GEOTECHNICAL INVESTIGATION REPORT
MAIN LIBRARY RENOVATIONS
INGLEWOOD CIVIC CENTER
1 W MANCHESTER BOULEVARD
INGLEWOOD, CALIFORNIA 90301

Prepared for:

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Project No. LA1653
October 30, 2024

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October 30, 2024
GD Project No. LA1653

Attention: Ms. Melody Tang
Senior Project Architect

Subject: Geotechnical Investigation Report
Main Library Renovations
Inglewood Civic Center
1 W Manchester Boulevard
Marina Del Rey, California 90292

Dear Ms. Tang,

Group Delta is pleased to submit this geotechnical investigation report in support of the planned improvements at the Main Library building of the Inglewood Civic Center in Inglewood, California. Our scope of work was conducted in general accordance with our proposal dated April 24, 2024.

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

Sincerely,

GROUP DELTA CONSULTANTS, INC.

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation by Group Delta Consultants, Inc. (Group Delta) to support the renovations at the Main Library building of the Inglewood Civic Center in Inglewood, California. The project site location is shown in the vicinity map in Figure 1, and the exploration locations are shown in Figure 2.

The purpose of this report is to provide geotechnical information to support the planned improvements for the project. This report provides interpretations of the geologic and geotechnical conditions observed and recommendations to support the planned improvements. Group Delta developed the recommendations from reviewing the previous studies referenced in this report, recent supplemental subsurface exploration, geologic and geotechnical engineering interpretation and analyses, and our previous experience with similar geologic conditions.

1.1 Project Description and Proposed Improvements

The project site includes the Main Library building at the southeastern side of the Inglewood Civic Center at 101 W Manchester Boulevard in Inglewood, California. The Main Library is bounded by Manchester Boulevard to the south, W Queen Street to the north, a grassy park area to the east, and parking lots to the west. The Main Library building is bordered by existing parking lots on the north, east and west, and landscaping and a walking path on the south.

The Main Library Building has four-stories above grade, and an additional mechanical penthouse at roof level. The first level shares a larger floor with an outdoor plaza area and lecture hall. The lecture hall has its own structural system that is different from the main library building portion, but they share the common first level diaphragm above grade. The library is supported by shallow spread footings.

We understand that a renovation and seismic upgrade project is currently underway led by LPA, the project architect at the Main Library building. Group Delta previously performed a geotechnical investigation in support of the seismic retrofit (Group Delta, 2021). The proposed new renovations to the Main Library require a supplemental geotechnical investigation to support the design.

The proposed improvements at the Main Library include a new elevator along the northeast side of the building, a relocated stair on the north side, new fencing, and locally some new pavements. The elevator will require mechanical equipment and an elevator pit, and the proposed location is immediately adjacent to an existing perimeter column. We understand from the project Structural Engineer that a portion of the existing footing will be cut to construct the elevator pit.

The current planned construction sequence includes enlarging the existing footing, combined with underpinning, demolishing the portion of the footing that needs to be removed and excavate for the elevator pit, and construct a new wall and footing for the elevator pit. Where the new elevator is located is adjacent to an existing exterior column and footing for the library, the footing will need to be cut to accommodate the construction of the elevator pit. Figure 3 shows the foundation plan with the location of the proposed elevator pit relative to the existing foundations.

Along the north side, the new relocated stair will require two lightly loaded foundations to be constructed that are directly adjacent to existing foundations. There is also an existing 12-inch diameter storm drain running in between the proposed footings, with the bottom of the pipe located about 2 feet below existing grade. Figure 3 shows the proposed new footings for the stair.

We understand from the record drawings for the Main Library that the existing foundations were designed for an allowable bearing pressure of 6,000 pounds per square foot (psf), with increases for depth and width up to a maximum allowable bearing capacity of 9,000 psf.

1.2 Scope of Services

Group Delta provided the following scope of services:

- Review of available background information, including existing geotechnical reports prepared by AMEC (2017) and their legacy company, LeRoy Crandall and Associates, as-built plans, and documents pertaining to the site conditions. Figure 2 shows the approximate locations of the relevant historical borings at the site. Appendix A contains relevant Previous Boring Records and pertinent laboratory test data.
- A geotechnical field investigation to obtain supplemental geotechnical data consisting of two hollow stem auger borings to a maximum depth of about 10 feet below ground surface. Figure 2 shows the approximate locations of these explorations. Appendix B provides current field exploration results.
- Laboratory testing on samples collected in the borings. Appendix C provides current laboratory test results.
- Engineering analysis of the field and laboratory data to develop geotechnical parameters for design of the proposed new improvements.
- Preparation of this report with our findings, conclusions and recommendations.

2.0 GEOTECHNICAL INVESTIGATION

2.1 Field Investigation

The subsurface conditions in the area of the proposed improvements were explored by drilling two (2) hollow stem auger borings to the maximum depths of about 10 feet below the existing ground surface at the locations shown in Figure 2.

The explorations were 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 were collected from soil cuttings from the entire length of the boring. Standard Penetration Test samples were taken in the borings at a 5-foot depth, and California Modified Split Spoon samples were taken in the borings at a depth of 8.5 feet. The soil samples were taken to our laboratory for further visual examination and testing. Borings were backfilled with Portland Cement grout and the surface was repaired with rapid-set concrete patch dyed black upon completion of the borings. Details of our field exploration program, including the boring logs, are presented in Appendix B.

2.2 Laboratory Testing Program

A laboratory testing program was performed on selected soil samples collected during our field investigation. The purpose of the laboratory tests is to classify soil samples and evaluate their static physical properties and engineering characteristics. Laboratory testing performed includes the following:

- Percent Passing No. 200 Sieve
- Atterberg Limits
- Direct Shear
- One-Dimensional Consolidation
- Soil Expansion Index
- Resistance R-Value
- Soil Corrosivity

Laboratory test results are included in Appendix C of this report.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

3.1 Geologic and Seismic Setting

Regionally, the site is located within the seismically active Los Angeles Basin area of the southern California Peninsular Ranges geomorphic province. The Peninsular Ranges are characterized by a series of northwest trending mountain ranges separated by valleys, with a coastal plain of subdued landforms. The Los Angeles Basin is filled with sediments thousands of feet thick, structurally influenced by thrusting fault blocks and strike-slip faults dividing the basin into northwest trending valleys and ridges.

Numerous faults are located in close proximity to the site which are sources of strong ground shaking, ground deformation, and surface fault rupture. The State of California define active faults as Holocene-active faults that have ruptured in the last approximately 11,000 years. The closest active fault is the Newport-Inglewood fault zone (NIFZ), which is mapped as crossing the Civic Center property.

3.2 Local Geology

Locally, the site is situated within an elevated alluvial plain at the southern edge of Baldwin Hills. Baldwin Hills are the result of faulting along the Newport-Inglewood fault zone. The entire site is underlain by Holocene to Pleistocene-aged Old Alluvial Valley Deposits (Qoa), which locally include dense sand, silty and clay. The geologic conditions in the site vicinity are depicted on the Regional Geologic Map, Figure 4.

3.3 Existing Pavement and Subsurface Conditions

Existing pavement was encountered at the site at both boring locations. The existing asphalt concrete was found to have a thickness of 3 to 4-inches, underlain by a base layer 5 to 8-inches thick. Below the pavements, subsurface soils consisting of interbedded clayey sand (SC), silty sands (SM), and sandy lean clay (CL) were encountered to the maximum explored depth of 10 feet below ground surface.

The R-value test in the clayey sand was 33, while the Expansion Index testing indicates the sandier soils have a very low expansion potential (EI of 0). Clayey soils generally have a low to medium expansion potential.

Subsurface conditions encountered during our field investigation were generally consistent with those encountered during LeRoy Crandall and Associate’s 1970 investigation, and with Group Delta’s prior field investigation (Group Delta, 2021). Prior boring logs in the vicinity of the proposed improvements indicate the presence of interbedded silt, sand, and clay. Sands are interpreted to be medium dense to dense, and fine-grained soils such as silt and clay are interpreted to be stiff to very stiff based on penetration resistances and descriptions of the soils encountered in the previous investigations.

AMEC (2017) noted that during the original site development, existing fill was encountered. The original existing fill was removed during construction activities for the entire Civic Center. Some existing fill is still present locally at the site, as wall backfill or pavement subgrade as part of existing site development. Without review of as-built reports of compaction of this fill, these materials are considered undocumented. During our subsurface investigation, any existing fill is undifferentiated from the underlying Old Alluvial Valley Deposits (Qoa).

Subsurface exploration from previous field investigations are presented Appendix A, while current boring exploration records are presented in Appendix B.

3.4 Groundwater

No groundwater was encountered in our geotechnical explorations nor in previous investigations in the 1960's and 1970's. Our previous experience in the site vicinity suggests that the local groundwater table is relatively deep.

4.0 GEOLOGIC HAZARDS

The publicly available USGS and CGS resources along with the Seismic Hazard Zone Report for the Inglewood 7.5-Minute Quadrangle (CDMG, 1998) and the City of Inglewood, 1995, Safety Element were reviewed for the evaluation of geologic hazards at the project site. A summary of our limited geologic hazard evaluation is presented in this section.

4.1 Strong Ground Motion

The primary geologic hazard at the site is the potential for strong ground shaking due to nearby or distant seismic event from one of the numerous faults in the vicinity of the site (Figure 5). The site could be subject to moderate to strong ground shaking from nearby or more distant, large magnitude earthquakes occurring during the expected continued life span of these buildings. This hazard is being managed through the seismic retrofit process following structural evaluation in accordance with ASCE 41-17. Seismic parameters are provided in the *Discussion and Recommendations* section of this report.

4.2 Earthquake Surface Fault-Rupture Hazard

Surface rupture is the result of movement on an active fault reaching the ground surface. The site is located within an Alquist-Priolo Earthquake Fault Zone associated with the Newport-Inglewood fault. The Newport-Inglewood fault zone (NIFZ) is a zone of discontinuities, folds and faults which stretches across the Los Angeles basin in a northwest/southeastern orientation from Beverly Hills to Newport Beach. The mapped location of the fault traverses the project site through the Police Department and parking structure to the south of the Police Department (Figure 6). Therefore, there is a potential for surface rupture at the site from movement of Newport-Inglewood fault reaching the ground surface.

An Alquist-Priolo Earthquake Fault Zone requires a special study for new structures, which is a site-specific surface fault rupture investigation. The requirements of this special study are governed by the California Public Resources Code (CPRC), Division 2, Chapter 7.5.

The Civic Center buildings have been in existence prior to May 4, 1975, and the new alterations only include seismic retrofitting of these structures, the proposed developments are exempt from site-specific fault rupture hazard investigation (Section 2621.6 of Chapter 7.5 of the CPRC).

Since the buildings were constructed prior to 1975 and the planned seismic retrofit does not exceed 50 percent of the value of the structure, it meets the exception stated in Section 2621.7, subdivision (b). In addition, the buildings are reinforced concrete moment resisting frame buildings undergoing a voluntary seismic retrofit, so they also meet the exception stated in Section 2621.7, subdivision (e). Therefore, a special study for potential surface fault rupture is not required for this current project.

4.3 Liquefaction and Seismic Settlement

Liquefaction is the sudden loss of soil shear strength within saturated, loose to medium dense, sands and non-plastic silts. Liquefaction is caused by the build-up of pore water pressure during strong ground shaking from an earthquake. Secondary effects of liquefaction are sand boils, settlement and instabilities within sloping ground that occur as lateral spreading, seismic deformation and flow sliding. Seismic shaking can also cause seismic compaction, which is the densification of loose to medium dense granular soils that are above groundwater. Loose unsaturated coarse-grained soils were generally not observed at the site.

Considering the dense condition of the soils underlying the site and the relatively deep groundwater, the potential for earthquake induced ground failure due to soil liquefaction and seismic compaction should be very low.

4.4 Landslides and Slope Stability

The site and site vicinity are relatively flat. Evidence of ancient landslides or slope instabilities were not observed during our literature review, site reconnaissance, or subsurface exploration. Based on our understanding of the current project, landslides and slope stability are not considered hazards for the site.

4.5 Tsunamis, Seiches, and Inundation

The site is located about 5.3 miles from the Pacific Ocean, with an approximate Elevation of 115 feet above Mean Sea Level (MSL). No bodies of water are located above or near the site. The site is not located within a Tsunami Inundation Area as mapped by the California Emergency Management Agency (2009). Therefore, the potential for a tsunami, seiche, and inundation is considered very low for this site.

4.6 Flooding

The site is located within a densely populated and developed area. Storm water is largely controlled by engineered drainage. The site is outside of the 100-year flood plain and it is located in an area of minimal flood hazard (Federal Emergency Management Agency, 2020). Therefore, the risk of flooding is considered very low.

4.7 Compressible Soils

In general, the site is underlain by Holocene to Pleistocene-aged alluvial deposits that are not considered compressible and provide good support for any existing or new foundations. However, any locally encountered undocumented fill is considered compressible due to the variable physical characteristics and apparent densities that can stem from uncontrolled placement and compaction of the fill. These soils are typically removed entirely and recompacted where the new footings or other improvements are placed.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

The existing Main Library is supported by shallow spread and continuous footings that are supported within the underlying Pleistocene-aged Old Alluvial Valley Deposits (Qoa). These soils are generally dense and firm, with relatively low compressibility and high shear strength, and are competent for support of foundations and other surface improvements.

Existing undocumented fill, where encountered, is expected to be relatively thin (less than 3 feet) in the vicinity of the Main Library as part of grading for pavements or other site improvements. New foundations, slabs, or walkways should not be supported directly on these undocumented fill soils without remedial grading.

5.2 Seismic Design

As this project is part of the seismic retrofit of the Main Library, we understand it will be evaluated in accordance with ASCE 41-17. Site-specific acceleration response spectra were developed for the project and our documented in our previous report (Group Delta, 2021). The site-specific seismic design parameters for the BSE 2E and BSE-1E seismic hazard levels are presented in the table below.

SITE-SPECIFIC SEISMIC DESIGN ACCELERATION PARAMETERS

Design Parameters	Site-Specific Seismic Design Parameters
Site Class	C
BSE-2E S_{XS} (g)	1.603
BSE-2E S_{X1} (g)	0.907
BSE-1E S_{XS} (g)	0.865
BSE-1E S_{X1} (g)	0.428

5.3 Demolition

Prior to the start of earthwork, demolition will be required to remove existing improvements that include but not limited to existing pavements, and other near-surface improvements. Any voids created from the demolition should be properly backfilled following the recommendations of Section 5.5 below to the limits determined by the project geotechnical engineer. 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 any existing utilities, as appropriate.

5.4 Removals

Any unsuitable soils, such as undocumented fill or expansive soils (EI > 50), should be removed and recompacted with properly compacted fill, to the limits directed by the project geotechnical engineer. The recompaction should extend for minimum a horizontal distance of 2 feet outside the spread footings.

5.5 Excavations

Vertical cuts for the temporary excavations may be used provided that adjacent underground utilities and structures are adequately supported. The sides of the temporary excavations made in fine grained soils should stand with vertical cuts to a max depth of 4 feet. Temporary excavation greater than four (4) feet may be constructed at an angle of 1H:1V (horizontal to vertical ratio), or flatter or shoring should be used.

Surcharge loads from equipment or stockpiled material should be kept behind the top of the temporary excavations a horizontal distance of at least twice the depth of the excavation. Surface drainage should be controlled and prevented from running down the slope face. Ponding water should not be allowed within or near the excavation. Even with the implementation of the above recommendations, some sloughing of slopes and unstable soil zones may still occur within temporary excavations, and workmen should be adequately protected. Construction equipment and foot traffic should be kept off excavation slopes to minimize disturbance/ sloughing.

Where there is insufficient room to excavate slopes, or where an existing structure or other improvement requires protection, temporary shoring should be used.

If the excavation is exposed during periods of rainfall, provisions for collection of the runoff should be made. All surface drainage should be controlled and prevented from running down into the excavation. Ponding water should not be allowed within the excavation.

The excavations should be readily accomplished using conventional heavy construction equipment. All excavation slopes and shoring systems should meet minimum requirements of the Occupational Safety and Health (OSHA) Standards. Maintaining safe and stable slopes on excavations is the responsibility of the contractor and will depend on the nature of the soils encountered and the contractor's method of excavation.

Excavations during construction should be carried out in such a manner that failure or ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsurface materials, height of the excavation, and length of time the excavation remains unsupported and exposed to equipment, vibrations, rainfall, and desiccation. The contractor should perform any additional studies deemed necessary to supplement the information contained in this report for planning and executing their excavation plan.

5.6 Earthwork and Grading

All grading should conform to the County of Los Angeles and City of Inglewood requirements, and the general grading recommendations outlined below.

1. The grading contractor is responsible for notifying the project geotechnical engineer of a pre-grading meeting prior to the start of grading operations and anytime that the operations are resumed after an interruption.

2. Prior to the start of earthwork existing improvements will require demolition. Existing utilities should be removed, relocated or protected, as appropriate.

3. Any unsuitable soils encountered during excavation should be removed and backfilled with properly compacted fill, as directed by the project geotechnical engineer. The actual limits for removals should be determined by the project geotechnical engineer depending on the actual conditions encountered.

4. The bottom of the completed excavation should be observed and evaluated by the project geotechnical engineer, as it is proof rolled with heavy equipment. Any loose or unstable soils should be over-excavated to the limits determined by the project geotechnical engineer.

5. Any fill or backfill placed under structures or pavement and any backfill placed adjacent to buried walls is "structural fill." New fill should be predominantly sandy soil, free of expansive clay, rock greater than 3 inches in maximum size, debris, and other deleterious materials. All structural fill and backfill should be placed in maximum 8-inch lifts, moisture conditioned to optimum moisture, and compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557.

6. All earthwork and grading should be performed under the observation of the project geotechnical engineer, including approval of the bottom of excavations, removal of existing fill, foundation excavations, and placement of fill and backfill.

7. Compaction testing of the fill soils shall be performed at the discretion of the project geotechnical engineer. Testing should be performed for approximately every 2 feet in fill thickness or 2,000 cubic yards of fill placed, whichever occurs first. If specified compaction is not achieved, additional compactive effort, moisture conditioning, and/or removal and recompaction of the fill soils will be required.

8. All materials used for asphalt, concrete, and base shall conform to the "Green Book" and shall be compacted to at least 95 percent relative compaction.

5.7 Foundation Recommendations

New shallow foundations planned as part of the retrofit should be embedded within the undisturbed native alluvial deposits for consistency with the other existing foundations supporting the structures.

5.7.1 Bearing Capacity

New shallow foundations planned as part of the improvements should be embedded within the undisturbed Old Alluvial Valley Deposit soils for consistency with other existing foundations supporting the structures. All new foundations should be at least 24-inches below the lowest adjacent surface grade and have a minimum width of 24 inches.

New footings that are constructed adjacent to (or in close proximity to) existing foundations may use a net allowable bearing capacity of 2,000 pounds per square foot (psf). This value may be increased by a factor of 3 for ultimate bearing capacity. The net allowable bearing capacity may be increased by one-third for short term seismic or wind loading.

Although the allowable bearing capacity is lower than the existing allowable bearing capacity shown in the record drawings, the reduced value is to reduce the potential for additional settlement of the existing foundations from the new loading. Group Delta should review the bearing capacities proposed to be used to design the new footings prior to final design.

5.7.2 Settlement

We estimate that the total and differential settlement of new foundations should be less than ½-inch and ¼-inch, respectively. Existing footings directly adjacent to new foundations should not experience more than ¼-inch of settlement.

5.7.3 Lateral Capacity

Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the soil, and passive pressure from the portion of vertical foundation members embedded into undisturbed Old Alluvial Valley Deposit soils. A coefficient of friction of 0.3 and a passive pressure of 300 psf per foot of embedment may be used. The allowable lateral resistance may be increased by a factor of 1.5 for ultimate lateral resistance.

Note that the planned new footings near the existing 12-inch diameter storm drain (for the relocated stair) should neglect any passive pressure and rely on friction only for lateral resistance.

5.8 Support of Existing Improvements (Underpinning)

An existing column footing will be impacted by the proposed elevator construction. We understand that the footing will need to be partially cut and modified to accommodate the new elevator pit. To avoid large lateral surcharge pressures on the new elevator pit walls and to support the existing column footing, underpinning support will be needed to transmit the vertical loading below the bottom of foundation for the new elevator pit. For foundation underpinning, either helical piles or micropiles may be used to support the existing foundation.

Construction of the helical piles or micropiles may require equipment with lower overhead clearance. Specialty contractors may be contacted for design and construction support for each of these underpinning systems.

5.8.1 Helical Piles

Helical piles often are designed in coordination with specialty contractors, depending on the type, helix diameter, and spacing needed for the project. Either helical piles or driven (displacement) friction piles may be used for underpinning support.

Axial capacity of helical piles should be based on end bearing. An allowable end bearing capacity of 6,000 psf is recommended for preliminary design of helical piles.

An allowable uplift capacity of 1 kip per foot of pile length is recommended for helical piles with an anchor diameter of 12 inches. Uplift capacities can be provided for alternate helix diameters upon request.

5.8.2 Micropiles

The following recommendations for pressure-grouted micropiles can be used:

- A single pressure-grouted micropile can use an allowable axial compressive capacity shown in the table below. These values have a minimum factor of safety of 2 for side friction.

- Micropiles should have a minimum design length of 10 feet, but also have tip elevations below the adjacent subgrade elevation for the elevator pit. An additional allowable axial capacity is provided per foot of additional length beyond 10 feet.
- Piles should be installed with a minimum of 30 inches or 3 pile diameters center-to-center spacing (whichever is greater) to use the full axial capacity without reduction.
- The allowable uplift capacity may be taken as 70-percent of the allowable axial capacities provided in the table below.
- Micropiles designed in accordance with these recommendations are anticipated to have minimal settlements under static loading.
- Lateral capacities may be provided, as needed.

PRELIMINARY ALLOWABLE MICROPILE AXIAL CAPACITIES

Micropile Diameter (inches)	Allowable Axial Capacity (kips)[1]	Allowable Axial Capacity Per Additional Foot of Length (kips/ft)
6	16	1.5
8	21	2
10	26	2.5

Note 1: Allowable axial capacity provided is for 10-foot length.

5.9 Elevator Pit Walls

Permanent subterranean walls that are restrained from lateral movement, such as the elevator pit walls, may be designed using an at-rest equivalent fluid pressure for static conditions of 55 pound per cubic foot (pcf). For seismic loading, the walls may be designed using an active-plus seismic equivalent fluid pressure under seismic loading of 35 pcf (active) plus a seismic increment of 15 pcf.

Surcharge loads resulting from traffic, live loads, adjacent foundations, or others should be included in the design as appropriate. Surcharge loading may be taken as a uniform lateral earth pressure of 0.4q where 'q' is the surcharge in pounds per square foot. Surcharges at least a distance of the wall height, H, away from the wall may be neglected (outside of a 1H:1V plane extending up from the bottom of the wall foundation).

Based on the distance between existing continuous footing "A" in Figure 3, we recommend that a surcharge from the existing footing be incorporated into the design. We understand that the footing is 18 inches wide, 24 inches below finished floor elevation, and has a dead plus live loading of 2,300 pounds per linear foot (lb/ft) of wall. Based on the distance from the wall, the surcharge distribution shown in Figure 7 can be incorporated into the design.

5.10 Passive Resistance of Existing Pile Foundations (Pole Foundations)

If the existing piles are to be evaluated for lateral loads, the embedded post and poles formula in Section 1807.3 of the California Building Code (CBC, 2022) could be used. The embedment length can be estimated by using Equation 18-1 for non-constrained elements and Equation 18-3 for constrained elements. The lateral soil-bearing pressure of 200 psf/ft can be used for lateral design.

5.11 Exterior Slabs

Exterior slabs and sidewalks subjected to pedestrian traffic and light vehicle loading (e.g. golf carts) should be at least 4 inches thick and supported by properly prepared subgrade. The upper 12 inches below new slabs should be scarified and recompacted to 90 percent of the maximum dry density per ASTM D1557. If any expansive soil is encountered (EI>50), then the upper 2 feet should be removed and replaced with granular, non-expansive soil in accordance with the *Earthwork and Grading* section of this report.

Control joints should be placed on a maximum spacing of 10-foot centers, each way, for slabs, and on 5-foot centers for sidewalks. The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-heigh of the slab.

5.12 Utility Trenches

Excavations for utility trenches should be readily accomplished with conventional excavating equipment. All shoring and excavation should comply with current OSHA regulations and observed by the designated competent person on site.

The bedding for any new sewer and water service pipelines should be a minimum of 4 inches thick and should consist of clean sand, No. 4 concrete aggregate or gravel, and should have a sand equivalent of not less than 30. The pipe zone material, which extends to a level 12 inches above the pipe should consist of sand and should have a sand equivalent of no less than 30, and a maximum rock size of 1 inch. All imported materials should be approved by the project geotechnical engineer before being brought onsite.

Trench zone backfill extends from a level 12 inches above the pipe to finished subgrade. In general, on-site excavated materials are suitable as backfill. Any boulders or cobbles larger than 3 inches in any dimensions, or any organics or other deleterious materials, should be removed before backfilling. We recommend that all backfill should be placed in lifts not exceeding six to eight inches in thickness and be compacted to at least 90 percent of relative compaction as determined by the ASTM D1557. Mechanical compaction will be required to accomplish compaction above the bedding along the entire pipeline alignments. Jetting or flooding of backfill should not be permitted.

In backfill areas, where mechanical compaction of soil backfill is impractical due to space constraints, 2-sack slurry (CLSM) may be substituted for compacted backfill.

5.13 Soil Corrosivity

A representative near surface bulk sample was tested to evaluate corrosion characteristics. results indicate the sample had a pH of 8.01, water-soluble sulfate content of less than 0.01% and soluble chloride content of less than 100 ppm. Based on these results, the site soils do not appear corrosive to concrete.

Results of laboratory electrical resistivity tests indicate a minimum resistivity value of 8,110 ohm-cm for the near-surface soils. To evaluate the corrosion potential of near-surface soils, we used the following correlation between electrical resistivity and corrosion potential:

CORROSION POTENTIAL CRITERIA

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

Based on this data, the onsite near-surface soils tested are considered moderately corrosive for buried metal. All underground metal pipes should consider this corrosion potential. A corrosion expert should be consulted for further evaluation and to develop optimum protection.

5.14 Pavement Design Recommendations

5.14.1 Existing Pavement

Existing pavement sections in the library parking lot were evaluated during our recent subsurface exploration. The following pavement sections were encountered:

- The asphalt pavement section was about 3 to 4 inches in thickness.
- The base thickness was about 5 to 8 inches.

5.14.2 Pavement Design Methodology

The design of pavement section, or the thickness, depends on the R-Value of the subgrade soil and type of traffic load. The R-Value indicates the strength of support provided by the subgrade soil. Higher R-Value indicates higher strength of support provided by the subgrade soil. The traffic load, which can be expressed in terms of Traffic Index (TI), ESAL (Equivalent Single Axle Load), or

ADTT (Average Daily Truck Traffic), represents the amount of vehicular traffic the roadway experiences in a design period. The design period, usually, is 20 years. Traffic Index values of 4 to 5 are generally recommended for car parking and non-truck areas. A traffic index of 6 or 7 may be used for heavier truck areas.

R-Value tests were conducted on subgrade samples collected during the field investigation. The testing was conducted in general accordance with CT301. The test results are presented in Appendix C. The clayey sand sample we tested had R-Value of 33. A design R-value of 20 was adopted for our analyses. We have provided design pavement sections for 20-year design Traffic Indices of 4 to 7, depending on the need for vehicle loading in different areas as determined by the project Civil Engineer.

5.14.3 Asphalt Concrete Pavement Design Sections

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Highway Design Manual for flexible pavement. Based on the Traffic Indices noted above, the following pavement sections would apply.

Traffic Index	Asphalt Section	Base Section (R=20)
4.0	3 Inches	5 Inches
5.0	3 Inches	7 Inches
6.0	3 Inches	11 Inches
7.0	4 Inches	12 Inches

Areas where complete pavement section reconstruction is conducted, the upper 12-inches of subgrade soil should be scarified immediately prior to constructing the new pavements, brought to about optimum moisture content, and compacted to at least 95 percent relative compaction per ASTM D1557. All aggregate base should also be compacted to at least 95 percent relative compaction. Aggregate base should conform to Section 200-2 of the Standard Specifications for Public Works Construction (*SSPWC)*. Asphalt concrete should conform to Section 203-6 of the *SSPWC* or Section 39 of the Caltrans Standard Specifications. We recommend that asphalt concrete be compacted to between 91 and 97 percent of the Rice density per ASTM D2041.

5.14.4 Portland Cement Concrete Pavement Sections

Rigid concrete pavements may be desirable in certain areas where heavy equipment or traffic may induce large pavement loads, such as an access road for emergency vehicles or near trash bin storage locations. Portland Cement Concrete (PCC) pavement design was conducted in accordance with a simplified design procedure (Chapter 4) of the Portland Cement Association. The methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The subgrade soils were assumed to provide "medium" subgrade support and the concrete modulus of rupture at 28-days was assumed to be 600 pounds per square inch (psi). Based on these assumptions, we recommend that the PCC pavement sections at the site consist of 6 inches of concrete placed over 6 inches of compact aggregate base.

Crack control joints should be constructed for all PCC slabs on a maximum spacing of 12-feet, each way. Concrete pavements with concentrated traffic should be reinforced with number 4 bars at 18-inch center-to-center in each direction.

5.14.5 Mill and Overlay Option

The existing pavements have been evaluated through back calculation of the pavement thickness considering an R-Value of 20 that indicates a 20-year design traffic index of about 4.0. A grind and overlay strategy will prolong the life of the pavement and would generally meet the existing condition TIs. Greater TIs can be achieved by raising the grade with additional overlay that could be considered if regrading for drainage is performed. Caltrans recommends that at least 1.8-inches of the existing pavement should remain in-place for a grind and overlay to ensure the milling machine does not loosen the base material and require additional removal and reconstruction, referenced as "digouts."

For those portions of the site where the existing pavement sections appear to have performed well to date, where a lower traffic index is anticipated, or where pavement maintenance is deemed acceptable, the existing asphalt concrete may be ground down and overlaid with at least 2-inches of a new dense-graded hot-mix asphalt concrete (HMA) or 1.5-inches of rubberized hot mix asphalt (RHMA). Note that the overlay may allow the project Civil Engineer to adjust drainage and finish grades in some portions of the site, but thinning the pavement section is not recommended.

Asphalt concrete should conform to Section 203-6 of the *SSPWC* or Section 39 of the Caltrans Standard Specifications. The overlay should be compacted to between 91 and 97 percent of the Rice density per ASTM D2041.

5.15 Site Drainage

Surface drainage during construction should be controlled and directed to appropriate drainage facilities. All surface drainage should be prevented from running down along the face of the excavation. Ponding water should not be allowed within any excavations.

6.0 LIMITATIONS

This investigation 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 the LPA, Inc., 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 for this project, to a high degree, are dependent upon proper quality control of site grading, fill and backfill placement, and pile foundation installation. The recommendations are made contingent on the opportunity for Group Delta to observe the earthwork operations. This firm should be notified of any pertinent changes in the project, or if conditions are encountered in the field, which differ from those described herein. If parties other than Group Delta are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project and must either concur with the recommendations in this report or provide alternate recommendations.

7.0 REFERENCES

Amec Foster Wheeler, (2017), "Report of Geotechnical Consultation Proposed Seismic Evaluation, Inglewood Civic Center Buildings, 1 West Manchester Boulevard, Inglewood, California", For Nabih Youssef and Associates.

American Public Works Association (2018). *Standard Specifications for Public Works Construction, Section 200-2.2*, Untreated Base Materials, Section 400-4, Asphalt Concrete: BNI, 761 p.

American Society for Testing and Materials (2019). *Annual Book of ASTM Standards, Section 4, Construction, Volume 04.08 Soil and Rock (I); Volume 04.09 Soil and Rock (II)*; Geosynthetics, ASTM, West Conshohocken, PA, Compact Disk.

American Society of Civil Engineers (ASCE). (2016). Minimum Design Loads for Buildings and Other Structures, ASCE 7-16.

American Society of Civil Engineers (ASCE). (2017). Seismic Evaluation and Retrofit of Existing Buildings, ASCE 41-17.

California Building Standards Commission (CBSC), (2019). *2022 California Building Code (CBC)*, California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, dated: July 1.

CDMG (1998), Seismic Hazard Zone Report 027, Seismic Hazard Zone Report for The Inglewood 7.5-Minute Quadrangle, Los Angeles County, California.

Caltrans. (2018). *Caltrans Highway Design Manual*.

City of Inglewood, 1995, Safety Element of the General Plan, Adopted July 1995.

Federal Emergency Management Agency (FEMA) (2020), Flood Map Service Center, accessed on March 11, 2020, at https://msc.fema.gov/portal/home

Group Delta, Inc. (2021). Report of *Geotechnical Investigation, City of Inglewood Civic Center Seismic Upgrades, City Hall, Main Library and Police Department Buildings, Inglewood, California.* Project No. LA1474. Dated March 3 (Rev. 1 April 22, 2021).

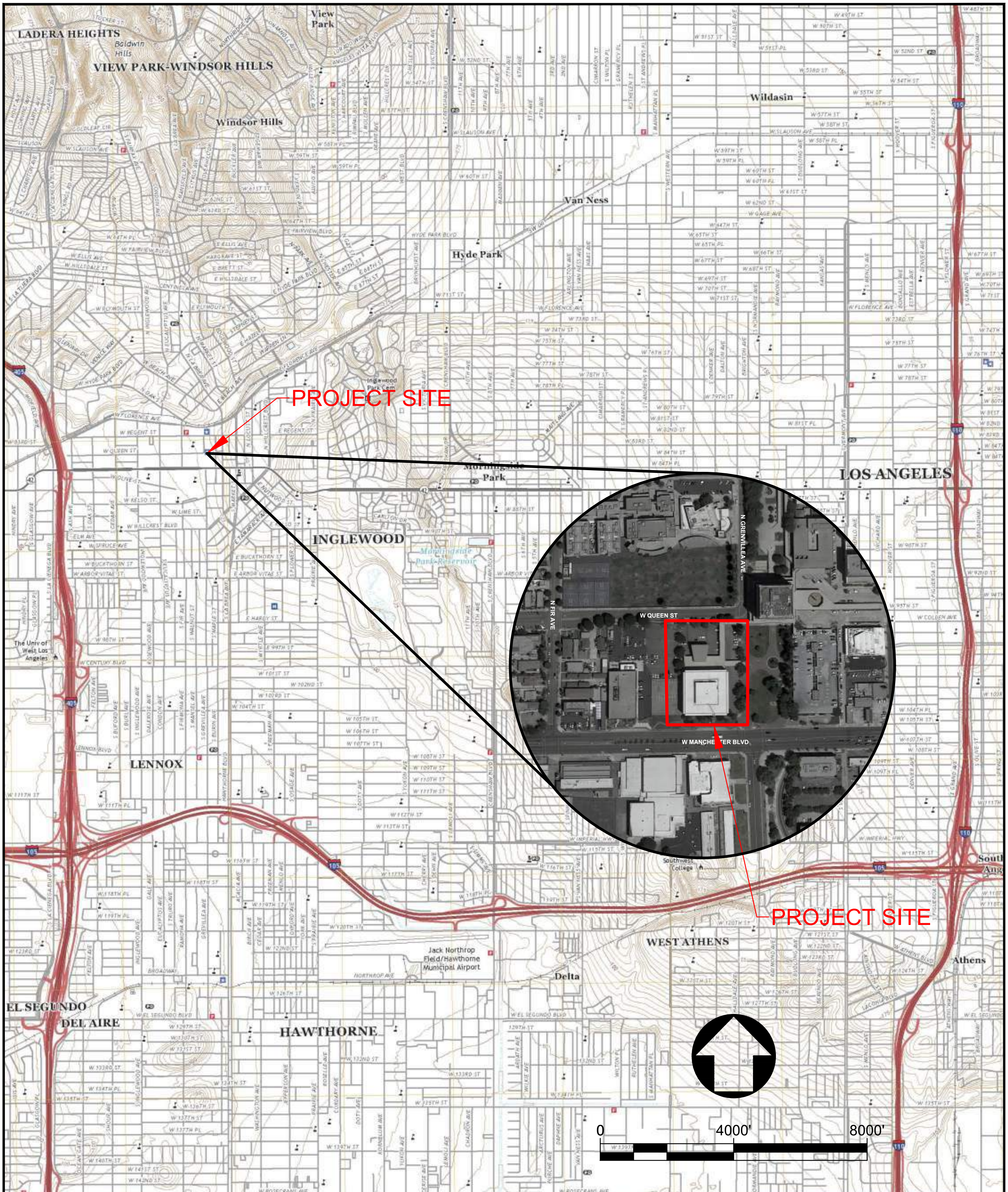
Kennedy, M. P., and Tan, S. S. (2008). *Geologic Map of the San Diego 30'x60' Quadrangle, California*: California Geologic Survey, Scale 1:100,000.

LeRoy Crandall and Associates, (1964), "Report of Foundation Investigation, Proposed Inglewood Civic Center Parking Structure, Grevillea Avenue between Queen Street and Manchester Boulevard, Inglewood, California", For the City of Inglewood.

LeRoy Crandall and Associates, (1970), "Report of Foundation Investigation, Proposed Civic Center, Grevillea Avenue Between Manchester Boulevard and Regent Street, Inglewood, California", For the City of Inglewood.

Portland Cement Association. (1984). *Thickness Design for Concrete Highway and Street Pavement,* EB109P.

FIGURES



REFERENCE: USGS TOPO, INGLEWOOD QUADRANGLE 7.5-MINUTE SERIES

DATE: 07/10/2024	DRAWN BY: BA	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	SITE VICINITY MAP	PROJECT NUMBER: LA1653
REVIEWED BY: IE	APPROVED BY: KC			INGLEWOOD CIVIC CENTER RENOVATIONS	SCALE: AS SHOWN
PREPARED BY: -				101W MANCHESTER BLVD INGLEWOOD, CA 90301	FIGURE NUMBER: 1

FILE PATH: \\192.168.0.2\Files\Projects\1600-1699\LA1653 LPA_Inglewood Civic Center Renovations\600 Drafting\CAD\LA1653 Fig 1 2.dwg
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GOOGLE EARTH 2023, AERIAL IMAGERY BY NAVIGATE LA, BUREAU OF ENGINEERING, CITY OF LOS ANGELES

LEGEND:

- CPT-2 — APPROXIMATE PRIOR CPT LOCATION (GROUP DELTA, 2021)
- SCPT-3 — APPROXIMATE PRIOR SCPT LOCATION (GROUP DELTA, 2021)
- 20 — APPROXIMATE PRIOR INVESTIGATION LOCATION (AMEC-1970)
- B-2 — APPROXIMATE BORING LOCATION (CURRENT INVESTIGATION)

DATE: 07/10/2024	DRAWN BY: BA
REVIEWED BY: IE	APPROVED BY: KC
REVISED: -	

GROUP DELTA

GROUP DELTA CONSULTANTS, INC
370 Amapola Ave.
Suite 212
Torrance, CA. 90501

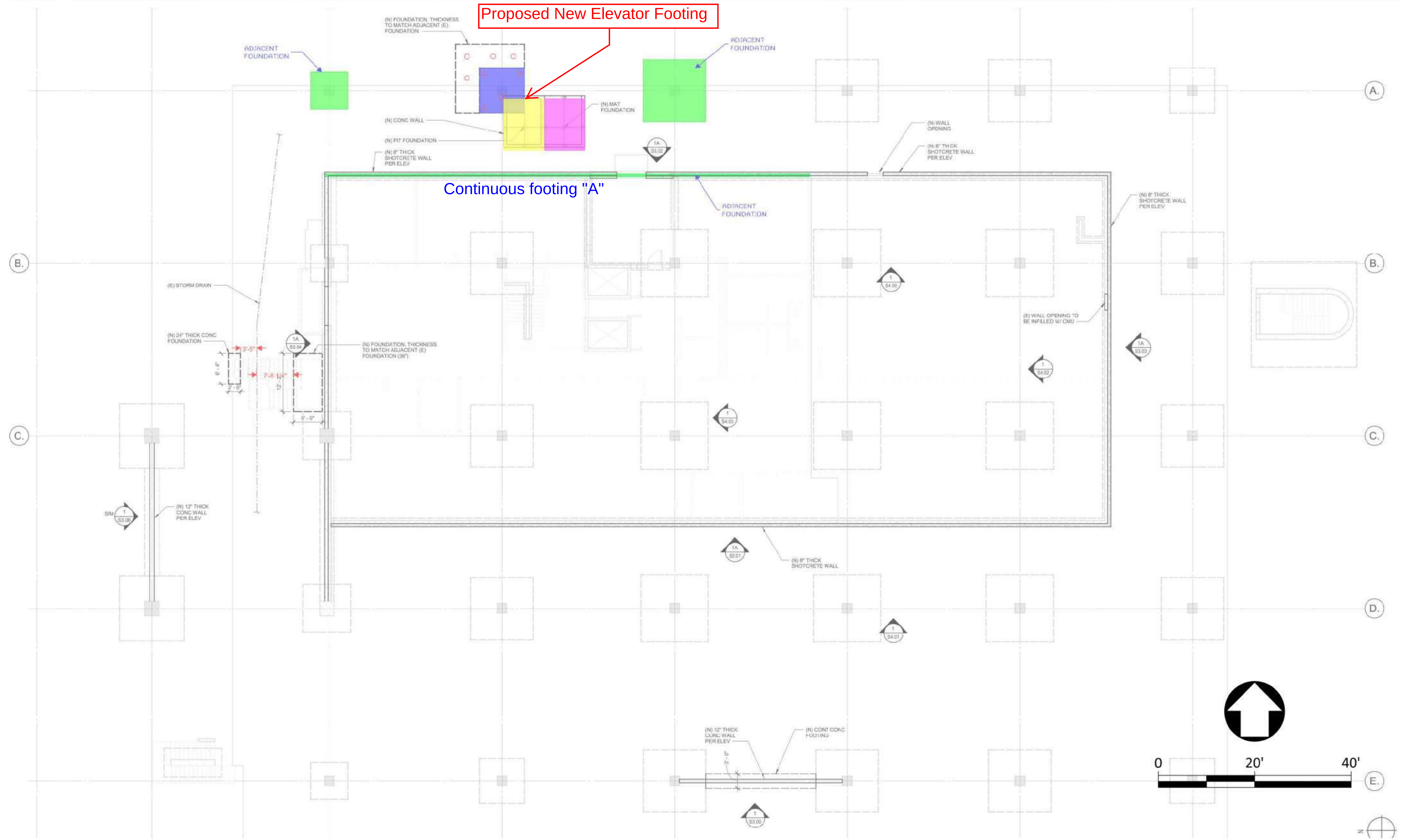
EXPLORATION LOCATION PLAN

INGLEWOOD CIVIC CENTER RENOVATIONS
101W MANCHESTER BLVD INGLEWOOD, CA 90301

PROJECT NUMBER: LA1653

SCALE: AS SHOWN

FIGURE NUMBER: 2



REFERENCE: FIRST FLOOR FOUNDATION PLAN, S2.01, LPA (2024)

DATE: 07/10/2024	DRAWN BY: BA	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	LIBRARY FOUNDATION PLAN	PROJECT NUMBER: LA1653
REVIEWED BY: IE	APPROVED BY: KC			INGLEWOOD CIVIC CENTER RENOVATIONS 101W MANCHESTER BLVD INGLEWOOD, CA 90301	SCALE: AS SHOWN
REVISED: -					FIGURE NUMBER: 3

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MAP UNITS
LATE HOLOCENE (SURFICIAL DEPOSITS

- af — **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
- Qsu — **Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
- Qls — **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
- Qb — **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
- Qw — **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
- Qf — **Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
- Qa — **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
- Qt — **Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel
- Ql — **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
- Qe — **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

- Qyf — **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qya — **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Late to Middle Pleistocene (Surficial Deposits)

- Qof — **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qoa — **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
- Qot — **Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits
- Qol — **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Middle to Early Pleistocene (Surficial Deposits)

- Qvof — **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qvoa — **Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed

Quaternary (Bedrock)

- Qss — **Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate
- Qsh — **Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

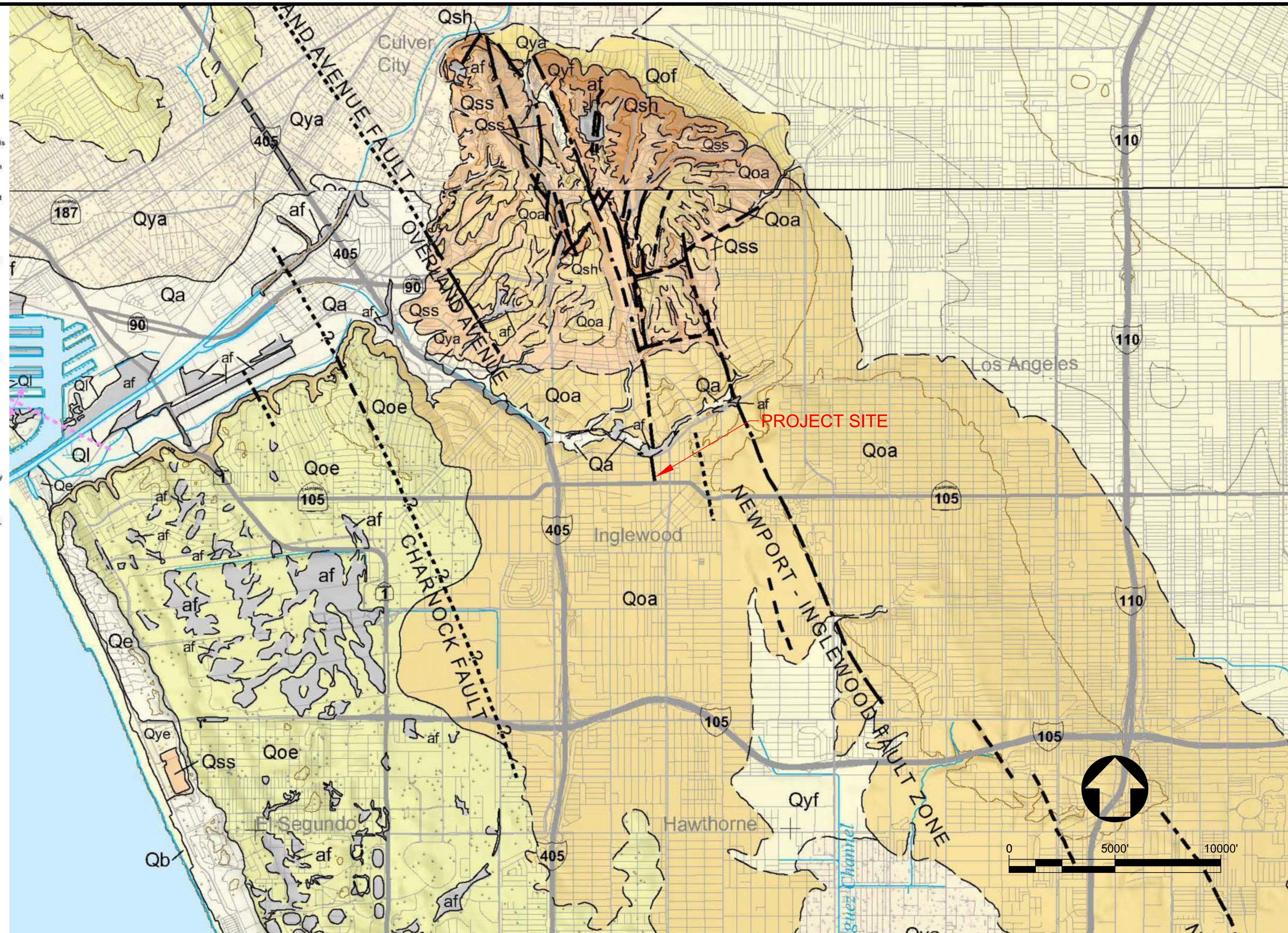
- Tss — **Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate
- Tsh — **Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
- Tv — **Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

- Kss — **Coarse-grained Cretaceous age formations of sedimentary origin**
- Ksh — **Fine-grained Cretaceous age formations of sedimentary origin**
- pKm — **Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**
- sp — **Serpentinite of all ages**
- gr — **Granitic and other intrusive crystalline rocks of all ages**

SYMBOL EXPLANATION

- Contact
- Gradational contact
- Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.
- Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip
- Lineament
- Folds -- Showing direction of plunge where appropriate
  - Anticline
  - Overturned anticline
  - Syncline
- Dike
- Stream
- Spring
- Road
- County boundary



REFERENCE: CGS, 2010, GEOLOGIC COMPILATION OF QUATERNARY SURFICIAL DEPOSITS IN SOUTHERN CALIFORNIA, LOS ANGELES AND LONG BEACH 30X60 QUADRANGLE

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REVIEWED BY: IE	APPROVED BY: KC		INGLEWOOD CIVIC CENTER RENOVATIONS 101W MANCHESTER BLVD INGLEWOOD, CA 90301	SCALE: AS SHOWN
REVISED: -				FIGURE NUMBER: 4

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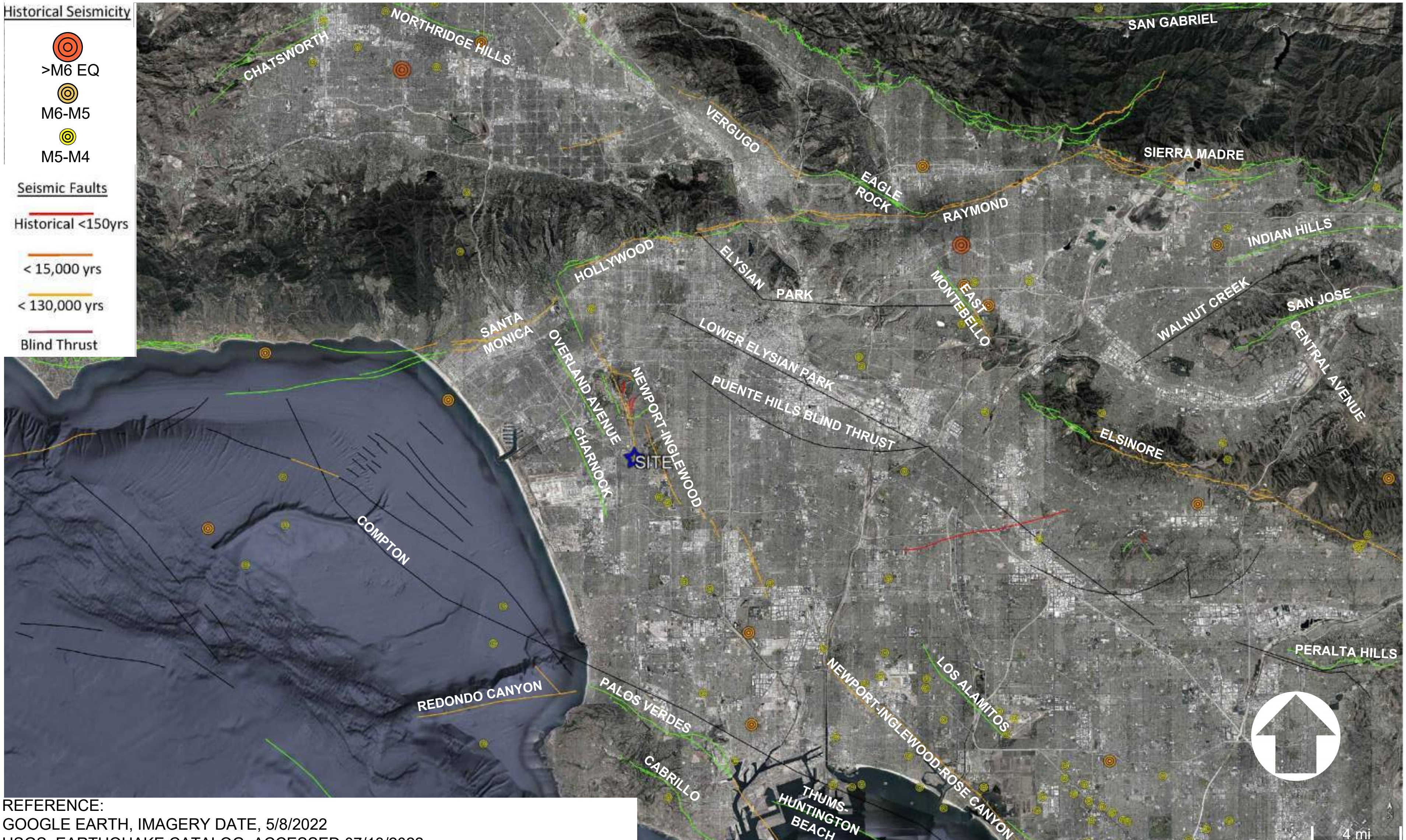

REFERENCE:
GOOGLE EARTH, IMAGERY DATE, 5/8/2022
USGS, EARTHQUAKE CATALOG, ACCESSED 07/13/2022
USGS & CGS, QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 07/13/2022

DATE: 07/10/2024	DRAWN BY: BA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL FAULT AND SEISMICITY MAP	PROJECT NUMBER: LA1653
REVIEWED BY: IE	APPROVED BY: KC		INGLEWOOD CIVIC CENTER RENOVATIONS	SCALE: AS SHOWN
PREPARED BY: -			101W MANCHESTER BLVD INGLEWOOD, CA 90301	FIGURE NUMBER: 5

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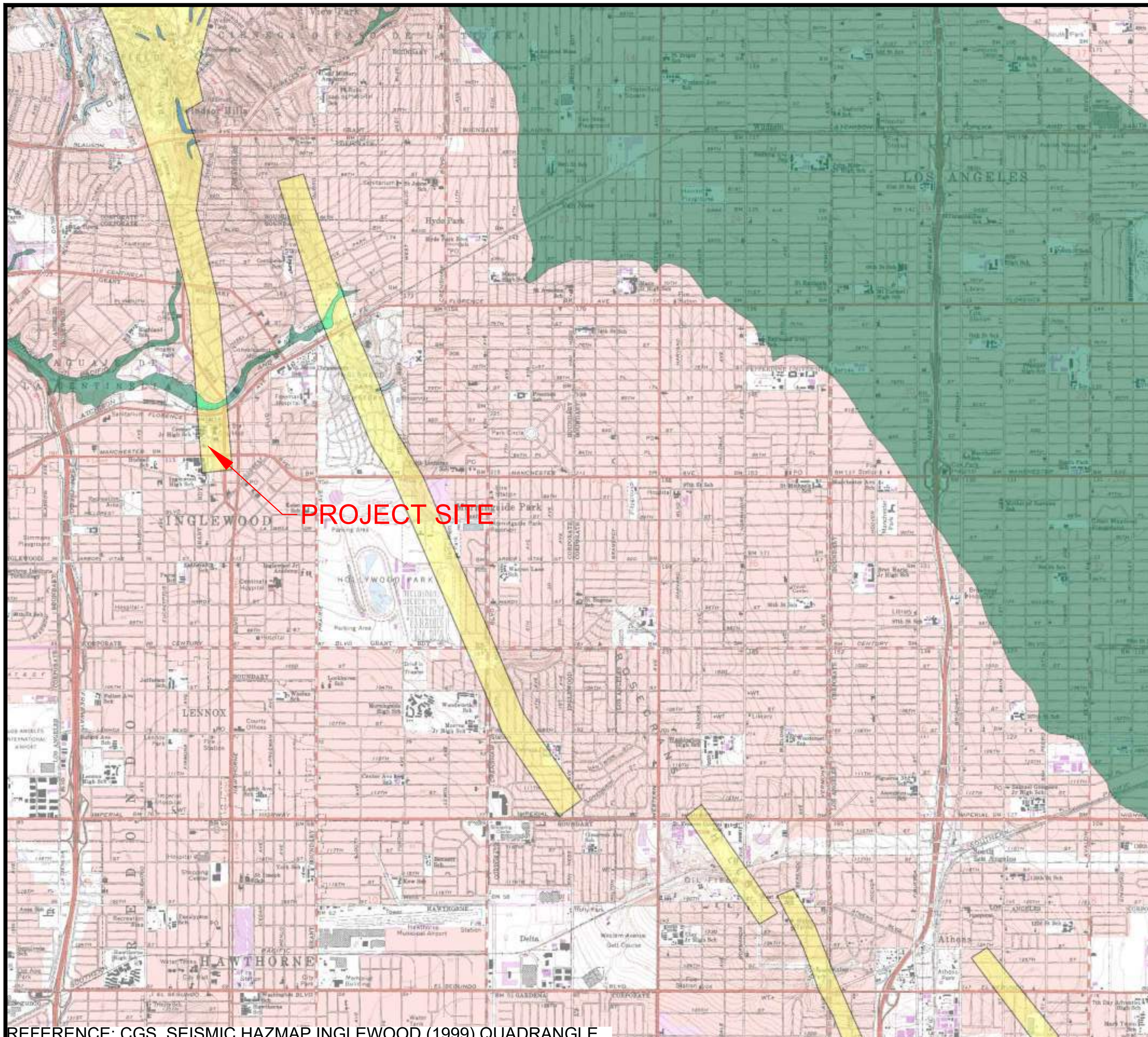


REFERENCE: CGS, SEISMIC HAZMAP INGLEWOOD (1999) QUADRANGLE

MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose 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.

1906 C ? ?

SEISMIC HAZARD ZONES

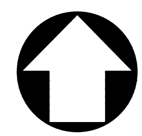
Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.



0 5000' 10000'

DATE: 07/10/2024	DRAWN BY: BA	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP
REVIEWED BY: IE	APPROVED BY: KC			INGLEWOOD CIVIC CENTER RENOVATIONS
PREPARED BY: -				101W MANCHESTER BLVD INGLEWOOD, CA 90301

PROJECT NUMBER:	LA1653
SCALE:	AS SHOWN
FIGURE NUMBER:	6

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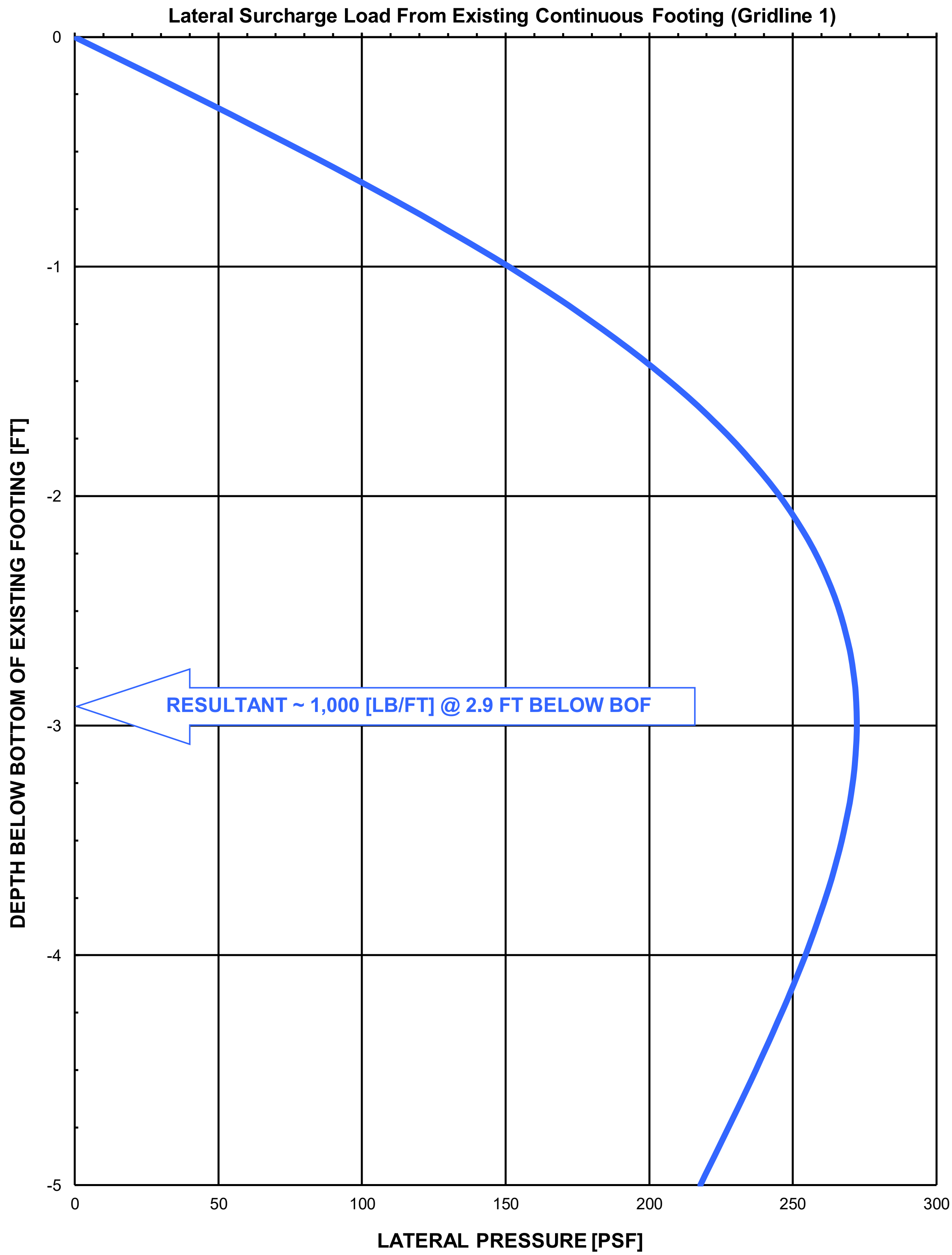


GROUP DELTA

SURCHARGE ANALYSIS

Project No. LA1653

FIGURE 7

APPENDIX A – PREVIOUS FIELD INVESTIGATION AND LABORATORY TESTING

JOB A-70230 DATE 10-27-70 DR O.E. CHKD.

BORING 13

DATE DRILLED October 7, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.8

Moisture (% of dry wt.)	Dry Density (lbs./cu. ft.)
9.7	117
13.1	121
16.3	113
16.3	112
18.4	112
28.8	95
12.4	115
16.2	110
34.3	86

Elevation (ft.): 115, 110, 105, 100, 95
Depth (ft.): 5, 10, 15, 20, 25

4½" Asphaltic Paving - 6" Base Course
SM SILTY SAND - fine, brown
CL SILTY CLAY - brown
SM SILTY SAND - fine, brown
ML SANDY SILT - greyish-brown
ML CLAYEY SILT - greyish-brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.T. O.E. JA CHKD [illegible]

BORING 13 (CONTINUED)

DATE DRILLED: October 7, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
90				
		14.7	104	■
85	30	5.7	111	■
80	35	4.7	95	■
75	40	11.7	125	■
70	45	11.7	123	■
65	50	9.1	118	■
60	55	5.3	110	■
55	60	4.4	104	■
	65			

SM — SILTY SAND - fine, brown

SP — SAND - fine, greyish-brown
Layer of Silty Sand
Light grey
Layer of Clayey Silt

SM — SILTY SAND - fine, brown

SP — SAND - fine, light grey
Layer of Silty Sand

NOTE: Water not encountered. No caving.

LOG OF BORING

A2-49 of 84

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR J.K. O.E. JM CHKD RCB

BORING 14

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket
ELEVATION 115.7

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
115				
		6.6	108	■
		4.6	108	■
	5			
110		7.2	113	■
		11.9	116	■
	10			
105		16.5	111	■
	15	18.5	109	■
100				
	20	16.5	97	■

Symbol	Description
ML	SANDY SILT - brown
SC	CLAYEY SAND - fine, brown
CL	SANDY CLAY - light brown
CL	SILTY CLAY - light brown
ML	SANDY SILT - light brown

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 15
DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.4

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
						3" Asphaltic Paving - 4" Base Course
115					SM	FILL - SILTY SAND - fine, few pieces of concrete, brown
		8.2	128	■	SM	SILTY SAND - fine, brown
	5	8.6	113	■		
110		7.7	118	■		
		8.1	117	■	SC	CLAYEY SAND - fine, brown
	10					
105		15.1	116	■	CL	SILTY CLAY - brown
	15	18.9	110	■		
100					ML	CLAYEY SILT - brown
		28.3	94	■		
	20					
95					ML	SANDY SILT - brown
						Layer of Sand
		16.2	113	■		
	25				SM	SILTY SAND - fine, brown
90					ML	CLAYEY SILT - brown
		18.5	105	■		
	30				SM	SILTY SAND - fine, brown
85						
		8.1	110	■	SP	SAND - fine, brown
	35					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

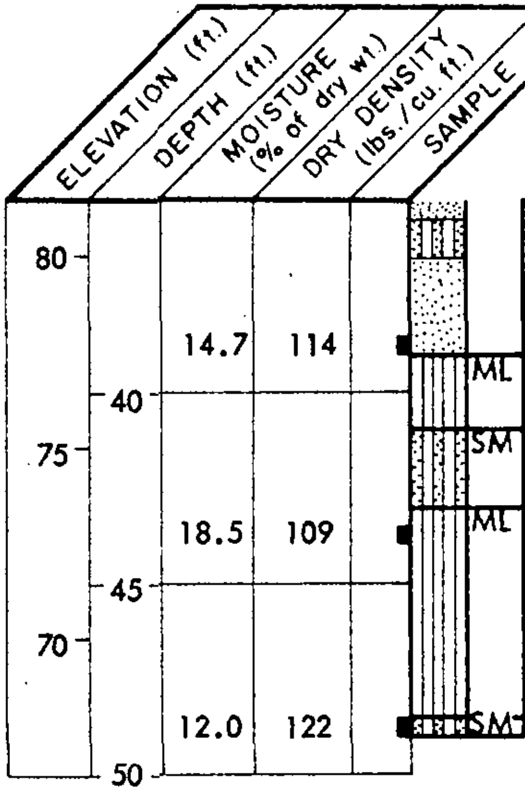
LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.F. O.E JM CHKD

BORING 15 (CONTINUED)

DATE DRILLED: October 9, 1970

EQUIPMENT USED: 18"-Diameter Bucket



Layer of Silty Sand

SANDY SILT - brown

SILTY SAND - fine, brown

CLAYEY SILT - greyish-brown

SILTY SAND - fine, greyish-brown

NOTE: Water not encountered. Caving from 36½' to 38½' (to 2½' in diameter).

LOG OF BORING

A2-52 of 84

LEROY CRANDALL AND ASSOCIATES

PLATE A-25

JOB A-70230 DATE 10-27-70 DR F.F. O.E. JH CHKD. [illegible]

JOB A-70230 DATE 10-27-70 DR J.K. O.E. J.H.

BORING 16

DATE DRILLED: October 12, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.1

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
						3" Asphaltic Paving - 6" Base Course
115					SM	SILTY SAND - fine, brown
		10.5	114	■		
		8.7	113	■		
	5					
110		8.9	107	■		
		6.4	114	■	SC	CLAYEY SAND - fine, brown
	10					
105		13.1	114	■		
	15					
100		14.7	111	■	CL	SILTY CLAY - light brown
					SM	SILTY SAND - fine, light brown
	20				SP	SAND - fine, light brown
95		4.0	105	■		
					SM	SILTY SAND - fine, light brown
	25					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR TL O.E. JH CHKD

BORING 16 (CONTINUED)

DATE DRILLED: October 12, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
90		17.0	114		ML	CLAYEY SILT - greyish-brown
					ML	SANDY SILT - light brown
85	30	28.6	88			Clayey
					SM	SILTY SAND - fine, light brown
80	35	7.5	111			
					SP	SAND - fine, light brown
75	40	20.2	102		ML	SANDY SILT - light greyish-brown
70	45	16.5	112			
	50	16.4	108			

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 17

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.5

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
115						3" Asphaltic Paving
					ML	SANDY SILT - brown
		5.3	105	■		
		7.2	119	■		
110	5					
					SM	SILTY SAND - fine, light brown
		6.1	106	■		
		7.8	120	■		
					SC	CLAYEY SAND - fine, brown
105	10					
					CL	SILTY CLAY - brown
		14.5	110	■		
100	15					
		16.6	111	■		
					ML	SANDY SILT - light brown
95	20					
		11.1	103	■		
	25					

(CONTINUED ON FOLLOWING PLATE)

JOB A-70230 DATE 10-27-70 DR T.F. O.E. JH CHKD. RLB

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 17 (CONTINUED)
DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE
90		20.0	98	■
85	30	10.3	105	■
80	35	5.1	102	■
75	40			
70	45	18.0	108	■
65	50	10.4	120	■
	55			

SM SILTY SAND - fine, brown

Layer of Clay

SP SAND - fine, light brown

ML SANDY SILT - grey

SC CLAYEY SAND - fine, brown

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-27-70 DR 77t O.E. JM CHKD. RCL

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 18

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.6

Elevation (ft.)	Depth (ft.)	Moisture (% of dry wt.)	Dry Density (lbs./cu. ft.)	Sample		Description
						2" Asphaltic Paving - 6" Base Course
115					ML	SANDY SILT - brown
		6.0	117	■		
	5	6.6	102	■	SM	SILTY SAND - fine, brown
110						
		8.6	116	■		
	10	7.3	109	■		
105		14.1	116	■	CL	SANDY CLAY - brown
	15	15.4	115	■		
100						
					CL	SILTY CLAY - brown
	20	20.6	102	■		
95					ML	SANDY SILT - light brown
					SM	SILTY SAND - fine, brown
	25	13.3	102	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR T.F. O.E. VM CHKD lkb

BORING 18 (CONTINUED)

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
90						
					CL	SILTY CLAY - brown
	30	36.2	82	■		
85					ML	CLAYEY SILT - light brown
					SM	SILTY SAND - fine, brown
	35	8.4	105	■		
80					SP	SAND - fine, light brown
	40	5.1	97	■		
75					ML	SANDY SILT - light brown
	45	14.7	110	■		
70						
	50	13.5	105	■		

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JH CHKD KB

BORING 19

DATE DRILLED October 9, 1970

EQUIPMENT USED 18"-Diameter Bucket

ELEVATION 115.2

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
115						2" Asphaltic Paving - 3" Silty Sand Base
					SM	SILTY SAND - fine, brown
		8.2	106			
		8.1	123			
110	5					
		7.7	117			
		6.8	113			
105	10					
		9.6	116			
					CL	SILTY CLAY - brown
		15.6	115			
100	15					
					ML	CLAYEY SILT - greyish-brown
		17.4	114			
95	20					
					SM	SILTY SAND - fine, brown
		9.1	108			
	25					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JM ECHKD [illegible]

BORING 19 (CONTINUED)

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu ft.)	SAMPLE		
90						
					ML	CLAYEY SILT - brown
		32.5	87	■		
85	30					
					SP	SAND - fine, light brown
		6.4	112	■		
80	35					
		3.1	101	■		
75	40				ML	SANDY SILT - greyish-brown
		16.0	117	■		
70	45					
		16.2	113	■		
	50					

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JM CHKD AK

LOG OF BORING

A2-60 of 84

LEROY CRANDALL AND ASSOCIATES

BORING 20
DATE DRILLED October 12, 1970
EQUIPMENT USED: 18"-Diameter Bucket
ELEVATION 115.1

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		Description
115						2" Asphaltic Paving
					SM	SILTY SAND - fine, brown
		8.0	97	■		
		8.1	114	■		
110	5	8.0	116	■		
		9.5	113	■		
105	10				SC	CLAYEY SAND - fine, brown
		9.3	107	■		
					ML	SANDY SILT - light brown
100	15	17.8	106	■		
					ML	CLAYEY SILT - light brown
95	20	17.9	104	■		
					SM	SILTY SAND - fine, light brown
	25	12.9	95	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

A2-61 of 84
LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR ___ O.E. JM ___ CHKD. MWB

BORING 20 (CONTINUED)

DATE DRILLED October 12, 1970
EQUIPMENT USED 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
90				
85	30	30.5	92	■
80	35	26.4	95	■
75	40	4.9	104	■
70	45	13.9	113	■
	50	12.8	114	■

ML CLAYEY SILT - light brown

SP SAND - fine, light brown

ML SANDY SILT - greyish-brown

NOTE: Water not encountered. No caving.

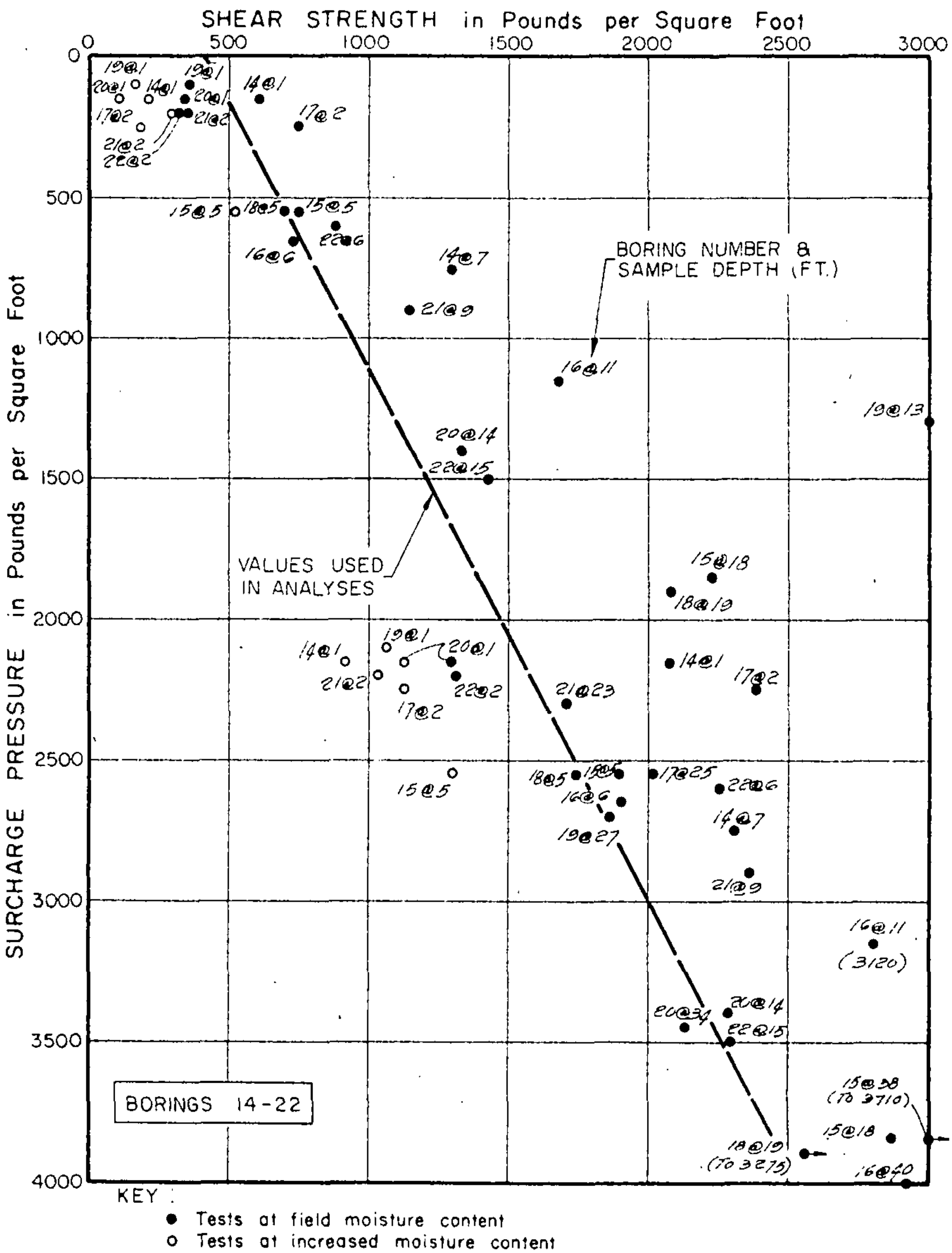
LOG OF BORING

A2-62 of 84

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR J.F. O.E. JM CHKD. [illegible]

JOB A-70230 DATE 10-22-70 DR. T.T. O.E. JM CHKD.

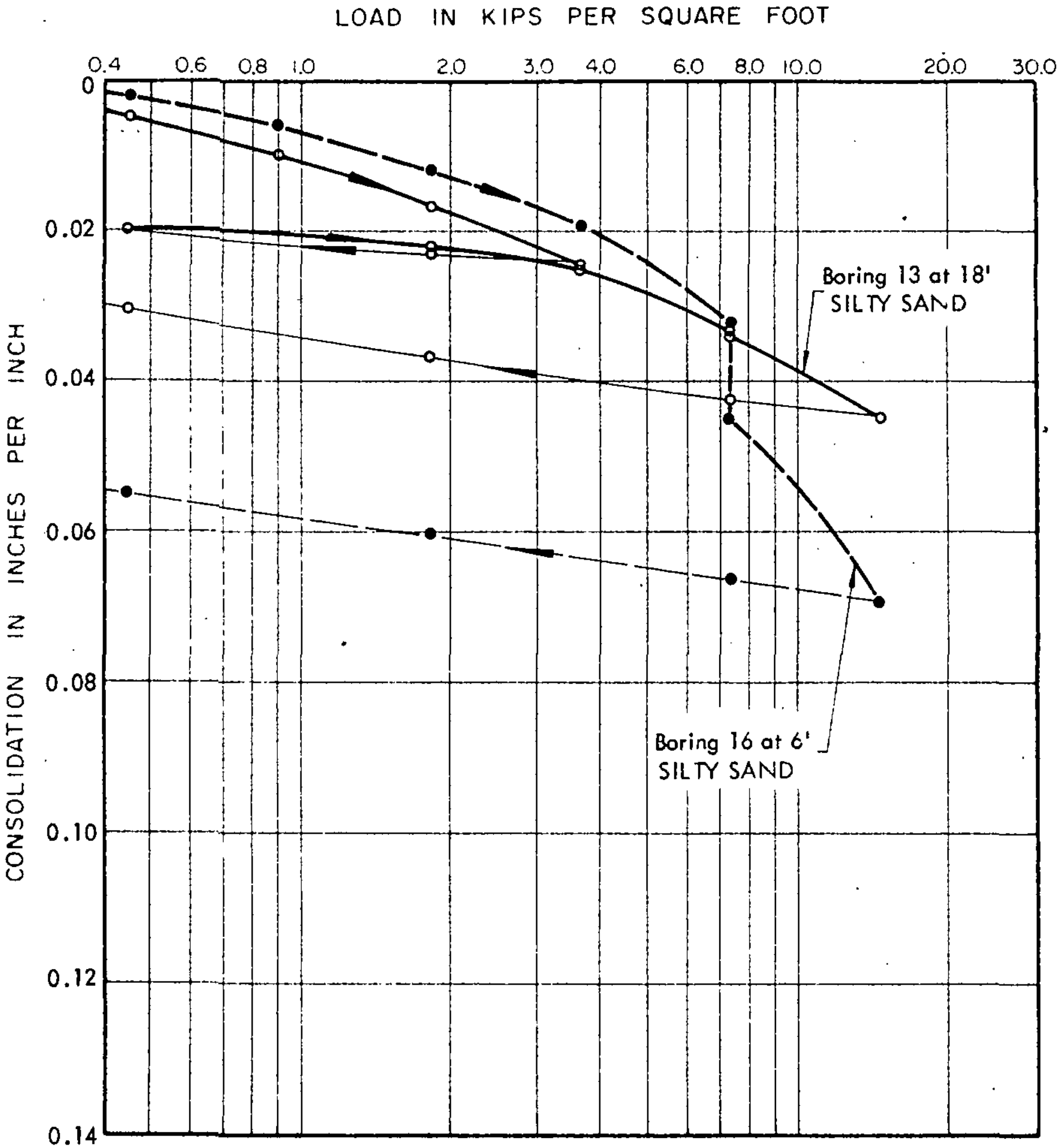


DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE C-2

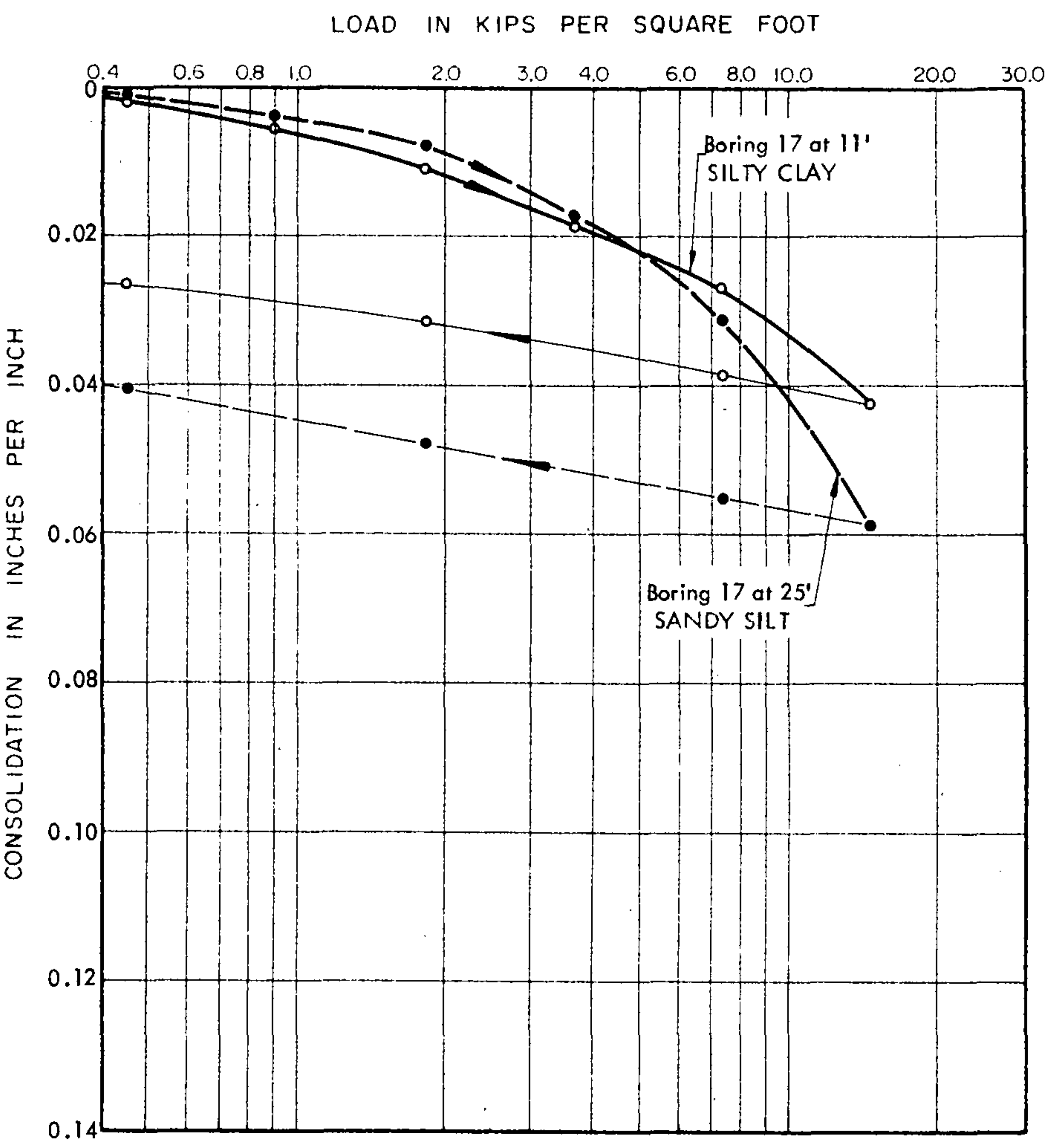
JOB A-70230 DATE 10-22-70 DR T.K. O.E. JM CHKD.



NOTE: Water added to samples after consolidation under a load of 7.2 kips per square foot.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES



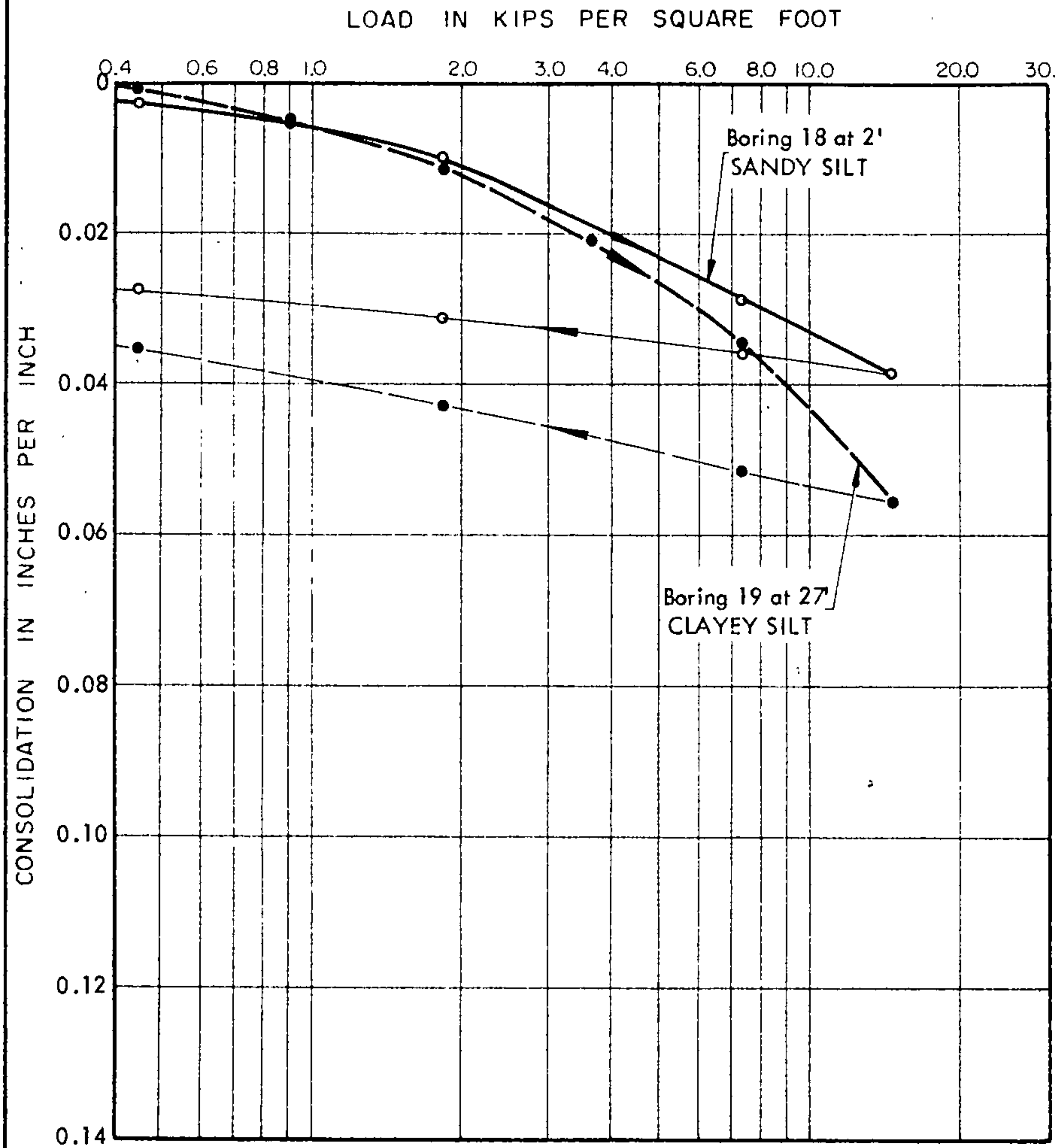
NOTE: Samples tested at field moisture content.

JOB A-70230 DATE 10-22-70 DR T.F. O.E. JM lb CHKD.

CONSOLIDATION TEST DATA

LeROY CRANDALL & ASSOCIATES

JOB 4-70230 DATE 10-22-70 DR. CHKD.



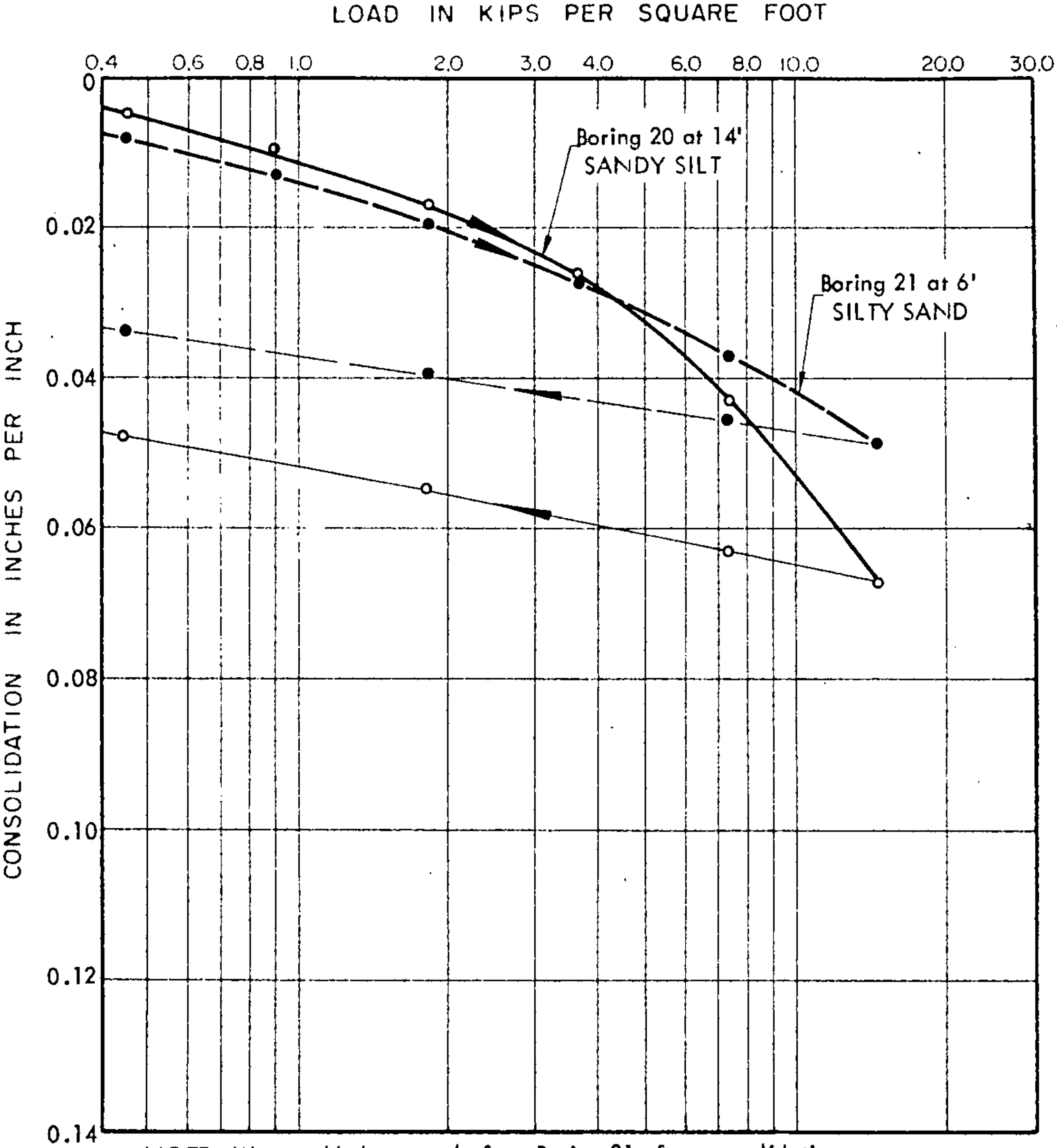
NOTE: Water added to sample from Boring 18 after consolidation under a load of 3.6 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL & ASSOCIATES

PLATE D-8

JOB A-70230 DATE 10-22-70 DR T.F. O.E. JM lb CHKD. ac



NOTE: Water added to sample from Boring 21 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

BORING NUMBER AND SAMPLE DEPTH.	13 at 9'	18 at 2'
SOIL TYPE:	SILTY CLAY	SANDY SILT
CONFINING PRESSURE: (Lbs./Sq.Ft.)	200	200
FIELD MOISTURE CONTENT: (%)	16.3	6.0
EXPANSION FROM FIELD TO SOAKED MOISTURE CONTENT: (%)	0.2	0
SOAKED MOISTURE CONTENT: (%)	17.4	13.0
SHRINKAGE FROM FIELD TO AIR-DRIED MOISTURE CONTENT: (%)	7.4	2.4
AIR-DRIED MOISTURE CONTENT: (%)	3.6	0.7
TOTAL VOLUME CHANGE: (%)	7.6	2.4

EXPANSION TEST DATA

LEROY CRANDALL AND ASSOCIATES

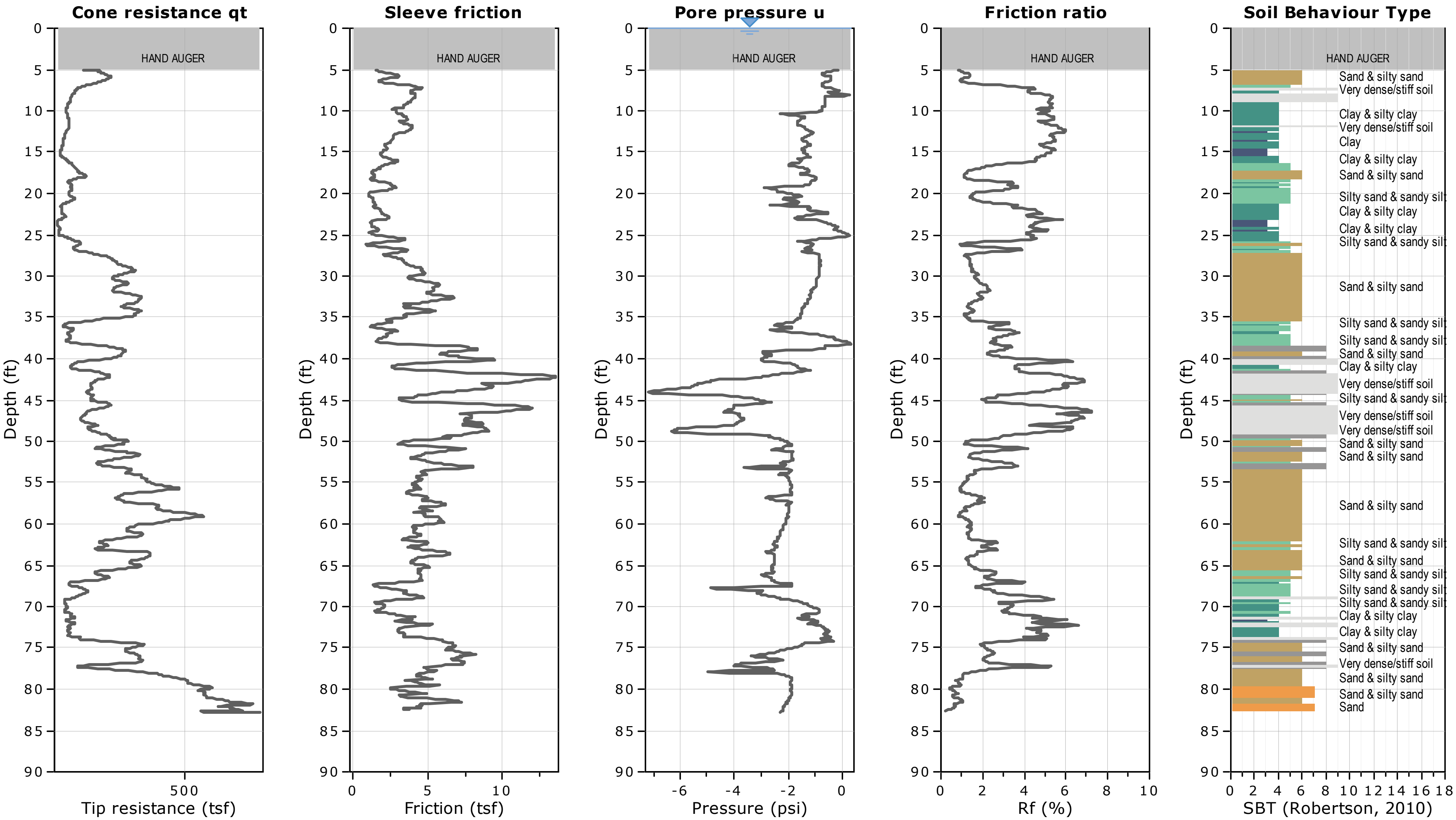
Group Delta Consultants
370 Amapola Avenue, Suite 212
Torrance, CA 90501
http://www.groupdelta.com

Project: **City of Inglewood Seismic Retrofits**

Location: **1 W Manchester Blvd., Inglewood, CA 90301**

CPT: SCPT-1

Total depth: 82.88 ft, Date: 10/23/2020



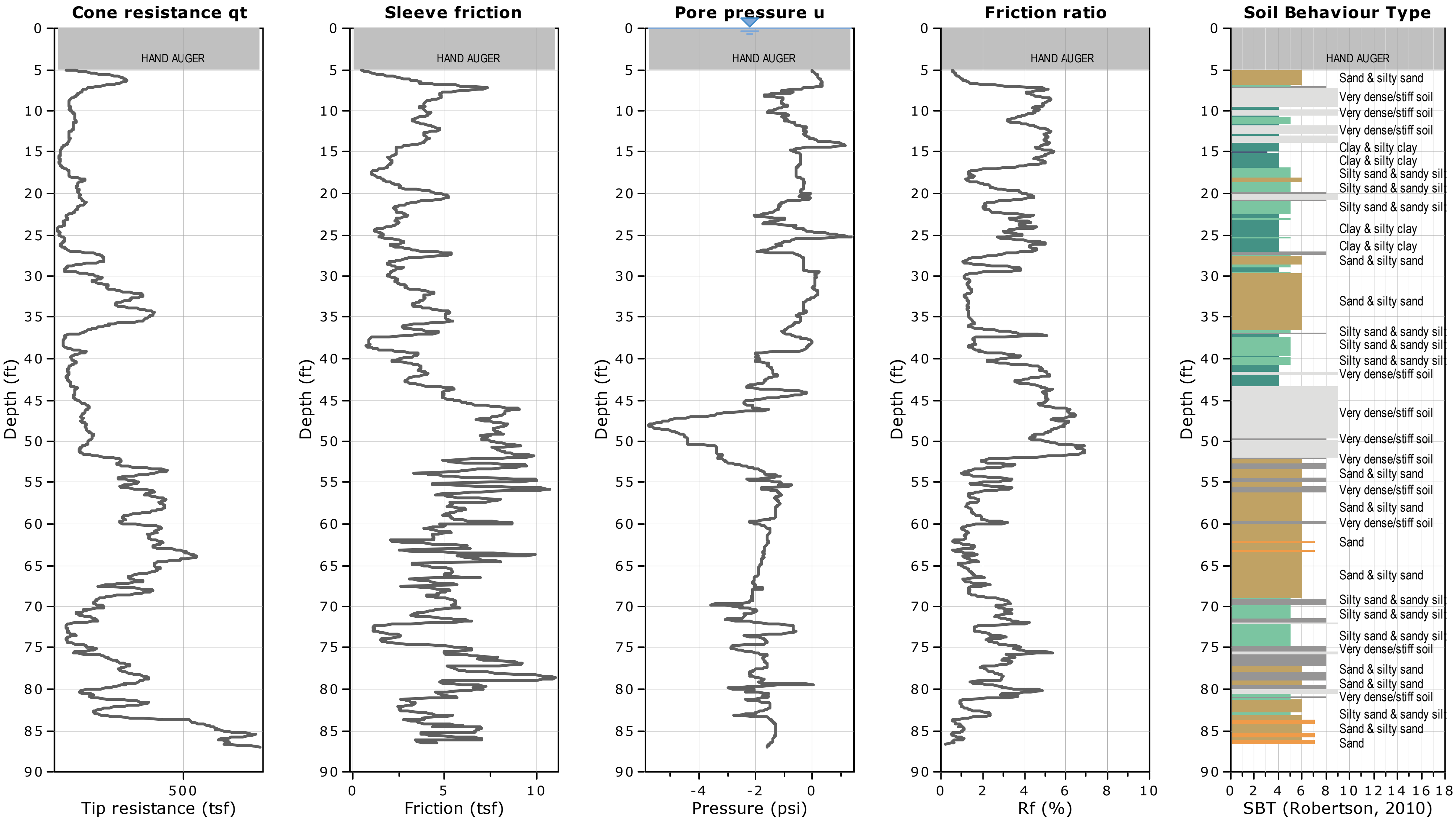
Group Delta Consultants
370 Amapola Avenue, Suite 212
Torrance, CA 90501
http://www.groupdelta.com

Project: City of Inglewood Seismic Retrofits

Location: 1 W Manchester Blvd., Inglewood, CA 90301

CPT: CPT-2

Total depth: 86.95 ft, Date: 10/23/2020



APPENDIX B – FIELD INVESTIGATION

APPENDIX B
Field Investigation

B.1 Introduction

A geotechnical subsurface investigation was conducted by Group Delta on June 17th, 2024 for geotechnical assessments of subgrade soils beneath the Project Site. The investigation consisted of two (2) hollow-stem auger (HSA) borings. The exploration locations and numbers are shown in Figure 2 of the main report. A summary table of Group Delta's current explorations is provided in Table B-1.

B.2 Borings

Two (2) HSA borings were drilled to depths of 10 feet below ground surface. Subsurface materials were visually classified and recorded by Group Delta's field technician in accordance with the Unified Soil Classification System (USCS).

The upper 5 feet of each boring was hand-augured to clear the location for utilities. Soil cuttings from the length of each boring were collected as bulk samples and transported to Group Delta's laboratory for further testing and classification.

Driven samples and bulk samples of encountered soils were obtained from the boring and recorded on the boring logs. 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.

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.

Modified California drive samples were collected with a 3.0 inch outside diameter (OD) ring-lined split barrel sampler with a 2.42-inch inside diameter (ID) cutting shoe. The sampler barrel is lined with 12-inches of metal rings for sample collection. 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 the 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.

B.3 List of Attached Tables and Figures

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

TABLE

Table B-1
Summary of Field Explorations

Exploration No.	Date Performed	Ground Surface Elevation (feet)	Total Depth (ft)	Groundwater Depth (ft)	Exploration Type
B-1	6/17/2024	113	10	NE	Hollow Stem Auger
B-2	6/17/2024	113	10	NE	Hollow Stem Auger

Notes:

1. NE = not encountered
2. Ground surface elevations are approximate and were obtained via Google Earth

FIGURES

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence		Refer to Section		Required	Optional
		Field	Lab		
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 CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER **B-1A**
PROJECT NAME INGLEWOOD CIVIC CENTER RENOVATIONS	PROJECT NUMBER LA1653
BORING RECORD LEGEND #1	

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	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
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and 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
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	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with 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	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
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL 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
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	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
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)	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
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)	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
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	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
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	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
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
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])

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 CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER B-1B
PROJECT NAME INGLEWOOD CIVIC CENTER RENOVATIONS	PROJECT NUMBER LA1653

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 B-1C
PROJECT NAME INGLEWOOD CIVIC CENTER RENOVATIONS	PROJECT NUMBER LA1653

BORING RECORD LEGEND #3

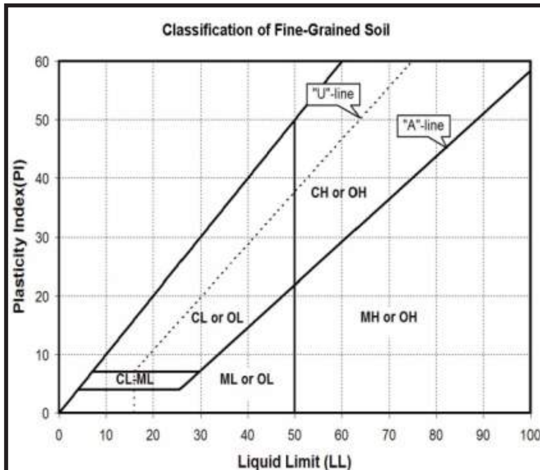
CLASSIFICATION OF INORGANIC FINE GRAINED SOILS (Soils with ≥50% finer than No. 200 Sieve)

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

Group Delta Project No. LA1653

INGLEWOOD CIVIC CENTER RENOVATIONS

KEY FOR SOIL CLASSIFICATION #1

Figure B-2A

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

				GROUP SYMBOL		GROUP NAME
GRAVEL % GRAVEL> % SAND	≤5%fines (<5% fines)	Well-graded		GW	<15% sand	Well-graded gravel
					≥15% sand	Well-graded gravel with sand
		Poorly graded		GP	<15% sand	Poorly graded gravel
					≥15% sand	Poorly graded gravel with sand
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	GW-GM	<15% sand	Well-graded gravel with silt
					≥15% sand	Well-graded gravel with silt and sand
			Fines=CL or CH	GW-GC	<15% sand	Well-graded gravel with clay
					≥15% sand	Well-graded gravel with clay and sand
		Poorly graded	Fines=ML or MH	GP-GM	<15% sand	Poorly graded gravel with silt
					≥15% sand	Poorly graded gravel with silt and sand
			Fines=CL or CH	GP-GC	<15% sand	Poorly graded gravel with clay
					≥15% sand	Poorly graded gravel with clay and sand
	≥15%fines (>12% fines)		Fines=ML or MH	GM	<15% sand	Silty gravel
					≥15% sand	Silty gravel with sand
			Fines=CL or CH	GC	<15% sand	Clayey gravel
					≥15% sand	Clayey gravel with sand
SAND % SAND≥ % GRAVEL	≤5%fines (<5% fines)	Well-graded		SW	<15% gravel	Well-graded sand
					≥15% gravel	Well-graded sand with gravel
		Poorly graded		SP	<15% gravel	Poorly-graded sand
					≥15% gravel	Poorly-graded sand with gravel
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	SW-SM	<15% gravel	Well-graded sand with silt
					≥15% gravel	Well-graded sand with silt and gravel
			Fines=CL or CH	SW-SC	<15% gravel	Well-graded sand with clay
					≥15% gravel	Well-graded sand with clay and gravel
		Poorly graded	Fines=ML or MH	SP-SM	<15% gravel	Poorly graded sand with silt
					≥15% gravel	Poorly graded sand with silt and gravel
			Fines=CL or CH	SP-SC	<15% gravel	Poorly graded sand with clay
					≥15% gravel	Poorly graded sand with clay and gravel
	≥15%fines (>12% fines)		Fines=ML or MH	SM	<15% gravel	Silty sand
					≥15% gravel	Silty sand with gravel
			Fines=CL or CH	SC	<15% gravel	Clayey sand
					≥15% gravel	Clayey sand with gravel

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 $1 \le C_c \le 3$
GW	$C_u > 4$ and $1 \le C_c \le 3$
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

GROUP DELTA

Group Delta Project No. LA1653

INGLEWOOD CIVIC CENTER RENOVATIONS

KEY FOR SOIL CLASSIFICATION #2

Figure B-2B

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Inglewood Civic Center Renovations	LA1653	B-1

SITE LOCATION	START	FINISH	SHEET NO.
101 W Manchester Blvd, Inglewood	6/17/2024	6/17/2024	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME-75	Hollow Stem Auger	IE	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in	N/A	8"	10	113	▽ NE / *na* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	
Bulk; SPT (ID: 1.4"); MC (ID: 2.4")		▼ NE / *na* AFTER DRILLING

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	113													ASPHALT (4")
														BASE (8")
														CLAYEY SAND (SC); brown; moist; mostly fine to medium grained SAND; some fines.
	110	Bulk	Bulk-1								Corr. R			
5														Sandy Lean CLAY (CL); stiff; brown; moist; mostly CLAY; some fine to medium grained SAND. (SAND = 36%, Fines = 64%)
		SPT	S-1	9 11 10	21			64.0	28:14					
	105	Bulk	Bulk-2											
		MC	R-1	3 9 17	26	14	120				Cons.			
10														Boring terminated at 10 feet below ground surface. Groundwater not encountered. Backfilled with grout and patched with quickset concrete dyed black.
	100													
15														
	95													

GDC_LOG_BORING_2011 LA1653.GPJ GDCLOG.GDT 7/5/24

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

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.

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Inglewood Civic Center Renovations	LA1653	B-2

SITE LOCATION	START	FINISH	SHEET NO.
101 W Manchester Blvd, Inglewood	6/17/2024	6/17/2024	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME-75	Hollow Stem Auger	IE	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in	N/A	8"	10	113	∇ NE / *na* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	
Bulk; SPT (ID: 1.4"); MC (ID: 2.4")		▼ NE / *na* AFTER DRILLING

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	113													ASPHALT (3")
														BASE (5")
														CLAYEY SAND (SC); brown; moist; mostly fine to medium grained SAND; some fines.
	110	Bulk	Bulk-1								EI			
5														Medium dense.
		SPT	S-1	9 11 10	21			24.4						(SAND = 75.6%, Fines = 24.4%).
		Bulk	Bulk-2											
	105													
		MC	R-1	6 14 19	33	18	114				DS			
10														Boring terminated at 10 feet below ground surface. Groundwater not encountered. Backfilled with grout and patched with quickset concrete dyed black.
	100													
15														
	95													

APPENDIX C – LABORATORY TESTING

APPENDIX C
Laboratory Testing

B.1 Introduction

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

Modified California drive samples, Standard Penetration Test (SPT) drive samples, and bulk samples collected during the field investigation were sealed in the field to conserve in-situ moisture. 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 D2487) and Visual Manual (ASTM D2488)
- Moisture content (ASTM D2216) and Dry Unit Weight (ASTM D2937)
- Percent Passing #200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D4318)
- One-Dimensional Consolidation (ASTM D2435)
- Direct Shear Test (ASTM D3080)
- R-Value (CTM 301)
- Soil Expansion Index (ASTM D4829)
- Soil Corrosivity:
    - pH (CTM 643)
    - Water-Soluble Sulfate (ASTM D516, CTM 417)
    - Water-Soluble Chloride (Ion-Specific Probe, CTM 422)
    - Minimum Electrical Resistivity (CTM 643)

Lab test results are included in this appendix. Brief descriptions of the laboratory testing program and test results are presented below.

B.2 Soil Classification

The subsurface materials were classified visually in the field using the Unified Soil Classification System (USCS), per ASTM Test Methods D2487 and D2488 and in general accordance per Caltrans Soil and Logging Classification and Presentation Manual (2010). Soil classifications were modified as necessary based on further inspection and testing in the laboratory. The soil classifications are presented on the key for soil classification and the boring logs in Appendix B.

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 D2216. Selected California Ring 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 were measured, and moisture content and dry density were calculated in general accordance with ASTM D2216 and D2937. The results of these tests are presented in the boring records in Appendix B.

B.4 Grain Size Distribution and Percent Passing #200 Sieve

The amount of material finer than the No. 200 sieve was determined according to ASTM D1140. The results of this test are presented in the boring logs in Appendix B and are attached to this appendix.

B.5 Atterberg Limits

Characterization of the fine-grained fractions of soils was evaluated using the Atterberg Limits. This test includes Liquid Limit (LL) and Plastic Limit (PL) tests to determine the Plasticity Index (PI) per ASTM D4318. Results of Atterberg Limit tests are illustrated in the plasticity charts attached to this appendix.

B.6 One-dimensional Consolidation Test

The consolidation characteristics of the soils were determined by performing one-dimensional consolidation in general accordance with ASTM D2435, using a floating ring consolidometer and dead weight system. The test results are attached to this appendix.

B.7 Direct Shear Test

The direct shear test was performed on a selected sample per ASTM D3080. After the initial weight and volume measurements were made, the sample was placed in a calibrated shear machine and a selected normal load was applied. The sample was then saturated and allowed to consolidate, and then were sheared under a constant strain to failure. Shear stress and sample deformations were monitored throughout the test. The test results are attached to this appendix.

B.8 R-Value

Resistance "R" value tests were performed using the stabilometer method on a selected bulk sample of the subgrade soils. The tests were conducted in general accordance with CTM 301. The test results are summarized in Table B-1 below and are attached to this appendix.

Table B-1
Resistance R-Value

Boring No.	Depth (ft)	Resistance R-Value
B-1	0-5	33

B.9 Expansion Index

The expansion potential of the site soils was estimated using the Expansion Index Test per ASTM D4829. The results of these tests are summarized in Table B-2 below and are attached to this appendix.

Table B-2
Expansion Index

Boring No.	Depth (ft)	Expansion Index	Expansion Potential
B-2	0-5	0	Very Low

B.10 Soil Corrosivity

Tests were performed to determine the 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 Probe or Caltrans Test Method 422), and water-soluble sulfates (ASTM D516 and CTM 417). The test results are summarized in Table B-3 below and are attached to this appendix.

Table B-3
Summary of Soil Corrosivity

Boring No.	Depth (ft)	pH	Sulfate Content (%)	Chloride Content (%)	Minimum Resistivity (ohm-cm)
B-1	0-5	8.01	<0.01	<0.01	8,110

B.11 List of Attached Figures

The following figures are attached and complete this appendix:

List of Current Laboratory Test Results

LABORATORY TESTING

GROUP DELTA
1320 South Simpson Circle
Anaheim, CA 92806
(714) 660-7500 office
(714) 660- 7550 fax

PERCENT PASSING NO. 200 SIEVE (ASTM D1140)

Lab No.: *SO7173*

Project Name: *Inglewood Civic Center* **Date Sampled:** *6/17/2024*

Project No.: *LA1653* **Sampled By:** *IE*

Tested By: *Eric Y.* **Date:** *6/19/2024*

	Boring No.	B-1		B-2									
	Sample No.	S-1		S-1									
	Depth	5'		5'									
	Soil Description	Brown Sandy Lean Clay		Brown Clayey Sand									
BEFORE WASH	Container No.	S-3		S-4									
BEFORE WASH	Wet Soil Weight & Tare	397.6		418.7									
BEFORE WASH	Dry Soil Weight & Tare	360.5		390.1									
BEFORE WASH	Tare Weight	126.1		126.3									
BEFORE WASH	Weight of Dry Soil	234.4		263.7									
BEFORE WASH	Moisture Loss	37.1		28.6									
BEFORE WASH	Percent Moisture	15.8		10.9									
AFTER WASH	Dry Weight of Soil Retained on #200 Sieve	84.3		199.5									
AFTER WASH	Dry Weight of Soil Retained on #4 Sieve	0.0		0.0									
AFTER WASH	Weight of Ret. #200 Soil Minus Ret. #4	84.3		199.5									
AFTER WASH	Percent Gravel	0.0		0.0									
AFTER WASH	Percent Sand	36.0		75.6									
AFTER WASH	Percent Fines	64.0		24.4									

Figure B-1

ATTERBERG LIMITS

ASTM D-4318 / AASHTO T-89 / CTM 204

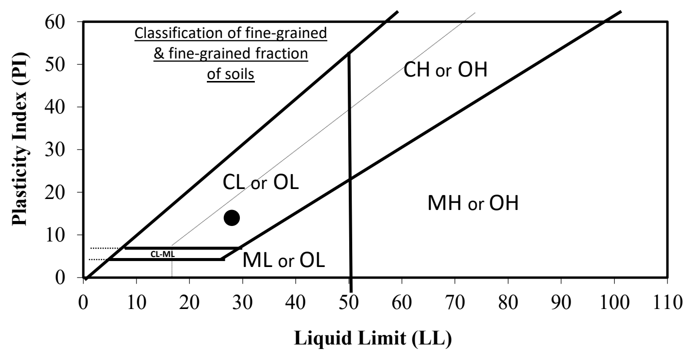
Project Name:	*Inglewood Civic Center*	Tested By :	*Eric Y.*	Date:	*06/21/24*
Project No. :	*LA1653*	Data Input By:	*Eric Y.*	Date:	*06/24/24*
Boring No.:	*B-1*	Checked By:		Date:	
Sample No. :	*S-1*	Depth (ft.) :	*5*		
Initial Moisture:		Container No.:	*AL-1*		
Description.:	*Brown Sandy Lean Clay - CL*				

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			31	25	17	
Container No.	A	B	C	D	E	
Wet Wt. of Soil + Cont. (gm.)	22.91	22.74	29.79	30.93	31.46	
Dry Wt. of Soil + Cont. (gm.)	21.95	21.79	26.68	27.51	27.72	
Wt. of Container (gm.)	15.27	15.17	15.24	15.39	15.01	
Moisture Content (%) [Wn]	14.37	14.35	27.19	28.22	29.43	

LIQUID LIMIT	**28**
PLASTIC LIMIT	**14**
PLASTICITY INDEX	**14**

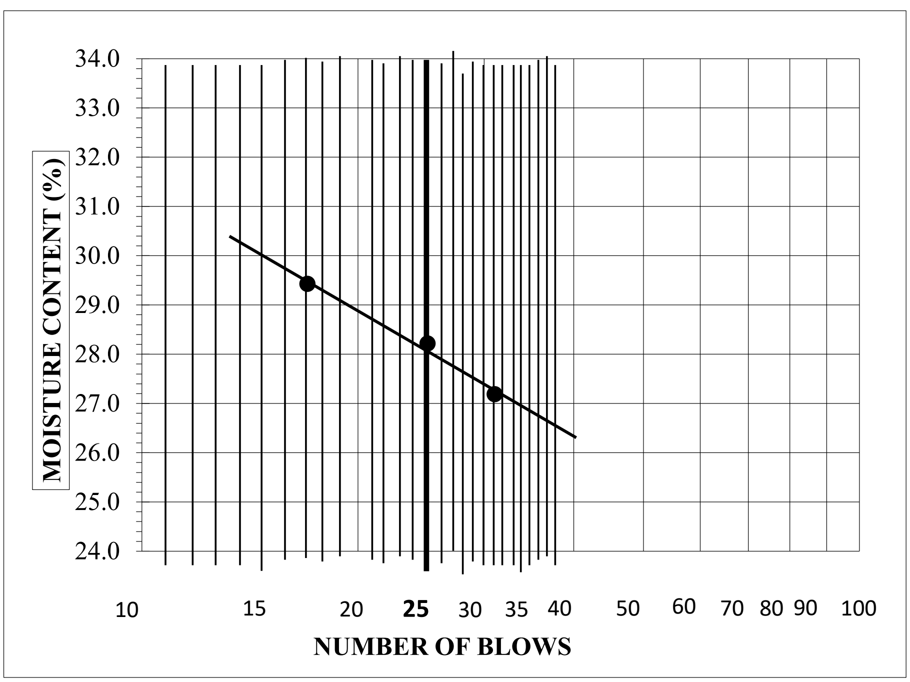
PI at "A" - Line = 0.73(LL-20) = **5.8**

One - Point Liquid Limit Calculation $LL=W_n(N/25)^{0.121}$



PROCEDURES USED

- ☐ Wet Preparation
  Multipoint Wet Preparation
- ☐ Dry Preparation
  Multipoint Dry Preparation
- ☐ Procedure A
  Multipoint Test
- ☐ Procedure B
  One-point Test



GROUP DELTA CONSULTANTS
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Anaheim, CA 92806
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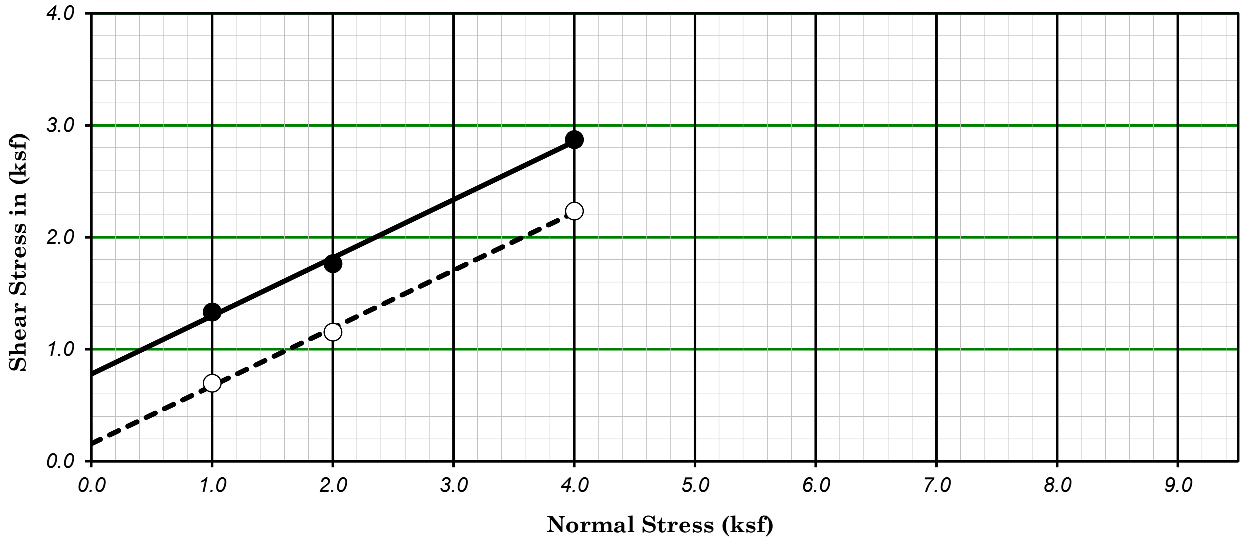
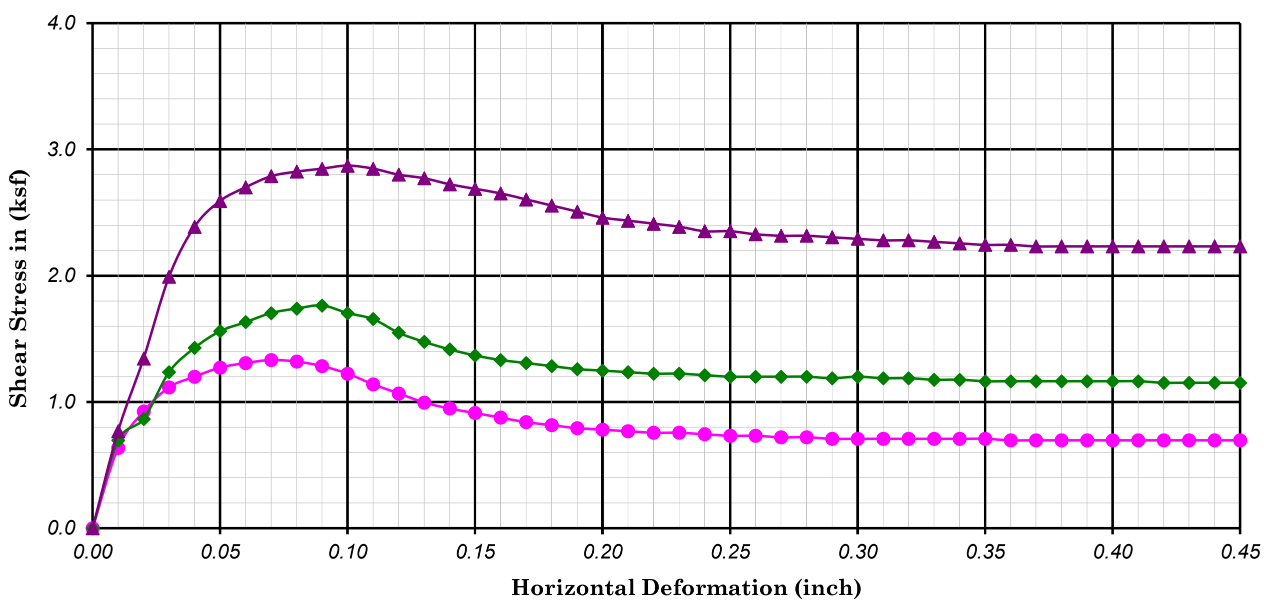
(714) 660-7550 fax

Figure B-2



Ultimate : ○ | Shear Type : Saturated | Undisturbed | Peak : ●



Boring No. : B-2	Strength Intercept (C) :	0.78	(ksf)	Peak	0.16	(ksf)	Ultimate
Sample No. : R-2		37.25	(kPa)		7.47	(kPa)	
Depth (ft/m) : 8.5 / 2.59	Friction Angle (ϕ) :	27.44	Degree		27.29	Degree	

Description : Brown Sandy Clay

Shear Rate (inch/minute) : 0.0002

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m³)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	20.41	114.24	17.98	0.48	1.00	47.88	1.33	63.78	0.70	33.32
◆	20.72	115.88	18.24	0.45	2.00	95.76	1.76	84.46	1.15	55.16
▲	21.48	116.86	18.39	0.44	4.00	191.52	2.87	137.51	2.23	106.87

GROUP DELTA

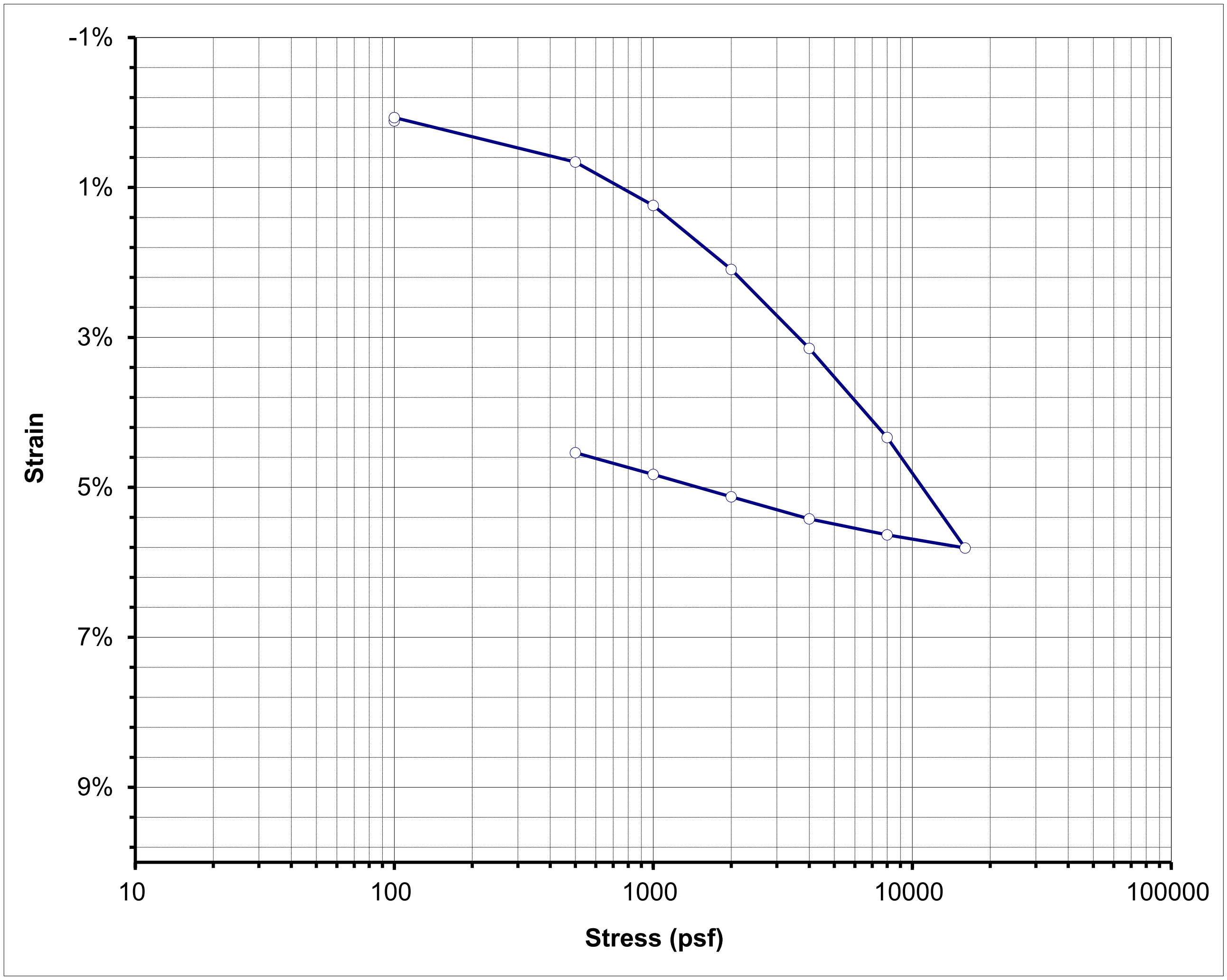
Inglewood Civic Center

Project No. : LA1653 | Date : 06/24/24

DIRECT SHEAR TEST
(ASTM D -3080)

Figure No. : B-3

CONSOLIDATION TEST RESULTS
ASTM D-2435



Boring No.	*B-1*	Sample Depth	*8.5'*
Sample No.	*R-2*	USCS	*SC*

BEFORE TEST

Initial Moisture Content:	*14.17%*	
Initial Dry Unit Wt:	*119.58*	pcf
Initial Total Unit Wt.:	*136.53*	pcf
Initial Void Ratio:	*0.42*	
Initial Degree of Saturation:	*91.3%*	

AFTER TEST

Final Moisture Content:	*13.18%*	
Final Dry Unit Wt:	*125.22*	pcf
Final Total Unit Wt.:	*141.72*	pcf
Final Void Ratio:	*0.36*	
Final Degree of Saturation:	*100.0%*	

Water Added at: *100* psf

ATTERBERG LIMITS					
LL=		PL=		PI=	

PRESSURE (psf)	SAMPLE STRAIN	VOID RATIO
100	*0.11%*	*0.422*
100	*0.07%*	*0.423*
500	*0.66%*	*0.414*
1000	*1.24%*	*0.406*
2000	*2.09%*	*0.394*
4000	*3.15%*	*0.379*
8000	*4.34%*	*0.362*
16000	*5.81%*	*0.341*
8000	*5.63%*	*0.343*
4000	*5.42%*	*0.346*
2000	*5.13%*	*0.351*
1000	*4.83%*	*0.355*
500	*4.54%*	*0.359*

Assumed Specific Gravity of Solids, Gs: *2.73*

PROJECT NUMBER:	*LA1653*	PROJECT NAME:	*Inglewood Civic Center*	FIGURE NO. *B-4*

GROUP DELTA

SAMPLE NO.: SO7173 **SAMPLE DATE:** 6/17/24
SAMPLE LOCATION: B-1 Bulk-1 @ 0 - 5' **TEST DATE:** 6/20/24
SAMPLE DESCRIPTION: Brown Clayey Sand

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	350	350	350			[PSI]
B	INITIAL MOISTURE	6.8	6.8	6.8			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	45.2	55.8	35.5			[ML]
E	WATER ADDED (D*(100+B)/C)	4.0	5.0	3.2			[%]
F	COMPACTION MOISTURE (B+E)	10.8	11.8	10.0			[%]
G	MOLD WEIGHT	2067.9	2065.7	2068.3			[G]
H	TOTAL BRIQUETTE WEIGHT	3303.5	3299.5	3306.0			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1235.6	1233.8	1237.7			[G]
J	BRIQUETTE HEIGHT	2.55	2.65	2.51			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	132.5	126.2	135.9			[PCF]
L	EXUDATION LOAD	5545	3445	7405			[LB]
M	EXUDATION PRESSURE (L/12.54)	442	275	591			[PSI]
N	STABILOMETER AT 1000 LBS	54	68	47			[PSI]
O	STABILOMETER AT 2000 LBS	67	101	50			[PSI]
P	DISPLACEMENT FOR 100 PSI	3.31	3.65	3.02			[Turns]
Q	R VALUE BY STABILOMETER	51	29	65			
R	CORRECTED R-VALUE (See Fig. 14)	51	30	65			
S	EXPANSION DIAL READING	0.0000	0.0000	0.0000			[IN]
T	EXPANSION PRESSURE (S*43,300)	0	0	0			[PSF]
U	COVER BY STABILOMETER	0.46	0.66	0.33			[FT]
V	COVER BY EXPANSION	0.00	0.00	0.00			[FT]

TRAFFIC INDEX:	4.5
GRAVEL FACTOR:	1.53
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	33
R-VALUE BY EXPANSION:	N/A
R-VALUE AT EQUILIBRIUM:	33

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

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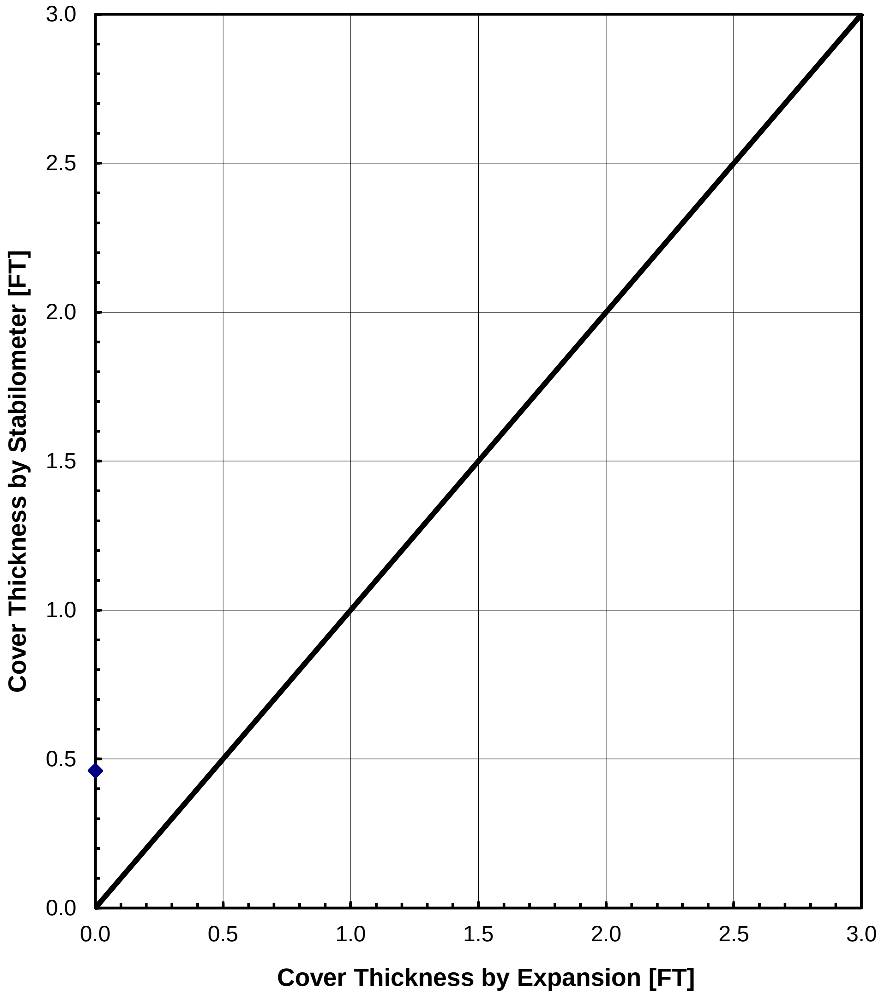
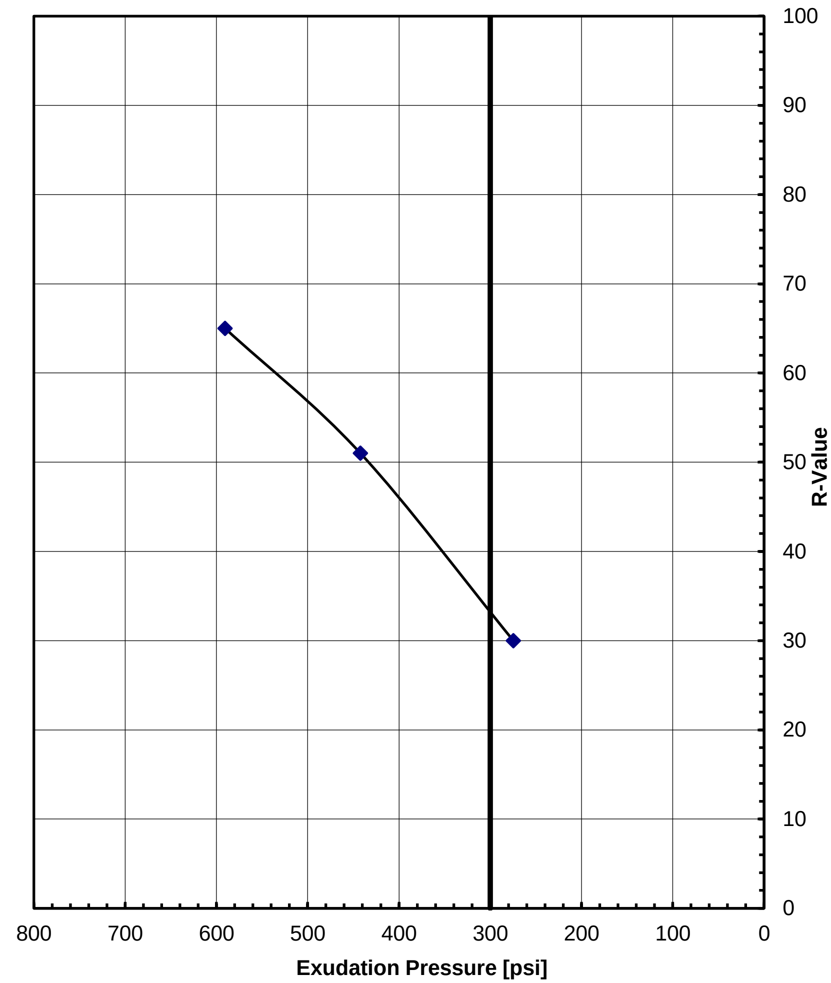
R-VALUE TEST RESULTS
CT301

Inglewood Civic Center
Project No. LA1653
FIGURE B-5A

Sample: SO7173, B-1 Bulk-1 @ 0 - 5'

R-Value at Equilibrium: 33





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COVER AND EXUDATION CHARTS

Project No. LA1653

FIGURE B-5B

GROUP DELTA

EXPANSION INDEX OF SOIL

ASTM D-4829-10 / UBC 29-2

Lab Number: SO7173

Project Name	: Inglewood Civic Center	Sampled By :		Date :	
Project No.	: LA1653	Prepared By :	Eric Y.	Date :	6/19/2024
Boring No.	: B-2	Tested By :	Eric Y.	Date :	6/20/2024
Sample No.	: Bulk-1	Calculated By :	Eric Y.	Date :	6/24/2024
Depth (ft.)	: 0 - 5	Checked By :		Date :	
Description	: Brown Clayey Sand				

Sample Preparation

1 1

Weight of Total Soil	3595.20	Weight of Soil Retained on No. 4 Sieve	9.50	% Passing No. 4 Sieve	99.74

Trail	1	2	3	4	Tested	M & D After Test	
Container No.	SB-2					Container No.	
Weight of Wet Soil + Container (gm)	796.21					Wet Soil+Cont.+Ring	
Weight of Dry Soil + Container (gm)	755.18					Dry Soil+Cont.+Ring	
Weight of Container (gm)	228.52					Wt. of Container	
Moisture Content (%)	7.79				7.79	Moisture Content	
Weight of Wet Soil + Ring (gm)	622.64						
Weight of Ring (gm) No. 2.0	198.61				198.61		
Weight of Wet Soil (gm)	424.03						
Wet Density of Soil (pcf)	127.91					Wet Density (pcf)	
Dry Density of Soil (pcf)	118.66					Dry Density (pcf)	
Precent Saturation of Soil $S_{(Meas.)}$	50.02				50.02	(%) Saturation	

Loading Machine No.			2	
Date	**Reading Time**	**Elapsed Time**	**Dial Reading**	**Expansion**
06/20/24	10:00:00	0:10:00		0.0000
06/20/24				
06/20/24	10:10:00	**0:00:00**	**0.3000**	**0.0000**
Add Distilled Water to Sample				
06/20/24	11:10:00	1:00:00	0.3000	0.0000
06/20/24	12:10:00	2:00:00	0.3000	0.0000
06/20/24	13:10:00	3:00:00	0.3000	0.0000
06/20/24	14:10:00	4:00:00	0.3000	0.0000
06/20/24	15:10:00	5:00:00	0.3000	0.0000
06/20/24	16:10:00	6:00:00	0.3000	0.0000
06/20/24	17:10:00	7:00:00	0.3000	0.0000
06/21/24	7:10:00	21:00:00	0.3000	0.0000
06/21/24	10:10:00	0:00:00	0.3000	0.0000

Remark :

1. Screen sample through **No. 4** *Sieve*

2. Sample should be compacted into a metal ring of the Degree of Saturation of **50 +/- 2% (48 - 52).**

3. Inundated sample in distilled water to 24 h, or until the rate of expansion > (0.0002 in./h), no less than 3 h.

Volume of Mold (ft^3)	0.00731	Specific Gravity	2.70
Rammer Weight (lb.)	5.0	Blows/Layer	15
Vertical Confining Pressure	1.0 (lbf/in^2) / 6.9 (kPa)		

$$(\%)\ S = \frac{S.G. \times W \times Dd}{Wd \times S.G. - Dd}$$

S.G.=Specific Gravity, **W**=Water Content

Dd=Dry Soil Density, **Wd**=Unit Wt. of Water

$$E.I._{(meas)} = \frac{\text{Change in High}}{\text{Initial Thickness}} \times 1000 = \mathbf{0.00}$$

$$\text{Expansion Index}_{(50)} = EI_{(meas.)} - (50 - S_{(meas.)}) \times \frac{65 + EI_{(meas.)}}{220 - S_{(meas.)}}$$

0	Very Low

Expansion Index	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
> 130	Very High

Figure B-6

CORROSIVITY TEST RESULTS
(ASTM D516, CTM 643)

SAMPLE	pH	RESISTIVITY (OHM-CM)	SULFATE CONTENT (%)	CHLORIDE CONTENT (%)
B-1 @ 0 - 5'	*8.01*	*8,110*	*<0.01*	*<0.01*

CORROSIVITY PARAMETERS

SULFATE CONTENT (%)	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	--
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY (OHM-CM)	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT (%)	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive

GROUP DELTA

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Project Name: Inglewood Civic Center
Project Number: LA1653

Figure B-7