	42
A-20-004	0-5	Silty Sand with Gravel (SM)	55

R-value

Two bulk soil samples were tested for resistance value (R-value) in accordance with the Caltrans Test Method 301. The test is designed to provide a relative measure of soil strength for use in pavement design. The test results are presented in the following table.

Table No. B-3, Summary of R-Value Test Results

Boring No.	Depth (feet)	Soil Classification	Measured R-value
A-20-002	1-5	Poorly graded Sand with Silt and Gravel (SP-SM)	79
A-20-005	1.5-5	Poorly graded Sand with Gravel (SP)	80

Soil Corrosivity

Four representative soil samples were tested by AP Engineering and Testing, Inc. (Pomona, California) in accordance with California Test Method (CTM) 643, 422, and 417, to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of the soils when placed in contact with common pipe materials. The test results are summarized in the table below.

Table No. B-4, Summary of Corrosivity Test Results

Boring No.	Depth (feet)	pH	Soluble Sulfates (CTM 417) (ppm)	Soluble Chlorides (CTM 422) (ppm)	Min. Resistivity (CTM 643) (Ohm-cm)
O-20-001	10-20	8.4	22	39	12,671
O-20-001	20-25	8.5	39	35	13,980
O-20-001	30-35	8.7	24	38	6,032
O-20-001	40-45	8.4	28	42	3,503

Grain Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on 7 selected samples in general accordance with the ASTM D6913 test method. Grain-size curves are shown in Figure Nos. B-2a and B-2b, *Grain Size Distribution Results* and summarized in the table below.

Table No. B-5, Summary Grain Size Distribution Test Results

Boring No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
A-20-001	1-5	Well-graded Sand with Silt and Gravel (SP-SM)	20.0	71.0	9.0	
A-20-003	0-5	Silty Sand with Gravel (SM)	24.0	59.0	17.0	
A-20-005	5-10	Poorly graded Sand with Silt and Gravel (SP-SM)	19.0	77.0	4.0	
O-20-001	10-20	Well-graded Sand with Silt and Gravel (SP-SM)	18.0	75.0	7.0	
O-20-001	20-25	Poorly graded Sand with Gravel (SP)	34.0	62.0	4.0	
O-20-001	30-35	Well-graded Sand with Silt and Gravel (SP-SM)	36.0	55.0	9.0	
O-20-001	40-45	Sandy Clay (CL)	8.0	18.0	74.0	

Maximum Dry Density and Optimum Moisture Content

Laboratory maximum dry density and optimum moisture content relationship tests were performed on 2 representative bulk soil samples. The tests were conducted in accordance with ASTM D1557 test method and CT 216 method. Test results are presented on Figure No. B-3, *Moisture-Density Relationship Results* and summarized in the following table.

Table No. B-6, Laboratory Maximum Density Test Results

Boring No.	Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
*A-20-003	5-10	Poorly graded Sand with Silt and Gravel (SP-SM), Yellowish Brown to Brown	135.0 (138.6*)	5.7 (5.0*)
*O-20-001	20-25	Poorly graded Sand with Gravel (SP), Grayish Brown	123.5 (130.9*)	6.5 (5.0*)
(*Rock correction: A-20-002= 13.06% and A-20-004= 21.00%)				

Direct Shear

One direct shear test was performed on relatively undisturbed samples and one (1) direct shear test was performed on sample remolded to 90% of the maximus dry density under soaked condition in accordance with ASTM Standard 3080. For each test, three samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. The test results, including average sample density and moisture content are shown in Figure Nos. B-4 and B-5, *Direct Shear Test Results,* and summarized in the following table.

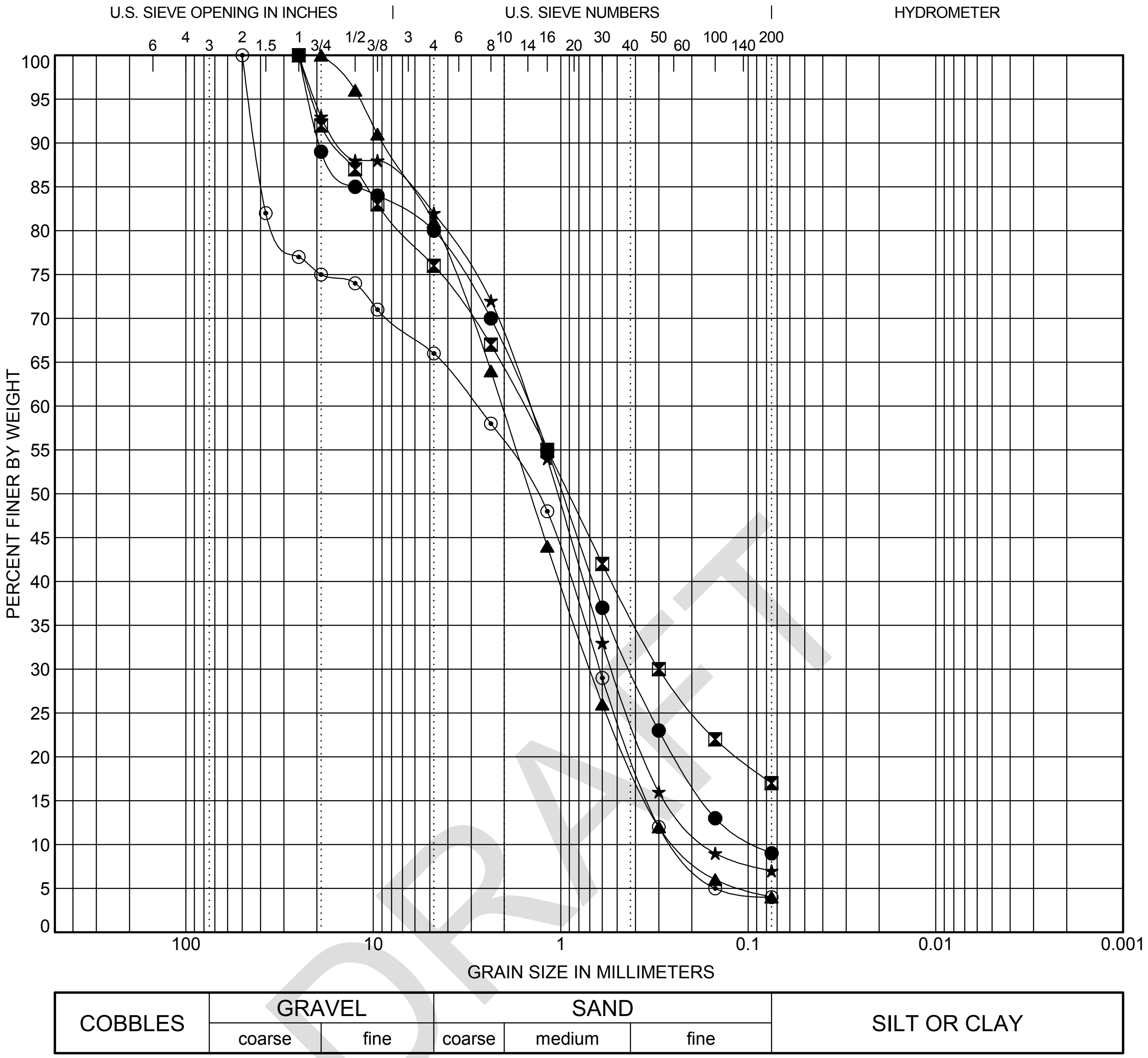
Table No. B-7, Summary of Direct Shear Test Results

Boring No.	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*O-20-001	20-25	Poorly graded Sand with Gravel (SP)	35	0
O-20-001	50.0-51.5	Poorly graded Sand with Gravel (SP)	32	90

(*Remolded to 90% of the maximum dry density)

Sample Storage

Soil samples currently stored in our laboratory will be discarded thirty days after the date of the final report, unless this office receives a specific request to retain the samples for a longer period.



	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**A-20-001**	**1-5**	**WELL-GRADED SAND with SILT and GRAVEL (SP-SM)**				**1.36**	**16.67**
☒	**A-20-003**	**0-5**	**SILTY SAND with GRAVEL (SM)**					
▲	**A-20-005**	**5-10**	**POORLY-GRADED SAND with SILT and GRAVEL (SP-SM)**				**0.99**	**8.63**
★	**O-20-001**	**10-20**	**WELL-GRADED SAND with SILT and GRAVEL (SP-SM)**				**1.14**	**8.98**
⊙	**O-20-001**	**20-25**	**POORLY-GRADED SAND with GRAVEL (SP)**				**0.56**	**11.42**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**A-20-001**	**1-5**	**25**	**1.487**	**0.424**	**0.089**	**20.0**	**71.0**	**9.0**	
☒	**A-20-003**	**0-5**	**25**	**1.575**	**0.3**		**24.0**	**59.0**	**17.0**	
▲	**A-20-005**	**5-10**	**19**	**2.054**	**0.697**	**0.238**	**19.0**	**77.0**	**4.0**	
★	**O-20-001**	**10-20**	**25**	**1.487**	**0.531**	**0.166**	**18.0**	**75.0**	**7.0**	
⊙	**O-20-001**	**20-25**	**50**	**2.811**	**0.622**	**0.246**	**34.0**	**62.0**	**4.0**	

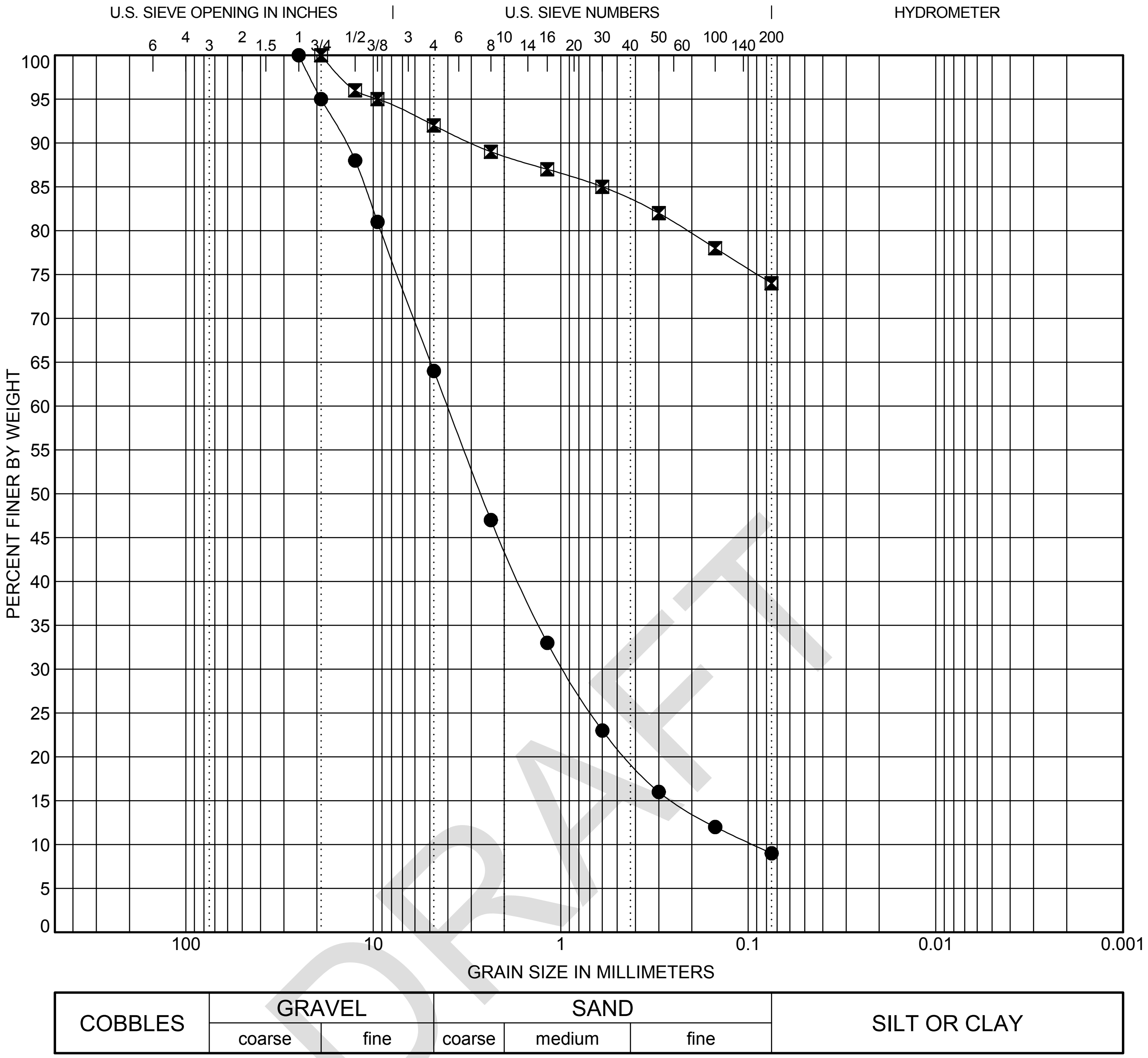
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Magnolia Avenue Bridge and Roadway Widening
El Camino Avenue to 1,000 feet East of All American Way
City of Corona, Riverside County, CA
For: CNS Engineering, Inc.

Project No.
18-81-147-03

Drawing No.
B-1a



	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**O-20-001**	**30-35**	**WELL-GRADED SAND with SILT and GRAVEL (SP-SM)**				**2.44**	**42.64**
☒	**O-20-001**	**40-45**	**SANDY CLAY (CL)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**O-20-001**	**30-35**	**25**	**4.029**	**0.963**	**0.094**	**36.0**	**55.0**	**9.0**	
☒	**O-20-001**	**40-45**	**19**				**8.0**	**18.0**	**74.0**	

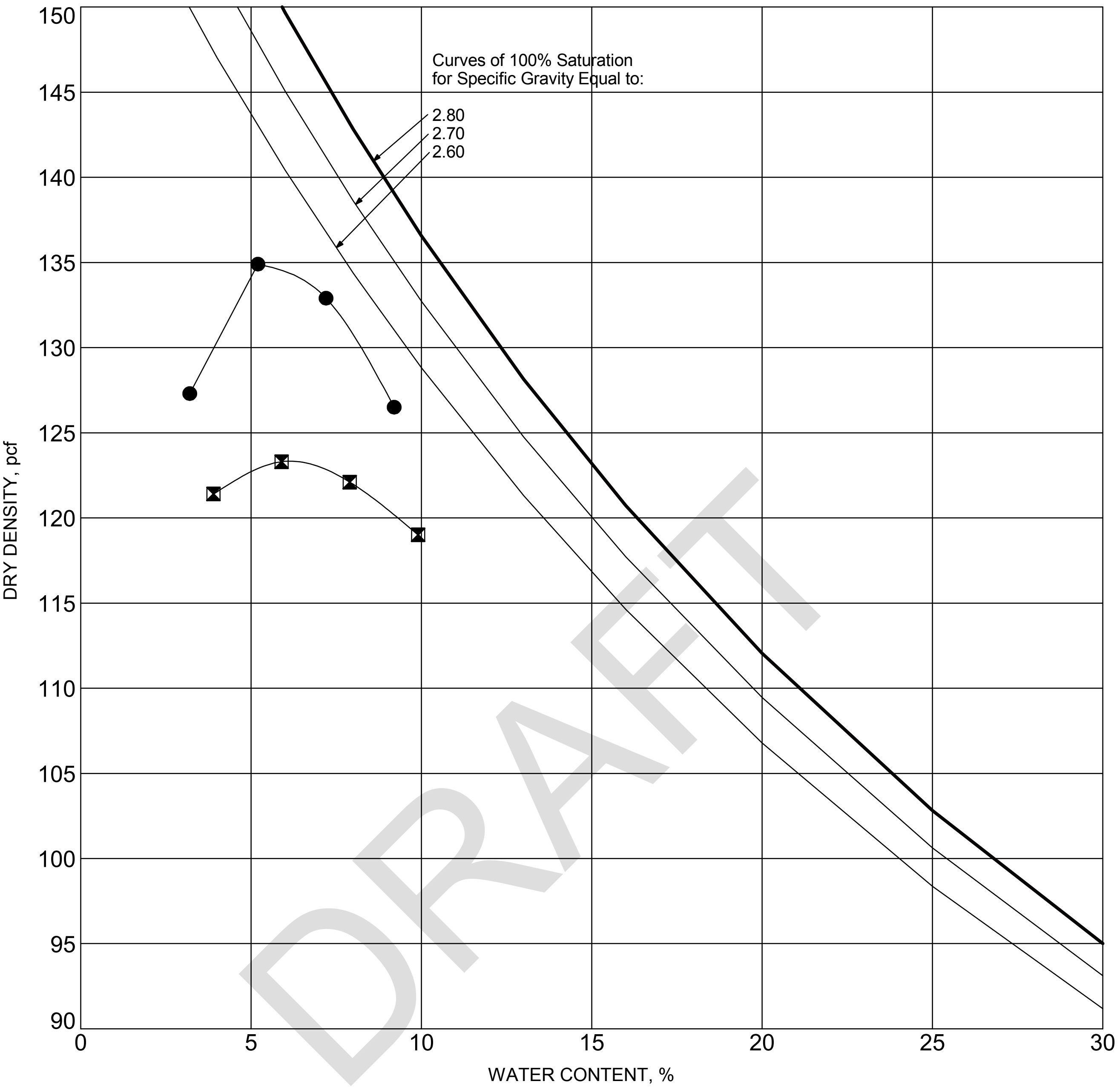
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Magnolia Avenue Bridge and Roadway Widening
El Camino Avenue to 1,000 feet East of All American Way
City of Corona, Riverside County, CA
For: CNS Engineering, Inc.

Project No.
18-81-147-03

Drawing No.
B-1b



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**A-20-003**	**5-10**	**POORLY-GRADED SAND WITH GRAVEL (SP), GRAYISH BROWN**	**D1557 - C**	**5.7 (5.0*)**	**135.0 (138.6*)**
☒	**O-20-001**	**20-25**	**POORLY-GRADED SAND with GRAVEL (SP), YELLOWISH BROWN TO BROWN**	**D1557 - C**	**6.5 (5.2*)**	**123.5 (130.9*)**

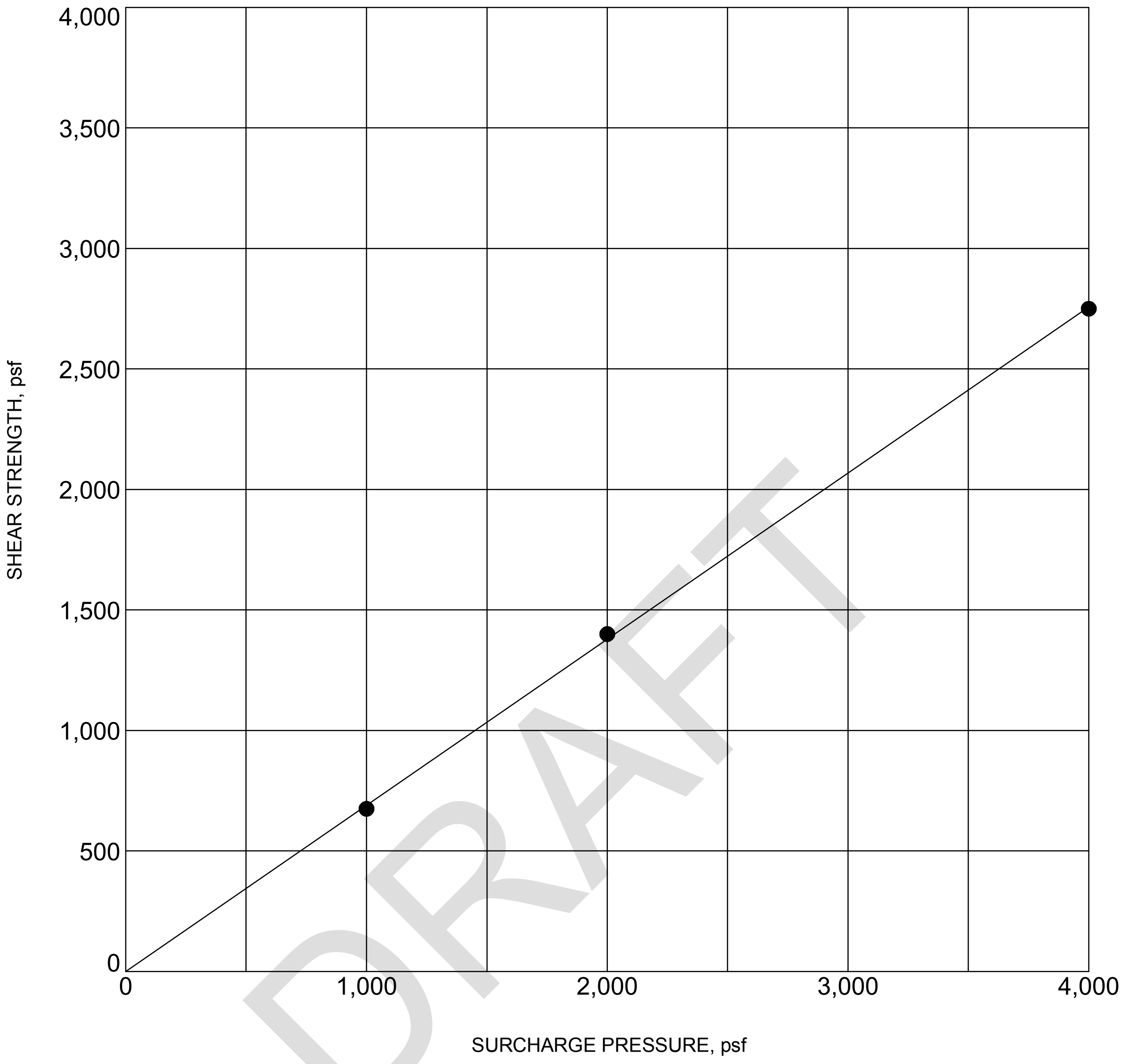
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Magnolia Avenue Bridge and Roadway Widening
El Camino Avenue to 1,000 feet East of All American Way
City of Corona, Riverside County, CA
For: CNS Engineering, Inc.

Project No.
18-81-147-03

Drawing No.
B-2



BORING NO. :	**O-20-001**	DEPTH (ft) :	**20-25**
DESCRIPTION :	**POORLY-GRADED SAND with GRAVEL (SP)**		
COHESION (psf) :	**0**	FRICTION ANGLE (degrees):	**35**
MOISTURE CONTENT (%) :	**6.5**	DRY DENSITY (pcf) :	**110.6**

(* Remolded to 90% of the laboratory maximum dry density)

NOTE: Ultimate Strength.

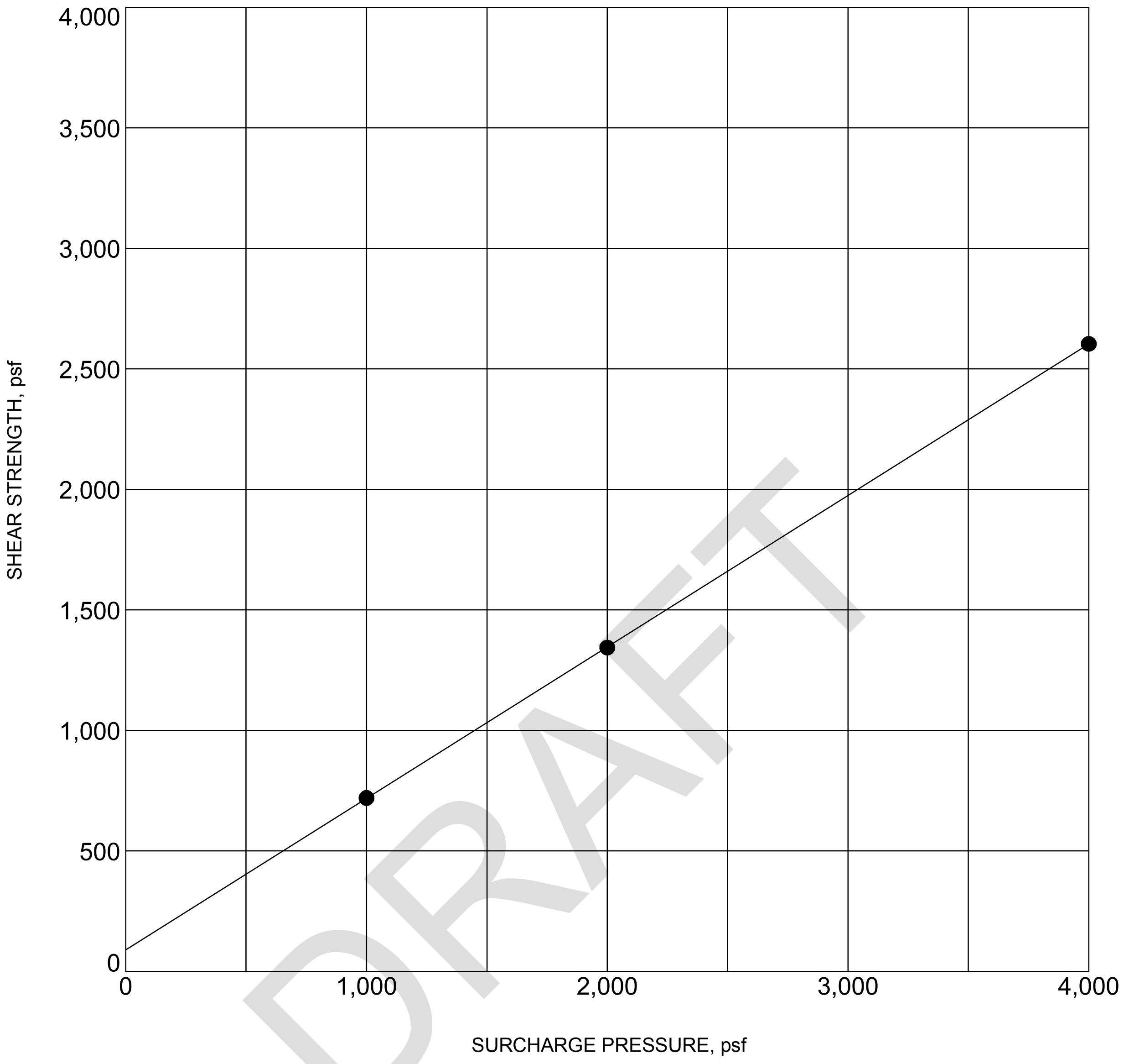
DIRECT SHEAR TEST RESULTS

Converse Consultants

Magnolia Avenue Bridge and Roadway Widening
El Camino Avenue to 1,000 feet East of All American Way
City of Corona, Riverside County, CA
For: CNS Engineering, Inc.

Project No.
18-81-147-03

Drawing No.
B-3



BORING NO. :	**O-20-001**	DEPTH (ft) :	**50.0-51.5**
DESCRIPTION :	**POORLY-GRADED SAND with GRAVEL (SP)**		
COHESION (psf) :	**90**	FRICTION ANGLE (degrees):	**32**
MOISTURE CONTENT (%) :	**17.9**	DRY DENSITY (pcf) :	**101.0**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Magnolia Avenue Bridge and Roadway Widening
El Camino Avenue to 1,000 feet East of All American Way
City of Corona, Riverside County, CA
For: CNS Engineering, Inc.

Project No.
18-81-147-03

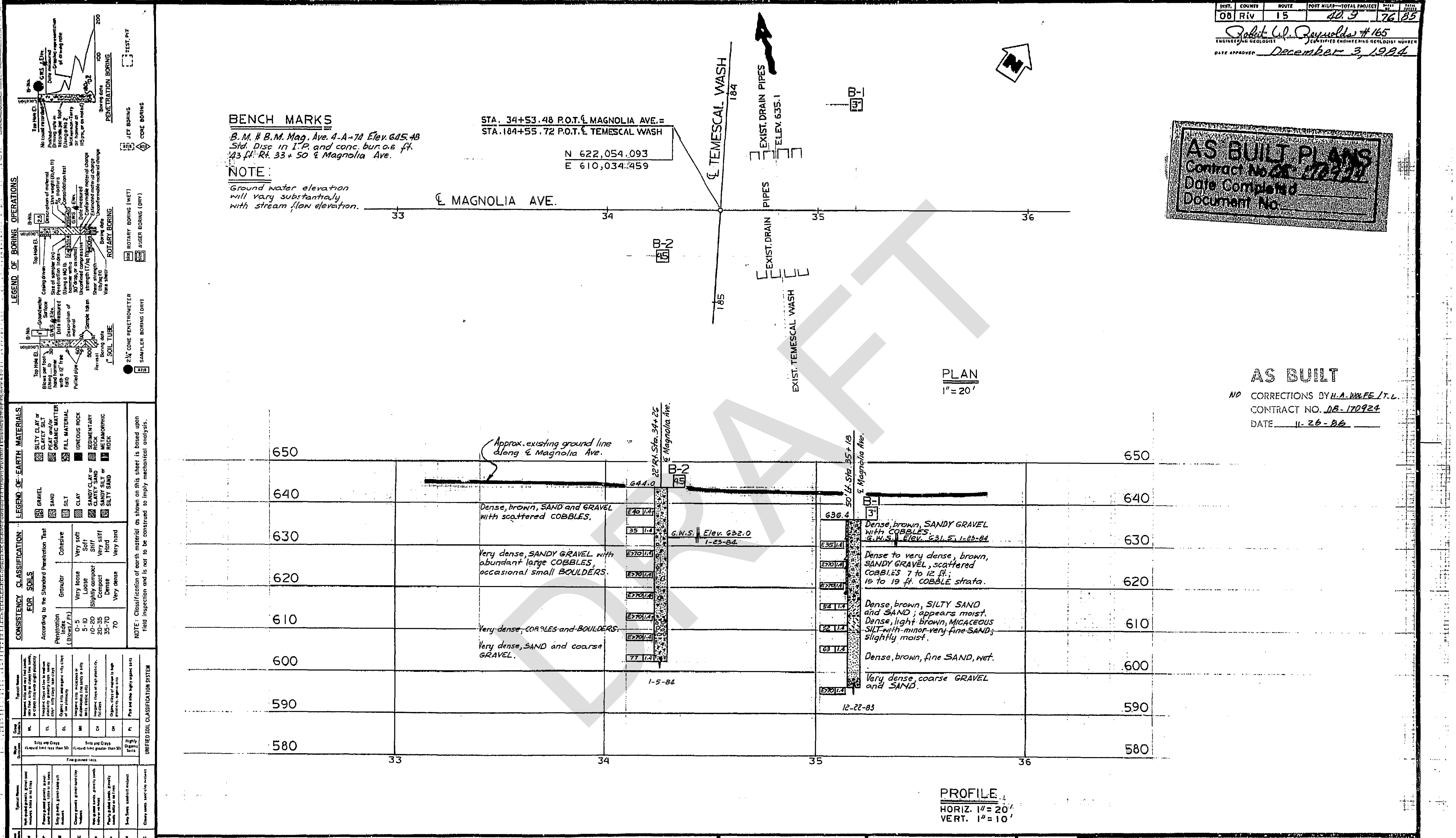
Drawing No.
B-4

Appendix C

Logs of Test Borings (As Built)

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
08	Riv	15	40.9	76	85

Robert W. Reynolds #165
ENGINEERING GEOLOGIST — CERTIFIED ENGINEERING GEOLOGIST NUMBER
DATE APPROVED December 3, 1984



BENCH MARKS

B.M. # B.M. Mag. Ave. 4-A-74 Elev. 645.48
Std. Disc in I.P. and conc. bur. 0.6 ft.
43 ft. Rt. 33 + 50 ℄ Magnolia Ave.

NOTE:

Ground water elevation will vary substantialy with stream flow elevation.

STA. 34+53.48 P.O.T. ℄ MAGNOLIA AVE. =
STA. 184+55.72 P.O.T. ℄ TEMESCAL WASH
N 622,054.093
E 610,034.459

PLAN
1" = 20'

Approx. existing ground line along ℄ Magnolia Ave.

B-2 — 22' Rt. Sta. 34+26 ℄ Magnolia Ave. — 644.0

Dense, brown, SAND and GRAVEL with scattered COBBLES.

G.W.S. Elev. 632.0 1-23-84

Very dense, SANDY GRAVEL with abundant large COBBLES, occasional small BOULDERS.

~~Very dense, COBBLES and BOULDERS.~~

Very dense, SAND and coarse GRAVEL.

E40	1.4
35	1.4
E>70	1.4
E>70	1.4
E>70	1.4
E>70	1.4
E>70	1.4
77	1.4

1-5-84

B-1 — 50' Lt. Sta. 35+18 ℄ Magnolia Ave. — 636.4

Dense, brown, SANDY GRAVEL with COBBLES.

G.W.S. Elev. 631.5, 1-23-84

Dense to very dense, brown, SANDY GRAVEL, scattered COBBLES 7 to 12 ft.; 15 to 19 ft. COBBLE strata.

Dense, brown, SILTY SAND and SAND; appears moist.

Dense, light brown, MICACEOUS ~~SILT with minor very fine SAND;~~ slightly moist.

Dense, brown, fine SAND, wet.

Very dense, coarse GRAVEL and SAND.

E35	1.4
E>70	1.4
E>70	1.4
54	1.4
52	1.4
63	1.4
E>70	1.4

12-22-83

PROFILE
HORIZ. 1" = 20'
VERT. 1" = 10'

AS BUILT PLANS
Contract No. 08-170924
Date Completed
Document No.

AS BUILT
NO CORRECTIONS BY H.A. WOLFE / T.L.
CONTRACT NO. 08-170924
DATE 11-26-86

ENGINEERING GEOLOGY AND TECHNICAL SERVICES BRANCH — TRANSPORTATION LABORATORY

				CHARGE UNIT	EXPEND. AUTHOR.	SPEC. DESIGN.
DRAWN BY	VICTOR SESTOKAS, 1-19-84	L.E. Rand 751	L.L. Champlin 368	08203	170901	456C 199
CHECKED BY	L.E. RAND 2-7-84	PROJECT GEOLOGIST — CEG NO.	ENGINEERING GEOLOGIST — CEG NO.			

State of
CALIFORNIA
DEPARTMENT OF TRANSPORTATION

STRUCTURES - DESIGN 9
F.M. Sarnan C17514
PROJECT ENGINEER — REGISTERED CIVIL ENGINEER NO.

BRIDGE NO. 56C-199
POST MILE 40.9

MAGNOLIA AVENUE BRIDGE
LOG OF TEST BORINGS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 08203
WO 170901

Disregard prints bearing earlier revision dates →
REVISION DATES (PRELIMINARY STAGE ONLY) 6/13/84
SHEET 8 OF 8

(17092 G)

76

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF TRANSPORTATION.
DATE 1-15-87 SIGNATURE Donald Blackford TITLE SUPERVISOR OF MICROFILM SERVICES

Appendix D

Percolation Testing

APPENDIX D

PERCOLATION TESTING

Percolation testing was performed at location (A-20-001) in general accordance with the Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011) for using a percolation testing method to estimate infiltration rate*.*

Upon completion of drilling the test holes, a 2-inch-thick gravel layer was placed at the bottom of each hole and a 3.13-inch diameter perforated pipe was installed above the gravel to the ground surface. The boring annulus around the pipe was filled with gravel. The purpose of the pipe and gravel was to reduce the potential for erosion and caving due to the addition of water to the hole.

The test hole was presoaked by filling with water to at least 5 times the radius of the test hole. More than 6 inches of water seeped away from the test hole within 25 minutes for 2 consecutive measurements, meeting the criteria for testing as “sandy soil”. Percolation testing was conducted immediately after these 2 measurements. During testing, the water level and total depth of the test hole were measured from the top of the pipe in every 10 minutes up to one hour. Following the completion of percolation testing, the pipe was cut below the asphalt surface, and the percolation test hole was backfilled with excavated soil.

Percolation rate describes the movement of water horizontally and downward into the soil from a boring. Infiltration rates describe the downward movement of water through a horizontal surface, such as the floor of a retention basin. Percolation rate is related to infiltration rate but is generally higher and require conversion before use in basin design. The percolation test data was used to estimate infiltration rate using the Porchet Inverse Borehole Method, in accordance with the Riverside County guidelines. A combined safety factor of 3.44, provided to us by Ceazar Aguilar with Aguilar Consulting, Inc. was applied to the measured infiltration rates to account suitability assessment and design factors. The designer should determine whether additional design-related safety factors are required and for design of the proposed infiltration system.

The measured percolation test data and calculations for conversion to infiltration rate, porosity correction, and factor of safety are shown on Plate No. 1, *Estimated Infiltration Rate from Percolation Test Data* is graphically represented on Plate No. 2, *Infiltration Rate Versus Elapsed Time*. The estimated infiltration rate at the test hole is presented in the following table.

Table No. D-1, Estimated Infiltration Rate

Infiltration Test	Depth (feet)	Soil Type	Infiltration Rate (inches/hour)
A-20-001	15	Silty Sand (SM)	2.55

Estimated Infiltration Rate from Percolation Test Data, PT-01

Project Name	Magnolia Ave. Bridge and Roadway Widening
Project Number	18-81-147-03
Test Number	PT-01
Test Location	33.868314, -117.538202
Personnel	Mahmoud Suliman
Presoak Date	10/15/2020
Test Date	10/15/2020

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	180.6
Inside Diameter of Pipe, I (inches)	3.00
Outside Diameter of Pipe, O (inches)	3.13
Factor of Safety (FOS), F	3.44

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	25.00	102	163.80	25.00	78.60	16.80	61.80	47.70	5.97		1.74
2	25.00	106.80	166.20	50.00	73.80	14.40	59.40	44.10	6.18		1.80
3	10.00	120.00	155.40	60.00	60.60	25.20	35.40	42.90	9.46		2.75
4	10.00	120.00	154.80	70.00	60.60	25.80	34.80	43.20	9.24		2.69
5	10.00	120.00	154.56	80.00	60.60	26.04	34.56	43.32	9.15		2.66
6	10.00	120.00	154.20	90.00	60.60	26.40	34.20	43.50	9.02		2.62
7	10.00	120.00	153.84	100.00	60.60	26.76	33.84	43.68	8.89		2.58
8	10.00	120.00	153.48	110.00	60.60	27.12	33.48	43.86	8.76		2.55

Recommended Design Infiltration Rate (inches/hr)	**2.55**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

Infiltration Rate versus Time, PT-01

Project Name	Magnolia Ave. Bridge and Roadway Widening
Project Number	18-81-147-03
Test Number	PT-01
Test Location	33.868314, -117.538202
Personnel	Mahmoud Suliman
Presoak Date	10/15/2020
Test Date	10/15/2020

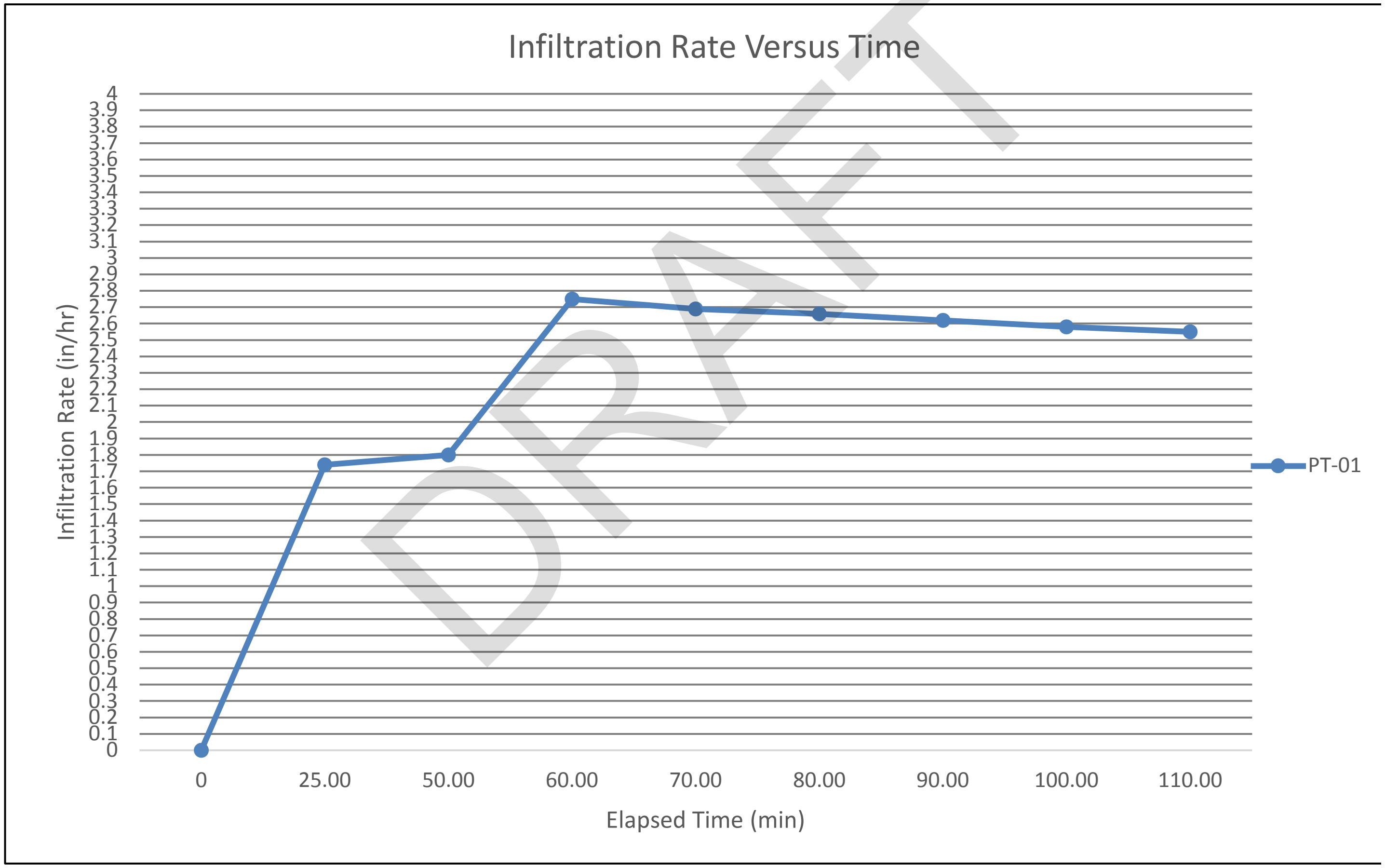


Plate No.
2

Appendix E

Liquefaction and Seismic Settlement Analysis

APPENDIX E

LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

The subsurface data obtained from the boring A-20-004/O-20-001 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ (InfraGEO Software, 2020) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A modal earthquake magnitude of M 6.47 was selected based on the results of seismic deaggregation analysis using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A peak ground acceleration (PGA_M) of 0.70g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on Section 8.6, *Seismicity.* The results of our analysis are presented on Sheet Nos. C-1 through C-3 and summarized in the following table.

Table E-1, Estimated Dynamic Settlements

Location	Groundwater Conditions	Groundwater Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
A-20-004/ O-20-001	Current	>50	Negligible	Negligible
	Historical			

Based on our analysis, the project site has up negligible potential of liquefaction and dry seismic settlement under current and historic groundwater conditions. However, we recommend a total of 1-inch total dynamic settlement and 0.5-inch of dynamic differential settlement should be used for the design purpose.

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Magnolia Avenue Bridge and Roadway Widening
Project No.	18-81-147-03
Project Location	City of Corona, CA
Analyzed By	Z. Alam
Reviewed By	C. Amante

SELECTED METHODS OF ANALYSIS

Analysis Description	Liquefation and Seismic Settlement
Triggering of Liquefaction	Boulanger-Idriss (2014)
Severity of Liquefaction	LPI: Liquefaction Potential Index based on Iwasaki et al. (1978)
Seismic Compression Settlement (Dry/Unsaturated Soil)	Pradel (1998)
Liquefaction-Induced Settlement (Saturated Soil)	Ishihara and Yoshimine (1992)
Liquefaction-Induced Lateral Spreading	Zhang et al. (2004)
Residual Shear Strength of Liquefied Soil	Idriss and Boulanger (2008)

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6.47	
Peak Ground Acceleration, A_{max}	0.70	g
Factor of Safety Against Liquefaction, FS	1.20	

BORING DATA AND SITE CONDITIONS

Boring No.	A-20-004/O-20-001	
Ground Surface Elevation	646.80	feet
Proposed Grade Elevation	646.80	feet
GWL Depth Measured During Test	50.00	feet
GWL Depth Used in Design	12.00	feet
Borehole Diameter	8.00	inches
Hammer Weight	140.00	pounds
Hammer Drop	30.00	inches
Hammer Energy Efficiency Ratio, ER (%)	80.00	%
Hammer Distance to Ground Surface	5.00	feet
Topographic Site Condition:	TSC1	(Level Ground with No Nearby Free Face)
- Ground Slope, S (%)		<<= Leave this blank
- Free Face Distance to Slope Height Ratio, (L/H)	5.00	<<= Leave this blank Set H to zero =>> feet

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	Y	112.0	MCal	52.0	17.00
5.00	10.00	SP-SM	Y	115.0	MCal	51.0	7.00
10.00	15.00	SP-SM	Y	114.0	MCal	55.0	7.00
15.00	20.00	SP-SM	Y	114.0	MCal	41.0	7.00
20.00	25.00	SP-SM	Y	118.0	SPT1	43.0	7.00
25.00	30.00	GP-GM	Y	118.0	MCal	65.0	4.00
30.00	36.50	SP-SM	Y	118.0	SPT1	33.0	7.00
36.50	40.00	CL	N	110.0	MCal	30.0	74.00
40.00	45.00	CL	N	110.0	SPT1	47.0	74.00
45.00	50.00	SP	Y	118.0	MCal	46.0	4.00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Magnolia Avenue Bridge and Roadway Widening
Project No.	18-81-147-03
Project Location	City of Corona, CA
Analyzed By	Z. Alam
Reviewed By	C. Amante

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6.47
Peak Ground Acceleration, A_{max}	0.70 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS

Boring No.	A-20-004/O-20-001
Ground Surface Elevation	646.80 feet
Proposed Grade Elevation	646.80 feet
GWL Depth Measured During Test	50.00 feet
GWL Depth Used in Design	12.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)
- Ground Slope, S	0.00 %
- Free Face (L/H) Ratio	5.00 H = feet
Average Total Unit Weight of New Fill	120.00 pcf (assumed)

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils: 0.00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 0.00 *** (Very low risk, with no surface manifestation of liquefaction)

Seismic Ground Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Seismic Compression Settlement:	Pradel (1998)	0.00 inches	0.00 inches	0.00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0.00 inches	0.00 inches	0.00 inches	(Saturated Soils)
Total Seismic Settlement:		0.00 inches	0.00 inches	0.00 inches	

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.00 inches	0.05 inches	0.05 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008).

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where $CRR = CRR_{7.5}\ MSF\ K_\sigma\ K_\alpha$, MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1.0, (level ground), CSR = Cyclic Stress Ratio = $0.65\ A_{max}\ (\sigma_{vo}/\sigma'_{vo})\ r_d$, and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field SPT Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	Y	112.00	MCal	52.00	17.00
5.00	10.00	SP-SM	Y	115.00	MCal	51.00	7.00
10.00	15.00	SP-SM	Y	114.00	MCal	55.00	7.00
15.00	20.00	SP-SM	Y	114.00	MCal	41.00	7.00
20.00	25.00	SP-SM	Y	118.00	SPT1	43.00	7.00
25.00	30.00	GP-GM	Y	118.00	MCal	65.00	4.00
30.00	36.50	SP-SM	Y	118.00	SPT1	33.00	7.00
36.50	40.00	CL	N	110.00	MCal	30.00	74.00
40.00	45.00	CL	N	110.00	SPT1	47.00	74.00
45.00	50.00	SP	Y	118.00	MCal	46.00	4.00

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. for Vert. Stress C_N	SPT Corr. for Hammer Energy C_E	SPT Corr. for Borehole Size C_B	SPT Corr. for Rod Length C_R	SPT Corr. for Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
280.00	280.00	1.483	1.333	1.150	0.750	0.650	38.9	57.6	61.5	0.999	1.100	0.454			Unsaturated Soil			0.00	0.05	0.00
847.50	847.50	1.230	1.333	1.150	0.800	0.650	40.7	50.0	50.1	0.978	1.100	0.445			Unsaturated Soil			0.00	0.05	0.00
1,420.00	1,326.40	1.085	1.333	1.150	0.850	0.650	46.6	50.6	50.7	0.953	1.100	0.464			Dense Soil			0.00	0.05	0.00
1,990.00	1,646.80	1.002	1.333	1.150	0.950	0.650	38.8	38.9	39.0	0.925	1.002	0.509			Dense Soil			0.00	0.05	0.00
2,570.00	1,914.80	0.953	1.333	1.150	0.950	1.000	62.6	59.7	59.8	0.895	0.925	0.547			Dense Soil			0.00	0.05	0.00
3,160.00	2,192.80	0.909	1.333	1.150	0.950	0.650	61.5	55.9	55.9	0.863	0.863	0.566			Dense Soil			0.00	0.05	0.00
3,838.50	2,512.50	0.830	1.333	1.150	1.000	1.000	50.6	42.0	42.1	0.825	0.804	0.574			Dense Soil			0.00	0.05	0.00
4,414.50	2,776.50									0.792		0.573			Clay-rich Soil			0.00	0.05	0.00
4,882.00	2,978.80									0.764		0.570			Clay-rich Soil			0.00	0.05	0.00
5,452.00	3,236.80	0.706	1.333	1.150	1.000	0.650	45.8	32.4	32.4	0.732	0.772	0.561			Dense Soil			0.00	0.05	0.00

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SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

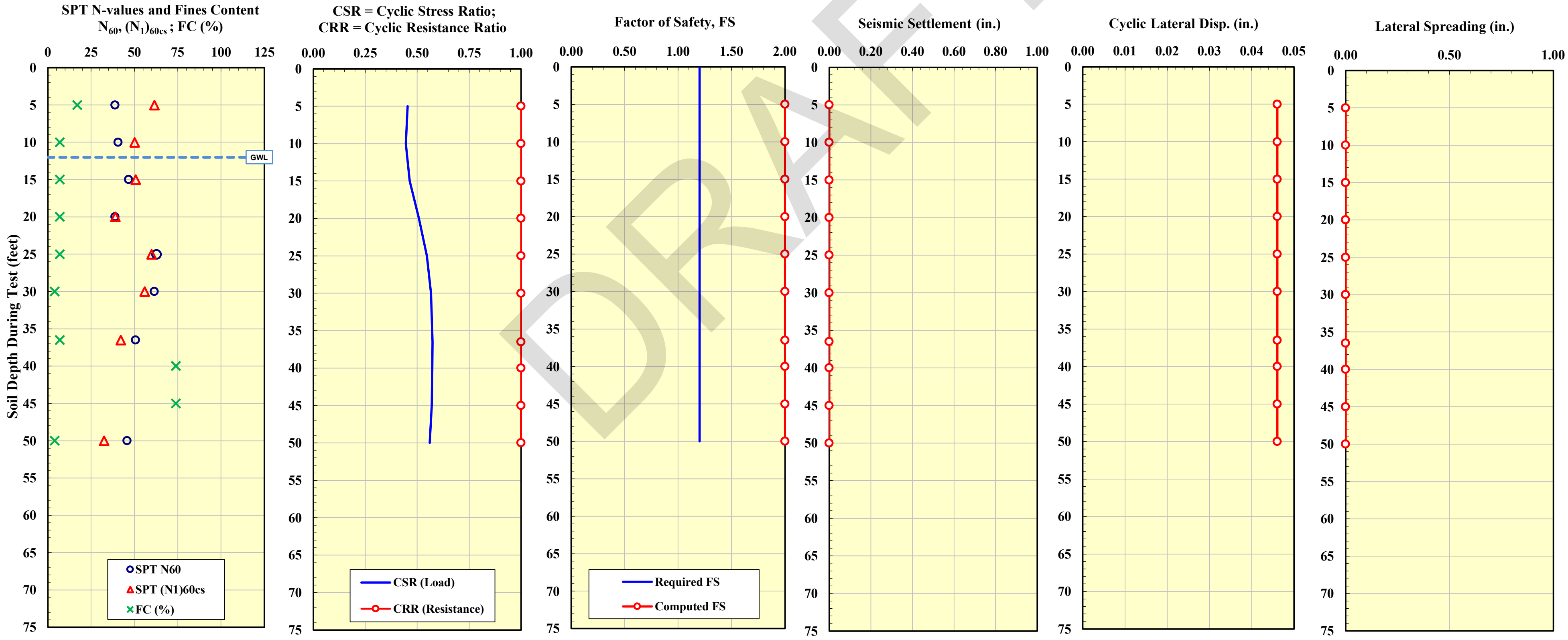
PROJECT INFORMATION	
Project Name	Magnolia Avenue Bridge and Roadway Widening
Project No.	18-81-147-03
Project Location	City of Corona, CA
Analyzed By	Z. Alam
Reviewed By	C. Amante

TOPOGRAPHIC CONDITIONS		
Ground Slope, S	0.00 %	
Free Face (L/H) Ratio	5.00	H = 0.00 feet

GROUNDWATER DATA	
GWL Depth Measured During Test	50.00 feet
GWL Depth Used in Design	12.00 feet

BORING DATA	
Boring No.	A-20-004/O-20-001
Ground Surface Elevation	646.80 feet
Proposed Grade Elevation	646.80 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.47
Peak Ground Acceleration, A_{max}	0.70 g
Factor of Safety Against Liquefaction, FS	1.20



Analysis Methods Used ==>>	**Liquefaction Triggering:**	**Seismic Settlements:**	**Cyclic Lateral Displacements:**	**Lateral Spreading:**
	Boulanger-Idriss (2014)	Above GWL: Pradel (1998)	Above GWL: Pradel (1998)	Zhang et al. (2004)
		Below GWL: Ishihara and Yoshimine (1992)	Below GWL: Tokimatsu and Asaka (1998)	

Appendix F

Site Class

SIMPLIFIED EVALUATION OF SITE CLASS AND GEOTECHNICAL DESIGN PARAMETERS USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Magnolia Avenue Bridge and Roadway Widening
Project No.	18-81-147-03
Project Location	City of Corona, CA
Analyzed By	Z. Alam
Reviewed By	C. Amante

GENERAL INPUT DATA

Analysis Description	Site Class and Vs30	
Boring ID No.	A-20-004/O-20-001	
Ground Surface Elevation	646.80	feet
Proposed Grade Elevation	646.80	feet
Total Unit Weight of New Fill	120.00	pcf
Borehole Diameter	8.00	inches
Hammer Weight	140.00	pounds
Hammer Drop	30.00	inches
Hammer Efficiency Ratio, ER	80.00	%
Hammer Dist. to Ground Surface	5.00	feet
Groundwater Depth During Test	12.00	feet

SPT BLOW COUNT AND RELATIVE DENSITY

- Based on the recommendations by Idriss and Boulanger (2008), the normalized SPT blow count is defined as $(N_1)_{60} = N_{60}\ C_N$

where $N_{60} = N_{field}\ C_E\ C_B\ C_R\ C_S$

and the relative density of granular soils is estimated as

$D_r = 15\ [(N_1)_{60}]^{0.5}$ in percent

SHEAR WAVE VELOCITY AND SITE CLASSIFICATION

- Shear wave velocities are estimated based on empirical correlations with SPT N_{60} values for various soil types, as derived by Brandenberg, Bellana and Shantz (2010) from regression analyses.
- Site classification is analyzed using the method by Boore (2004).

Ave. Shear Wave Velocity (Top Depth d), $V_{s,d}$ = 231.80 m/s

Ave. Shear Wave Velocity (Top 30 m), $V_{s,30} = 10^{a + b \log (Vs,d)}$

where a = 0.01380

b = 1.02630

Coefficients a and b vary with depth, as derived by Boore (2004).

Computed $V_{s,30}$ = 276.0 m/s

Site Class = D

SOIL STRENGTH AND DEFORMATION MODULUS PARAMETERS

- For granular soils, the effective peak friction angle, ϕ', is estimated from correlations with the normalized SPT blow count, $(N_1)_{60}$ from Bowles (1996) and recommended adjustments from Caltrans Geotechnical Manual (2014).
- For cohesive soils, the undrained shear strength, S_u, is based on field measurements with torvane or pocket penetrometer. When only SPT values are available, S_u is estimated using the correlation chart with $(N_1)_{60}$ value provided in the Caltrans Geotechnical Manual (2014).
- Modulus of Elasticity, E_s, values for granular soils and cohesive soils are estimated from correlations with SPT N_{60} and undrained shear strength, S_u, respectively summarized by Bowles (1996).
- Shear Modulus, $G = E_s / [3\ (1 - 2\mu)]$ and Bulk Modulus, $K = E_s / [2\ (1 + \mu)]$ based on theory of elasticity where μ is the Poisson's ratio of the soil. Typical values of Poisson's ratio are estimated from various references.

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INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Pocket Penetrometer Shear Test Results PP (tsf)	Torvane Shear Test Results TV (tsf)
0.00	5.00	SM	112.0	MCal	52.0		
5.00	10.00	SP-SM	115.0	MCal	51.0		
10.00	15.00	SP-SM	114.0	MCal	55.0		
15.00	20.00	SP-SM	114.0	MCal	41.0		
20.00	25.00	SP-SM	118.0	SPT1	43.0		
25.00	30.00	GP-GM	118.0	MCal	65.0		
30.00	36.50	SP-SM	118.0	SPT1	33.0		
36.50	40.00	CL	110.0	MCal	30.0		
40.00	45.00	CL	110.0	SPT1	47.0		
45.00	50.00	SP	118.0	MCal	46.0		

ESTIMATED GEOTECHNICAL DESIGN PARAMETERS

Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Relative Density D_r (%)	Shear Wave Velocity V_s (ft/s)	Effective Peak Friction Angle ϕ' (deg)	Undrained Shear Strength S_u (ksf)	Apparent Density / Soil Consistency Description FHWA (2002) and AASHTO (1988)	Poisson's Ratio μ	Modulus of Elasticity E_s (ksf)	Shear Modulus G (ksf)	Bulk Modulus K (ksf)
641.80	2.50	1.700	1.333	1.150	0.750	0.650	38.9	66.1	100.00	491.18	42.00		Dense Sand	0.35	671.00	745.55	248.52
636.80	7.50	1.536	1.333	1.150	0.800	0.650	40.7	62.5	100.00	640.67	43.00		Dense Sand	0.35	688.75	765.00	255.00
631.80	12.50	1.228	1.333	1.150	0.850	0.650	46.6	57.2	100.00	721.48	42.00		Dense Sand	0.35	746.03	829.00	276.00
626.80	17.50	1.102	1.333	1.150	0.950	0.650	38.8	42.8	98.00	746.08	40.00		Dense Sand	0.35	670.50	745.00	248.00
621.80	22.50	1.022	1.333	1.150	0.950	1.000	62.6	64.0	100.00	809.45	43.00		Very Dense Sand	0.40	892.50	1,488.00	319.00
616.80	27.50	0.955	1.333	1.150	0.950	0.650	61.5	58.8	100.00	834.35	43.00		Very Dense Gravel	0.40	2,123.48	3,539.00	758.00
610.30	33.25	0.892	1.333	1.150	1.000	1.000	50.6	45.1	100.00	845.54	40.00		Very Dense Sand	0.40	783.65	1,306.00	280.00
606.80	38.25	0.849	1.333	1.150	1.000	0.650	29.9	25.4		868.96		3.67	Very Stiff Clay	0.45	2,725.49	9,085.00	940.00
601.80	42.50	0.819	1.333	1.150	1.000	1.000	72.1	59.1		1,076.17		9.03	Hard Clay	0.45	13,537.94	45,126.00	4,668.00
596.80	47.50	0.786	1.333	1.150	1.000	0.650	45.8	36.0	90.00	889.17	39.00		Dense Sand	0.35	738.92	821.00	274.00