

Geotechnical Evaluation S-7 Sewer Lift Station Rehabilitation Long Beach Water Department Long Beach, California

DRP Engineering

2550 W. Main Street, Suite 212 | Alhambra, California 91801

July 15, 2022 | Project No. 211941001



Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation S-7 Sewer Lift Station Rehabilitation Long Beach Water Department Long Beach, California

Mr. Saik-Choon Poh

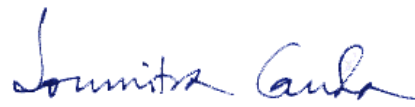
DRP Engineering

2550 W. Main Street, Suite 212 | Alhambra, California 91801

July 15, 2022 | Project No. 211941001


Julianne Padgett, PE
Project Engineer

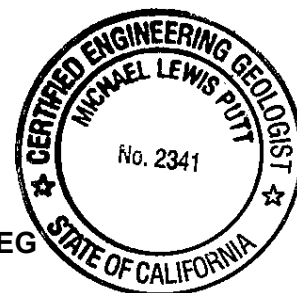



Soumitra Guha, PhD, PE, GE
Principal Engineer



JKP/SG/MLP/sc


Michael Putt, PG, CEG
Principal Geologist



CONTENTS

1	INTRODUCTION	1
2	SCOPE OF SERVICES	1
3	SITE DESCRIPTION AND BACKGROUND	1
4	PROPOSED CONSTRUCTION	2
5	OBSERVATIONS	3
6	ELEVATION SURVEY	3
7	SUBSURFACE EVALUATION AND LABORATORY TESTING	4
8	GEOLOGIC AND SUBSURFACE CONDITIONS	4
8.1	Regional Geology	4
8.2	Site Geology	5
9	GROUNDWATER	5
10	FAULTING AND SEISMICITY	6
10.1	Surface Rupture	6
10.2	Ground Motion	6
10.3	Liquefaction Evaluation	7
10.4	Tsunamis and Seiches	7
11	CONCLUSIONS	8
12	RECOMMENDATIONS	9
12.1	Earthwork	10
12.1.1	Pre-Construction Conference	10
12.1.2	Site Preparation	10
12.1.3	Treatment of Near-Surface Soils	10
12.1.4	Excavation Characteristics	11
12.1.5	Temporary Excavations and Shoring	11
12.1.6	Construction Dewatering	13
12.1.7	Excavation Bottom Stability	13
12.1.8	Fill Material	14
12.1.9	Fill Placement and Compaction	14
12.1.10	Pipe Bedding	15
12.1.11	Modulus of Soil Reaction	15

12.1.12	Lateral Earth Pressures for Thrust Blocks	15
12.2	Seismic Design Considerations	15
12.3	Hardscape	16
12.4	Corrosivity	16
12.5	Concrete Placement	16
12.6	Drainage	17
12.7	Landscaping	17
13	CONSTRUCTION OBSERVATION	17
14	LIMITATIONS	18
15	REFERENCES	20

TABLE

1 – 2019 California Building Code Seismic Design Criteria	15
---	----

FIGURES

1 – Site Location
2 – Boring Locations
3 – Floor Level Survey
4 – Regional Geology
5 – Fault Locations
6 – Seismic Hazard Zones
7 – Lateral Earth Pressures for Braced Excavation Below Groundwater
8 – Lateral Earth Pressures for Temporary Cantilevered Shoring Below Groundwater
9 – Thrust Block Lateral Earth Pressure Diagram

APPENDICES

A – Boring Logs
B – Laboratory Testing
C – Photographs

1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the Long Beach Water Department's (LBWD) S-7 Sewer Lift Station Rehabilitation project (SC-0410) in Long Beach, California (Figure 1). Our services included evaluation of the soil and groundwater conditions at the site and preparation of geotechnical recommendations for the design and construction of the new improvements at the sewer lift station site. We also evaluated the cause of the cracking in the walls and settlement in the upper level of the lift station and exterior sidewalk on the southern side of the building. This report presents our geotechnical findings, conclusions, and recommendations for the project.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of readily available background material, including published geologic maps, fault and seismic hazards maps, groundwater data, topographic maps, stereoscopic aerial photographs, and project-related plans provided by the client.
- Permit acquisition from the City of Long Beach Department of Human and Health Services for drilling into groundwater.
- A field reconnaissance to observe and photograph the existing site conditions, perform a floor level survey of the upper floor and exterior sidewalk, and mark the boring locations for utility clearance by Underground Service Alert.
- Subsurface exploration consisting of the drilling, sampling, and logging of two small-diameter borings to depths ranging from approximately 36½ to 56 feet below the ground surface. The borings were logged by a representative of our firm, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.
- Laboratory testing of representative soil samples to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, Atterberg limits, consolidation, direct shear strength, expansion index, and soil corrosivity.
- Analytical testing of soil samples to characterize drummed materials from our exploratory borings for disposal purposes.
- Compilation and geotechnical engineering analysis of data from our background review, subsurface exploration, laboratory testing, and the results of our relative elevation measurements.
- Preparation of this report presenting our findings, conclusions, and recommendations.

3 SITE DESCRIPTION AND BACKGROUND

The existing S-7 sewer lift station is located in a residential neighborhood at 349 Lakeview Avenue in Long Beach, California (Figure 2). A concrete sidewalk and driveway provide access to the lift

station on the south side of the property. The existing lift station was constructed in 1929 and last rehabilitated in 2000. The existing lift station is a two-level structure, with the upper level being an at-grade building. The lower level sump consists of two dry pits, a 2,600-gallon wet well, submersible pumps and a single 8-inch diameter ductile iron pipe force main. A concrete stairwell provides access to the lower sump from the upper building. The walls of the at-grade building consist of 9-inch unreinforced brick on all four sides (VCA Engineers, 2020). The foundation of the exterior walls for the at-grade building rests partially on grade and partially on the lower level sump structure. The southeast wall is supported on a shallow strip footing on grade, while the remaining walls are supported on the concrete slab that is part of the lower level structure. Three corners of the building foundation are supported with caissons, located at the south, southeast, and northeast corners. The caissons at the south and southeast corners extend approximately 13 feet deep into the native soil. The caisson at the northeast corner is supported on the lower level concrete structure. The bottom of the lower level is approximately 25 feet deep. The basement walls, stair wells, and sump structure are comprised of reinforced concrete.

Based on discussion with DRP, we understand that the upper level of the existing lift station has experienced cracking and spalling on the interior walls on southern side of the lift station as well as settlement and cracking of the exterior walls and sidewalk slab on the southern side. Detailed records of when the cracks and settlement began were not available; however, we understand that they were noted in the last 10 years. We also understand that the project site experiences frequent flooding, and ground settlement around a street sign on the north side of the property has been observed. In 2018, an emergency repair was performed on a leak on the force main, located at the front of the driveway.

A structural assessment was performed for the lift station in 2020, in which VCA Engineers observed interior cracks near the entry door to the upper level and a crack on the interior wall which divides the wet well and stairway wall, approximately 11 feet below the upper floor slab. They also observed that the exterior slab-on-grade sidewalk settled approximately 4 inches at the far end corner of the slab. They concluded that the wall crack near the entry door is a type of structural crack related to the past seismic activities and the brick walls being unreinforced. For the crack on the stairway interior wall, they concluded that it is not likely that the crack developed by the interior water pressure from the well, but from a reinforcement steel volume increase due to rusting from moisture from the wet well side (VCA Engineers, 2020).

4 PROPOSED CONSTRUCTION

LBWD plans to rehabilitate the existing lift station, which may include improvements to equipment, replacing existing pumps, piping repair, wet well restoration, dry well repair work and SCADA

upgrades. We understand that LBWD is also considering installing a second force main for redundancy. Additional improvements related to the new force main may include a new manhole. Plans regarding the new force main were not available at the time of this report. However, based on the depth of the existing force main, we estimate that the new force main will have an invert depth of approximately 8 to 10 feet below the ground surface.

5 OBSERVATIONS

Geotechnical reconnaissance of the subject site was performed by Ninyo & Moore on May 16, 2022. Selected photographs of the site conditions are included in Appendix C. Exterior distress features generally consisted of settlement of the sidewalk slab on the south side of the lift station (Photographs 1 through 3), vertical and horizontal separation of the sidewalk slab from the building (Photographs 4 through 6), a ½-inch-wide crack on the south corner of the building (Photographs 7, 9, and 10), vertical separation between the sidewalk and the ground surface (Photographs 7 and 8), and settlement of the ground surface on the north side of the property (Photographs 11 and 12). Interior distress features generally consisted of a ¼-inch-wide horizontal crack on the upper floor of the lift station, above the doorway on the southeast wall (Photographs 13 and 14), and a horizontal crack extending approximately halfway across the southwest wall of the upper floor (Photographs 15 and 16). We also observed the crack noted by VCA engineers in their structural assessment, located on the interior wall dividing the wet well and stairway wall. Based on discussions with DRP, no cracks or structural issues have been observed within the sump of the lift station.

6 ELEVATION SURVEY

On May 16, 2022, elevation measurements were taken on the upper interior floor and exterior sidewalk slab of the S-7 lift station. The upper interior floor and exterior sidewalk slab were contoured separately. The results of the surveys performed are presented on Figure 3. The relative low area of the upper interior floor in the survey is indicated by the 0.0-inch contour line near the south corner of the building, at the doorway. The relative high area of the upper interior floor is indicated by the 2.5-inch contour line at the northwest side of the floor. The relative low area of the exterior sidewalk slab in the survey is indicated by the 0.0-inch contour line at the south corner of the slab. The relative high area of the exterior sidewalk slab is indicated by the 4.0-inch contour line at the northwest corner of the slab.

The survey results show that the upper interior floor of the lift station slopes approximately 2.5 inches from the north corner towards the south corner (i.e., difference between the 2.5-inch and 0.0-inch contour lines). The survey of the exterior sidewalk slab shows that the slab slopes

approximately 4.0 inches from the northwest corner to the south corner (i.e. from the doorway to the southernmost corner of the sidewalk slab).

7 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on May 20, 2022, and consisted of drilling, logging, and sampling of two small-diameter borings using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers. The borings were drilled to depths ranging from approximately 36½ to 56 feet below the ground surface. The borings were logged by a representative from our firm and bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The borings were backfilled with cement-bentonite grout in accordance with our City of Long Beach well permit. The approximate locations of the borings are presented on Figures 2 and 3. The boring logs are presented in Appendix A.

Laboratory testing was performed to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, Atterberg limits, consolidation, direct shear strength, expansion index, and soil corrosivity. The results of in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory tests are presented in Appendix B.

For the purpose of characterizing the auger cuttings prior to disposal off-site, composite samples of the soils were collected in glass jars, placed into a chilled container, and submitted to SunStar Laboratories, a state-certified laboratory, for analysis. The soil samples were analyzed for the presence of Total Petroleum Hydrocarbons, Title 22 Metals, and volatile organic compounds in general accordance with United States Environmental Protection Agency Methods 8015M, 6010B/7471A, and 8260B, respectively. The soil samples were classified as non-hazardous waste and the drums were disposed of at a legal dump facility.

8 GEOLOGIC AND SUBSURFACE CONDITIONS

8.1 Regional Geology

The subject site is located within the Los Angeles Basin in the Transverse Ranges geomorphic province of southern California (Norris and Webb, 1990), which is characterized by east to west trending mountain ranges and intervening valleys. The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent fault systems. The site is located within the Southwestern block, the majority of which is beneath the Pacific Ocean. The exposed part of the Southwestern block mostly consists of a low plain that is approximately 28-miles long and extends from Santa Monica at the northwest to Long Beach at the Southeast. Topographically,

the site is relatively flat with an elevation of approximately 10 feet above mean sea level (United States Geological Survey [USGS], 2021).

Published geologic maps of the area indicate that the site is underlain by artificial fill that was placed over Holocene-age unconsolidated shelf sediments. The artificial fill includes compacted engineered and non-compacted, non-engineered fill associated with the Alamitos Bay Marina Center. The unconsolidated shelf deposits are described as consisting of unconsolidated sand and silt (Saucedo et al., 2016) (Figure 4). However, based on the findings from our subsurface exploration further described in the following section, we did not encounter significant amounts of artificial fill in our borings and the fill associated the Alamitos Bay Marina Center construction may not have encroached into the area of the lift station.

8.2 Site Geology

The materials encountered in our exploratory borings consisted of relatively small amount of fill and marine shelf sediments. Fill associated with previous site grading was encountered from the ground surface to a depth of approximately 2 feet. The fill generally consisted of moist, very loose to loose, clayey sand. Marine shelf sediments were encountered beneath the fill to the total depths explored of up to approximately 56 feet. The shelf sediments generally consisted of moist to wet, soft to very stiff, fat clay and lean clay, and wet, loose to very dense, sandy silt, silty sand, poorly graded sand, and poorly graded sand with silt. More detailed descriptions of the subsurface materials encountered during our exploration are presented on the boring logs in Appendix A.

9 GROUNDWATER

Groundwater was encountered in our exploratory borings during drilling at depths ranging from approximately 8 to 10 feet below the ground surface. The groundwater depth encountered during drilling is not considered a stabilized water level.

Regional maps indicate that the historic high groundwater at the site is at approximately 10 feet below the ground surface (California Division of Mines and Geology [CDMG], 1998). In addition, seepage and perched groundwater may occur at relatively shallow depths and are commonly associated with coarse structural backfill or pipe bedding zone materials. Due to the relatively close proximity of the site to Alamitos Bay, the depth to groundwater may vary due to tidal fluctuations and should be planned for by the dewatering contractor. Fluctuations in the level of groundwater in the project area will also occur due to relatively slow rate of seepage in clay, variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors which may not have been evident at the time of field evaluation.

10 FAULTING AND SEISMICITY

The project site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (California Geological Survey [CGS], 2018). However, the site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed improvements. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the CGS, active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major faults in the site vicinity and their geographic relationship to the site are shown on Figure 5. The nearest active fault is the Newport Inglewood fault, located approximately ½ mile northeast of the site (USGS, 2008).

The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, and tsunamis and seiches. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

10.1 Surface Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

10.2 Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration that corresponds to the MCE_R for the project area was calculated as 0.620g using the 2022 Applied Technology Council (ATC) seismic design tool (web-based).

Spectral response acceleration parameters, consistent with the 2019 CBC, are also provided in Section 13.2 for the evaluation of seismic loads on buildings and other structures.

The 2019 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the mapped maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA_M) with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard. The MCE_G PGA is based on the geometric mean PGA with a 2 percent probability of exceedance in 50 years. The PGA_M was calculated as 0.748g using the 2022 ATC seismic design tool.

Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R , 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures. A site-specific ground motion hazard analysis was beyond the scope of this evaluation. Furthermore, the planned improvements do not include new buildings or structures for human occupancy.

10.3 Liquefaction Evaluation

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The State of California Seismic Hazard Zone Map (CDMG, 1999) indicates that the site is located within an area considered to be susceptible to seismically induced liquefaction (Figure 6). However, the proposed improvements do not include new structures for human occupancy and evaluation of liquefaction potential at the site is not within the scope of our services for this study.

10.4 Tsunamis and Seiches

Tsunamis are long wavelength, seismic, sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Seiches are waves generated in a large, enclosed body of water. According to a

2009 State of California tsunami inundation map, the site is mapped within an area considered susceptible to tsunami inundation (California Emergency Management Agency, 2009). Therefore, tsunami related damage may occur during the design life of the project.

11 CONCLUSIONS

The purpose of our study was to evaluate the cause of the cracking and settlement in the exterior sidewalk and upper level of the lift station, as well as provide recommendations for the construction of a new force main. Our study included review of background materials, geotechnical reconnaissance, relative elevation survey, subsurface exploration including two borings, laboratory testing, and geotechnical analysis.

Based on our evaluation, we conclude that the settlement and cracks in the south side of the upper building were caused by long-term consolidation settlement of the soft to very stiff clay soils underlying the building footings. The caissons on the south side of the building, which have experienced the most settlement, extend to a depth of 13 feet and are supported on native fat clay. The north side of the building is supported on the lower level sump structure, which extends to a depth of approximately 25 feet into a sandy silt layer. Long-term differential settlement due to the different foundation structures could contribute to the cracks and settlement in the upper building. Our laboratory consolidation test results indicate that after almost 100 years since the construction of the lift station, most of the long-term consolidation settlement of the existing building has already taken place.

In general, the following conclusions were made:

- The site is underlain by fill and marine shelf deposits. The fill generally consisted of moist, very loose to loose, clayey sand. The shelf deposits generally consisted of moist to wet, soft to very stiff, fat clay and lean clay, and wet, loose to very dense, sandy silt, silty sand, poorly graded sand, and poorly graded sand with silt.
- Excavations into the soils at the site should be feasible with earthmoving equipment in good working order. We anticipate that excavated site soils will not be suitable for use as fill.
- On-site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications. Temporary shoring should be provided in accordance with OSHA regulations. Excavations that extend below groundwater will be unstable.
- Groundwater was measured after completion of drilling at approximately 8 feet below the ground surface. Historical high groundwater levels at the project site have been mapped at a depth of approximately 10 feet below the ground surface. Variations in the depth to groundwater are anticipated to occur due to tidal fluctuations and the relatively slow rate of seepage in clay. Groundwater and seepage should be anticipated and planned for by the contractor during construction. Dewatering of temporary excavations that extend to the groundwater table will be involved in order to construct the proposed improvements under a dry condition.

- The project is located within an area mapped by the State of California (CDMG, 1999), as being susceptible to earthquake-induced liquefaction. Soil liquefaction during a significant earthquake event could result in pipeline damage.
- The site is not located within a State of California Earthquake Fault Zone (CGS, 2018). Based on our review of the published geologic maps and aerial photographs, no known active or potentially active faults transect the site. The potential for surface fault rupture at the site is considered low.
- Our limited laboratory test results indicate that the near-surface site soils can be classified as corrosive based on California Department of Transportation (Caltrans, 2021) corrosion guidelines.
- The walls of the upper building are constructed of unreinforced brick, and past seismic activity since the construction of the building in 1929 could have caused cracking in the walls.
- We conclude that the settlement of the exterior sidewalk slab was caused by flooding of the site and poor drainage, poor subsurface soil conditions (very loose clayey sand and fat clay), and possible overwatering of the nearby landscaping.

12 RECOMMENDATIONS

The following sections present our geotechnical recommendations for construction of the proposed force main and related improvements, new exterior concrete sidewalk, and establishment of site drainage. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the proposed construction. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

Due to the relatively minor amount of remaining anticipated future settlement and the age of the structure, extensive foundation recommendations are not warranted. However, it is imperative that appropriate surface drainage be established and maintained to reduce potential water infiltration into the subsurface soils. Our recommendations are intended to reduce the potential for future settlement of exterior sidewalks and the foundation of the building.

Due to the cracking and settlement of the exterior sidewalk slab, we recommend that the existing sidewalk be removed, and the soil beneath be overexcavated and replaced with clean, non-expansive structural backfill. The subgrade and compacted fill should be prepared in accordance with the recommendations presented in the Earthwork section of this report.

Drainage improvements should be performed to mitigate the infiltration of surface water into the subsurface soils at the site. Drainage improvements should include surficial regrading to facilitate positive flow away from the building and hardscape, correcting areas of ponding, and connecting roof drain downspouts to closed drainage systems that outlet away from the building. We further

recommend that the drainage conditions be evaluated and recommendations be provided by a qualified Civil engineer.

12.1 Earthwork

Earthwork at the site is anticipated to consist of site clearing, excavations and trenching of approximately 8 to 10 feet deep to construct the new force main and any related improvements, remedial grading of the near-surface soils for new hardscape, and finish grading for establishment of site drainage. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections.

12.1.1 Pre-Construction Conference

We recommend that the project plans be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. We further recommend that a pre-construction conference be held in order to discuss the recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

12.1.2 Site Preparation

Prior to performing the site excavations, the project area should be cleared of pavements and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soils. Materials generated from the clearing operations should be removed from the project site and disposed at a legal dumpsite.

12.1.3 Treatment of Near-Surface Soils

In order to provide suitable support for new exterior flatwork, we recommend that the soil beneath flatwork be overexcavated and replaced with clean, non-expansive structural backfill. The limits of the excavation should extend 1 foot laterally beyond the outside edge of the flatwork. The overexcavation should consist of 1) removing 2 feet of soil below the planned flatwork subgrade elevation, 2) scarifying the exposed remedial grading bottom to a depth of 8 inches, moisture-conditioning to slightly above the optimum moisture content, and recompacting the soil to 90 percent relative compaction as evaluated by ASTM International (ASTM) test method D 1557, and 3) replacing the excavated soils with clean, non-expansive, structural backfill. Due to the relatively high moisture contents of the subgrade soils, drying-

back of the subgrade is anticipated. This may involve scarifying the bottom and leaving the scarified bottom exposed for several days to allow the moisture content to dissipate. The backfill should be moisture conditioned to slightly above the optimum moisture content and compacted to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. The depth of removals may be deeper and should be evaluated in the field by Ninyo & Moore's representative based on the materials exposed.

12.1.4 Excavation Characteristics

Based on our subsurface exploration at the site, we anticipate that excavations in the undocumented fill and shelf deposits may be accomplished with earthmoving equipment, including backhoes, excavators, or other trenching/excavating equipment in good working condition. We anticipate that the materials excavated at the site will be predominantly moist, loose, clayey sand and moist to wet, firm to stiff, fat clay.

Excavations close to or below the groundwater (before or after dewatering) are anticipated to encounter wet and potentially loose or soft ground conditions. Unsupported excavations will be prone to caving. Wet soils may be subject to pumping under heavy equipment loads.

12.1.5 Temporary Excavations and Shoring

Temporary near-vertical excavations above groundwater and not exceeding a depth of approximately 4 feet should be feasible where existing structures that may surcharge the excavations are not present. However, excavations that expose friable sand with low cohesion may be subject to caving. In particular, bedding materials for existing pipelines, if encountered, may be prone to caving. Excavations that are unstable or deeper than 4 feet should be sloped at an inclination no steeper than 1½:1 (horizontal to vertical). Some surficial sloughing may occur, and temporary excavations should be evaluated in the field in accordance with OSHA guidelines. The surficial soils should be considered as OSHA Soil Type C, and temporary excavations should conform with OSHA regulations. Temporary slope surfaces should be kept moist to retard raveling and sloughing. Water should not be allowed to flow over the top of excavations in an uncontrolled manner. Stockpiled material and/or equipment should be kept back from the top of excavations a distance equivalent to the depth of the excavation or more. Temporary excavations should be observed by the geotechnical consultant so that appropriate additional recommendations necessitated by actual field conditions may be provided. Temporary excavations are time sensitive, and failures are possible.

Shored trenches are anticipated for the construction of the new force main due to anticipated trench instability, shallow groundwater, and to reduce the potential for damage to adjacent utilities and improvements that will be protected during construction activities. Appropriate shoring systems may include cantilevered shoring, shoring with internal bracing, driven sheet piles, or similar systems. Where excavations extend below groundwater, the shoring systems should be designed to act as a cofferdam to cut-off groundwater flow from outside of the excavations and it is recommended that dewatering be performed from inside the excavations to reduce the potential for ground settlement. Sheet piles will act as a cutoff wall for excavations extending below groundwater, but they will not completely seal the excavation from groundwater. Reduced production rates and/or modified shoring systems may be appropriate where caving is severe, groundwater is encountered, and/or adjacent utilities are at risk. Shoring installed in advance of trenching should be provided for excavations below groundwater and where damage to existing improvements is at risk. The shoring system should be designed using the lateral earth pressure values shown on Figures 7 and 8 for braced and cantilevered excavations, respectively. The recommended design pressures are based on the assumption that the shoring system is constructed without raising the ground surface elevation behind the shored sidewalls of the excavations, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls.

The selection of shoring systems and construction installation should also consider the protection of adjacent improvements. Soils with low cohesion have a potential for caving and shoring systems should be installed and removed such that adjacent improvements are not left unsupported.

We anticipate that settlement of the ground surface will occur behind the shored excavations. The amount of settlement depends heavily on the type of shoring system, the contractor's workmanship, and soil conditions. To reduce the potential for distress to adjacent improvements, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to ½ inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring elements, excavation for storm drain, construction vibrations, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during

construction if beam and/or sheet pile driving is performed within 50 feet from existing structures.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make the appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

12.1.6 Construction Dewatering

Dewatering is anticipated for the construction of the proposed improvements so that work can be performed in a dry condition. Based on our subsurface evaluation, the groundwater level measured at completion of drilling was approximately 8 feet below the ground surface. Shallower depths to groundwater due to relatively slow rate of seepage in clay and tidal fluctuations should also be anticipated and planned for by the dewatering contractor.

The dewatering scheme may include pumping of groundwater from well points within or outside of the shoring. Dewatering design and pump selection should be performed by a specialty dewatering contractor. Lowering the water table may result in an increase in effective stresses and may induce settlement of the soils underlying adjacent utilities and/or structures. Accordingly, we recommend that dewatering be performed from inside the shoring system and the groundwater level be lowered no more than 3 feet below the excavation.

Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board and the National Pollutant Discharge Elimination System's (NPDES's) screening levels for discharge. Design of the groundwater control system is the responsibility of the contractor. Prior to dewatering, water quality testing should be performed for acceptance to discharge.

During dewatering, groundwater should be monitored and tested for potential contaminants from off-site sources. Details regarding the monitoring and testing frequency should be based on the contractors' means and methods as well as the Soil and Groundwater Management Plan.

12.1.7 Excavation Bottom Stability

Excavations close to or below the groundwater (before or after dewatering) will encounter wet and loose or soft ground conditions. Excavations that expose loose/soft soils or encounter seepage or groundwater, or that become disturbed during excavation, may be

unstable. In general, unstable bottom conditions may be mitigated by over-excavating to a depth of approximately 1 to 2 feet below the proposed subgrade and replacing the excavated soil with crushed aggregate base or gravel wrapped in geofabric. If aggregate base is used, it should consist of either Caltrans Class II base or crushed miscellaneous base. Caltrans Class II base should conform to the State of California Standard Specifications, Section 26 1.02A. Crushed miscellaneous base should conform to the Standard Specifications for Public Works Construction, Section 200 2.4. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by a Ninyo & Moore representative at the time of construction.

12.1.8 Fill Material

The onsite clay soils are not suitable for reuse as fill, including trench backfill. Import fill will be needed for support of new exterior hardscape, trench backfill, and general fill. Import soils to be placed as fill should be free of rocks or lumps of material in excess of 3 inches in diameter. Oversize cobbles or hard lumps larger than 3 inches in diameter should be broken into smaller pieces (less than 3 inches in diameter) or removed from the site. The near-surface clayey soils are not anticipated to meet the “Greenbook” criteria for structure backfill.

Imported fill material should consist of clean, non-expansive, granular material, which conforms to the “Greenbook” for structure backfill. “Non-expansive” can be defined as soil having an expansion index of 20 or less in accordance with ASTM test method D4829. The imported materials should also meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.15 percent (1,500 ppm) or less, a pH value of 5.5 or higher, or an electrical resistivity of 1,500 ohm-centimeters or more). Materials for use as fill should be evaluated by Ninyo & Moore prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

12.1.9 Fill Placement and Compaction

Fill material, including trench backfill, should be moisture-conditioned and compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. Fill material should be moisture-conditioned to slightly above the laboratory optimum moisture content. The lift thickness for fill soils will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Special care should be exercised to avoid damaging pipes during compaction of trench backfill. Placement and compaction of the fill soils should be in general accordance with applicable grading ordinances and good construction practice.

12.1.10 Pipe Bedding

We recommend that pipes be supported on 6 inches or more of granular bedding material. Bedding material should be placed around pipe zones to 12 inches or more above the top of the pipes in accordance with the current “Greenbook” Standard Specifications for Public Works. The bedding material should be classified as sand, should be free of organic material, and have a sand equivalent (SE) of 30 or more. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently. Trench backfill, including bedding material, should be placed and compacted with mechanical equipment in accordance with the recommendations presented in the Earthwork section of this report.

12.1.11 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1,000 pounds per square inch be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

12.1.12 Lateral Earth Pressures for Thrust Blocks

Thrust restraint for pipes under pressure may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the lateral passive earth pressures presented on Figure 9.

12.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with the CBC (2019) guidelines and adjusted MCE_R spectral response acceleration parameters (ATC, 2022).

Table 1 – 2019 California Building Code Seismic Design Criteria	
Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.739
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	1.551g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.561g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.551g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.976g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.034g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.650g
Site Modified Peak Ground Acceleration, PGA_M	0.748g

12.3 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a minimum thickness of 4 inches and be reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab. The hardscape should be underlain by 6 inches of granular material such as Crushed Aggregate Base or Crushed Miscellaneous Base and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

12.4 Corrosivity

Laboratory testing was performed on a representative sample of near-surface soil to evaluate pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix B.

The soil pH was measured at approximately 7.4 and the electrical resistivity was measured to be approximately 969 ohm-cm. The chloride content of the sample was measured to be approximately 155 ppm. The sulfate content of the tested sample was approximately 0.017 percent (170 ppm). Based on the laboratory test results and Caltrans (2021) criteria, the project site can be classified as a corrosive site, which is defined as having earth materials with a chloride concentration of 500 ppm or greater, sulfate concentration of 1,500 ppm or greater, a pH of 5.5 or less, or an electrical resistivity of 1,500 ohm-cm or less.

12.5 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the CBC (2019), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using Caltrans Test Method 417, indicates a water-soluble sulfate content of 0.017 percent by weight (i.e., 170

ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the on-site soils, consideration should be given to using Type II/V cement for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

12.6 Drainage

Positive surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage is defined as a slope of 5 percent or more (2 percent or more if paved) for a distance of 10 feet or more away from foundations. Runoff should then be transported by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to foundations or on pavements. Area drains for landscaped and paved areas are recommended.

12.7 Landscaping

Project landscaping should consist of drought tolerant plants. Landscape irrigation should be kept to a level just sufficient to maintain plant vigor. Over-watering should not be permitted.

13 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the subsurface conditions during construction. Observation and testing of compacted fill and backfill should also be performed by the project geotechnical consultant during construction. We recommend that Ninyo & Moore review the project plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report may be revised or modified.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing clearing, grubbing, and removals.

- Observing remedial grading bottoms.
- Observe the placement and compaction of fill and trench backfill.
- Evaluating imported materials prior to their use as fill.
- Performing field tests to evaluate fill compaction.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report.

14 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analysis presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

15 REFERENCES

- American Concrete Institute, 2016, ACI Manual of Concrete Practice.
- American Concrete Institute, 2019, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19).
- American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads and Associated Criteria for Building and Other Structures, ASCE Standard 7-16.
- ASTM International (ASTM), 2019, Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.
- Applied Technology Council (ATC), 2022, Hazards by Location, <https://hazards.atcouncil.org/>
- California Building Standards Commission, 2019, California Building Code (CBC): California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, dated July 1.
- California Department of Conservation, Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California: Seismic Hazard Zone Report 028.
- California Department of Conservation, Division of Mines and Geology (CDMG), 1999, Seismic Hazard Zones Official Map, Long Beach Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-19, dated March 25.
- California Department of Transportation, 2021, Corrosion Guidelines, Version 3.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch, Version 3.2, dated May.
- California Emergency Management Agency, California Geological Survey, University of Southern California, 2009, Tsunami Inundation Map for Emergency Planning, Long Beach Quadrangle, Scale 1:24,000, dated March 1.
- California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, dated September 11.
- California Geological Survey (CGS), 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42.
- City Engineers Office, Long Beach, California, Sewer Department, 1929, Belmont Park Pump Station, Drawing No. B-622, dated July.
- Google Earth, 2022, Website: <http://earth.google.com>.
- Historical Aerials, 2022, www.historicaerials.com.
- Long Beach Water Department (LBWD), 2021, Request for Proposal (RFP) to Provide Engineering Services for the S-7 Sewer Lift Station Rehabilitation Project (SC-0410), dated September 2.
- Ninyo & Moore, 2021, Proposal for Geotechnical Consulting Services, S-7 Sewer Lift Station Rehabilitation Project (SC-0410), Long Beach Water Department, Long Beach, California, Proposal No. 04-03394, dated September 27.
- Norris, R.M., and Webb, R.W., 1990, Geology of California, Second Edition: John Wiley & Sons.
- Public Works Standard, Inc., 2018, The “Greenbook”: Standard Specifications for Public Works Construction, with Errata No. 1, dated 2019: BNI Building News, Vista, California.
- Saucedo, G.J., Greene, H.G., Kennedy, M.P., and Bezore, S.P., 2016, Geologic Map of the Long Beach 30' x 60' Quadrangle, California, Version 2.0, Scale 1:100,000.

- Tran Consulting Engineers, 2000, Plans for Construction of Rehabilitation of S-4, S-7, and S-11 Sewer Lift Stations, Long Beach Water Department, Long Beach, California, WD-22-00, dated August.
- United States Geological Survey (USGS), 2008, National Seismic Hazard Maps, http://geohazards.usgs.gov/cfusion/hazfaults_search/hf_search_main.cfm.
- United States Geological Survey (USGS), 2021, USGS US Topo 7.5-Minute Map for Long Beach, CA: USGS - National Geospatial Technical Operations Center (NGTOC).
- United States Geological Survey and California Geological Survey, 2022, Quaternary fault and fold database for the United States, accessed June 16, at: <https://www.usgs.gov/natural-hazards/earthquake-hazards/faults>.
- VCA Engineers, Inc., 2020, Preliminary report for Pump House S-7 structural assessment at 349 Lakeview Avenue, Long Beach, CA, Long Beach Water Department, dated November 11.



FIGURES



Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

S-7 SEWER LIFT STATION REHABILITATION
349 LAKEVIEW AVENUE
LONG BEACH, CALIFORNIA
211941001 | 7/22

211941001_BLD.dwg 06/21/2022 JDP



LEGEND

B-2
TD=36.5



BORING:
TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: GOOGLE EARTH, 2022.

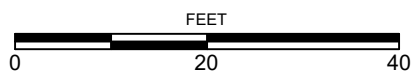


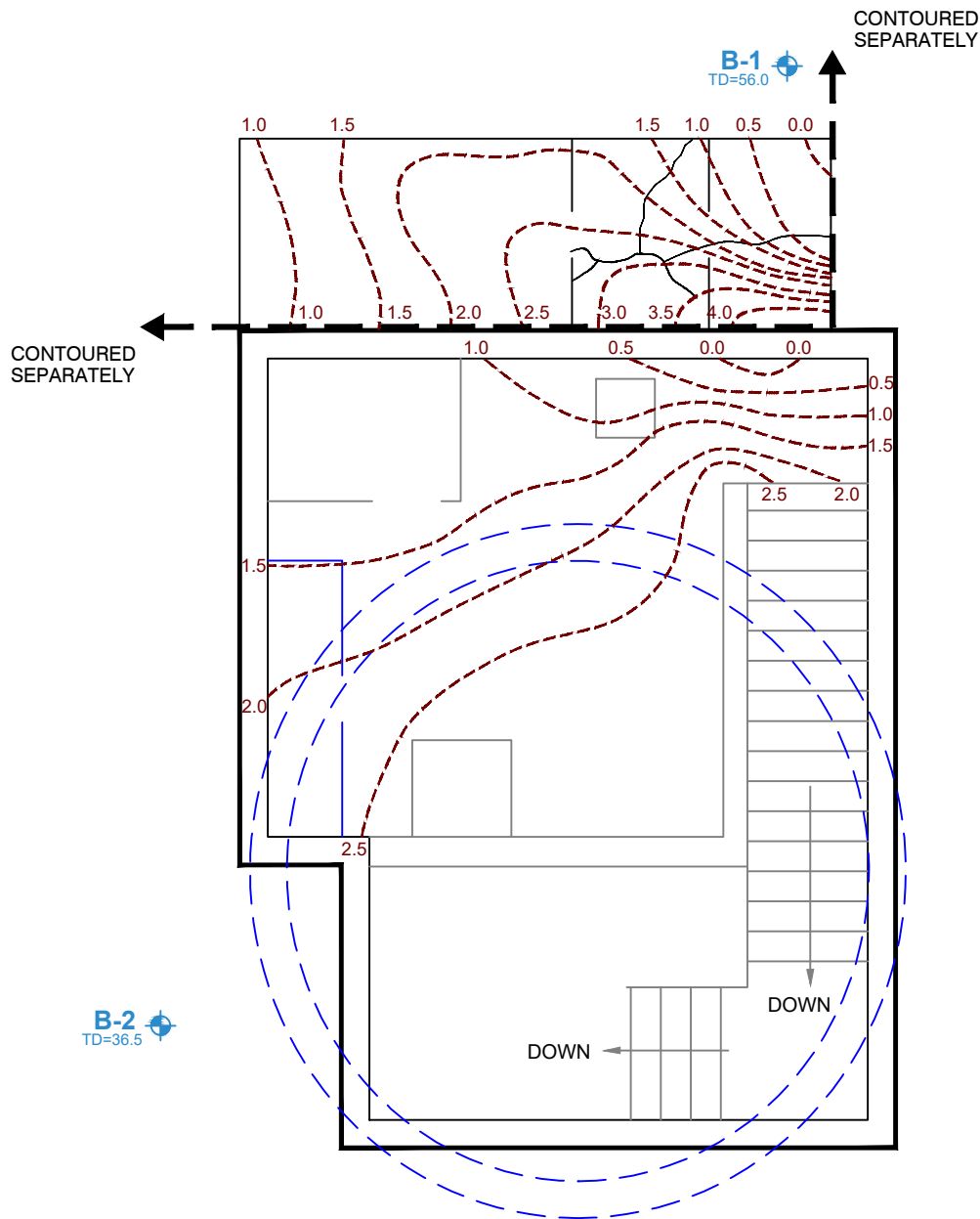
FIGURE 2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

BORING LOCATIONS

S-7 SEWER LIFT STATION REHABILITATION
349 LAKEVIEW AVENUE
LONG BEACH, CALIFORNIA
211941001 | 7/22

211941001_FLS.dwg 07/08/2022 GK, JDP



LEGEND

--- 2.5 --- RELATIVE FLOOR ELEVATION IN INCHES

— RELATIVE FLOOR ELEVATION IN INCHES

B-2 TD=36.5 BORING;
TD=TOTAL DEPTH IN FEET

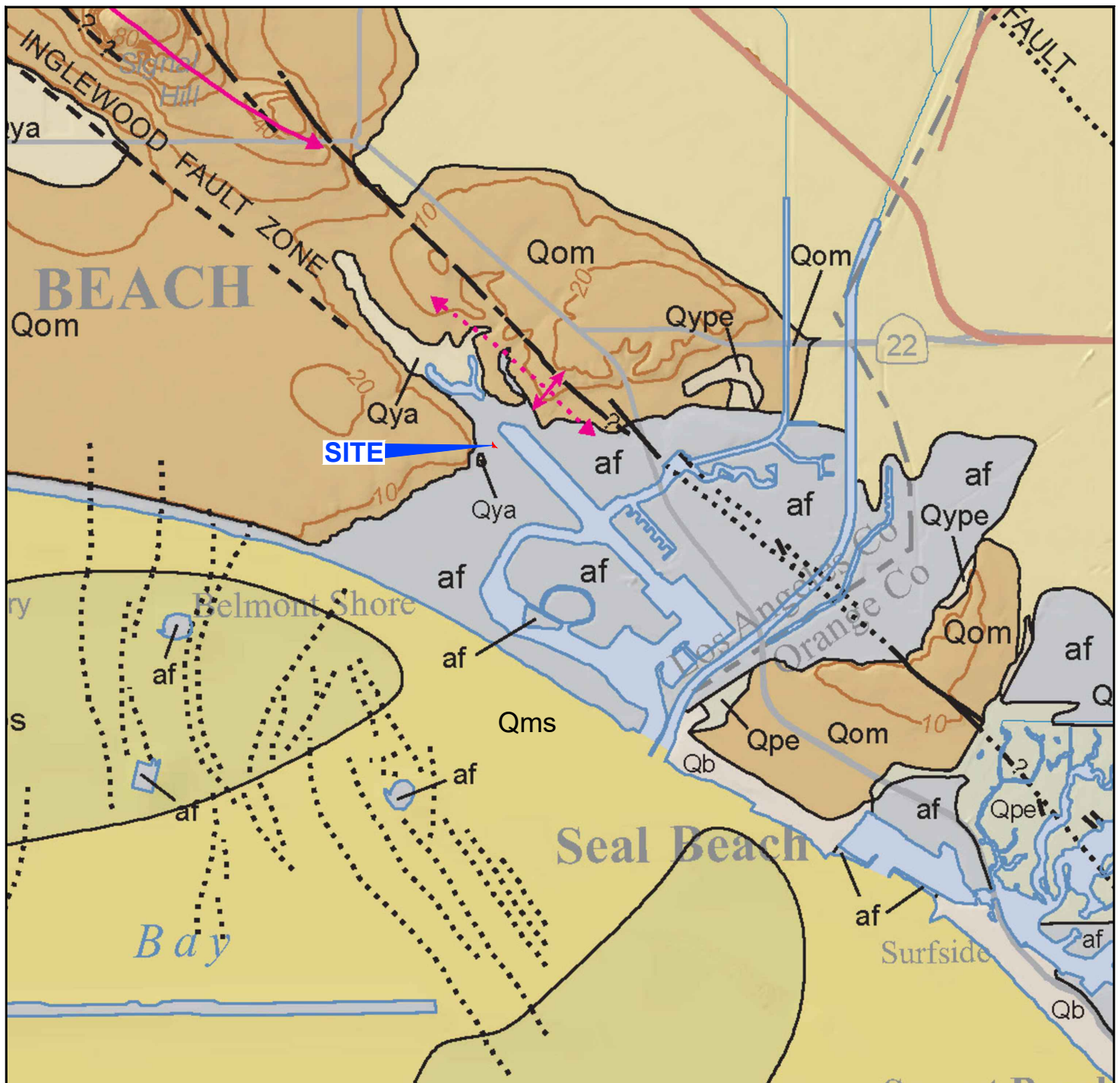
SUMP

DATE SURVEYED: MAY 16, 2022

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.



FIGURE 3



LEGEND

af	ARTIFICIAL FILL	Qms	SHELF SEDIMENTS
Qya	YOUNG ALLUVIUM	—	GEOLOGIC CONTACT
Qom	OLD SHALLOW MARINE DEPOSITS	— • —	FAULT

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: SAUCEDO, ET AL., 2016.

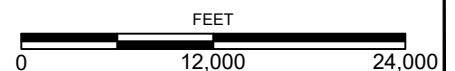


FIGURE 4

REGIONAL GEOLOGY

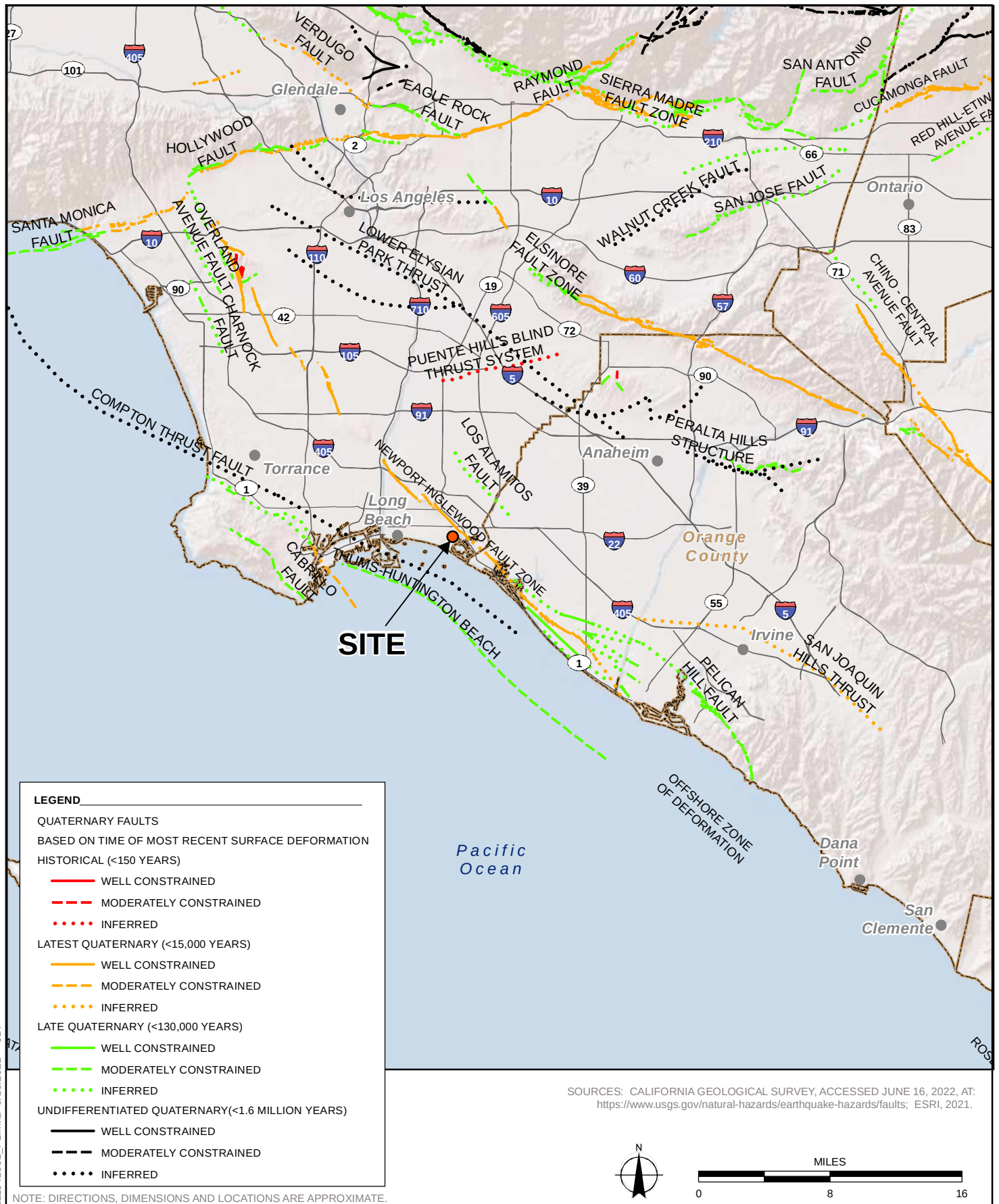
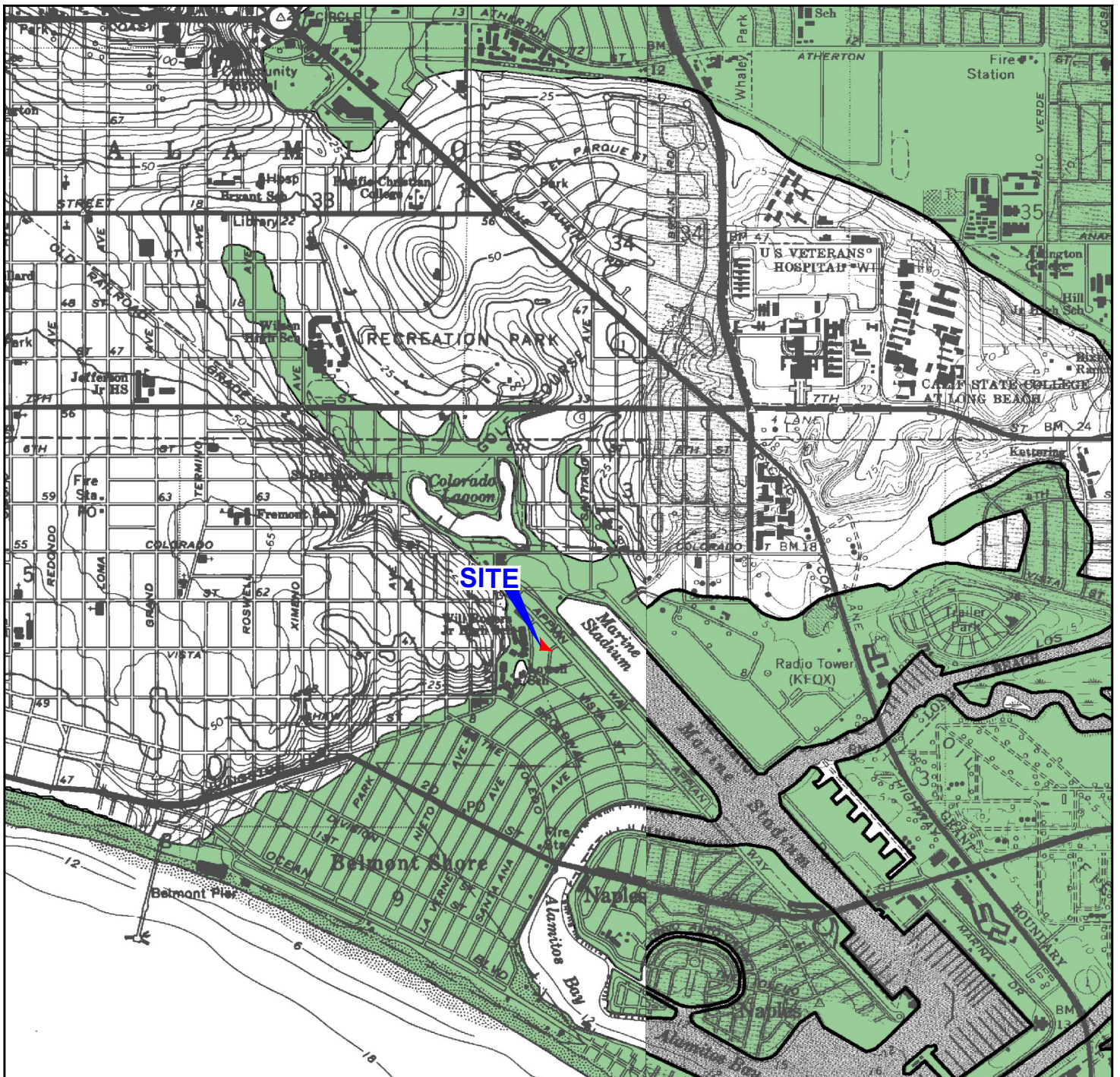


FIGURE 5

FAULT LOCATIONS

S-7 SEWER LIFT STATION REHABILITATION
 349 LAKEVIEW AVENUE
 LONG BEACH, CALIFORNIA
 211941001 | 7/22



LEGEND

LIQUEFACTION



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

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: CDMG, 1999.

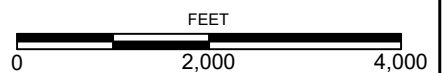
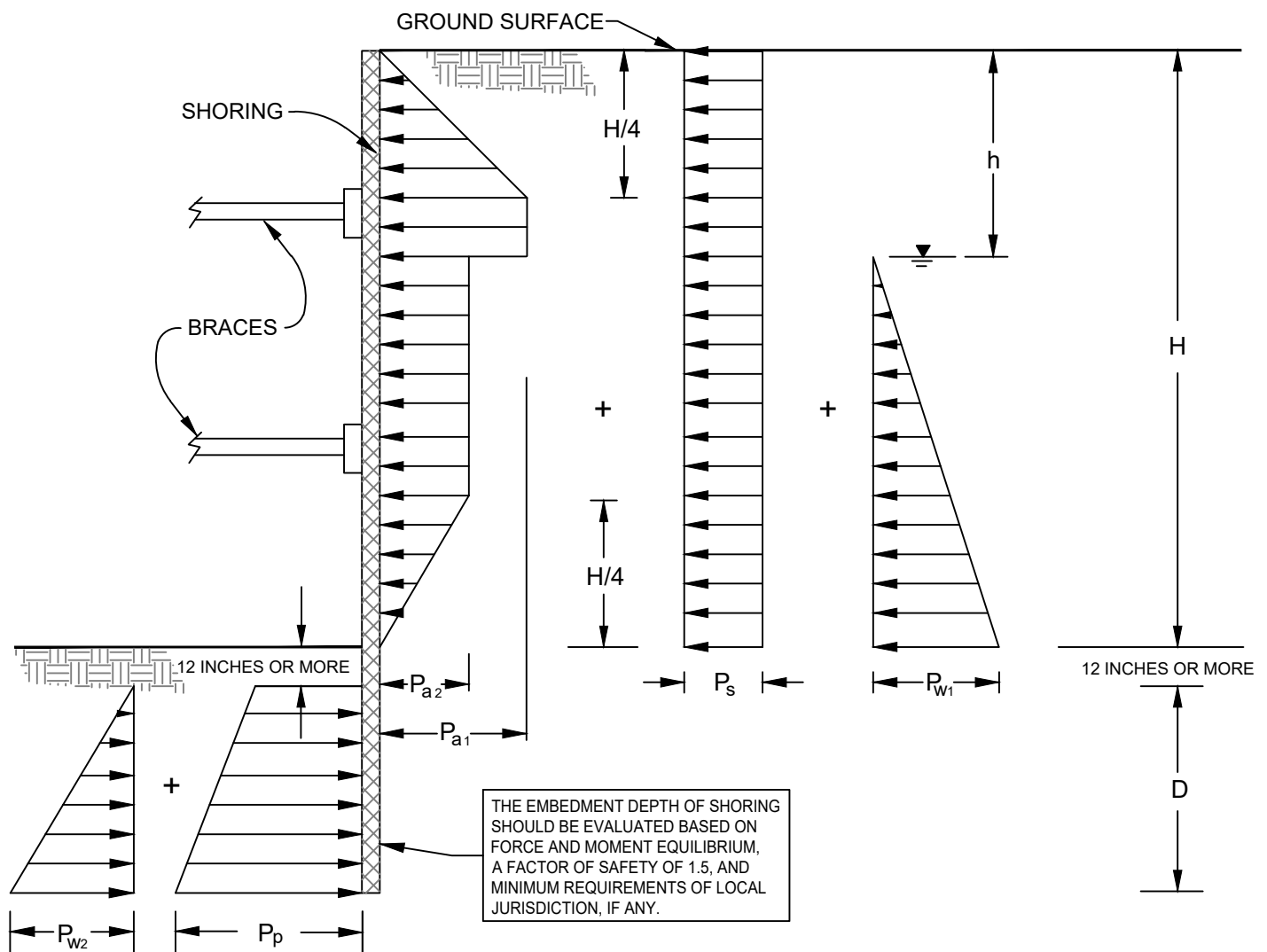


FIGURE 6



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
 $P_{a1} = 48H$ psf; $P_{a2} = 23H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 120$ psf
3. HYDROSTATIC PRESSURE, P_{w1} & P_{w2}
 $P_{w1} = 62.4(H - h)$ psf
 $P_{w2} = 62.4D$ psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 57.6D + 1000$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H , h AND D ARE IN FEET
7. GROUNDWATER TABLE
8. P_{w2} IS APPLICABLE WHEN DEWATERING IS TO BE PERFORMED FROM INSIDE OR OUTSIDE OF THE EXCAVATION

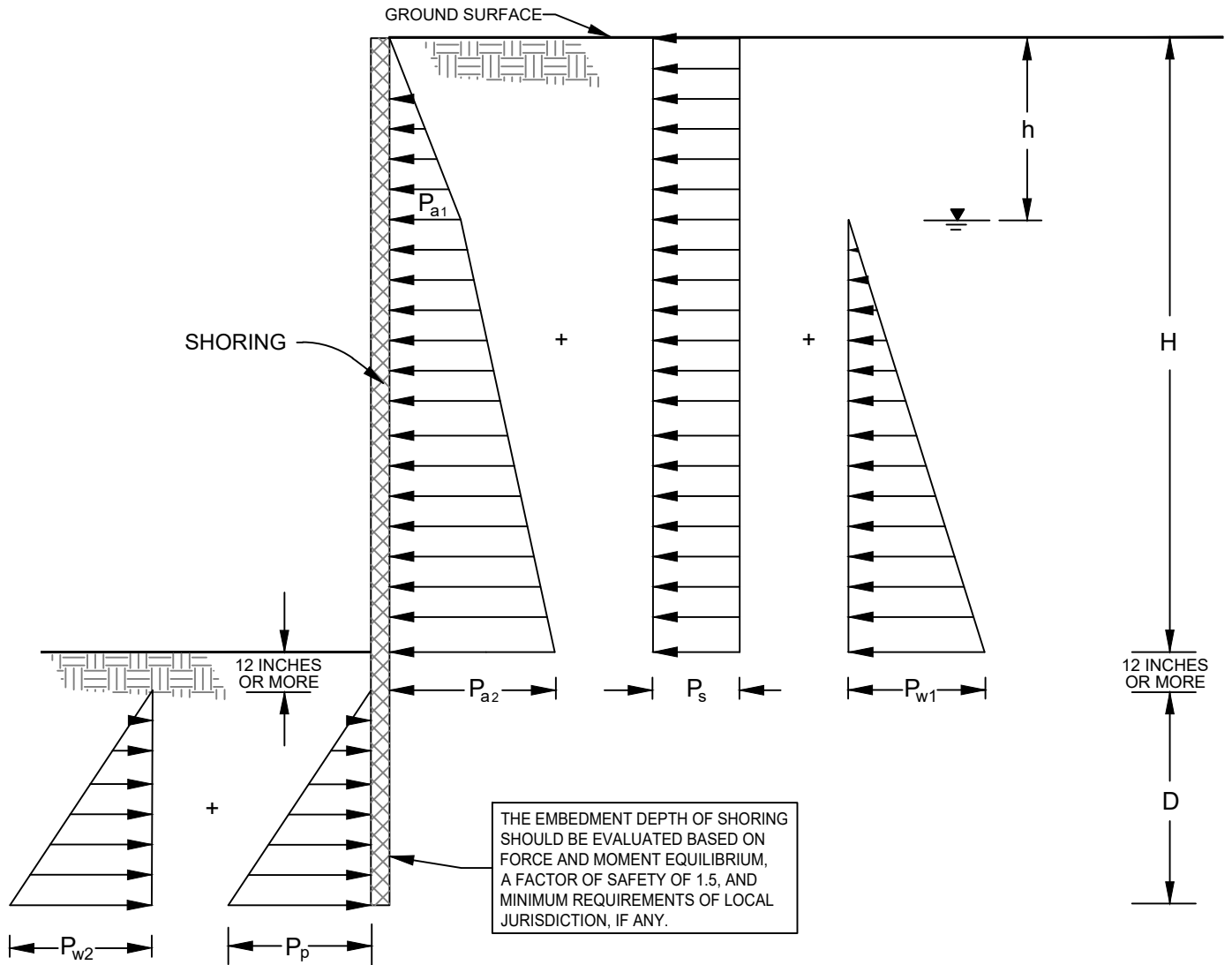
NOT TO SCALE

FIGURE 7

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION BELOW GROUNDWATER

S-7 SEWER LIFT STATION REHABILITATION
 349 LAKEVIEW AVENUE
 LONG BEACH, CALIFORNIA

211941001 | 7/22



NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
 $P_{a1} = 37h$ psf; $P_{a2} = P_{a1} + 18(H - h)$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 72$ psf
3. HYDROSTATIC PRESSURE, P_{w1} & P_{w2}
 $P_{w1} = 62.4(H - h)$ psf
 $P_{w2} = 62.4D$ psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 180D$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H , h AND D ARE IN FEET
7. GROUNDWATER TABLE
8. P_{w2} IS APPLICABLE WHEN DEWATERING IS TO BE PERFORMED FROM INSIDE OR OUTSIDE OF THE EXCAVATION

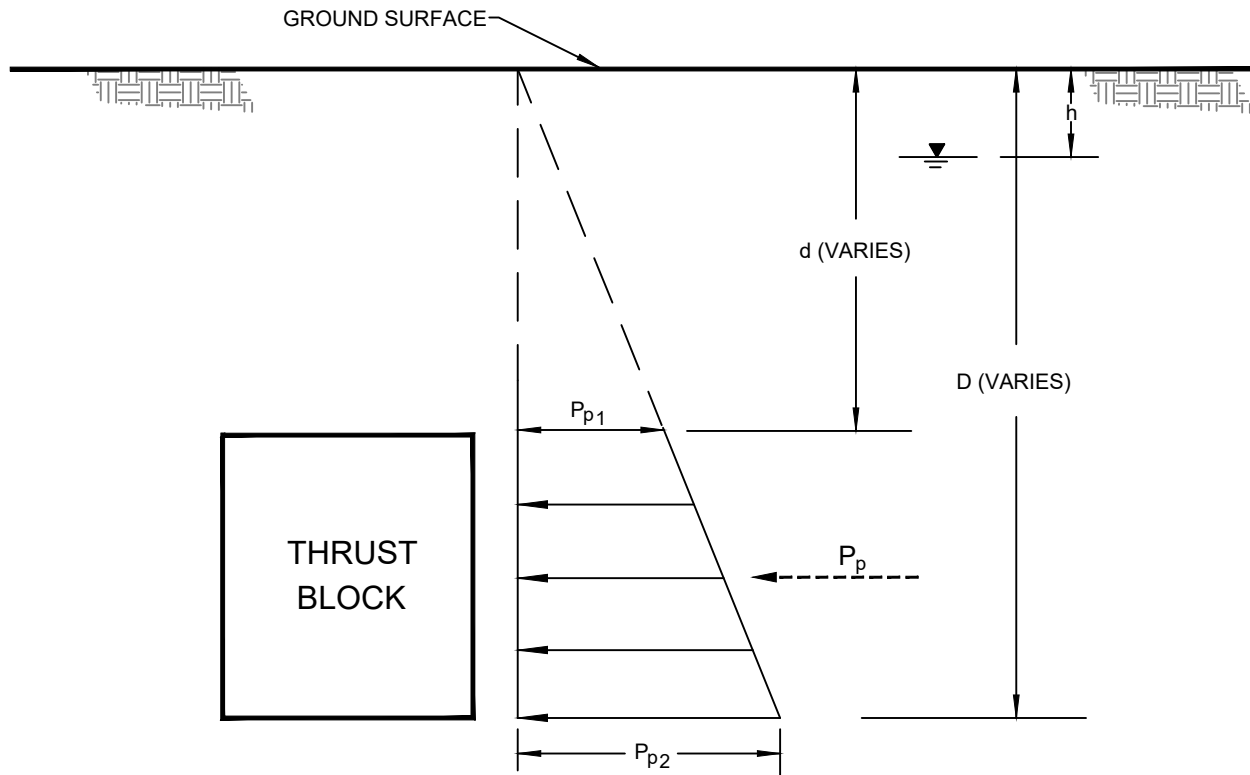
NOT TO SCALE

FIGURE 8

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING BELOW GROUNDWATER

S-7 SEWER LIFT STATION REHABILITATION
 349 LAKEVIEW AVENUE
 LONG BEACH, CALIFORNIA

211941001 | 7/22



NOTES:

1. GROUNDWATER BELOW BLOCK

$$P_p = 195(D^2 - d^2) \text{ lb/ft}$$
2. GROUNDWATER ABOVE BLOCK

$$P_p = 1.6(D - d)[124.8h + 58(D + d)] \text{ lb/ft}$$
3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6.  GROUNDWATER TABLE

NOT TO SCALE

FIGURE 9

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

S-7 SEWER LIFT STATION REHABILITATION
 349 LAKEVIEW AVENUE
 LONG BEACH, CALIFORNIA

211941001 | 7/22



APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

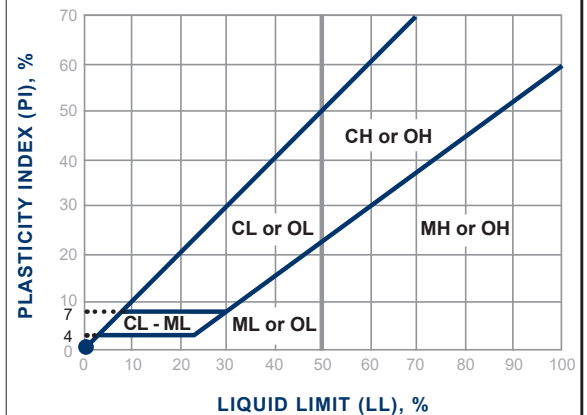
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	 GW	well-graded GRAVEL
			 GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	 GW-GM	well-graded GRAVEL with silt
			 GP-GM	poorly graded GRAVEL with silt
			 GW-GC	well-graded GRAVEL with clay
			 GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	 GM	silty GRAVEL
			 GC	clayey GRAVEL
			 GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	 SW	well-graded SAND
			 SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	 SW-SM	well-graded SAND with silt
			 SP-SM	poorly graded SAND with silt
			 SW-SC	well-graded SAND with clay
			 SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	 SM	silty SAND
			 SC	clayey SAND
			 SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	 CL	lean CLAY
			 ML	SILT
			 CL-ML	silty CLAY
		ORGANIC	 OL (PI > 4)	organic CLAY
			 OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	 CH	fat CLAY
			 MH	elastic SILT
			 OH (plots on or above "A"-line)	organic CLAY
		ORGANIC	 OH (plots below "A"-line)	organic SILT
			 PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.075 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.075"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

DEPTH (feet)	Bulk Driven SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0							Bulk sample.
							Modified split-barrel drive sampler.
							No recovery with modified split-barrel drive sampler.
							Sample retained by others.
							Standard Penetration Test (SPT).
5							No recovery with a SPT.
	XX/XX						Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
							No recovery with Shelby tube sampler.
							Continuous Push Sample.
10							Seepage.
							Groundwater encountered during drilling.
							Groundwater measured after drilling.
						SM	MAJOR MATERIAL TYPE (SOIL):
							Solid line denotes unit change.
						CL	Dashed line denotes material change.
15							Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20							The total depth line is a solid line that is drawn at the bottom of the boring.

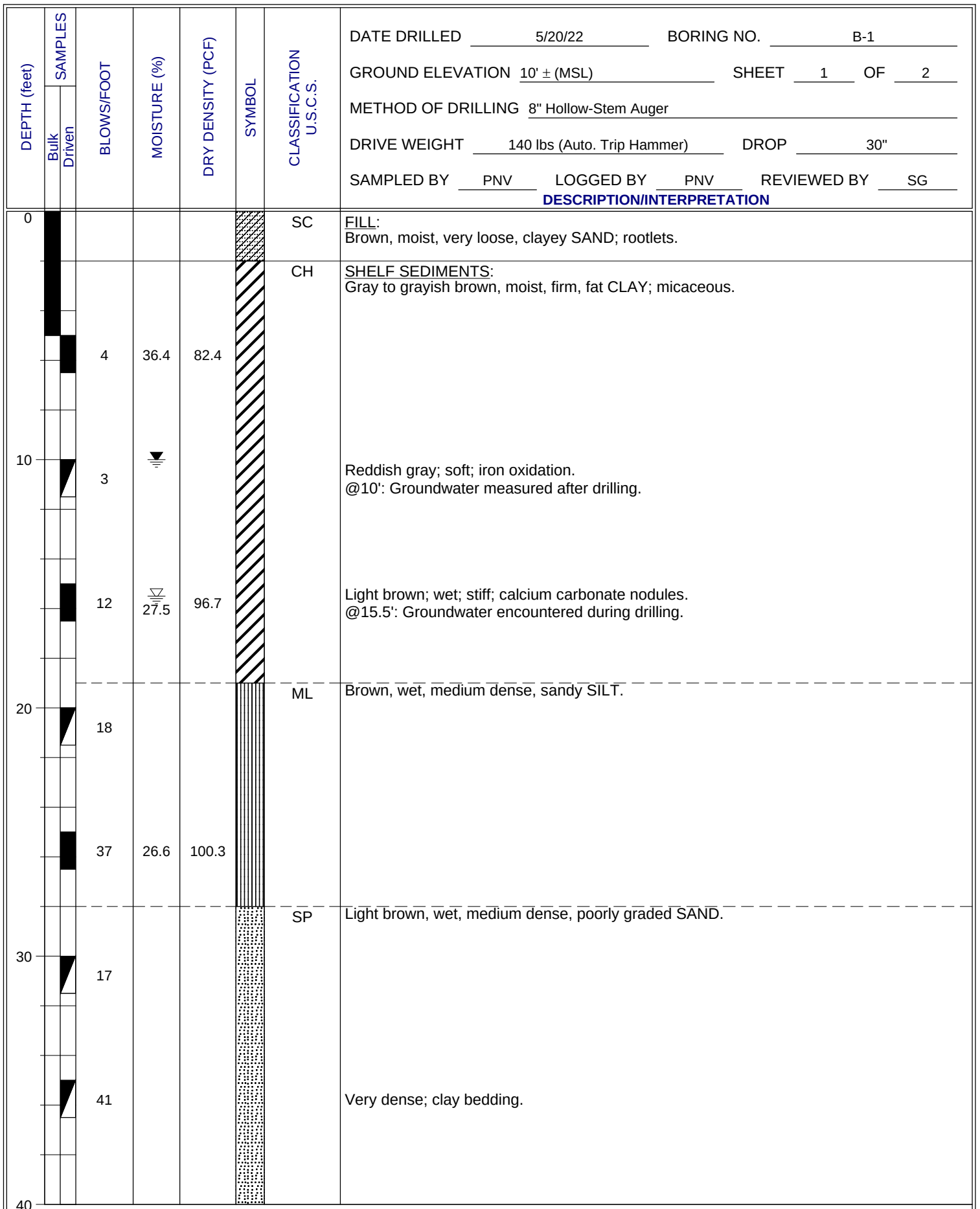


FIGURE A- 1

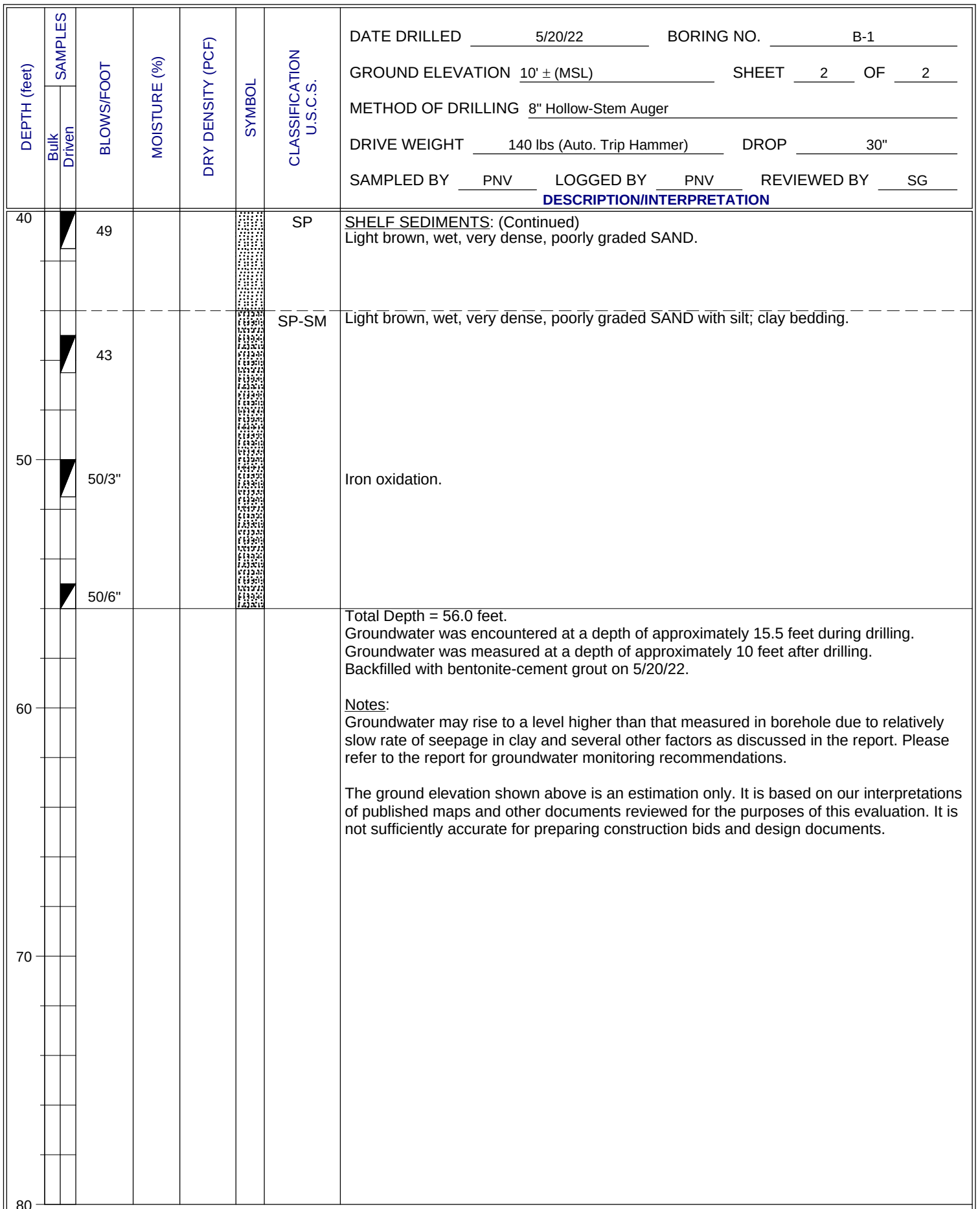


FIGURE A- 2

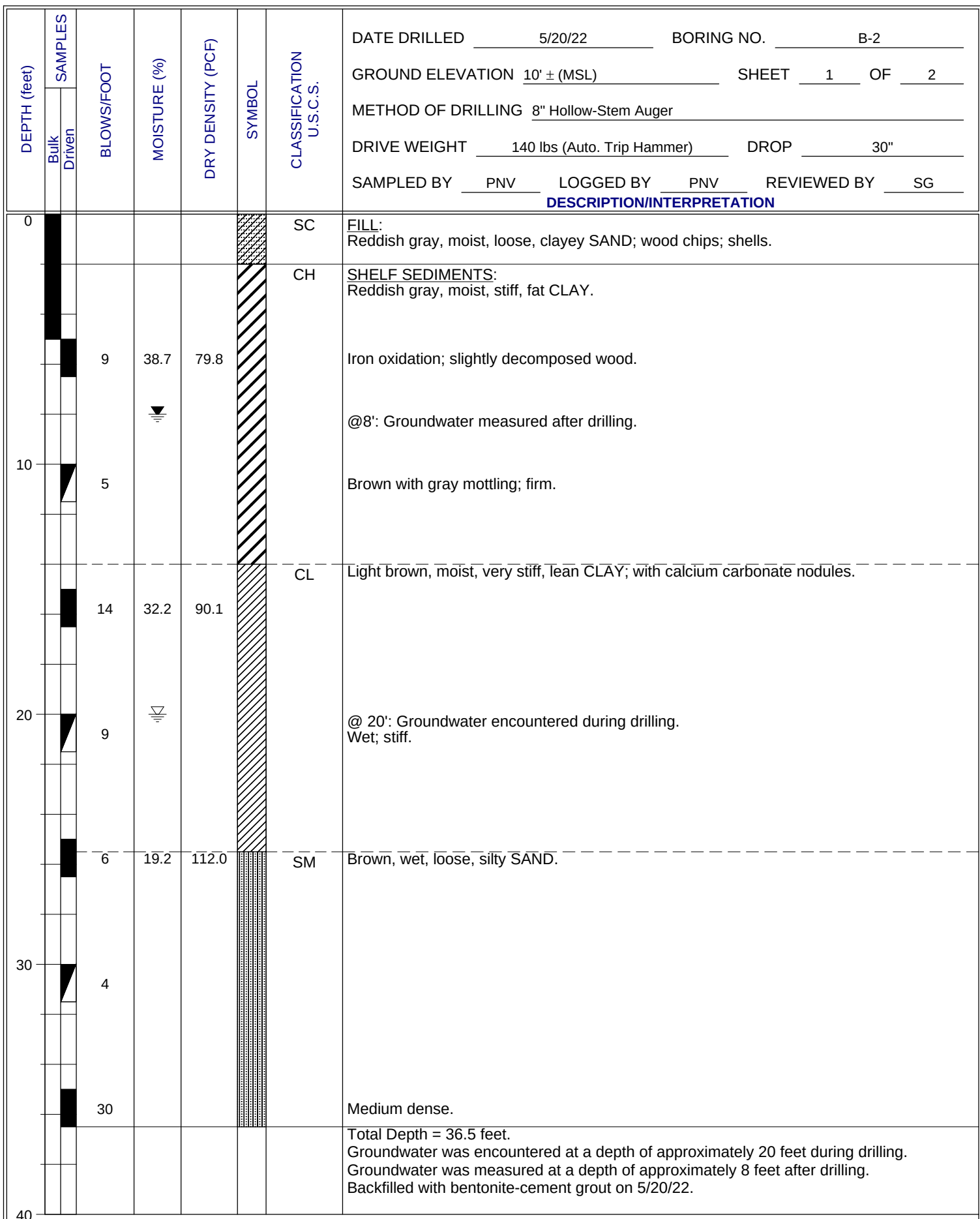


FIGURE A- 3

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>5/20/22</u> BORING NO. <u>B-2</u> GROUND ELEVATION <u>10' ± (MSL)</u> SHEET <u>2</u> OF <u>2</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger</u> DRIVE WEIGHT <u>140 lbs (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>PNV</u> LOGGED BY <u>PNV</u> REVIEWED BY <u>SG</u>		
	Bulk	Driven						DESCRIPTION/INTERPRETATION		
40								<u>Notes:</u> Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. Please refer to the report for groundwater monitoring recommendations. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.		
80										

FIGURE A- 4



APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-1.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. The test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure B-2.

Consolidation Tests

Consolidation tests, including time rate of consolidation, were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-3 through B-6.

Direct Shear Test

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-7.

Expansion Index Test

The expansion index of a selected soil sample was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of this test is presented on Figure B-8.

Soil Corrosivity Tests

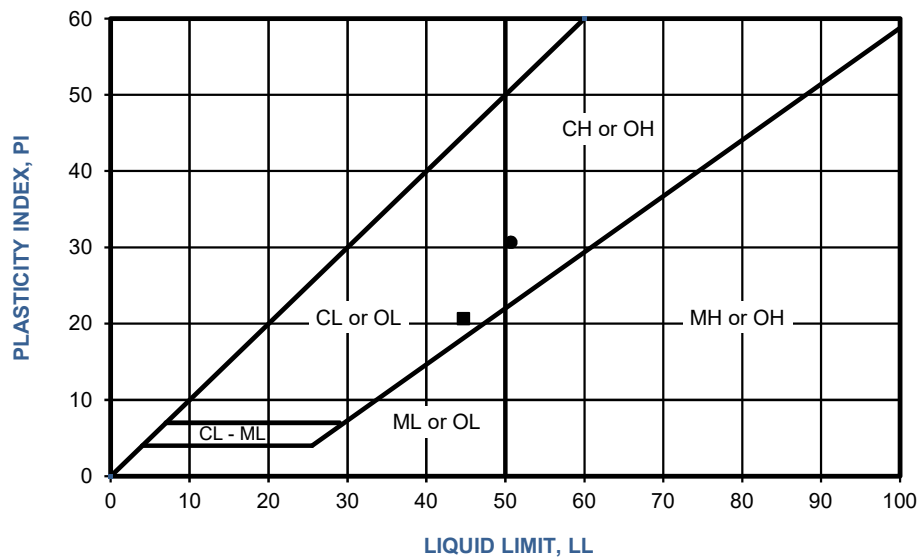
Soil pH and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of the selected sample was evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-9.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	20.0-21.5	SANDY SILT	100	60	ML
B-1	30.0-31.5	POORLY GRADED SAND	100	3	SP
B-1	45.0-46.5	POORLY GRADED SAND WITH SILT	100	6	SP-SM
B-2	30.0-31.5	SILTY SAND	100	13	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

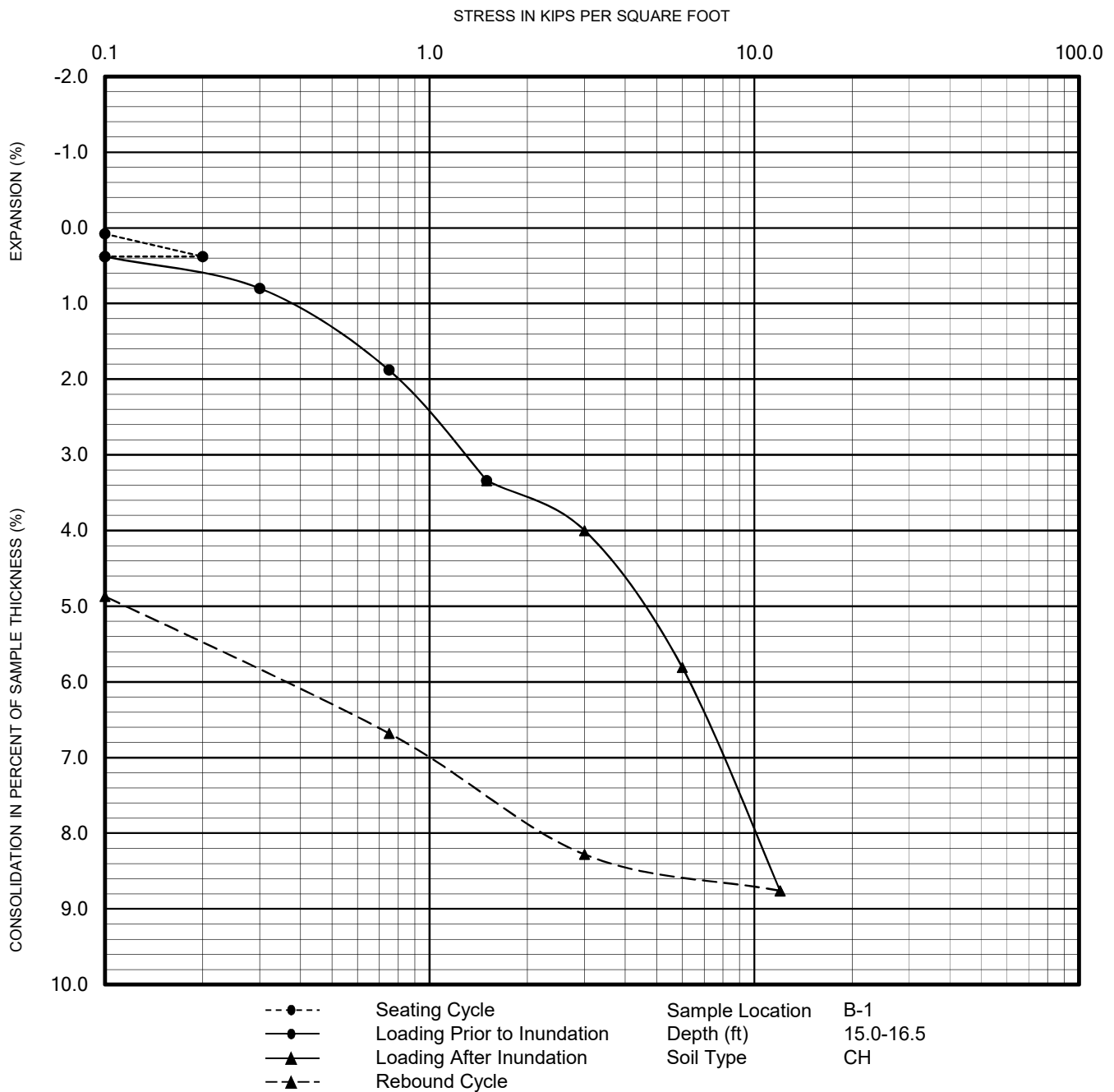
FIGURE B-1

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	15.0-16.5	51	20	31	CH	CH
■	B-2	15.0-16.5	45	24	21	CL	CL



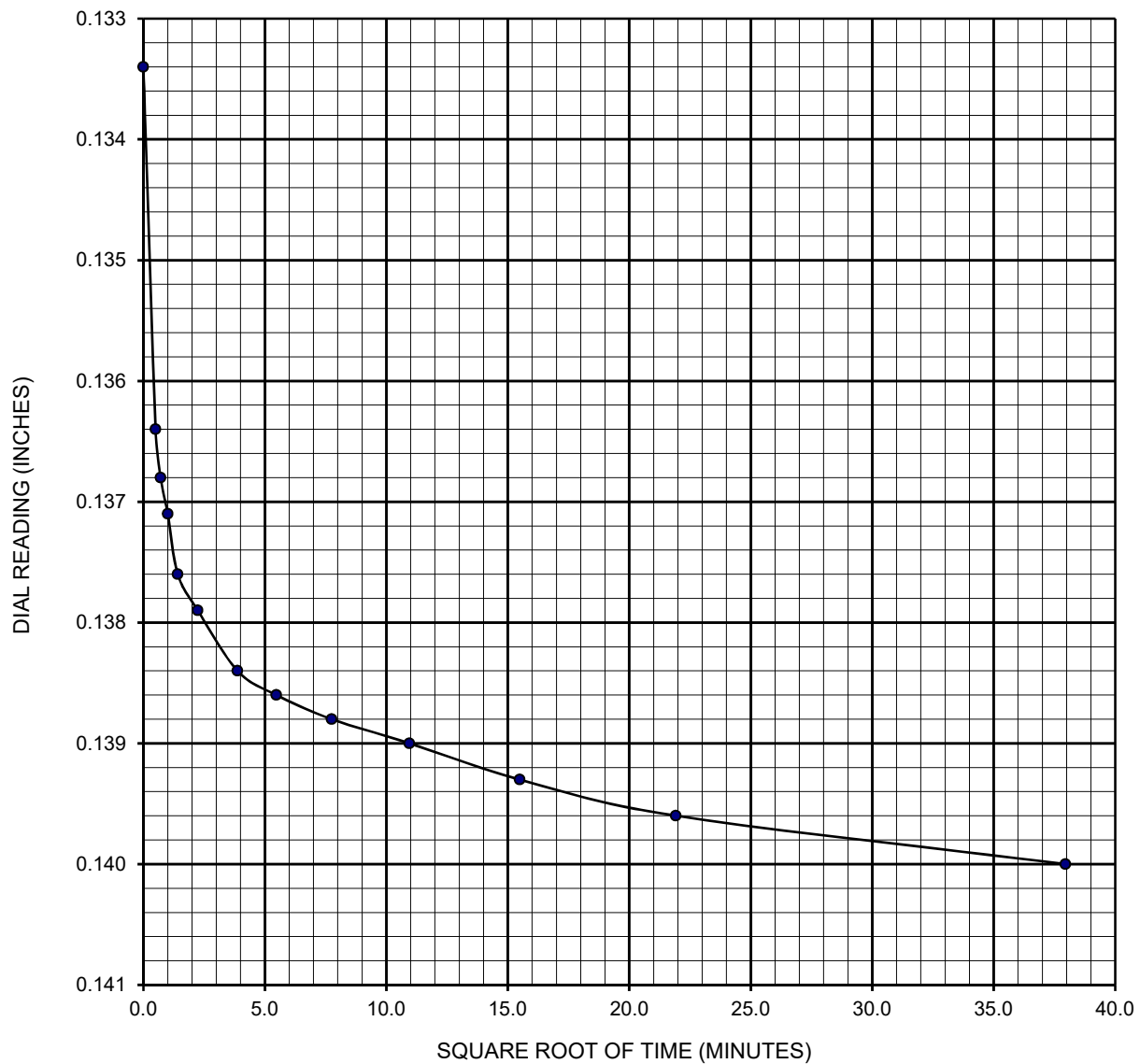
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-2



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-3



Sample Location B-1
Depth (ft) 15.0-16.5

Load (ksf) 3.0
Soil Type CH

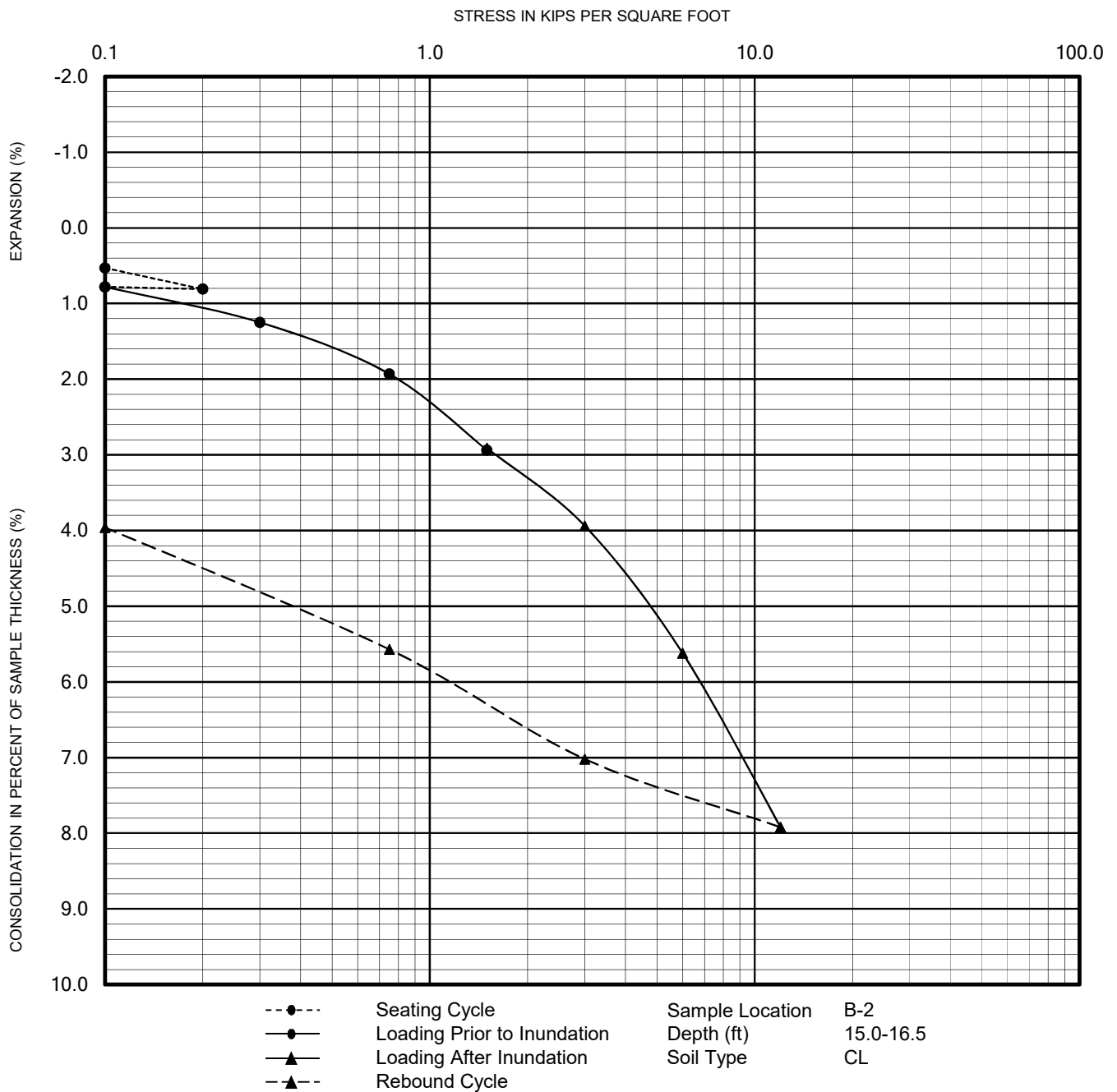
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435 - SQUARE ROOT OF TIME METHOD

FIGURE B-4

TIME RATE OF CONSOLIDATION TEST RESULTS

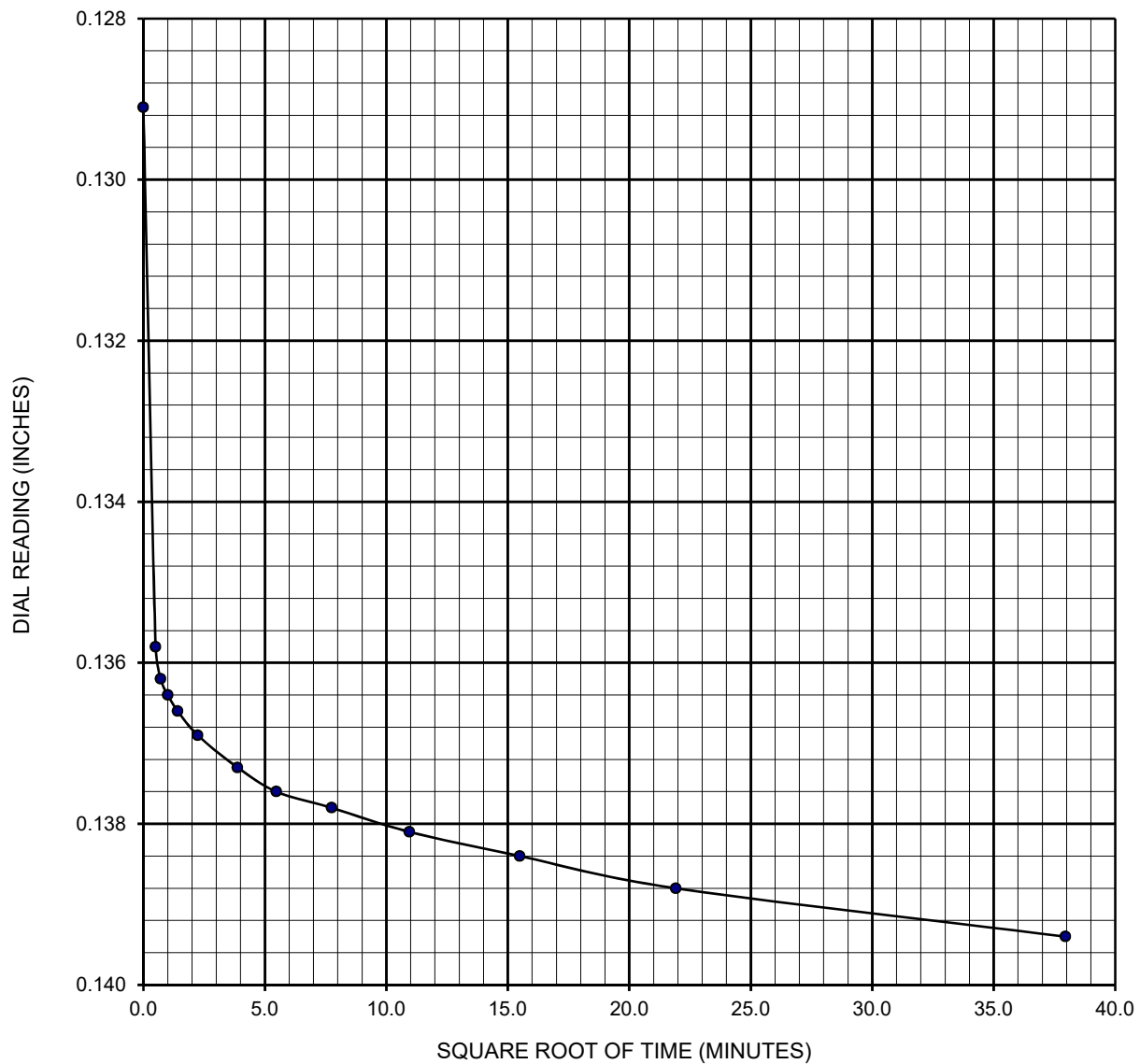
S-7 SEWER LIFT STATION REHABILITATION
349 LAKEVIEW AVENUE, LONG BEACH, CALIFORNIA

211941001 | 7/22



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-5



Sample Location B-2
Depth (ft) 15.0-16.5

Load (ksf) 3.0
Soil Type CL

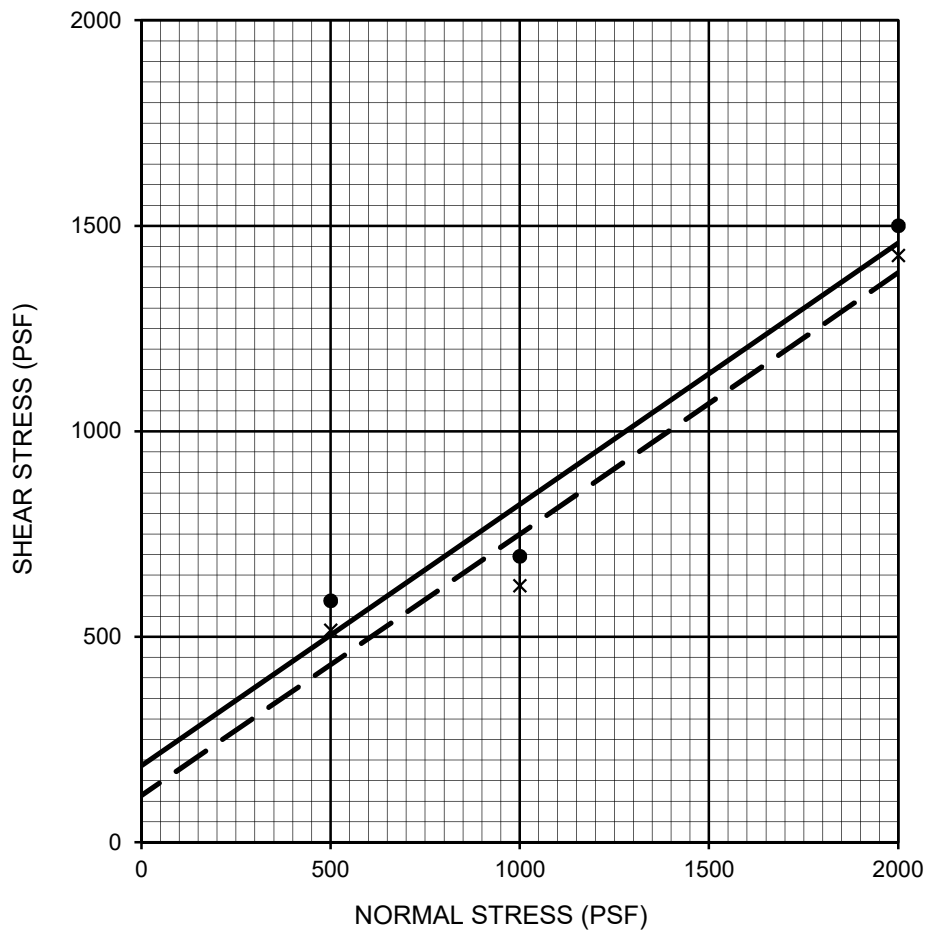
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435 - SQUARE ROOT OF TIME METHOD

FIGURE B-6

TIME RATE OF CONSOLIDATION TEST RESULTS

S-7 SEWER LIFT STATION REHABILITATION
349 LAKEVIEW AVENUE, LONG BEACH, CALIFORNIA

211941001 | 7/22



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
FAT CLAY	—●—	B-2	5.0-6.5	Peak	186	32	CL
FAT CLAY	- - X - -	B-2	5.0-6.5	Ultimate	114	32	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-7

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH ¹	RESISTIVITY ¹ (ohm-cm)	SULFATE CONTENT ²		CHLORIDE CONTENT ³ (ppm)
				(ppm)	(%)	
B-2	0.0-5.0	7.4	969	170	0.017	155

¹ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

² PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

³ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-9

CORROSIVITY TEST RESULTS

S-7 SEWER LIFT STATION REHABILITATION
349 LAKEVIEW AVENUE, LONG BEACH, CALIFORNIA



APPENDIX C

Photographs



Photograph 1: Southeast side of S-7 sewer lift station, looking northeast.



Photograph 2: Southeast side of S-7 sewer lift station, looking southwest.

FIGURE C-1



Photograph 3: Southeast side of S-7 sewer lift station, looking northwest.

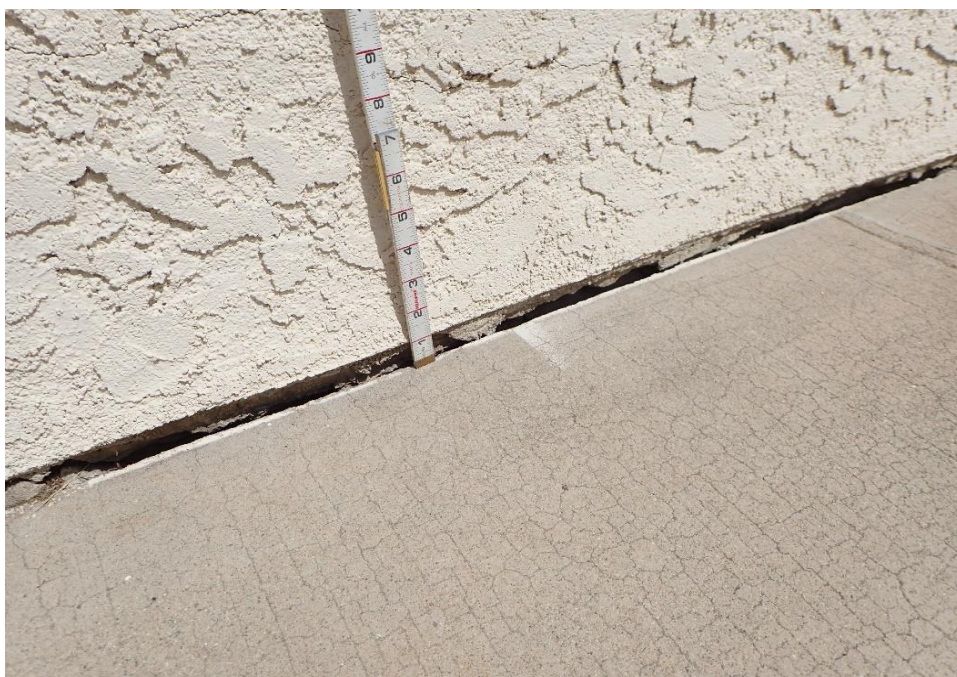


Photograph 4: Separation between the building entrance and the sidewalk.

FIGURE C-2



Photograph 5: Close up of 1/2-inch horizontal separation in photograph 4.



Photograph 6: 1-inch vertical separation between the southeast side of the building and the sidewalk.

FIGURE C-3



Photograph 7: A crack on the south corner of the building was noted, as well as a vertical separation between the sidewalk and the ground surface.



Photograph 8: Close up of 2½ inches of separation between sidewalk and ground surface in photograph 7.

FIGURE C-4



Photograph 9: Close up of ½-inch-wide crack on the south corner of the building in photograph 7.



Photograph 10: Close up of 2-inch-deep crack on the south corner of the building in photograph 7.

FIGURE C-5



Photograph 11: Settlement of the ground surface around a street sign on the north corner of the property, looking north.



Photograph 12: Close up of 2 inches of settlement of the ground surface in photograph 11.

FIGURE C-6



Photograph 13: Horizontal crack on the upper floor of the lift station, above the doorway on the southeast wall of the building, looking southeast.



Photograph 14: Close up of 1/4-inch-wide crack in photograph 13, above the doorway on the southeast wall of the building.

FIGURE C-7

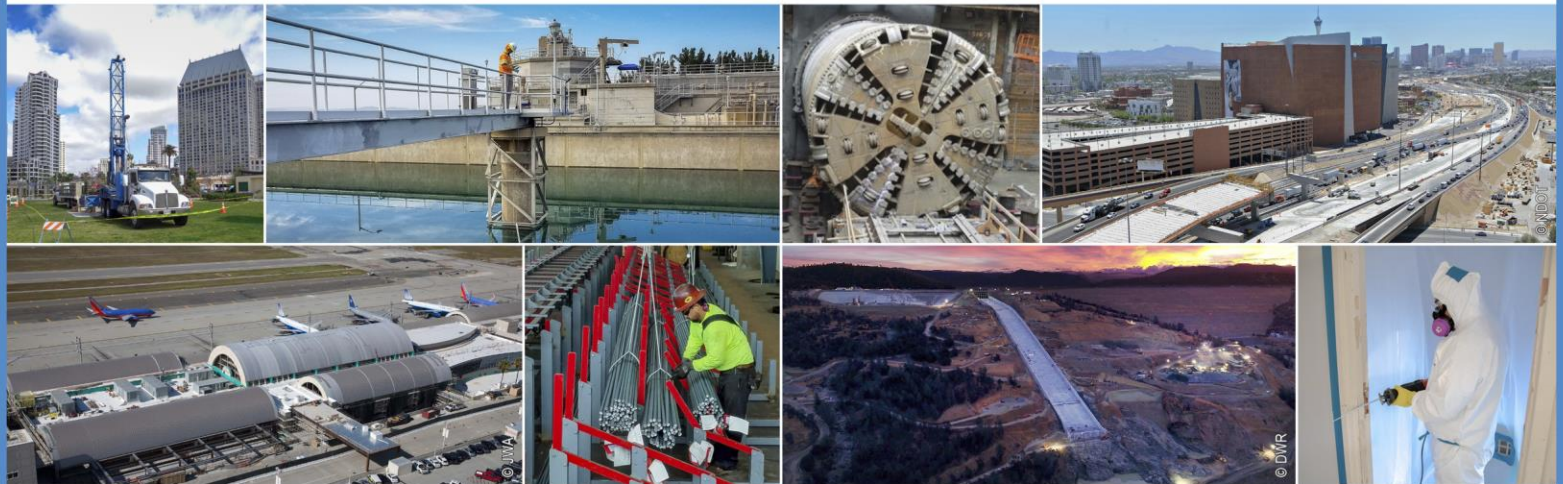


Photograph 15: Horizontal crack on the upper floor of the lift station, on the southwest wall of the building, looking south.



Photograph 16: Continuation of crack in photograph 15, on the southwest wall.

FIGURE C-8



475 Goddard, Suite 200 | Irvine, California 92618 | p. 949.753.7070

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

ninyoandmoore.com

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants