Appendix E

Geotechnical Report

GROUP DELTA

Preliminary Geotechnical Report
Proposed Residential Development
1024 West Workman Avenue
West Covina, California

MLC Holdings, Inc.

March 13, 2020
Group Delta Project No. IR739

MLC Holdings, Inc.
5 Peters Canyon Drive, Suite 310
Irvine, California 92618

March 13, 2020
Group Delta Project No. IR739

Attention: Mr. Matt Maehara
Forward Planning Manger

Subject: Preliminary Geotechnical Report
1024 West Workman Avenue
West Covina, California 91790

Dear Mr. Maehara,

Group Delta Consultants (Group Delta) is pleased to submit this preliminary geotechnical report for the proposed residential development at 1024 West Workman Avenue in West Covina, California. Our scope of work was performed in general accordance with our proposal dated February 7, 2020, and your authorization issued as part of Agreement Number No. 69146822 dated February 12, 2020.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions pertaining to this report, or if we can be of further service, please do not hesitate to contact us.

Sincerely,
Group Delta Consultants, Inc.

DRAFT
Michael Givens, PhD, P.E., G.E., P.G.
Associate Engineer

DRAFT
Katherine Reyes, Ph.D.
Senior Engineer

Distribution: Emailed to Addressee

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PRELIMINARY GEOTECHNICAL REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
1024 West Workman Avenue
West Covina, California

1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed development located at 1024 West Workman Avenue, West Covina, California (Site).

1.1 Background

The Site is approximately 8.05 acres located at the southwest intersection of Workman Avenue and North Vincent Avenue in the City of West Covina, California, as shown in Figure 1 – Site Location Map. The Site is occupied by the Vincent Children's Center that includes one story building, one story portables, playgrounds, paved parking lots and grass areas.

1.2 Project Description

Based on the conceptual plans provided by the Client, the proposed development will consist of demolition of the existing structures, grading and construction of single-family homes and two- to three-story townhomes.

1.3 Scope of Work

The main intent of this report is to present the preliminary geotechnical factors that may potentially impact the redevelopment of the Site. Our scope of work included the following:

- Review of relevant United States Geological Survey (USGS) and California Geological Survey (CGS) maps and reports for the site and surrounding area;
- Perform a limited geotechnical field investigation to evaluate subsurface conditions, which includes drilling two (2) hollow-stem auger (HSA) borings (B-1 and B-2) to depths of approximately 51.5 feet and 21.5 feet below ground surface (bgs) with one boring (B-1) converted to a percolation test at a depth of 5 to 10 feet bgs;
- Perform limited laboratory testing program on selected soil samples to evaluate on selected soil samples to evaluate physical, engineering, and chemical (corrosion) properties of the onsite soils;
- Evaluate limited geologic and seismic hazard including local seismicity, surface fault rupture, ground shaking, liquefaction, and other considered geologic hazards;
- Evaluate seismic design parameters in accordance with the 2019 California Building Code;

- Evaluate preliminary geotechnical data to provide preliminary recommendations for foundation type and design parameters (allowable bearing pressure, minimum size, and anticipated settlement);
- Identify primary geotechnical factors that may affect the proposed development; and
- Prepared this preliminary geotechnical report.

This report will not be sufficient for final design or to obtain a building permit. A design level geotechnical report will be required during design phase of this project.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

The preliminary subsurface ground investigation for the project site was performed by Group Delta on February 29, 2020. Laboratory tests were conducted on selected soil samples obtained during our field investigation. A brief description of the field investigation and laboratory testing is provided below.

2.1 Field Exploration

Prior to beginning the field investigation, well permits were obtained from County of Los Angeles, Environmental Health Department. Underground Service Alert (USA) was notified at each exploration location to check subsurface utilities. In addition, geophysical surveys were performed by Southwest Geophysics of San Diego, California (a Group Delta subconsultant), to identify any potential subsurface utilities.

The field exploration program was performed on February 28, 2020 and consistent on drilling two (2) HSA exploratory borings (B-1 and B-2) to depths of about 51.5 and 21.5 feet bgs. Boring B-1 was converted to perform an in-hole permeability test with the percolation zone depth between 5 to 10 feet bgs. The percolation test followed the Los Angeles County Administrative Manual (GS200.1) and ASTM 5912-96.

The exploration was performed under the supervision of a Group Delta Engineer, who maintained logs of the soils encountered, visually classified the material and assisted in obtaining soil samples. Bulk samples of drill cuttings were collected at depths of about 0 to 5 feet. Relatively undisturbed samples were taken in the boring at about 5-foot depth thereafter. Samples were obtained with alternating Modified California (MC) Split Spoon and Standard Penetration Tests (SPT).

The locations of our field exploratory borings are shown on Figure 2. The detailed field investigation and observation procedures along with the logs are presented in Appendix A, and the testing results are presented in Appendix B.

2.2 Laboratory Testing

Limited laboratory testing was performed on selected soil samples collected from the borings to characterize the subsurface materials and to evaluate their index and engineering properties. The laboratory testing program consisted of the following:

- Soil classification
- Moisture content and dry density
- Atterberg limits
- Percent passing No. 200 sieve and grain size distribution
- Corrosion suite

The performed tests are identified on the boring logs in Appendix A and laboratory test results are presented in Appendix B.

2.3 Percolation Testing

On-site falling head percolation test was performed to estimate average infiltration rate of water at a percolation zone depth of 5 to 10 feet bgs at the location of boring B-1 (Figure 2). The percolation test was performed using the falling head permeability test procedure in accordance with Los Angeles County Administrative Manual (GS200.1) and ASTM 5912-96. Percolation test procedures and rate calculations are described in Section 5.5.

3.0 SITE CONDITIONS

3.1 Surface Conditions

The Site is located in a well-developed urbanized area of the City of West Covina and is currently occupied by the Vincent Children's Center and its associated parking and lawn areas. The northern half of the property has been developed with an existing one-story school building, portables, asphalt paved parking lots, playgrounds and associated flat work. The site is fairly level with the improved portion of the property, with the school facilities, situated approximately 5 to 7 feet higher than the lawn area. Elevation ranges at the Site from approximately 403 to 407 feet at the lawn area and from approximately 407 to 410 feet at the building and parking areas. A historical topographic map of the surrounding area is presented in Figure 3.

3.2 Subsurface Conditions

Preliminary evaluation of the onsite soil profile was performed considering field exploration borings discussed in Section 2.02.1. The field explorations performed at the Site indicated the presence of fill at the northern portion where the building and parking areas are located. The historic topographic map indicates the northern portion of the property had previously been graded with 5 to 8 feet of fill. Fill materials encountered during drilling consisted of medium dense Silty Sand (SM) with gravels and cobbles. Native material encountered below the fills at the northern portion and from ground surface at the lawn areas consisted mostly of medium dense to very dense Silty Sand (SM),Poorly-graded Sand (SP), and Well-graded Sand (SW) interlayered with Lean Clay with Sand (CL) and Clayey Sand (SC) to the maximum explored depth of 51.5 feet bgs. The sand is mostly fine to medium with some coarse grained and the few gravels were fine grained.

3.3 Groundwater

Groundwater was not encountered in borings drilled to maximum depth of 51.5 feet bgs. According to the Seismic Hazard Zone Report (CGS, 1998), the historic high ground water is about 100 feet deep at the project site, as shown in Figure 4.

Review of available data from the California Groundwater Elevation Monitoring Program (CASGEM) indicate 3 wells within 1 to 2 miles from the Site that report continuous groundwater depths deeper than 100 feet between the period from 2011 to 2019.

4.0 GEOLOGICAL HAZARD EVALUATION

Preliminary evaluation of potential geologic hazards for the project site included review of available published maps, reports, and data. The main potential geologic hazards evaluated for the site include seismicity, ground rupture, liquefaction, and landslides. Our preliminary findings and conclusions are discussed below. A detailed geologic and seismic hazard evaluation should be performed during the design-level geotechnical investigation.

4.1 Geologic Setting

The Site is situated within the southeast central San Gabriel Valley area of the Peninsular Ranges. The valley rests on a triangular shaped basin at depth, which formed through Pleistocene-Pliocene convergence of northwest trending faults within the Peninsular Ranges approaching the Transverse Ranges east-west faults (Yeats, 2004). The basin is bordered by fault bound hills and mountains. The San Gabriel Mountains are to the north, bound by the Raymond and Sierra Madre Fault Zone. The San Jose Hills are to the southeast, bound by the Walnut Creek Fault, and the Montebello Hills are to the southwest, bound by the East Montebello Fault.

Locally, the Site is located within the Big Dalton Wash depositional area of the San Gabriel Valley. The Wash extends southwest from the San Gabriel Mountains and blankets the southern portion of the broad valley. The geologic map in Figure 5 shows that the near surface deposition is mapped as Holocene Young Alluvial Fan and Valley Deposits (Qyfa) that are predominantly composed of sand. Sedimentation is hundreds of feet thick below the site. Since the development of the valley, mass sedimentation is largely confined to debris-controlled basins and channels. Walnut Creek flows west through an engineered concrete lined channel about 0.4 miles south of the site and an engineered channel for the Big Dalton Wash flows southwest about 1.4 miles north and west of the site.

4.2 Local Seismicity and Earthquake Faults

The Site is located within the seismically active area of southern California and there is a high potential for the Site to experience strong ground shaking from local and regional faults. A fault that has ruptured in at least the last 11,700 years is considered to have a higher potential of future seismicity and is considered an active fault by the Alquist-Priolo Earthquake Fault Zoning Act. Faults with evidence of longer earthquake frequency events are considered to have a lower potential of future seismicity. The location of the Site with respect to regional faults with the potential for future seismic activity is presented in Figure 6, Regional Fault and Seismicity Map. Significant seismically active faults nearest to the are presented in Table 1.

Table 1. List of Earthquake Faults Closest to the Subject Site

Abbreviated Fault Name	Fault Type	Max. Magnitude (Mw)	Approximate Closest Distance (Km)
Indian Hill	Normal	6.1	3.7
San Jose	Strike Slip	6.7	5.7
Sierra Madre Fault (Sierra Madre C)	Reverse	7.2	8.2
Sierra Madre Fault (Sierra Madre E)	Reverse	7.2	11.4
Raymond	Strike Slip	6.7	11.4
Puente Hills (Coyote Hills)	Reverse	6.6	12.3

Note: Data collected from Caltrans ARS Online Tool Version 3.0

The San Andreas Fault is the most significant seismically active fault in the region. It stretches over 800 miles across the state of California and represents the boundary of the North American Tectonic Plate and the Pacific Tectonic Plate. It is over 25 miles (40 kilometers) north east of the Site and considered capable of M7.9 earthquakes.

Historical seismicity recorded by the U.S. Geological Service (USGS, accessed 3/11/2020) within a 100-kilometer radius of the Site includes 395 earthquakes of magnitude (M) 4.0 and greater since 1812. Nine (9) of these earthquakes are of M6.0 and greater, including the M6.7 January 17, 1994 Northridge earthquake located at 36 miles (58 kilometers) from the Site. The closest recorded earthquake to the Site was a M5.2 on August 28, 1889, which epicenter was about 4 kilometers to the east of the Site. While not within the search radius, earthquakes of M7.0 and greater have been recorded in Southern California. The M7.5 Kern County earthquake occurred in July 21, 1952, the Landers M7.1 earthquake in June 28, 1992, and the recent Ridgecrest M7.1 earthquake in July 5, 2019 was located about 119 miles (192 kilometers) northeast of the Site.

Construction in this area should be designed with accepted engineering practices and in compliance with current building codes that accommodates strong seismic ground motion.

4.3 Preliminary Seismic Design Parameters

Preliminary seismic design parameters were developed in accordance with the 2019 California Building Code. Based on the subsurface exploration and underlying geology, the site classification for seismic design is Site Class D, in accordance with Chapter 20 of ASCE 7-16.

Per Section 11.4.8 of ASCE 7-16, a site-specific ground motion hazard analysis is required for "structures on Site Class D and E sites with S_1 greater than or equal to 0.2", unless certain exceptions are met. Based on the site subsurface conditions and the mapped seismic demand (S_1 > 0.2), the mapped design acceleration parameters (presented in Table 2) can only be used if Exception 2 of ASCE 7-16 Section 11.4.8 is met:

- **If $T \leq 1.5\ T_S$**: The value of the seismic response coefficient C_S is determined by Eq. (12.8-2), i.e., S_{DS} is used to obtain C_S, or
- **If $T_L \geq T > 1.5\ T_S$**: The value of seismic response coefficient C_S is taken as ***1.5 times*** the value computed in Eq. (12.8-3), i.e., ***1.5****S_{D1} is used to obtain C_S, or
- **If $T > T_L$**: The value of seismic response coefficient C_S is taken as ***1.5 times*** the value computed in Eq. (12.8-4), i.e., ***1.5****S_{D1} is used to obtain C_S.

Based on Exception 2, if the fundamental period is less than or equal to $1.5T_S$, S_{DS} must be used to determine the seismic response coefficient, C_S, with equation 12.8-2. If the fundamental period is higher than 1.5 T_S (longer period structures), then the determination of C_S is increased by a factor of 1.5.

Table 2. CBC 2019 /ASCE 7-16 Mapped Seismic Design Acceleration Parameters

Design Parameters	Seismic Design Parameter Mapped Value (ASCE 7-16 Section 11.4)
Site Latitude	34.074490
Site Longitude	-117.927507
S_s (g)	1.664
S_1 (g)	0.610
Site Class	D
F_a	1.008
F_v	1.70
T_S (sec)	0.623
T_L (sec)	8
S_{MS} (g)	1.664
S_{M1} (g)	1.037
S_{DS} (g)	1.109 (1)
S_{D1} (g)	0.691 (2)

Notes:
(2) For $T \leq 1.5\ T_s$, S_{DS} should only be used to obtain Cs using Equation 12.8-2.
(1) If S_{D1} is used to obtain C_S with either equation 12.8-3 or 12.8-4 of ASCE 7-16, the value must be increased by a factor of 1.5. This may only be used for $T > 1.5\ T_S$.

4.4 Liquefaction and Seismic Settlement

Liquefaction involves sudden loss in strength of a saturated, cohesionless soil caused by the build-up of pore water pressure during cyclic loading, such as that produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in differential settlements and ground deformations. Typically, liquefaction occurs in areas where there are loose soils and the depth to groundwater is less than 50 feet from the surface.

According to the State of California Seismic Hazards Zone Map (CGS, 1998), the site is not located within an Earthquake Required Investigation Zone for liquefaction. Furthermore, ground water was not encounter during our field investigation to maximum depth of about 51.5 feet bgs and historical highest groundwater level is 100 feet deep. Therefore, the potential liquefaction during earthquake is considered low.

4.5 Landslides

As mentioned above, the Site is situated centrally within an alluvial valley. The valley floor is relatively level with a gentle slope to the southwest. There are no significant slopes that can present a landslide hazard at or near the site. Therefore, landslides are not considered a hazard at the Site.

4.6 Fault Surface Rupture

Faults that show evidence of a surface rupture event in the last 11,700 years are defined by the California Geological Survey (CGS) to be a potential source of fault surface rupture hazard. To mitigate this potential hazard the State regulates new development, requiring structures planned for human occupancy to be setback from recent ruptured fault traces. The regulated faults are mapped within in Alquist-Priolo Earthquake Fault Zones by the State and Fault Hazard Management Zones by the City. There are no mapped zoned faults across the site or trending directly toward the site. The closest fault to the site is the Indian Hill Fault which traverses east-west about 3.7 km east of the site, on trend toward the project site. However, evidence of recent surface rupture is unknown at this time. Therefore, the potential for surface fault rupture hazard at the site is considered low.

4.7 Other Geologic Hazards Considered

4.7.1 Flooding and Inundation

Flooding and inundation potential at the Site were evaluated through review of the Safety Element for the City of West Covina and FEMA National Flood Hazard Layer (NFHL) maps (FEMA, 2008). The City Safety Element indicates the site is located within a dam breech inundation zone which may source from several dams upgradient of the site, including: San Gabriel River Flood Control dams, Big Dalton Wash, San Dimas Wash, and Puddingstone Dam. Flood control across the San Gabriel Valley is accomplished through the maintenance and monitoring of the Los Angeles County Public Works Department. The FEMA NHFL indicates the Site is in an area of 0.2% Annual Chance Flood Hazard, Flood Hazard Zone X. New development at the site may significantly reduce the potential for flood hazard with proper drainage design.

Site is located over 25 miles east from the nearest coastline at an Elevation of about 410 feet. Tsunami is not considered a hazard at the Site.

4.7.2 Naturally Occurring Hazardous Elements

Naturally occurring hazardous elements within subsurface materials, can include corrosivity, asbestos, radon, and oil and methane gas. Corrosivity testing has been performed for one sample from the site soils at depth, however, corrosivity will need to be evaluated for the Site during the future design level geotechnical investigation for the project. The CGS Map Sheet 59, of known sites with naturally occurring asbestos does not indicate there is a potential for naturally occurring asbestos to be at the site (USGS, 2011). The CGS Special Radon Potential Zone Map indicates the site has low potential for indoor radon levels above four (4) picocuries per liter (CGS, 2005). Four picocuries per liter is recommended to be an action level for radon reduction by the U.S. Environmental Protection Agency.

4.8 Expansive Soils

The on-site near surface sandy soils are expected to have a very low expansion potential (Expansion Index, EI<20) materials. Therefore, expansive soils are not likely a design concern.

4.9 Corrosivity

A representative sample collected on the site soils was tested to evaluate preliminary corrosion characteristics. The results indicate the test sample had a pH of 7.18; a water-soluble sulfate content of less t han 0.01%, and a soluble chloride content of less than 0.01%. The sulfate results indicate that sulfate exposure to Portland cement is negligible.

The test sample was also found to have a minimum measured electrical resistivity of 4,808 Ohm-cm. The following correlation can generally be used between electrical resistivity and corrosion potential:

Electrical Resistivity (Ohm-Cm)	Corrosion Potential
Less than 1,000	Severe
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderate
Greater than 10,000	Mild

Based on the laboratory test results, the test sample is classified as moderately corrosive to buried metals. Further evaluation and testing should be performed during final design and recommendations for corrosion protection should be provided by a corrosion consultant.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

Based on a review of existing subsurface information and the current conceptual project information provided to us (2- to 3-story townhomes and single-family homes), it is our opinion that the proposed project is feasible from a geotechnical standpoint. Following proper site development grading, the proposed construction can be supported on shallow foundations or a post-tensioned slab on properly compacted fill or undisturbed native soils. A design level geotechnical report will be required to develop geotechnical recommendations for final design including drilling and sampling geotechnical borings, performing laboratory testing to confirm engineering parameters and for detailed engineering analyses.

A conceptual grading plan Is not available at the time of this report preparation. The proposed finish grade is assumed to be close to the existing grade.

5.2 Site Preparation

The site should be cleared and grubbed of all existing footings, pavements, and other improvements in general accordance with Section 300-1 of the Standard Specification for Public Works Construction [SSPWC] (Green Book, 2018).

Approximately 5 to 8 feet of undocumented fill was identified at the developed (northern) portion of the site. The northern portion of the site with the school facilities has grades ranging between approximately elevation 407 and 410 feet, whereas the original grade of the site is consistent with the southern portion between elevations of approximately 402 to 405 feet. No debris was identified in the undocumented fill, however, it should be anticipated that the remnants of previous construction could be encountered anywhere on the Site. The undocumented fill below the structures should be removed and replaced with compacted fill. Select on-site materials may be used as fill. The removal areas should extend laterally at least 5 feet beyond the edge of footings in all directions.

A firm and unyielding subgrade should be established below the footings and slab, and demonstrated by proof-rolling with loaded heavy equipment. Prior to placement of the first lift of engineered fill or prior to final grading, the upper 8 inches of the exposed soil subgrade should be brought to slightly wet of optimum moisture contest and compacted to a minimum 90 percent of its maximum dry density as determined by ASTM D1557 to provide a uniform bearing surface. A minimum of 95 percent relative compaction is recommended for the foundations and pavement subgrade.

The civil engineer should identify the presence and location of all existing utilities on and adjacent to the Site. Precautions will be required to remove, relocate or protect existing utilities as appropriate.

5.3 Foundation Design Recommendations

Following proper site development grading/excavation, the proposed structures may be situated on conventional spread footings or post-tensioned slabs supported on native soils or structural fill. Our recommendations are based on the assumption that the structure foundations will be located at the ground level.

5.3.1 Shallow Foundations

The following design criteria are recommended for the footings founded on engineered fill or competent sandy soils:

- Shallow spread footings should have a minimum width of 2 feet;

- Shallow continuous footings should have a minimum width of 1.5 feet;
- Bottom of footings should be placed at least 2 feet below the adjacent grade; and
- Bearing design of footings per an allowable pressure of 2 ksf.

The allowable bearing pressure may be increased by one-third for transient loading conditions.

5.3.2 Post-Tensioned Slab

The existing soils at the site are predominantly cohesionless with some clayey soils. These materials generally have a low expansion potential. However, due to the presence of some clayey soils above groundwater, for planning purposes design parameters for the post tensioned slab to resist expansive soils is provided in Table 3. The post-tensioned slab thickness and reinforcement should be designed by the project structural engineer.

Table 3. Post-Tensioned Slab Foundation Preliminary Design Recommendations

<table>
<tr><th colspan="2">Design Parameter</th><th>Value</th></tr>
<tr><td colspan="2">Plasticity Index</td><td>0-15</td></tr>
<tr><td colspan="2">Expansion Index</td><td>0-20</td></tr>
<tr><td colspan="2">Percent Passing No. 200 Sieve</td><td>15-40</td></tr>
<tr><td colspan="2">Thornthwaite Moisture Index</td><td>-20</td></tr>
<tr><td colspan="2">Depth of Constant Soil Suction (feet)</td><td>3.6</td></tr>
<tr><td rowspan="2">Center Lift</td><td>9.0</td><td>9.0</td></tr>
<tr><td>-0.15</td><td>-0.3</td></tr>
<tr><td rowspan="2">Edge Lift</td><td>5.0</td><td>5.0</td></tr>
<tr><td>0.25</td><td>0.5</td></tr>
</table>

For preliminary design, a post-tensioned slab may be designed for an allowable dead-plus-live load pressure of 2,000 psf. The allowable bearing pressure may be increased by one-third when considering temporary loads associated with wind and seismic loading.

5.3.3 Settlement

We estimate the settlement of the structures supported on shallow foundations or a post-tensioned slab in the manner recommended is expected to be less than one (1) inch. The differential settlement is anticipated to be equal to one-half of the total settlement over a distance of 30 feet.

Seismically induced settlements of the foundations due to dry sand settlement during the design earthquake are expected to be less than 0.5 inch.

5.3.4 Lateral Resistance

For resistance of lateral loads, an allowable passive fluid pressure of 300 pcf and an allowable sliding friction coefficient of 0.35 may be used for design, for foundations and slabs place in structural fill or undisturbed native soils. Both values include a factor of safety of at least 1.5 and both passive and sliding resistance may be used in combination without reduction.

5.4 Construction Observation and Testing

A Geotechnical Engineer's representative should observe subgrade preparation, backfill and fill placement. Footing excavations should be observed before placement of concrete to verify that the foundation conditions meet the requirements of the final geotechnical report. The project Geotechnical Engineer may perform compaction tests, probing, or use other methods, to verify that the foundations will be supported in competent soils.

5.5 Infiltration Tests

Boring percolation testing was performed at the boring location B-1 shown in Figure 2. After the boring reached the test depth it was converted into a boring percolation test within a test zone from 5 to 10 feet deep. A test well was installed on the boring at the test zone and consisted of a 4-inch diameter PVC pipe with a solid end cap. The pipe was slotted within the zone and fine gravel was used. Just above the slotted test zone bentonite chips were used as backfill to seal the test zone.

Before performing the boring percolation tests, the well was filled with water to saturate the soils with the purpose of developing a steady state flow within the test zone. After completion of the boring infiltration test, the test wells were abandoned. The casing was removed, soil cuttings were collected in drums for later disposal.

Following saturation, falling head permeability tests were conducted in each test well in accordance with Los Angeles County Administrative Manual (GS200.1) and ASTM 5912-96. The well casing was filled with water and then the level of water in the well was recorded at 10-minute intervals. The water levels were recorded a minimum of eight times. A stabilized rate was achieved in the last three readings and were within ten percent of each other.

Details of the percolation testing procedure are included in Appendix A and percolation rate calculation sheets are included in Appendix C. The field infiltration rates were calculated based on the percolation rate data in the following manner:

- Calculate the field percolation rate as the rate of drop in water level in inches per hour.
- Convert the percolation rate to a raw infiltration rate by accounting for flow out of the sides of the borehole and the volume of water in the pipe.

Reduction Factors may be applied to the raw percolation rate based on the following:

- Use of the Boring Percolation Procedure;
- Site Variability; and
- Long-term siltation, plugging, and maintenance.

A reduction factor of 2 was added for using the boring percolation procedure. A reduction factor of 2 was used for site variability and a reduction factor of 1 was used for long-term siltation, plugging, and maintenance. Therefore, a total reduction factor of 4 was used on the raw percolation rates. A summary of the recommended design infiltration rates is shown in Table 4 below.

Table 4. Summary of Boring Infiltration Tests

Test Well	Soil Type	Zone Evaluated (feet bgs)	Raw Percolation Rate (in/hr)	Recommended Design Infiltration Rate (in/hr)
B-1	Silty Sand (SM)	5-10	0.10	0.05

The following summarizes the findings and our comments regarding this study:

1. Based on the County of Los Angeles Department of Public Works, Geotechnical and Materials Engineering Division, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, the required minimum design infiltration rate is 0.3 inches per hour.

2. The infiltration test was performed at a depth of 5 to 10 feet bgs at the boring B-1. The infiltration test results indicate that the site soils near the test zones do not meet the permeability requirements for proper infiltration. Therefore, shallow infiltration facilities are not feasible within these test zone. It should be cautioned that fines present in the infiltrating water will decrease the soil permeability over time, as fines are carried into the soil. Our experience suggests that this can happen quickly, and the decrease in infiltration rates of silty soils can be significant.

3. The site soils at the test location, B-1 between 5 to 10 feet, consist predominantly of Silty Sand (SM) with fines content measured at 44 percent. The underlying 10 feet of soil consisted of poorly graded Sand with Silt (SP) over a lean Clay with Sand (CL) layer. The fine content present in the sand within the test zone could have decrease the soil permeability, also, the presence of the clay material could have contributed to the decrease in the raw infiltration rate.

4. Based on review of the two explorations advanced as part of this study, there may be an opportunity to identify zones with less fine grained soils as indicated in boring B-2 or target a deeper zone that is more advantageous to infiltration. Additional testing is recommended.

6.0 LIMITATIONS

This consultation was performed in accordance with generally accepted Geotechnical Engineering principles and practice. The professional engineering work and judgments presented in this report meet the standard of care of our profession at this time. No other warranty, expressed or implied, is made. This report has been prepared for MLC Holdings, and their design consultants. It may not contain sufficient information for other parties or other purposes and should not be used for other projects or other purposes without review and approval by Group Delta.

The recommendations contained in this report are preliminary and based on conceptual plans for the project. This report is not sufficient to obtain a building permit. A design-level geotechnical report is required before final design plans can be developed.

7.0 REFERENCES

California Department of Water Resources (DWR), California Groundwater Elevation Monitoring Program (CASGEM), https://water.ca.gov/Programs/Groundwater-Management/Groundwater-Elevation-Monitoring--CASGEM (accessed March 10, 2020)

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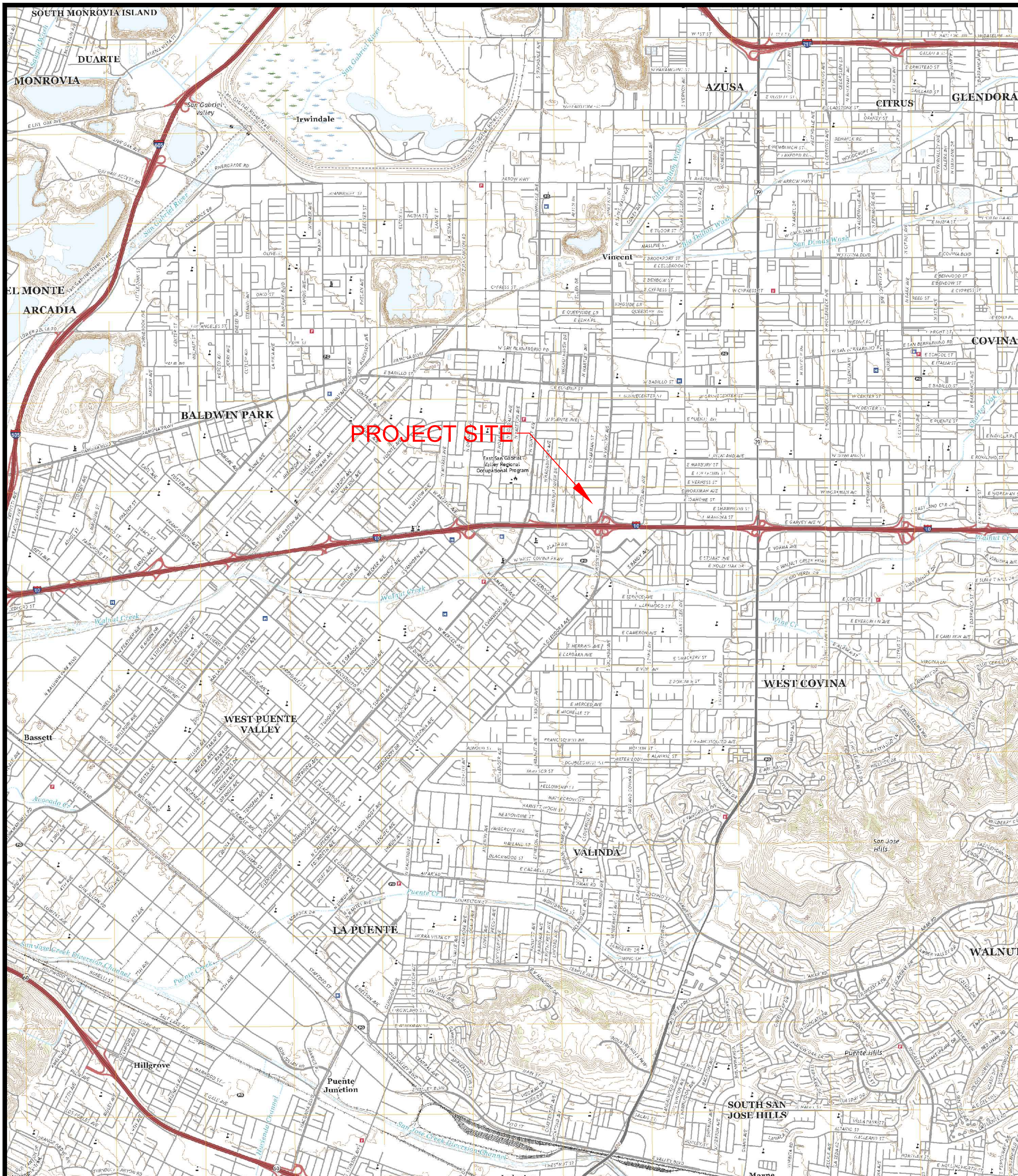
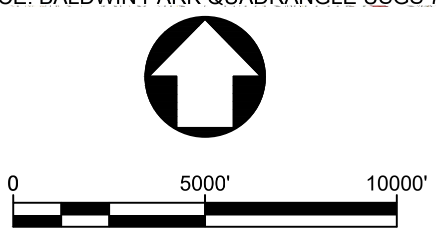
FIGURES



REFERENCE: BALDWIN PARK QUADRANGLE USGS 7.5 MINUTE SERIES



GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100		FIGURE NUMBER: 1
	PREPARED BY: JMT	PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE 1024 W. WORKMAN AVENUE WEST COVINA, CALIFORNIA	PROJECT NUMBER: IR739
	REVIEWED BY: KR	SITE LOCATION MAP	

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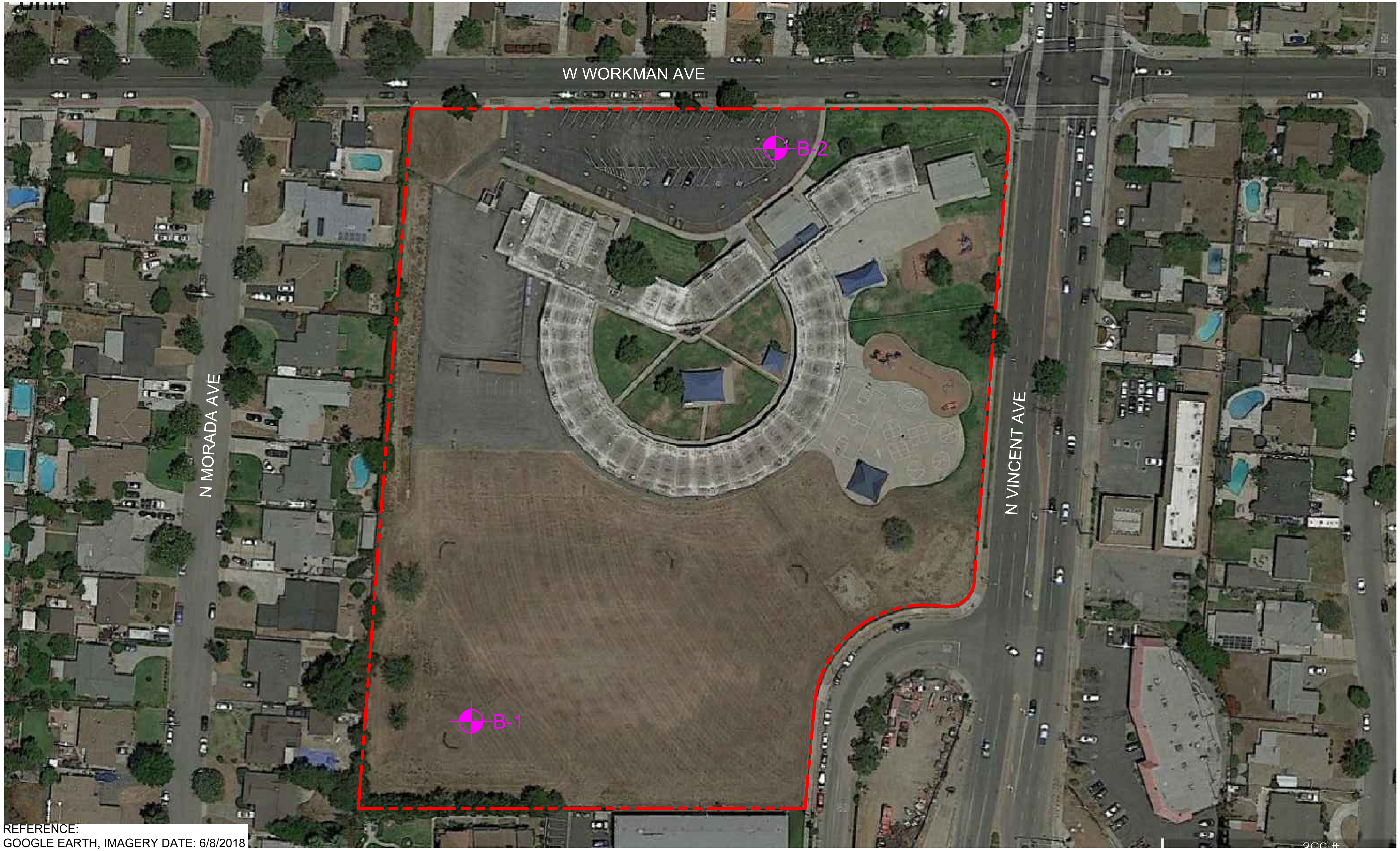


REFERENCE:
GOOGLE EARTH, IMAGERY DATE: 6/8/2018

LEGEND:

B-1 APPROXIMATE GROUP DELTA LOCATION OF HSA BORING AND PERCOLATION TEST

B-2 APPROXIMATE GROUP DELTA LOCATION OF HSA BORING

SITE BOUNDARY

0 100' 200'

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
32 MAUCHLY, SUITE B
IRVINE, CALIFORNIA (949) 450-2100

FIGURE NUMBER: 2

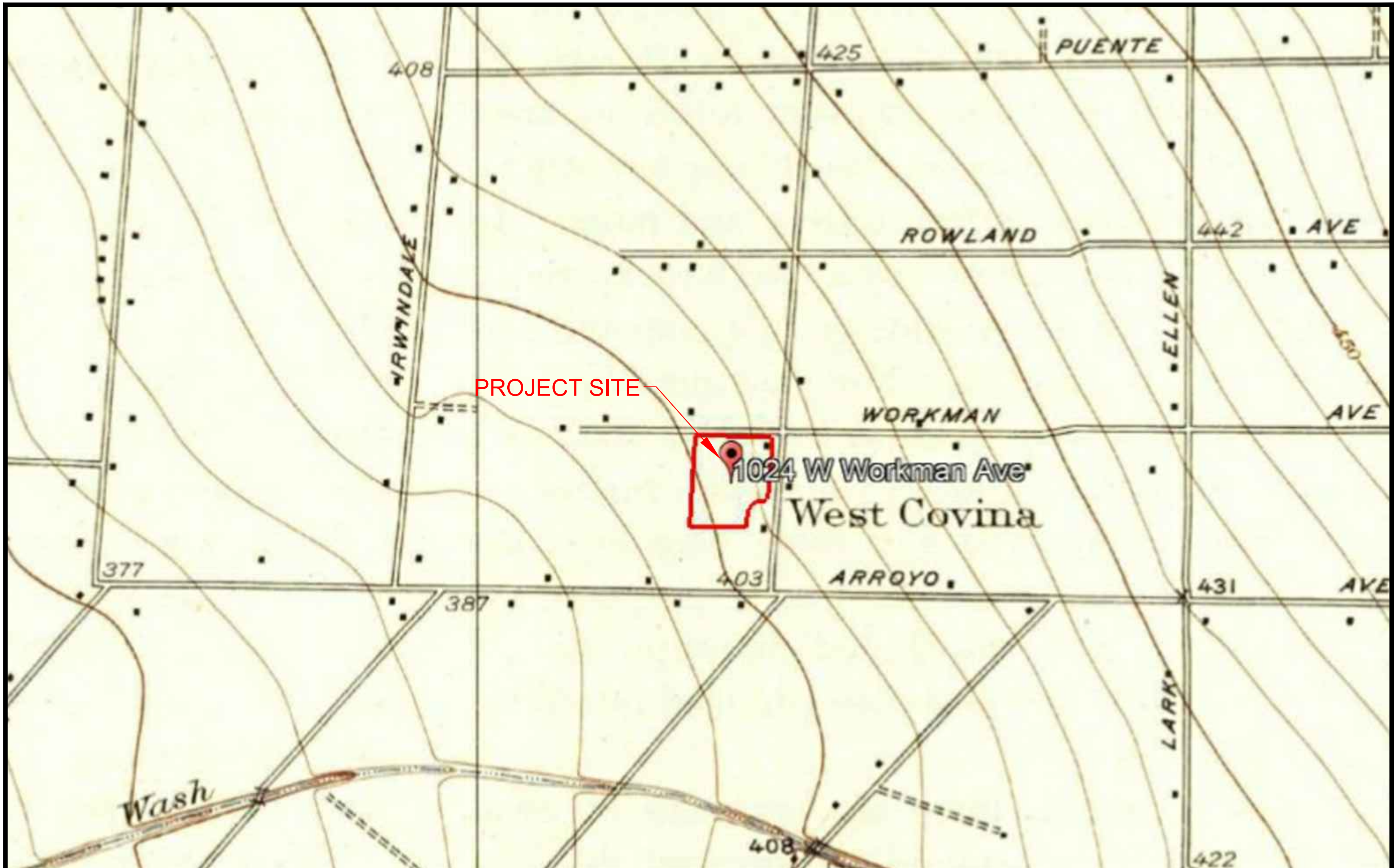
PREPARED BY: JMT

PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE
1024 W. WORKMAN AVENUE
WEST COVINA, CALIFORNIA

PROJECT NUMBER: IR739

REVIEWED BY: KR

EXPLORATION LOCATION PLAN



REFERENCE:
USGS, 1927 HISTORICAL TOPOGRAPHIC MAP,
LA PUNTED 7.5 MINUTE QUADRANGLE

0 1000' 2000'

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100		FIGURE NUMBER: 3
PREPARED BY: JMT	PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE 1024 W. WORKMAN AVENUE WEST COVINA, CALIFORNIA	PROJECT NUMBER: IR739
REVIEWED BY: KR	HISTORIC TOPOGRAPHIC MAP	

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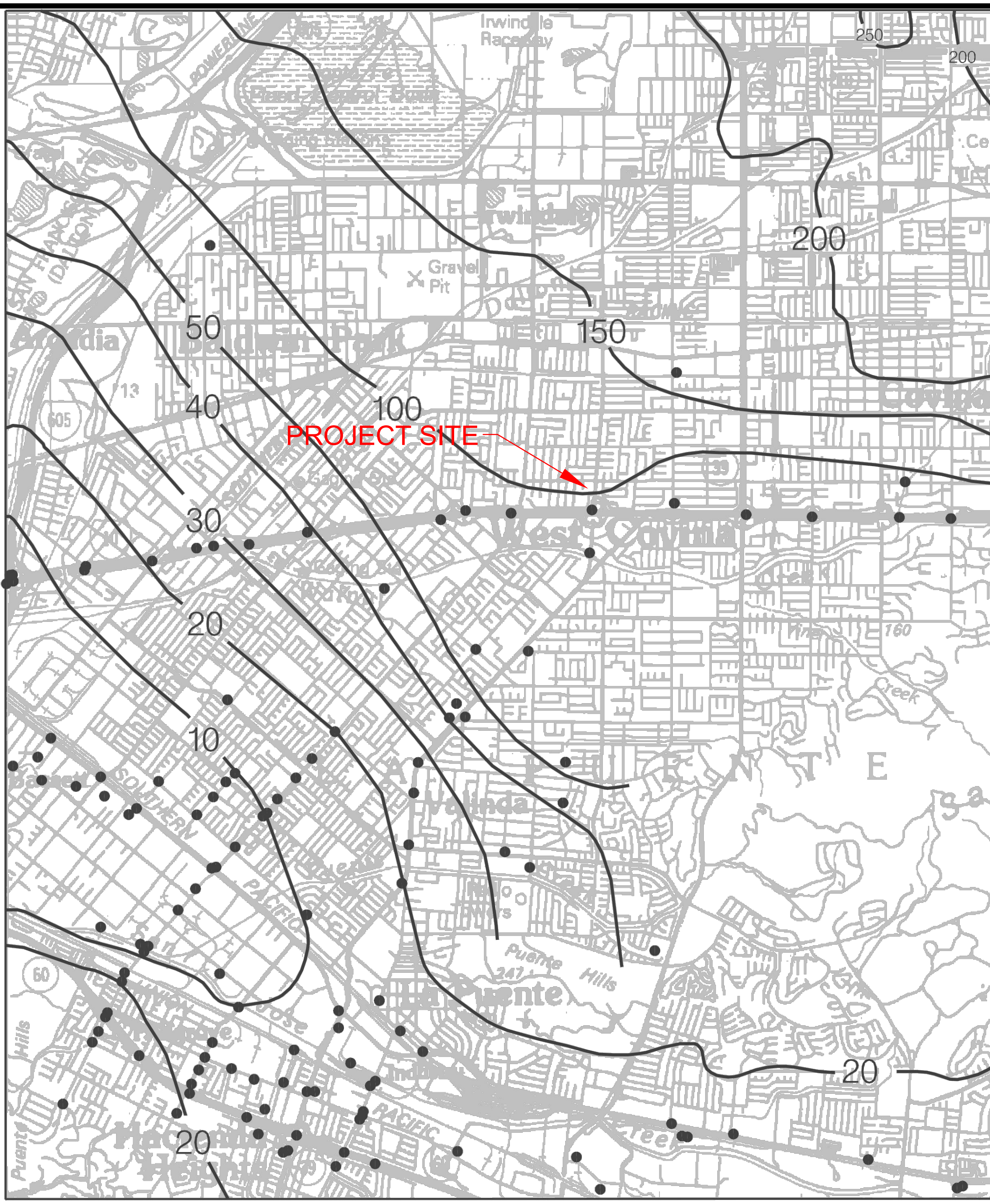


Base map enlarged from U.S.G.S. 30 x 60-minute series

Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Baldwin Park Quadrangle.

● Borehole Site

30 Depth to ground water in feet

ONE MILE

SCALE

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100	FIGURE NUMBER: 4
	PREPARED BY: JMT / PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE 1024 W. WORKMAN AVENUE WEST COVINA, CALIFORNIA	PROJECT NUMBER: IR739
	REVIEWED BY: KR / HISTORICALLY HIGHEST GROUNDWATER CONTOURS	

REFERENCE: CGS, 1998, SEISMIC HAZARD ZONE REPORT, PLATE 1.2 GROUND WATER, FOR THE BALDWAIN PARK 7.5-MINUTE QUADRANGLE, LOS ANGELES, CALIFORNIA, SHZR 022.

EXPLANATION OF MAP UNITS

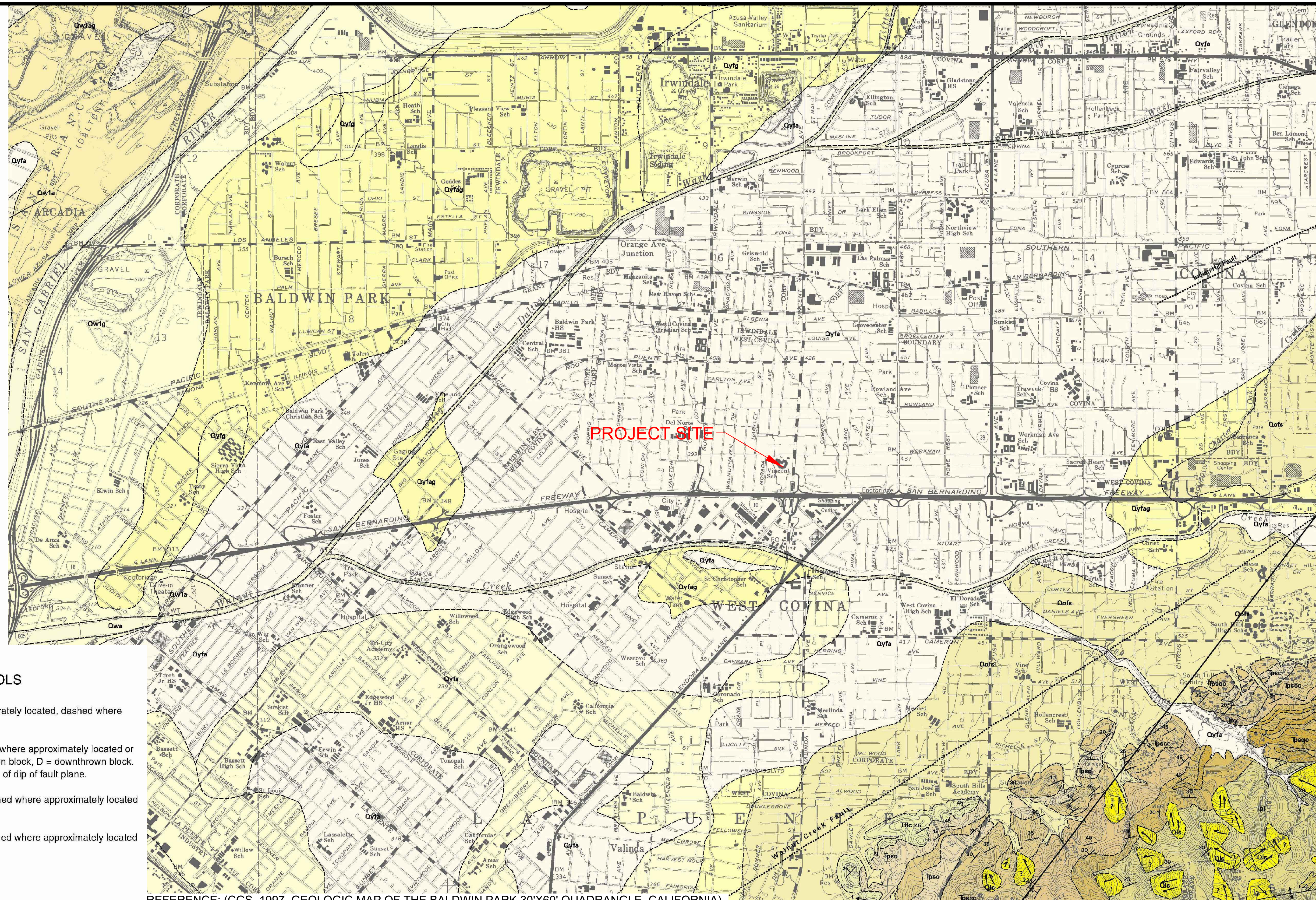
Qw	Active channel and wash deposits, unconsolidated silt and coarser material, mostly artificially channelized; a = sand, g = gravel.
Ql	Active lacustrine deposits, unconsolidated, submerged; c = clay, s = silt, g = gravel.
Qw1	Modern river sediments prior to the artificial channelization of river, unconsolidated; a = sand, g = gravel.
Qyf	Younger (Holocene) undivided alluvial fan and valley deposits, unconsolidated; c = clay, s = silt, a = sand, g = gravel.
Qls	Landslide deposits; landslide, broken-up and weathered material; queried where existence is uncertain. It is subject to renewed slope failure.
Qof	Older (Pleistocene) undivided alluvial fan and valley deposits, moderately to well-consolidated; s = silt, a = sand, g = gravel; numbers 1 to 4 indicate relative levels of terraces with 1 being the highest and oldest.

Fernando Formation (Pliocene)

Tfu	Upper Member; massive, friable silty and pebbly sandstone, with interbedded thin beds of siltstone.
Tfuc	Upper Member; conglomerate and pebbly sandstone, interbedded with Tfu.
Tfl	Lower Member; massive silty sandstone, with interbedded pebbly sandstone and conglomerate.
Tflc	Lower Member; conglomerate and pebbly sandstone, interbedded with Tfl.

Puente Formation (late Miocene)

Tpsc	Sycamore Canyon Member; sandstone, with interbedded pebble-cobble conglomerate and sandy siltstone.
Tpscc	Sycamore Canyon Member; pebble-cobble conglomerate and pebbly sandstone, interbedded with Tpsc.
Tpy	Yorba Member; platy diatomaceous siltstone with interbeds of sandstone, limestone and marl.
Tpsq	Soquel Member; sandstone with concretions, siltstone and conglomerate.
Tpscq	Soquel Member; conglomerate and pebbly sandstone, interbedded with Tpsq.
Tpl	La Vida Member; platy siltstone contains foraminifera and fish scales, with interbeds of sandstone, conglomerate, limestone and tuff.



REFERENCE: (CGS, 1997, GEOLOGIC MAP OF THE BALDWIN PARK 30'X60' QUADRANGLE, CALIFORNIA)

MAP SYMBOLS

Contact between map units - solid where accurately located, dashed where approximately located or inferred.

Fault - solid where accurately located, dashed where approximately located or inferred, dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Anticline - solid where accurately located, dashed where approximately located or inferred.

Syncline - solid where accurately located; dashed where approximately located or inferred.

Strike and dip of inclined sedimentary beds.

Strike and dip of overturned sedimentary beds.

Landslide - arrows indicate principal direction of movement; headscarp area is indicated by hachures.
Landslides were mapped from pre-graded (natural) conditions; some slides may have been subsequently altered by mitigation and stabilization activities; questionable landslides are denoted by single arrow and a query.

A-A'-A" Geologic cross section.

0 3000' 6000'

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100	FIGURE NUMBER: 5
PREPARED BY: JMT	PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE 1024 W. WORKMAN AVENUE WEST COVINA, CALIFORNIA	PROJECT NUMBER: IR739
REVIEWED BY: KR	REGIONAL GEOLOGY MAP	

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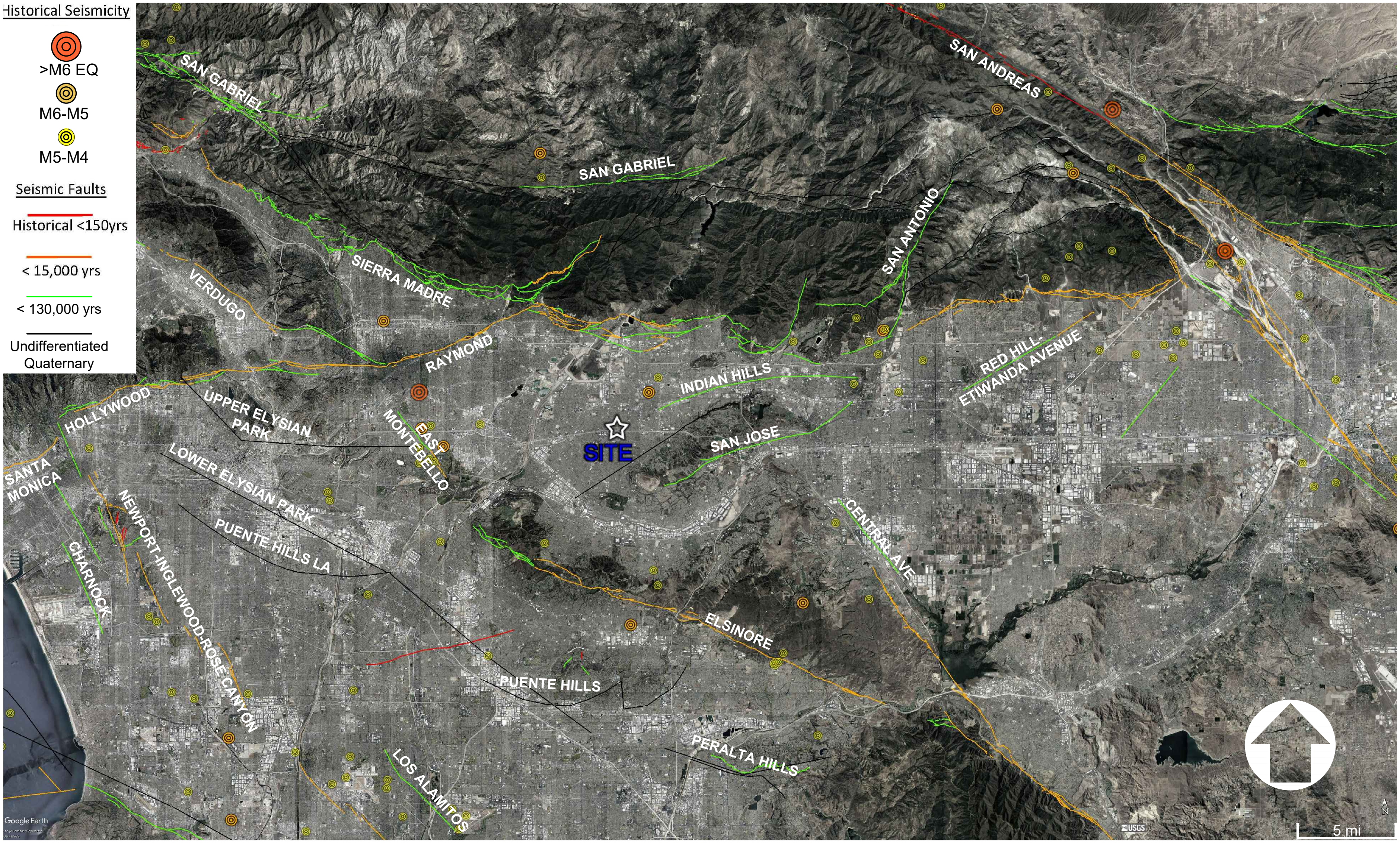

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100	FIGURE NUMBER: 6
PREPARED BY: JMT	PROJECT NAME: MLC HOLDINGS-WEST COVINA SITE 1024 W. WORKMAN AVENUE WEST COVINA, CALIFORNIA	PROJECT NUMBER: IR739
REVIEWED BY: KR	REGIONAL FAULT ACTIVITY MAP	

REFERENCE:
GOOGLE EARTH, IMAGINARY DATE, 6/8/2018
USGS, EARTHQUAKE CATALOG, ACCESSED 11/26/2019
USGS & CGS, QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 3/4/2020

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APPENDIX A
FIELD INVESTIGATION

APPENDIX A
FIELD INVESTIGATION

A.1 Introduction

The subsurface conditions at the West Covina project site were investigated by performing 2 hollow stem auger borings with 1 percolation test on February 28, 2020. The locations of the explorations are presented in Figure 2 of the main report. A summary of field explorations is presented in Table A-1.

Prior to beginning the exploration program, access permission and drilling permits were obtained as necessary from MLC Holdings, Inc. Underground Service Alert (USA) was notified and each exploration location was cleared for underground utilities. Approved traffic control plans were implemented where necessary during field activities. The exploration methods are described in the following sections.

A.2 Soil Drilling and Sampling

Drilling, Logging, and Soil Classification

Borings were performed by GDC's drilling subcontractors ABC Liovin Drilling under the continuous technical supervision of a GDC field engineer, who visually inspected the soil samples, measured groundwater levels, maintained detailed records of the borings, and visually / manually classified the soils in accordance with the ASTM D 2488 and the Unified Soil Classification System (USCS). Logging and classification was performed in general accordance with Caltrans "Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition)". A Boring Record Legend and Key for Soil Classification are presented in Figures A-1A through A-1E. The boring records are presented in Figures A-2A through Figure A-3A.

Sampling

Bulk samples of soil cuttings were collected at selected depths and drive samples were collected at a typical interval of 5 feet from the borings. The sampling was performed using Standard Penetration Test (SPT) samplers in accordance with ASTM D 1586 and Ring-Lined "California" Split Barrel samplers in accordance with ASTM D 3550.

Bulk samples were collected from auger cuttings and placed in plastic bags.

SPT drive samples were obtained using a 2-inch outside diameter and 1.375-inch inside diameter split-spoon sampler without lining. The soil recovered from the SPT sampling was sealed in plastic bags to preserve the natural moisture content.

California drive samples were collected with a 3-inch outside diameter 2.5-inch inside diameter split barrel sampler with a 2.42-inch inside diameter cutting shoe. The sampler barrel is lined with 18-inches of metal rings for sample collection and has an additional length of waste barrel. Stainless steel or brass liner rings for sample collection are 1-inch high, 2.42-inch inside diameter, and 2.5-inch outside diameter. California samples were removed from the sampler, retained in the metal rings and placed in sealed plastic canisters to prevent loss of moisture.

At each sampling interval, the drive samplers were fitted onto sampling rod, lowered to the bottom of the boring, and driven 18 inches or to refusal (50 blows per 6 inches) with a 140-lb hammer free-falling a height of 30-inches using an automatic hammer.

Compared to the SPT, the California sampler provides less disturbed samples.

<u>Penetration Resistance</u>

SPT blow counts adjusted to 60% hammer efficiency (N_{60}) are routinely used as an index of the relative density of coarse grained soils, and are sometimes used (but less reliable) to estimate consistency of cohesive soils. For samples collected using non-SPT samplers, different hammer weight and drop height, and/or efficiency different than 60%, correction factors can be applied to estimate the equivalent SPT N_{60} value following the approach of Burmister (1948) as follows:

$$N^*_{60} = N_R * C_E * C_H * C_S$$

where

N^*_{60} = equivalent SPT N_{60}

N_R = Raw Field Blowcount (blows per foot)

C_E = Hammer Efficiency Correction = Er_i / 60%

C_H = Hammer Energy Correction = (W * H) / (140 lb * 30 in)

C_S = Sampler Size Correction = $[(2.0\text{ in})^2-(1.375\text{ in})^2]/[D_o^2-D_i^2]$

Er_i = hammer efficiency, %

W= actual drive hammer weight, lbs

H = actual drive hammer drop, inch

D_o, D_i = actual sampler outside and inside diameter, respectively, inches

Burmister's correction assumes that penetration resistance (blowcount) is inversely proportional to the hammer energy. For a hammer other than a 140# hammer with 30" drop the hammer energy correction is equal to the ratio of the theoretical hammer energy (weight times drop) to the theoretical SPT hammer energy, or C_H = (W * H) / (140 lb * 30 in).

Burmister's correction assumes that penetration resistance (blowcount) is proportional to the annular end area of the drive sampler. For California drive samplers with D_o=3 inch and D_i=2.42 inch the sampler size correction factor is the ratio of the annular area of an SPT split spoon to that of the California Sampler, or $C_S = [2.0^2 - 1.375^2]/[3^2 - 2.42^2] = 0.67$.

To normalize the field SPT and California blowcounts to a hammer with 60% efficiency, an energy correction factor equal to Hammer Efficiency (%) / 60% was applied to the field blowcounts. Hammer efficiency was determined by Pile Driving Analyzer (PDA) measurement. Hammer efficiency measurements are presented in Figures A-4A through A-4B.

The correction factors applied to obtain N^*_{60} are summarized in the following table:

Borings	Hammer Type	Hammer Weight and Drop	C_H	Hammer Efficiency (%)	C_E	Cal Sampler Dimensions	C_S	Combined Correction Factor SPT Samples	Combined Correction Factor CAL Samples
B-1	CME Auto	140# 30"	1	62.6	1.043	D_o=3.0" D_i=2.42"	0.67	1.043	0.696
B-2	CME Auto	140# 30"	1	62.6	1.043	D_o=3.0" D_i=2.42"	0.67	1.043	0.696

Corrected N^*_{60} are generally used, with due engineering judgment, only for qualitative assessment of in place density or consistency and are not used for other more critical analyses such as liquefaction.

Relative Density and Consistency

Equivalent SPT N_{60} values were used as the basis for classifying relative density of granular/cohesionless soils. Wherever possible consistency classification of cohesive soils was based on undrained shear strength estimated in the field with a pocket penetrometer or by testing in the laboratory. Where pocket penetrometer or other tests could not be performed, consistency of cohesive soils was estimated by correlations to Equivalent SPT N_{60}. The correlations for consistency and relative density are shown in the Boring Record Legend, Figures A-1A through A-1C. Drive sample field blow counts, SPT N^*_{60} values, pocket penetrometer readings, and corresponding density/consistency classifications are presented on the boring records.

Borehole Abandonment

At the completion of the drilling groundwater was measured (where possible) and the borings were abandoned by backfilling the borehole with Bentonite grout. Excess cuttings and drilling fluids were placed in 55 gallon drums, sampled and tested for contaminants, temporarily stored at an approved location, and legally disposed of off-site. The surface was patched with cold mix asphalt concrete or quickset concrete, as necessary. Notes describing the borehole abandonment are presented at the bottom of each boring record.

Sample Handling and Transport

Geotechnical samples were sealed to prevent moisture loss, packed in appropriate protective containers, and transported to the geotechnical laboratory for further examination and geotechnical testing.

Laboratory Testing

The soils were further examined and tested in the laboratory and classified in accordance with the Unified Soil Classification System following ASTM D 2487 and D 2488 (see Figures A-1D and A-1E). Field classifications presented on the records were modified where necessary on the basis of the laboratory test results. Descriptions of the laboratory tests performed and a summary of the results are presented in Appendix B.

A.3 List of Attached Tables and Figures

The following tables and figures are attached and complete this appendix:

List of Tables

Table A-1 Summary of Field Explorations

List of Figures

Figure A-1A through A-1C Boring Record Legend
Figure A-1D and A-1E Key for Soil Classification
Figures A-2A through A-3A Boring Records
Figure A-4A through A-4B Hammer Efficiency Calibrations

TABLE A-1
SUMMARY OF FIELD EXPLORATIONS

Exploration No.	Approximate Exploration Location		Exploration			Groundwater		Figure No.
	Longitude	Latitude	Type	Surface Elevation (ft)	Total Depth (ft)	Depth (ft)	Elevation (ft)	
B-1	-117.9282	34.0738	HSA	150.0	51.5	NE	NE	A-2 (A-C)
B-2	-117.9272	34.0753	HSA	150.0	11.5	NE	NE	A-3 (A)

Notes: 1) Boring locations are illustrated in Figure 2 of the main report.
2) Elevations reported to 0.01 ft were surveyed, other elevations estimated to nearest 0.5 ft using tape measure and topographic map.
HSA = Hollow-Stem Auger   NE = Not Encountered

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence		Refer to Section: Field	Refer to Section: Lab	Required	Optional
1	Group Name	2.5.2	3.2.2	●	
2	Group Symbol	2.5.2	3.2.2	●	
	Description Components				
3	Consistency of Cohesive Soil	2.5.3	3.2.3	●	
4	Apparent Density of Cohesionless Soil	2.5.4		●	
5	Color	2.5.5		●	
6	Moisture	2.5.6		●	
7	Percent or Proportion of Soil	2.5.7	3.2.4	●	◉
	Particle Size	2.5.8	2.5.8	●	◉
	Particle Angularity	2.5.9			○
	Particle Shape	2.5.10			○
8	Plasticity (for fine-grained soil)	2.5.11	3.2.5		○
9	Dry Strength (for fine-grained soil)	2.5.12			○
10	Dilatency (for fine-grained soil)	2.5.13			○
11	Toughness (for fine-grained soil)	2.5.14			○
12	Structure	2.5.15			○
13	Cementation	2.5.16		●	
14	Percent of Cobbles and Boulders	2.5.17		●	
	Description of Cobbles and Boulders	2.5.18		●	
15	Consistency Field Test Result	2.5.3		●	
16	Additional Comments	2.5.19			○

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

USCS Group Name (Group Symbol); Consistency or Density; Color; Moisture; Percent or Proportion of Soil; Particle Size; Plasticity (optional).

◉ = optional for non-Caltrans projects

Where applicable:

Cementation; % cobbles & boulders;
Description of cobbles & boulders;
Consistency field test result

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

HOLE IDENTIFICATION

Holes are identified using the following convention:

H-YY-NNN

Where:
H: Hole Type Code
YY: 2-digit year
NNN: 3-digit number (001-999)

Hole Type Code	Description
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (conventional)
RC	Rotary core (self-cased wire-line, continuously-sampled)
RW	Rotary core (self-cased wire-line, not continuously sampled)
P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
HA	Hand auger
D	Driven (dynamic cone penetrometer)
CPT	Cone Penetration Test
O	Other (note on LOTB)

Description Sequence Examples:

SANDY lean CLAY (CL); very stiff; yellowish brown; moist; mostly fines; some SAND, from fine to medium; few gravels; medium plasticity; PP=2.75.

Well-graded SAND with SILT and GRAVEL and COBBLES (SW-SM); dense; brown; moist; mostly SAND, from fine to coarse; some fine GRAVEL; few fines; weak cementation; 10% GRANITE COBBLES; 3 to 6 inches; hard; subrounded.

Clayey SAND (SC); medium dense, light brown; wet; mostly fine sand,; little fines; low plasticity.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER A-1A
	PROJECT NAME Miles Avenue Bridge, Indian Wells, California	PROJECT NUMBER IR702
	BORING RECORD LEGEND #1	

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
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 with 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 Names
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 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 D1557-12)
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	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [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])
#200	Percent Passing #4, #200 sieves (ASTM D1140)

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

WATER LEVEL SYMBOLS

- ▽ First Water Level Reading (during drilling)
- ▼ Static Water Level Reading (after drilling, date)

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

DEFINITIONS FOR CHANGE IN MATERIAL

Term	Definition	Symbol
Material Change	Change in material is observed in the sample or core, and the location of change can be accurately measured.	———
Estimated Material Change	Change in material cannot be accurately located because either the change is gradational or because of limitations in the drilling/sampling methods used.	- - - - - - - - -
Soil/Rock Boundary	Material changes from soil characteristics to rock characteristics.	

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER A-1B	
PROJECT NAME Miles Avenue Bridge, Indian Wells, California	PROJECT NUMBER IR702	

BORING RECORD LEGEND #2

CONSISTENCY OF COHESIVE SOILS

Descriptor	Shear Strength (tsf)	Pocket Penetrometer, PP Measurement (tsf)	Torvane, TV. Measurement (tsf)	Vane Shear, VS. Measurement (tsf)
Very Soft	< 0.12	< 0.25	< 0.12	< 0.12
Soft	0.12 - 0.25	0.25 - 0.50	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.50	0.50 - 1.0	0.25 - 0.50	0.25 - 0.50
Stiff	0.50 - 1.0	1.0 - 2.0	0.50 - 1.0	0.50 - 1.0
Very Stiff	1.0 - 2.0	2.0 - 4.0	1.0 - 2.0	1.0 - 2.0
Hard	> 2.0	> 4.0	> 2.0	> 2.0

APPARENT DENSITY OF COHESIONLESS SOILS

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

MOISTURE

Descriptor	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR 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%

PARTICLE SIZE

Descriptor		Size (in)
Boulder		> 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		< 1/300

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.

CONSISTENCY OF COHESIVE SOILS VS. N_{60}

Description	SPT N_{60} (blows / foot)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	> 30

Ref: Peck, Hansen, and Thornburn, 1974, "Foundation Engineering", Second Edition

Note: Only to be used (with caution) when pocket penetrometer or other data on undrained shear strength are unavailable. Not allowed by Caltrans Soil and Rock Logging and Classificaton Manual, 2010

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.

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010), with the exception of consistency of cohesive soils vs. N_{60}.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER A-1C
	PROJECT NAME Miles Avenue Bridge, Indian Wells, California	PROJECT NUMBER IR702
	BORING RECORD LEGEND #3	

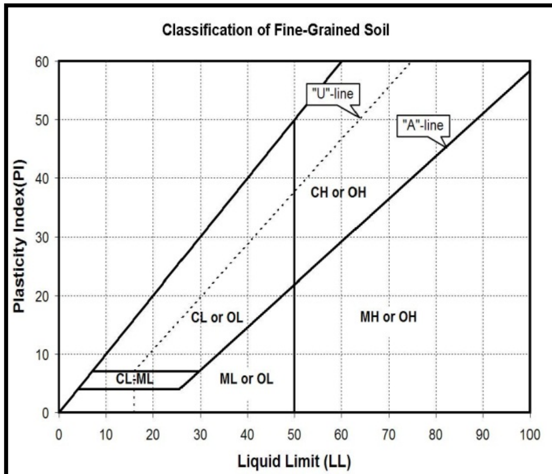
CLASSIFICATION OF INORGANIC FINE GRAINED SOILS (Soils with ≥50% finer than No. 200 Sieve)

GROUP SYMBOL				GROUP NAME
CL	<30% plus No. 200	<15% plus No. 200		Lean clay
		15-25% plus No. 200	%sand ≥ %gravel	Lean clay with sand
			%sand < %gravel	Lean clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy lean clay
			≥15% gravel	Sandy lean clay with gravel
		% sand < % gravel	<15% sand	Gravelly lean clay
			≥15% sand	Gravelly lean clay with sand
ML	<30% plus No. 200	<15% plus No. 200		Silt
		15-25% plus No. 200	%sand ≥ %gravel	Silt with sand
			%sand < %gravel	Silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy silt
			≥15% gravel	Sandy silt with gravel
		% sand < % gravel	<15% sand	Gravelly silt
			≥15% sand	Gravelly silt with sand
CH	<30% plus No. 200	<15% plus No. 200		Fat clay
		15-25% plus No. 200	%sand ≥ %gravel	Fat clay with sand
			%sand < %gravel	Fat clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy fat clay
			≥15% gravel	Sandy fat clay with gravel
		% sand < % gravel	<15% sand	Gravelly fat clay
			≥15% sand	Gravelly fat clay with sand
MH	<30% plus No. 200	<15% plus No. 200		Elastic silt
		15-25% plus No. 200	%sand ≥ %gravel	Elastic silt with sand
			%sand < %gravel	Elastic silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy elastic silt
			≥15% gravel	Sandy elastic silt with gravel
		% sand < % gravel	<15% sand	Gravelly elastic silt
			≥15% sand	Gravelly elastic silt with sand

Reference:
ASTM D 2487 and 2488

Laboratory Classification of Clay and Silt

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).



CL: LL<50; above A-Line.

CH: LL≥50; above A-Line.

ML: LL<50; below A-Line, or PI<4, or Non-Plastic

MH: LL≥50; below A-Line.

CL-ML: above A-Line and PI=4 to 7

CL/CH, ML/MH: at or near LL=50

ML/CL, MH/CH: at or near the A-Line

Field Identification of Clays and Silts

Group Symbol	Dry Strength	Dilatancy	Toughness	Plasticity
ML	None to low	Slow to rapid	Low or thread cannot be formed	Low to nonplastic
CL	Medium to high	None to slow	Medium	Medium
MH	Low to medium	None to slow	Low to medium	Low to medium
CH	High to very high	None	High	High

GROUP DELTA

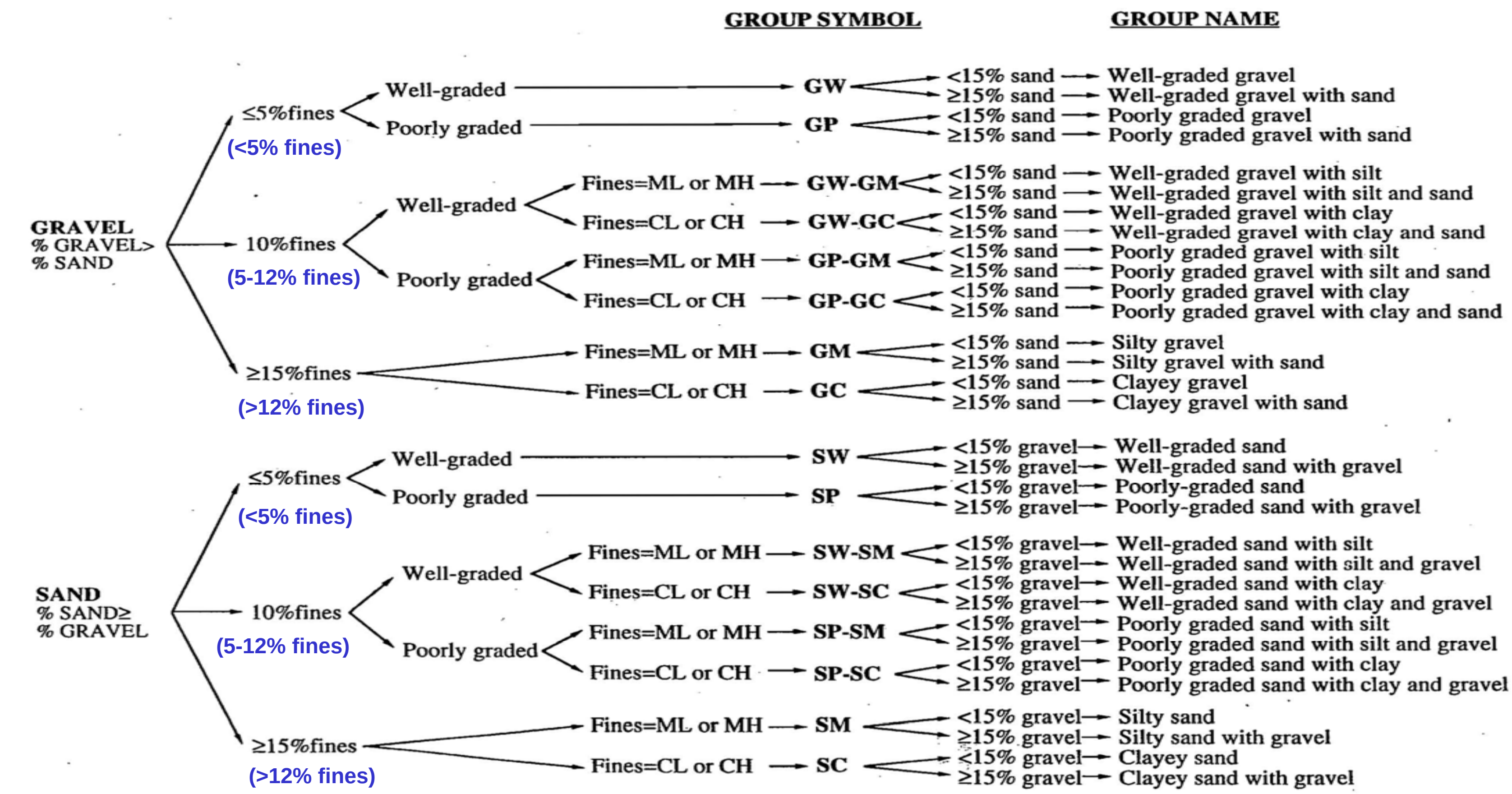
GDC Project No. IR702

Miles Avenue Bridge, Indian Wells, California

KEY FOR SOIL CLASSIFICATION #1

Figure A-1D

CLASSIFICATION OF COARSE-GRAINED SOILS (Soils with <50% "fines" passing No. 200 Sieve)



Reference:

ASTM D 2487 and 2488

Note: Values estimated to nearest 5% to be used for visual identification, values in parentheses to be used for classification when based on laboratory grain size data.

Granular Soil Gradation Parameters

Coefficient of Uniformity: $C_u = D_{60}/D_{10}$

Coefficient of Curvature: $Cc = D_{30}^2 / (D_{60} \times D_{10})$

D_{10} = 10% of soil is finer than this diameter

D_{30} = 30% of soil is finer than this diameter

D_{60} = 60% of soil is finer than this diameter

Group Symbol	Gradation or Plasticity Requirement
SW	$C_u > 6$ and $1 \leq C_c \leq 3$
GW	$C_u > 4$ and $1 \leq C_c \leq 3$
GP or SP	Clean gravel or sand not meeting requirement for SW or GW
SM or GM	Non-plastic fines or below A-Line or PI<4
SC or GC	Plastic fines or above A-Line and PI>7

GDC Project No. IR702

Miles Avenue Bridge, Indian Wells, California

KEY FOR SOIL CLASSIFICATION #2

Figure A-1E

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
1024 West Workman Avenue	IR739	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
1024 West Workman Ave, West Covina	2/28/2020	2/28/2020	1 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Drilling/Wong	CME85	Hollow Stem Auger	Y.G.	K.R.

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Automatic (140 lbs, 30 inch)	62.6%	8	51.5	402	∇ NE / *na* DURING DRILLING; ▼ / *na* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, MC (2.4"), SPT (1.4")	$N_{60}^{*} = 1.043N_{SPT} = 0.696N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS
	400	B-1								CR
5		R-2	3, 7, 12	19	3.5	98.4	44			PA
	395									
10		S-3	9, 9, 9	18						
	390									
15		R-4	3, 3, 10	13	12.7	102.4		40:21:19	2.5	
	385									
20		S-5	4, 5, 9	14						
	380									

DRILLING METHOD: Hollow Stem Auger (0 to 25 ft)

DEPTH (feet)	DESCRIPTION AND CLASSIFICATION
0	**NATIVE** SILTY SAND (SM); medium dense; reddish brown; moist; few plant roots; mostly fine to medium SAND; some fines.
5	56% SAND; 44% Fines.
10	Poorly-graded SAND with SILT (SP); medium dense; light reddish brown; moist; mostly fine SAND; little fines; little fine to coarse subangular to subrounded GRAVEL; few medium SAND; trace coarse SAND.
15	Lean CLAY with SAND (CL); stiff; dark reddish brown; moist; mostly fines; few fine to medium SAND; few fine to coarse subrounded to rounded GRAVEL; medium plasticity. PP=2.5 tsf
	Moisture increases.
20	SILTY SAND (SM); medium dense; reddish brown; moist; mostly fine to medium SAND; trace coarse SAND; little fines.
	Trace fine subangular to subrounded GRAVEL.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2A

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
1024 West Workman Avenue	IR739	B-1

SITE LOCATION	START	FINISH	SHEET NO.
1024 West Workman Ave, West Covina	2/28/2020	2/28/2020	2 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Drilling/Wong	CME85	Hollow Stem Auger	Y.G.	K.R.

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Automatic (140 lbs, 30 inch)	62.6%	8	51.5	402	∇ NE / *na* DURING DRILLING; ▼ / *na* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, MC (2.4"), SPT (1.4")	$N_{60}^* = 1.043N_{SPT} = 0.696N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		MC	R-6	5 9 10	19	3.6	92.9	16			PA			SILTY SAND (SM); medium dense; light yellowish brown; moist; mostly fine to medium SAND; little fines; trace coarse SAND. 84% SAND; 16% Fines.
	375													
														More coarse SAND. More fine to coarse subrounded GRAVEL. Fewer fines.
30		SPT	S-7	12 11 9	20									Poorly-graded SAND (SP); medium dense; moist; light yellowish brown; mostly fine to medium SAND; little fine to coarse subrounded to rounded GRAVEL; trace fines; trace coarse SAND.
	370													
35		MC	R-8	6 12 16	28	10.0	97.4							SANDY Lean CLAY (CL); stiff; reddish brown; moist; mostly fines; few fine to medium SAND; medium plasticity.
														Poorly-graded SAND (SP); medium dense; yellowish brown; moist; mostly fine to medium SAND; few fines; trace coarse SAND.
	365													
40		SPT	S-9	5 7 10	17									Lean CLAY with SAND (CL); stiff; reddish brown; moist; mostly fines; few fine to medium SAND; medium plasticity.
														SANDY Lean CLAY (CL); little fine to medium SAND.
	360													
45		MC	R-10	28 28 50/3	78/9	1.8								Well-graded SAND with GRAVEL (SW); very dense; light yellowish brown; moist; mostly fine to medium SAN; little fine to coarse subrounded to rounded GRAVEL; few coarse SAND; trace fines.
	355													

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2B

BORING RECORD

PROJECT NAME: 1024 West Workman Avenue		PROJECT NUMBER: IR739	HOLE ID: B-1	
SITE LOCATION: 1024 West Workman Ave, West Covina		START: 2/28/2020	FINISH: 2/28/2020	SHEET NO. 3 of 3
DRILLING COMPANY: ABC Drilling/Wong	DRILL RIG: CME85	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: Y.G.	CHECKED BY: K.R.
HAMMER TYPE (WEIGHT/DROP): Automatic (140 lbs, 30 inch)	HAMMER EFFICIENCY (ERi): 62.6%	BORING DIA. (in): 8	TOTAL DEPTH (ft): 51.5	GROUND ELEV (ft): 402
DEPTH/*ELEV.* GW (ft): ∇ NE / *na* DURING DRILLING; ▼ / *na* AFTER DRILLING				
DRIVE SAMPLER TYPE(S) & SIZE (ID): Bulk, MC (2.4"), SPT (1.4")	NOTES: $N_{60}^{*} = 1.043N_{SPT} = 0.696N_{MC}$			

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			S-11	10 16 12	28									Poorly-graded SAND with SILT (SP-SM); medium dense; reddish brown; moist; mostly fine to medium SAND; few fines; few fine subrounded to rounded GRAVEL; trace coarse SAND.
	350													Hole terminated at 51.5 feet. Groundwater not encountered. Backfilled with Bentonite-Cement Grout.
55														
	345													
60														
	340													
65														
	335													
70														
	330													

BORING RECORD

PROJECT NAME 1024 West Workman Avenue			**PROJECT NUMBER** IR739		**HOLE ID** B-2
SITE LOCATION 1024 West Workman Ave, West Covina		**START** 2/28/2020	**FINISH** 2/28/2020		**SHEET NO.** 1 of 1
DRILLING COMPANY ABC Drilling/Wong	**DRILL RIG** CME85	**DRILLING METHOD** Hollow Stem Auger		**LOGGED BY** Y.G.	**CHECKED BY** K.R.
HAMMER TYPE (WEIGHT/DROP) Automatic (140 lbs, 30 inch)	**HAMMER EFFICIENCY (ERi)** 62.6%	**BORING DIA. (in)** 8	**TOTAL DEPTH (ft)** 21.5	**GROUND ELEV (ft)** 407	**DEPTH/*ELEV.* GW (ft)** ⊻ NE / *na* DURING DRILLING; ▼ / *na* AFTER DRILLING
DRIVE SAMPLER TYPE(S) & SIZE (ID) Bulk, MC (2.4"), SPT (1.4")		**NOTES** $N_{60}* = 1.043N_{SPT} = 0.696N_{MC}$			

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	407													ASPHALT (3") OVER BASE (3").
	405	Bulk	B-1											**FILL** SILTY SAND (SM); medium dense; reddish brown; moist; mostly fine SAND; little fines; little COBBLES (4" - 6"); trace medium to coarse SAND. Fewer fines. More medium to coarse SAND.
5		MC	R-2	7 7 8	15									
6	400													**NATIVE** SILTY SAND (SM); medium dense; reddish brown; moist; mostly fine to medium SAND; little fines; few coarse SAND; few fine subrounded to rounded GRAVEL.
10		MC	R-3	6 5 5	10	5.4	109.2	17			PA			77% SAND; 17% Fines; 6% GRAVEL.
15	395 / 390	MC	R-4	12 18 18	36			5.9			PA			Dense.
16														Poorly-graded SAND with SILT and GRAVEL(SP-SM); dense; light reddish brown; moist; mostly fine SAND; little medium to coarse SAND; little fine subangular to rounded GRAVEL; few fines. 77.3% SAND; 16.8% GRAVEL; 5.9% Fines.
20	385	MC	R-5	7 8 8	16	9.5	105.4							CLAYEY SAND (SC); medium dense; dark reddish brown; moist; mostly fine SAND; little fines; trace medium to coarse SAND; trace fine subangular to rounded GRAVEL.
21.5														Hole terminated at 21.5 feet. Groundwater not encountered. Backfilled with Bentonite-Cement Grout.

GDC_LOG_BORING_2011 IR739.GPJ GDCLOG.GDT 3/12/20

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.

370 Amapola Ave., Suite 212

Torrance, CA 90501

FIGURE

A-3A

EARTHSPECTIVES
250 Goddard
Irvine, California 92618
Phone: (949) 777-1270
Fax: (949) 777-1283

October 29, 2019

ABC Liovin Drilling Inc.
1180 East Burnett Street
Signal Hill, California 90755

Attention: Mr. Ivan Liovin

Dear Mr. Liovin:

SPT Hammer Energy Measurement
Drill Rigs R-1 (CME-85) and R-5 (CME-85)
ES Project No. 190806-365

INTRODUCTION

This letter report summarizes the results of EarthSpectives' (ES) SPT hammer energy measurements performed on October 12, 2019. It provides a description of the test program and the results. Testing was performed on two CME 85 Drill Rigs equipped with Auto Trip hammers.

SPT energy measurements were accomplished using a Pile Driving Analyzer (PDA) system manufactured by Pile Dynamics, Inc. and was conducted in general accordance with ASTM 4945 and 6066 test standards. Results are summarized in Table 1, while more details regarding energy records are provided in Appendix A.

TESTING CONDITIONS

SPT hammer energy measurements were performed on two drill rig/hammer combination that were equipped with an automatic trip hammer. Drill rigs R-1 and R-5 were both CME-85 Rigs. Samplings were performed using NWJ drilling rod.

INSTRUMENTATION

SPT energy measurements were performed by placing a 2 ft instrumented section of drill rod at the top of the drill string between the hammer and the sampling rods. The instruments consist of two sets of accelerometers and strain transducers, mounted on opposite sides of the drill rod, with a view to evaluate normal and eccentric effects. The analyzer acquired and processed the signals during sampling, and provided real-time evaluations of the maximum SPT hammer transferred energy. The raw data were stored directly on a portable field computer for subsequent analysis in the office.

RESULTS

Results from SPT hammer energy measurements are summarized in Tables 1. It shows the Energy Transfer Ratio (ETR) for every sampling depth for the tested drill rig/hammer. 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 fall (140 lb x 30 inches = 4200 lb-in = 0.35 kip-ft).

Plots of the maximum transferred energy, energy transfer ratio, and blow rate is provided as function of depth in Appendix A. Table immediately following the plot also provides the minimum, maximum, and average values at every sampling depth. In general, average ETR value for the tested hammers were 83.5% and 62.6% for Drill Rigs R-1 and R-5, respectively, over all the sampling intervals as shown in Table 1.

TABLE 1 – SUMMARY OF SPT HAMMER ENERGY MEASUREMTS

Drill Rig Number Type and Model	**AVERAGE SPT HAMMER EFFICIENCY (ENERGY TRANSFER RATIO)**			
	Data Set # 1	Data Set # 2	Data Set # 3	Data Set # 4
Drill Rig R-1 CME 85	80.5%	87.5%	84%	82.1%
Drill Rig R-5 CME 85	63.7%	65.1%	61.4%	60.1%

LIMITATIONS

Professional judgments represented in this report are based on evaluations of the technical information gathered, our understanding of the proposed construction, and our general experience in the geotechnical field. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments are rendered while striving to meet the standard of care of our profession at this time.

CLOSURE

We hope the above information satisfies the project needs at this time. Please call if you have any question or need more information.

Sincerely submitted for EarthSpectives,

Hossein Rashidi

Hossein K. Rashidi, PhD, PE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
HOSSEIN K. RASHIDI
No. C 055140
Exp. 6/30/18
CIVIL
STATE OF CALIFORNIA

A-4A

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

B.1 General

The laboratory testing was performed using appropriate American Society for Testing and Materials (ASTM) and Caltrans Test Methods (CTM).

Modified California drive samples, Standard Penetration Test (SPT) drive samples, and bulk samples collected during the field investigation were carefully sealed in the field to prevent moisture loss. The samples of earth materials were then transported to the laboratory for further examination and testing. Tests were performed on selected samples as an aid in classifying the earth materials and to evaluate their physical properties and engineering characteristics. Laboratory testing for this investigation included:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Grain Size Distribution (ASTM D 422) & % Passing #200 Sieve (ASTM D 1140);
- Expansion Index (D 4829);
- Soil Corrosivity:
  - pH (CTM 643);
  - Water-Soluble Sulfate (ASTM D 516);
  - Water-Soluble Chloride (Ion-Specific Probe);
  - Minimum Electrical Resistivity (CTM 643);

A summary of laboratory test results is presented in Table B-1. Brief descriptions of the laboratory testing program and test results are presented below.

B.2 Soil Classification

Earth materials recovered from subsurface explorations were classified in general accordance with Caltrans' "Soil and Rock Logging Classification Manual, 2010". The subsurface soils were classified visually / manually in the field in accordance with the Unified Soil Classification System (USCS) following ASTM D 2488; soil classifications were modified as necessary based on testing in the laboratory in accordance with ASTM D 2487. The details of the soil classification system and boring records presenting the classifications are presented in Appendix A.

B.3 Moisture Content and Dry Unit Weight

The in-situ moisture content of selected bulk, SPT, and Ring samples was determined by oven drying in general accordance with ASTM D 2216. Selected California samples were trimmed flush in the metal rings and wet weight was measured. After drying, the dry weight of each sample was measured, volume and weight of the metal containers was measured, and moisture content and dry density were calculated in general accordance with ASTM D 2216 and D 2937. Results of these tests are presented in Table B-1 and on the boring records in Appendix A.

B.4 Grain Size Distribution and Percent Passing No. 200 Sieve:

Representative samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The percentage of fines (soil passing No. 200 sieve) was determined for selected samples in accordance with ASTM D 1140. For selected samples the washed fraction retained on the No. 200 sieve was then screened on a No. 4 sieve, and the fraction retained on No. 4 was weighed to determine the percentage of gravel. For selected samples, the washed material retained on No. 200 sieve was shaken through a standard stack of sieves in accordance with ASTM D 422 to determine the grain size distribution. For selected samples, the grain size distribution of the fraction finer than No. 200 sieve was determined by Hydrometer Analysis in accordance with ASTM D 422. The results of grain size distribution test are plotted in Figure B-1A of this appendix. The relative proportion (or percentage) by dry weight of gravel (retained on No. 4 sieve), sand (passing No. 4 and retained on No. 200 sieve), and fines (passing No. 200 sieve) are listed on the boring records in Appendix A and summarized in Table B-1.

B.5 Expansion Index

The expansion potential of the site soils was estimated using the Expansion Index Test in accordance with ASTM D 4829. The results of this test are listed in Table B-1.

B.6 Soil Corrosivity

Tests were performed in order to determine corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity and soil pH (Caltrans method 643), water-soluble chlorides (Orion 170A+ Ion), and water-soluble sulfates (ASTM D 516). The test results will be summarized in the final report.

B.7 List of Attached Figures

The following tables and figures are attached and complete this appendix:

List of Tables

Table B-1 Summary of Laboratory Test Results

List of Figures

Figure B-1A through B-1C Grain Size Analysis Test Results

Boring No.	Sample No.	Depth (ft)	Sample Type	USCS Group Symbol	SPT N*60 (blows/ft)	Undrained Shear Strength, Su (tsf)			Moisture Content (%)	Dry Unit Weight (pcf)	Total Unit Wt (pcf)	Atterberg Limits			Grain Size Distribution (%) by dry weight			Other Tests
						Pocket Pen.	Mini Vane	UU Test				LL	PL	PI	Gravel	Sand	Fines	
B-1	B-1	0.0	BULK	SM														CR
B-1	R-2	5.0	MC	SM	19				3.5	98	102				0	56	44	PA
B-1	S-3	10.0	SPT	SP	13													
B-1	R-4	15.0	MC	CL	13	2.5			12.7	102	115	40	21	19				
B-1	S-5	20.0	SPT	SM	10													
B-1	R-6	25.0	MC	SM	19				3.6	93	96				0	84	16	PA
B-1	S-7	30.0	SPT	SP	14													
B-1	R-8	35.0	MC	SP	28				10.0	97	107							
B-1	S-9	40.0	SPT	CL	12													
B-1	R-10	45.0	MC	SW	78/9				1.8									
B-1	S-11	50.0	SPT	SP-SM	20													
B-2	B-1	0.5	BULK	SM														
B-2	R-2	5.0	MC	SM	15													
B-2	R-3	10.0	MC	SM	10				5.4	109	115				6	77	17	PA
B-2	R-4	15.0	MC	SP-SM	36										16.8	77.3	5.9	PA
B-2	R-5	20.0	MC	SC	16				9.5	105	115							

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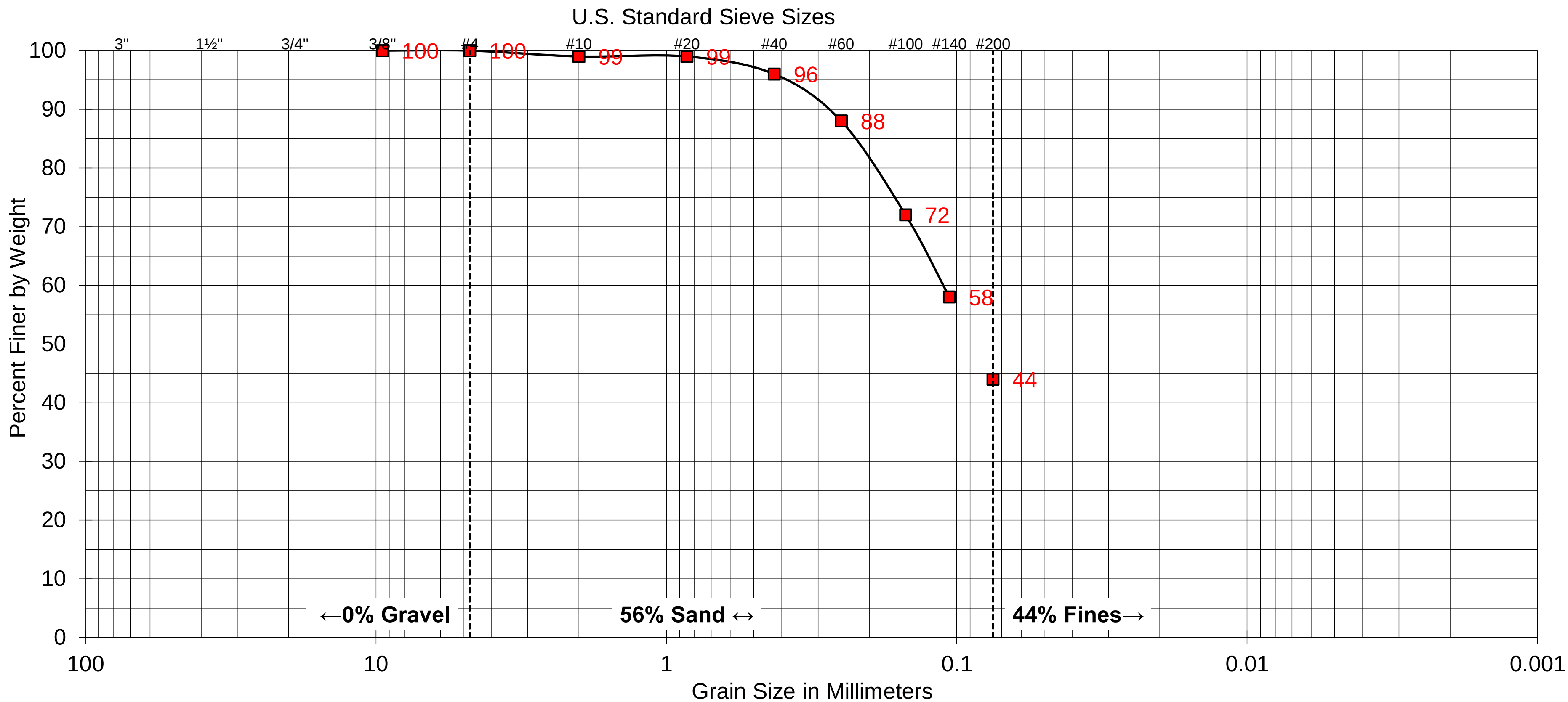
TABLE B-1: Summary of Laboratory Results

Project: 1024 West Workman Avenue

Location: 1024 West Workman Ave, West Covina

Number: IR739



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	B-1
SAMPLE NUMBER:	R-2
SAMPLE DEPTH:	6' - 6.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	0
PLASTIC LIMIT:	0
PLASTICITY INDEX:	0

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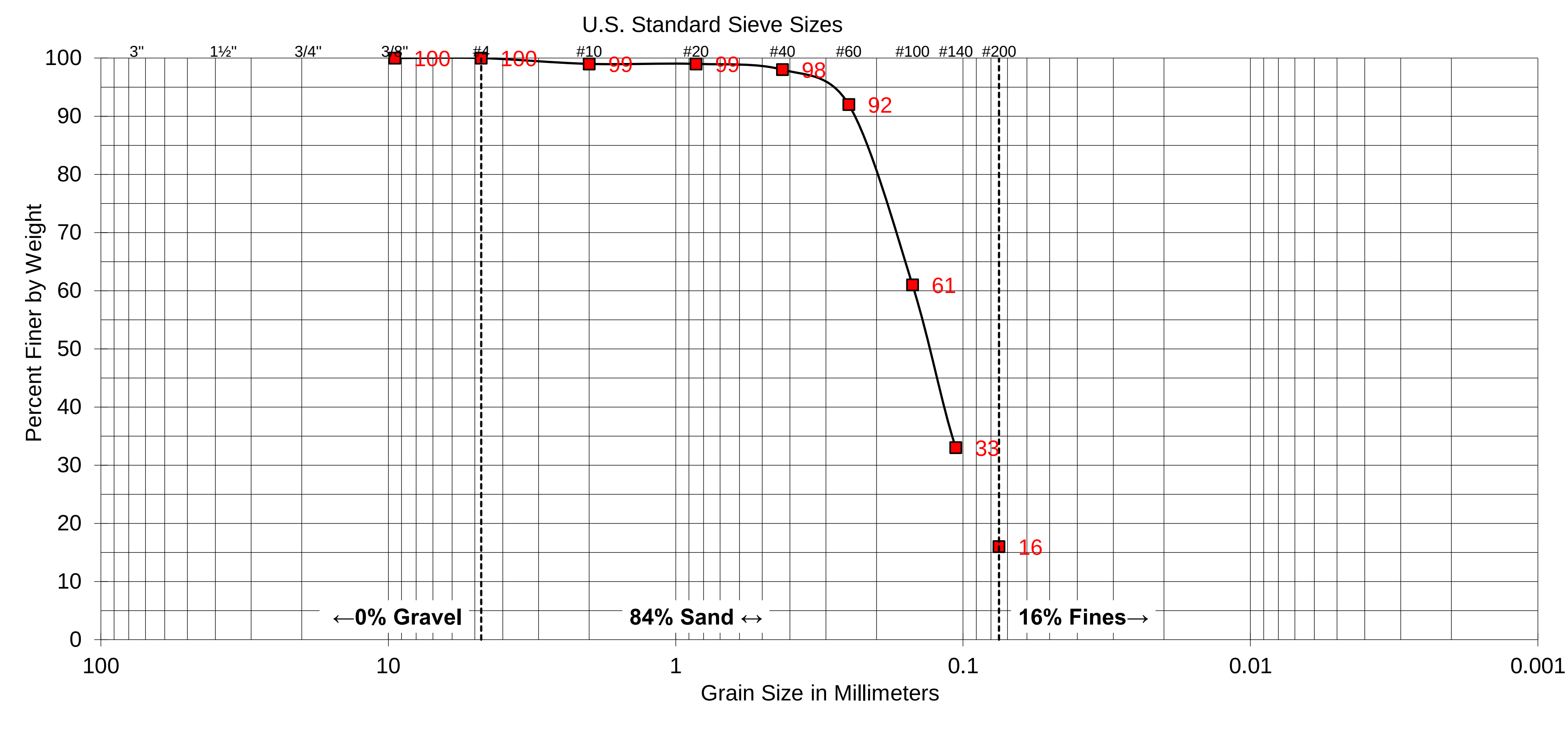
SOIL CLASSIFICATION

Laboratory No. SO5664
Project No. IR739
FIGURE B-1A



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE B-1
SAMPLE NUMBER: R-6
SAMPLE DEPTH: 26' - 26.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	0
PLASTIC LIMIT:	0
PLASTICITY INDEX:	0

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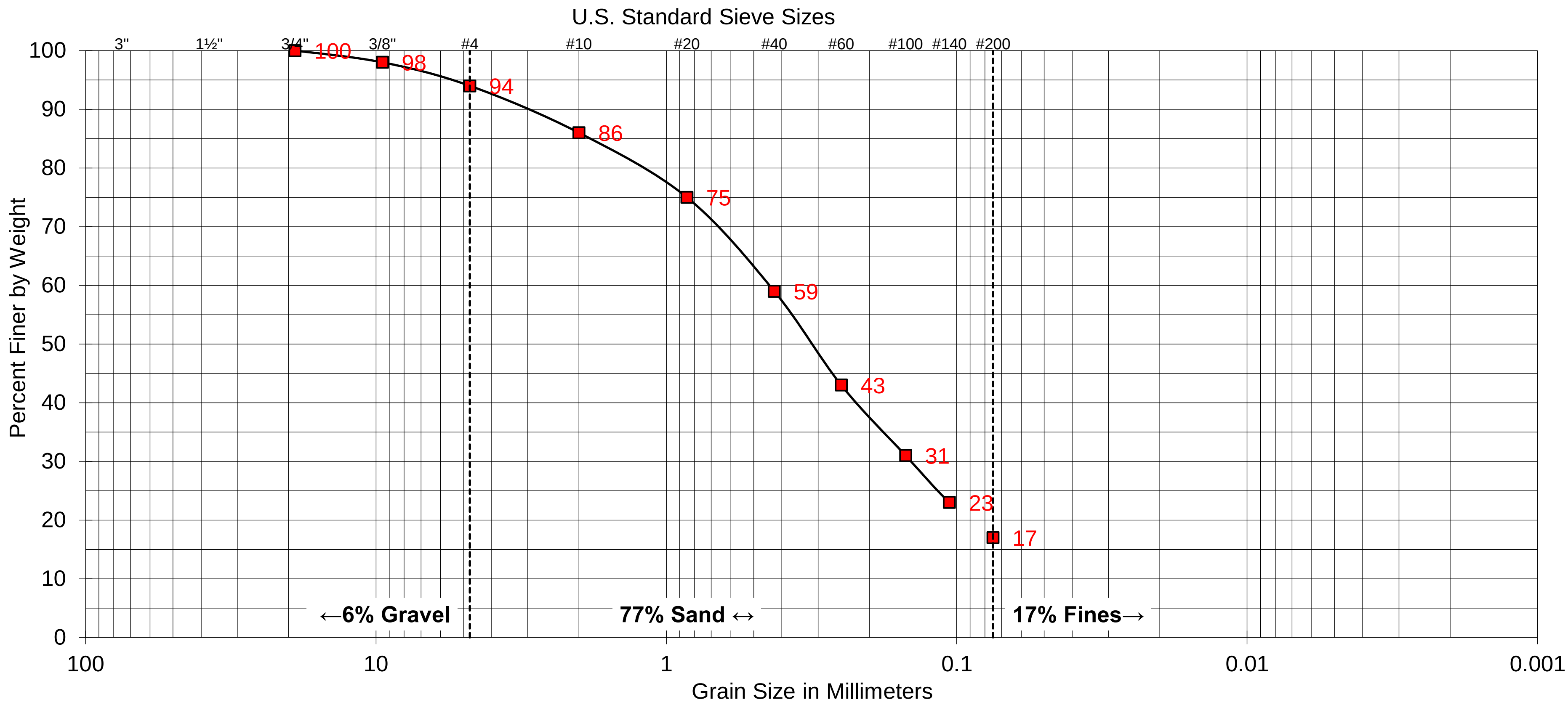
SOIL CLASSIFICATION

Laboratory No. SO5664
Project No. IR739
FIGURE B-1B



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE B-2
SAMPLE NUMBER: R-3
SAMPLE DEPTH: 11' - 11.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	0
PLASTIC LIMIT:	0
PLASTICITY INDEX:	0

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SOIL CLASSIFICATION

Laboratory No. SO5664
Project No. IR739
FIGURE B-1C

APPENDIX C
PERCOLATION TEST RESULTS

Boring Percolation Test Data Sheet

Project Number:	IR739	**Test Hole Number:**	B-1
Project Name:	W 1024 Workman	**Date Excavated:**	28-Feb-20
Soil Description:	SM	**Date Tested:**	28-Feb-20
Liquid Description:	Clean Water	**Depth of Boring (ft):**	50
Tested By:	Y Gao	**Diameter of boring (in):**	8
Test Time Interval:	10 minutes	**Diameter of casing (in):**	4
Start Time for Pre-Soak:	12:10 PM	**Length of perforated casing (ft):**	5
Start Time for Test:	13:10 pm	**Depth to Initial Water Depth (ft):**	5.5
Screened Interval :	5.0 ft to 10.0 ft bgs		

Percolation Data

Sandy Soil Criteria Test

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level ΔD (in)	Greater than or equal to 6 inches ?
1	1:30 PM	1:55 PM	25	223.2	310.8	87.6	Yes
2	11:30 AM	11:55 AM	25	168.0	235.2	67.2	Yes

Deep Percolation Test

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (ft)	Final Depth of Water (ft)	Change in Water Level ΔD (in)	Percolation Rate (in/hr)
1	1:53 PM	2:03 PM	10	5.50	5.98	5.7	0.29
2	2:04 PM	2:14 PM	10	5.50	5.60	1.2	0.06
3	2:14 PM	2:24 PM	10	5.50	5.77	3.2	0.16
4	2:26 PM	2:36 PM	10	5.50	5.66	1.9	0.10
5	2:39 PM	2:49 PM	10	5.50	5.65	1.8	0.09
6	2:50 PM	3:00 PM	10	5.50	5.67	2.0	0.10
7	3:03 PM	3:13 PM	10	5.40	5.56	1.9	0.10
8	3:14 PM	3:24 PM	10	5.52	5.69	2.0	0.10

Infiltration Rate, I = **0.1** **in/hr**

Design Infiltration Rate

Project Number:	IR739	Infiltration Number:	B-1
Project Name:	1024 West Workman Ave, West Covina		

Diameter of Boring (in):	8
Diameter of Casing (in):	4
Depth of Casing Above Ground (ft):	0
Depth of Boring (ft):	50
Bentonite Plug at bottom of test section?:	Yes
Length of Test Section (ft):	5
Standard Time Interval Between Readings (min):	10

Average Water Drop (in): **2**

Volume of Water Dischaged (in^3): **25.13** in^3

Surface Area of Test Section (in^2): **1507.96** in^2

Raw Percolation Rate (in/hr): 0.10 in/hr

Reduction Factors

Boring Percolation (RFt=2)	RF_t=	2
Site variability, number of tests, and thoroughness of subsurface investigation (RFv= 1 to 3)	RF_v=	1
Long-term siltation, plugging and maintenance (RFs= 1 to 3):	RF_s=	1
Total Reduction Factor, RF = RFt x RFv x RFs	RF=	2

Design Infiltration Rate = Raw Percolation Rate / RF

Design Infiltration Rate (in/hr): 0.05 in/hr

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Calculation method taken from the "Administrative Manual County of Los Angeles Department of Public Works Geotechnical and Materials Engineering Division" (GS200.2 6/30/17)