Report

Geotechnical Investigation Report

Martin Luther King, Jr. Medical Campus

Augustus F. Hawkins Building Demolition Project

1720 E 120th Street

Los Angeles, California



Prepared for

Los Angeles County Public Works

900 South Fremont Avenue, 4th Floor

Alhambra, California 91803

May 21, 2026

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Project No.: 20044A-TO1

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Subject: **Geotechnical Investigation Report**
Martin Luther King, Jr. Medical Campus
Augustus F. Hawkins Building Demolition Project
1720 E 120th Street
Los Angeles, California 90059

Dear Mr. Schwartz,

In general accordance with the terms of GeoPentech's On-Call Geotechnical and Geohazard Services for Various County Health Care Projects contract with LACPW (No. PW15482 dated April 8, 2021) and the Notice to Proceed issued on March 16, 2023, GeoPentech has completed a geotechnical investigation to support the proposed Augustus F. Hawkins Building Demolition and post-demolition configuration at the site. This report presents the results of our investigation, including descriptions of our work, boring logs, geophysical survey, and laboratory test results for the collected soil samples, and our recommendations for demolition and rough grading at the project site.

We trust that this report meets the needs of the project. If you should have any questions, please call.

Very truly yours,

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1 INTRODUCTION

This report presents the results of GeoPentech's geotechnical investigation to support the Augustus F. Hawkins Building Demolition ("Project") on the Martin Luther King Jr. Medical Campus located at 1720 E 120th Street in Los Angeles, California within the county of Los Angeles (33.924149° N, -118.242361° E). The location of the project site is shown on Figure 1.

Our work has been performed in general accordance with the terms of GeoPentech's On-Call Geotechnical and Geohazard Services for Various County Health Care Projects contract with LACPW (No. PW15482 dated April 8, 2021) and the Notice to Proceed issued on March 16, 2023.

2 PROJECT DESCRIPTION

We understand that the proposed Project includes the demolition of the existing Augustus F. Hawkins Building, as well as the removal of all surface features within the site, such as paving, landscaping walls, and light poles. The extent of the Project site and the existing building are shown on Figure 2.

The existing Hawkins building is a three-story building with an approximate height of 45 ft above ground. Based on the available civil plans (RBB Architects Inc, dated March 21st, 2023), the building's first floor slab-on-grade lies on level ground at an approximate elevation of 93.84 feet (NAVD 1988[1]) with a partial basement to the southwest extending to elevation 79.35 ft. The Project site contains walls along the west, east, and south sides separating it from the surrounding sloping ground, which ranges between elevation 91 ft on the north to 104 ft on the south. The garden area is at current grade on the southern edge of the building at approximate elevation of 93 ft. The building is supported on pile foundations with lengths ranging between 31 and 42 feet across 196 pile caps. The typical top elevation of first-floor pile caps is between approximately 92.3 to 83.3 ft. The top elevation of the basement pile caps is typically between 78.3 to 77.3 ft. The pile cap depths are typically between 3 to 5.5 ft per the available structural plans. The existing foundation plans are shown and discussed further in Appendix A. Additionally, a utility tunnel enters the building on the southwestern side carrying utilities from other nearby buildings on the medical campus to the Hawkins building. The tunnel is 9 ft wide by 8 ft tall with an invert elevation of approximately 79.35 ft (i.e., basement level slab on grade).

The current project consists of the demolition of the building and some of the surrounding features. Following the demolition of the building, the pile caps supporting the building will be removed, and top of the piles will be surveyed by others and then left in place. The site will then be roughly graded to near

[1] All elevations provided are based on NAVD 88 vertical datum unless specially indicated.

the elevation of the existing first floor of the building. The Project will also involve capping of the utility tunnel that enters the site on the southwestern corner (see Figure 2). The existing garden area on the south of the building will remain as-is and will not be demolished as part of the Project. The roughly graded site may be used for a future development, details of which are unknown at this time. The future development is not part of the scope of work in this report.

We understand that the Project, including geotechnical aspects of the design, may be submitted for review and approval to the Department of Health Care Access and Information (HCAI). This report presents the results of GeoPentech's geotechnical investigation (including field exploration) as well as recommendations for the demolition of the proposed Project.

3 SCOPE OF WORK

GeoPentech's scope of work included the following tasks:

- Reviewed available geotechnical, geologic, and seismic information at the site, as well as available architectural and structural plans of the existing building, and civil plans to support the demolition and grading of the site.
- Completed a site investigation program which included drilling three (3) hollow-stem auger borings, three (3) cone penetration tests (CPT), three (3) geophysical surveys to evaluate subsurface conditions, obtain samples for laboratory testing, and characterize the shear wave velocity profile at the site (see Figure 2). Percolation testing was also performed in two boring locations to measure surficial infiltration rates at the site.
- Performed laboratory testing of soil samples obtained from the boreholes.
  Evaluated the subsurface conditions, geologic conditions, and seismic setting affecting the project, and performed engineering analyses to develop recommendations for the Project and post-demolition site configuration.
- Prepared this report to present the results of the geotechnical investigation.

4 FIELD INVESTIGATION AND LABORATORY TESTING

4.1 Available Subsurface Data

Available subsurface data at the Project site consists of previous explorations by LeRoy Crandall and Associates (1972) which include seven (7) bucket auger borings with associated soil descriptions, moisture content, and density values. The approximate location and depth of the previous investigations are shown on Figure 2, and the available boring logs and laboratory test results are included in Appendix B.

4.2 Boring Exploration and Laboratory Testing

GeoPentech's exploratory borings were performed between April 19 and April 21, 2023, and consisted of drilling three borings, GP-1 through GP-3, at the locations shown on Figure 2 to depths of about 101.5 feet below ground surface (bgs). All locations were hand-augured to 5-ft bgs before drilling to check for potential underground utilities. Soils encountered during drilling were visually classified in general accordance with the Unified Soil Classification System (USCS). Soil samples were obtained at approximately 2.5- to 5-foot intervals using either a Standard Penetration Test (SPT) sampler or a Modified California (MC) sampler. The work was performed under the supervision of an engineer or geologist who monitored the drilling operations and prepared a field record of soils observed and drilling conditions. The drilling was subcontracted to Gregg Drilling Inc., who provided all drilling equipment, crew, and supplies. The boring logs and more details of the drilling program are included in Appendix C.

Selected soil samples were submitted to a geotechnical laboratory to evaluate their pertinent physical and engineering properties. The laboratory testing included moisture content and unit weight, grain-size distribution, passing #200 sieve, Atterberg limits, shear strength, consolidation, corrosion, and expansion index. A description of the laboratory tests and their results are included in Appendix D. The key results are also summarized on the boring logs in Appendix C.

4.3 Cone Penetration Tests

Cone penetration tests (CPTs) were performed on April 17th, 2023 and consisted of advancing three CPTs, CPT-1 through CPT-3, at the locations shown on Figure 2 to depths of about 105 to 108 feet bgs. All locations were hand-augured to 5-ft bgs to check for potential utilities before advancing the cone.

The CPTs were subcontracted to Gregg Drilling Inc. who provided all CPT equipment, crew, and supplies. The work was performed under the supervision of a registered civil engineer who monitored the testing. Details and results of the CPTs are presented in Appendix E.

4.4 Geophysical Surveys

Geophysical surveys were performed to measure the shear-wave (S-wave) velocity profile of the subsurface soil strata underlying the Project site. The geophysical investigation consisted of active and passive surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and microtremor array methods, respectively. The geophysical measurements were performed along three (3) survey lines, SW23-1 through SW23-3, on April 17th, 2023. The locations of the survey lines are shown on Figure 2.

The geophysical data were collected and interpreted under the supervision of a California-licensed Professional Geophysicist. Details and results of the geophysical survey can be found in Appendix F.

5 GEOLOGIC CONDITIONS

5.1 Regional Geology and Seismicity

The site is located within the Peninsular Ranges geomorphic province, which is dominated by northwest-trending, strike-slip faults. The site is located within the relatively flat coastal plain of southern Los Angeles County. The site area is within the prehistoric lowlands of the Los Angeles basin that consisted of various river channels, lakes, and swamps.

Figure 3 shows the Project location on a geologic map of the area by Saucedo et al. (2016). As indicated on Figure 3, the site area is generally underlain by Holocene and late Pleistocene-age young alluvium (Qya_2). As described in Figure 3's geologic legend, the alluvium within the area is generally poorly consolidated, poorly sorted, and consists of permeable flood-plain deposits of loose to moderately dense sand, silt, and soft clay.

The site is located within a seismically active region of southern California. Recent examples of significant seismic activity in the region include the 1987 M6 Whittier Narrows earthquake, the 1994 M6.7 Northridge earthquake, and the 2009 M4.7 Inglewood earthquake. Figure 4a shows the site location relative to mapped active faults in the region, as identified by the USGS (2010) as well as historic seismicity in the region including significant past earthquakes. Significant faults near the site that have displaced the ground surface include the Newport Inglewood Fault located about 3 km west, Palos Verdes Fault located about 18 km to the southwest; the Whittier Fault located about 19 km to the northeast; the Hollywood Fault located about 21 km to the north; the Santa Monica Fault located about 22 km to the northwest; and the Verdugo Fault located about 23 km to the north. The San Andreas Fault is located approximately 68 km to the northeast.

Potentially active blind thrust faults are also believed to exist in the region. These blind thrust faults are not expressed at the surface, but are inferred to exist based on indirect information, such as seismicity and folded stratigraphy. Recognition of the existence of blind thrust faults in the region was largely triggered by the occurrence of the 1987 Whittier Narrows earthquake. As shown on Figure 4b, the site is located on the hanging wall of the potentially active Compton blind thrust fault.

5.2 Site Geology and Subsurface Conditions

The current field investigation encountered artificial fill and young alluvium during borehole drilling. Figures 5a and 5b show geologic cross sections A-A' and B-B', respectively, and the locations of the sections are shown on Figure 2. The following presents a description of the encountered geologic units at the site.

Artificial Fill

Artificial Fill was encountered below the ground surface to depth of about 5ft bgs in borings GP-1 and GP-2, and to 11ft bgs in boring GP-3. The observed fill predominantly consisted of dark brown clayey sand (SC) and sandy silt and clay (ML, CL). Given that no records of placement of the fill is available to us, the fill is considered undocumented fill from a geotechnical point of view.

Alluvium

Young alluvium was encountered within all borings below the artificial fill to the maximum depth drilled (i.e., 101.5 feet bgs). The encountered alluvium predominantly consisted of light brown, loose to very dense sands (SM, SP-SM, SP, and SC) and medium stiff to hard silts and clays (ML, CL, CL-ML). The clay layers encountered generally had low to medium plasticity.

5.3 Groundwater

Groundwater was encountered during drilling within all borings at the site; at a depth of about 58 ft bgs in boring GP-1, 56 ft bgs in GP-2, and 65 ft bgs in GP-3 which correspond to an approximate elevation of 36 ft. Previous investigations at the site had also encountered groundwater at depths between 54 and 58 ft bgs (LeRoy Crandall and Associates, 1972) consistent with GeoPentech's observations. The observed ground water levels are indicated on the cross sections in Figures 5a and 5b.

Based on a review of the groundwater contour maps in the Seismic Hazard Zone Report for the South Gate Quadrangle (CGS, 1998), the historic high groundwater in the vicinity of the site is estimated to be at a depth of approximately 10 feet bgs (i.e., elevation 82 ft).

It should be recognized that groundwater levels can fluctuate over time, depending on seasonal rainfall and other influences. Furthermore, there may be a potential for perched water to occur locally in sandy zones above the groundwater level. In addition, recent changes in policies for the use of stormwater infiltration could result in changing seepage conditions at shallow depths across the region.

6 POTENTIAL GEOLOGIC AND SEISMIC HAZARDS

An evaluation of the potential geologic hazards is presented in the following sections.

6.1 Surface Fault Rupture

The site is not located within a currently established Alquist-Priolo (AP) Zone based on a review of the Earthquake Zones of Required Investigation for the South Gate Quadrangle (CGS, 1998). Additionally, the site is not located within 1,000 feet of a mapped Holocene-active fault based on a review of mapping by

(USGS, 2010), as shown on Figure 4a. Therefore, the site is not considered susceptible to surface fault rupture hazards.

6.2 Seismic Shaking

The site is within a seismically active region of southern California and should be designed in accordance with the seismic design requirements of governing codes and regulations. The site could be subject to strong ground shaking in the event of future earthquakes generated by the various active or potentially active faults mapped in the region, therefore, it is considered a hazard for this site. Seismic design recommendations are provided in Section 7.

6.3 Liquefaction Potential

Liquefaction potential is greatest where the groundwater level is shallow and submerged loose to medium-dense sand occur within a depth of about 50 feet or less below the ground surface. Liquefaction potential generally decreases as fines and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the CGS map of Earthquake Zones of Required Investigation for the South Gate Quadrangle (CGS, 1998), the site is located within an area identified as having a potential for liquefaction. During the field investigations at the site, pockets of medium dense sandy layers below the historic high groundwater (approximately 10 ft bgs) were encountered within the alluvium that could potentially liquefy during a strong shaking event. We estimate up to approximately 2.5 inches of liquefaction-induced settlement in the event of strong shaking at the site. Details and results of our liquefaction analysis are presented in Appendix G.

6.4 Seismically-Induced Dry Sand Settlement

Seismically-induced settlement may be caused by unsaturated loose to medium-dense granular soils densifying during ground shaking. Our field investigation encountered occasional loose to medium dense sandy layers within the artificial fill and alluvium above the historic high ground water table, which could be susceptible to seismically-induced settlements if not mitigated. We estimate up to approximately 1 inch of dry sand settlement within the artificial fill if strong shaking were to occur at the site. Note that at the location of CPT-3 we have calculated approximately 4 inches of dry sand settlement, however, based on other available subsurface data from the site, it appears that potential for this level of seismic-induced settlement may be localized and not spread out through the site. Details and results of the dry sand settlement analysis are presented in Appendix G.

6.5 Subsidence

Ground surface subsidence generally results from the extraction of fluids or gas from the subsurface that can result in the gradual lowering of the overlying ground surface. Subsidence can also occur when subsurface peat deposits oxidize and undergo volume loss. The site is not located over any existing oil or gas fields. Additionally, the subsurface soils are not known to contain significant quantities of peat that would create a potential for subsidence. Therefore, the potential for subsidence at the site is considered low.

6.6 Flooding

According to FEMA (2008), the site is not located within a defined flood zone. Therefore, flooding is not considered to be a potential hazard at the site.

6.7 Seiches and Inundation (Water Storage Facilities)

This potential hazard is associated with seiches (water waves created when a body of water is shaken that have the potential to overtop a water storage facility) and inundation due to water storage facility failure. The site is not located within a mapped dam inundation zone; therefore, inundation is not considered a hazard.

6.8 Landslide

A potential for landsliding is often indicated in areas of moderate to steep terrain that are underlain by unfavorably oriented geologic discontinuities. The site is located on relatively level terrain and no landslides are mapped in the vicinity of the site (CGS, 1998). In addition, the site is not in a designated earthquake-induced landslide hazard zone (CGS, 2018). Therefore, the potential for landsliding is considered negligible.

6.9 Erosion

Most of the site is covered with asphalt or concrete pavements, so damage from erosion is not considered a possible hazard in these areas.

6.10 Corrosion

The corrosion potential of surficial fill soils at the site were evaluated through laboratory tests on collected samples (see Appendix E). Based on available guidelines (LACPW, 2013), a corrosive soil is defined as material with a sulfate concentration equal to or greater than 2,000 ppm, a chloride concentration greater than or equal to 500 ppm, a pH of 5.5 or less, or a minimum resistivity less than 1,000 ohm-centimeters.

Based on these requirements and the laboratory test results in Appendix E, the on-site soils are not considered corrosive to metals, therefore, corrosion potential at the site is considered low.

6.11 Expansive Soils

Results of expansion index tests presented in Appendix E indicate low expansion potential of the surficial artificial fill at the project site. Therefore, expansive soil potential at the site is considered low.

6.12 Other Geologic Hazards

Other geologic conditions including oil wells, potential hazardous oil and gas, tsunami, and volcanic eruptions are not considered to be hazards at the site.

7 GEOTECHNICAL RECOMMENDATIONS

Based on our understanding of the Project and the results of our investigation, the proposed Project is feasible from a geotechnical point of view. The key geotechnical considerations for the proposed project include:

- **Removal of pile caps and capping of utility tunnel**
  Following the demolition of the Hawkins building, pile caps supporting the existing building will be removed while the piles themselves will be left in place. We recommend surveying the top of the piles for record keeping purposes to aid future development planning efforts. Additionally, we understand that the utility tunnel on the southwest corner of the site will be capped. The elevation of the pile caps at the building are summarized in Appendix A for reference. Upon completion of demolition and pile cap removal, the site will be roughly graded to an approximate elevation of 92ft (i.e., the current adjacent grade on the north side of the site).

- **Site Grades**
  Final site slopes should be constructed at slopes no steeper than 2:1 (horizontal to vertical) in native alluvium or engineered fill. If desired changes in site grades cannot be achieved by slopes, permanent retaining walls should be considered.

- **Liquefaction and seismic-induced settlements (for potential future development)**
  The site is located within an area indicated as Zones of Required Investigation for liquefaction and consists of sandy soils below the historic high ground water table that could potentially liquefy during a major earthquake. Furthermore, the site consists of zones of loose to medium dense soils above the design groundwater level that are considered susceptible to seismically-induced

settlement. We estimate the total seismically induced settlements could be on the order 3½ inches. Given that the site will be a roughly graded with no planned development at this time, these settlements are not considered to impact the current project, however, they should be considered for potential future development.

The following sections provide further recommendations for the Project.

7.1 Seismic Design Parameters

Seismic design parameters in accordance with the 2022 California Building Code (CBC 2022) and ASCE 7-16 Standard were developed for a potential future development at the site. Given the presence of the liquefiable soils at the site, the site is classified as Site Class F as per Chapter 20 of ASCE 7-16. However, if liquefaction potential is mitigated at the site, the future building can be designed to Site Class D (Stiff Soil) or better, depending on the selected mitigation option.

The shear wave velocity measurements at the site indicate a V_{S30} of 936 ft/s from 0 to 100 ft bgs, which is consistent with seismic Site Class D, and will be the governing Site Class if the liquefaction hazard at the site is mitigated. The corresponding code-based seismic design parameters for Site Class D, considering that the site will be mitigated against liquefaction hazard are provided below:

Mapped seismic parameters S_s = 1.771g and S_1 = 0.629g were provided by OSHPD, and F_a = 1.0, and F_v = 1.7 were obtained from Table 11.4-1 and 11.4-2 of ASCE 7-16, respectively. Hence, from Sections 11.4 and 11.8 of ASCE 7-16 (located at 33.924078° N, -118.242276° E):

$S_{MS} = F_a S_S$ = 1.771g
$S_{M1} = F_v S_1$ = 1.07g
S_{DS} = 2/3 S_{MS} = 1.181g
S_{D1} = 2/3 S_{M1} = .713g
PGA = 0.76g
F_{PGA} = 1.1
PGA_M = 0.836g

Note that to obtain values for S_{M1} and S_{D1}, it was assumed that a site-specific ground motion hazard analysis will not be required as indicated in Section 11.4.8[2] of ASCE 7-16. The project structural engineer

[2] Section 11.4.8 of ASCE 7-16 indicates that structures on Site Class D sites with S_1 greater than or equal to 0.2 may be exempted from performing a site-specific ground motion hazard analysis, provided that the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.

should review these assumptions and evaluate whether performing a site-specific ground motion hazard analysis would be beneficial for the project design.

7.2 Walls Below Grade

The utility tunnel cap (i.e., wall) should be designed to resist lateral earth pressures plus any surcharges from adjacent loads. Furthermore, if needed desired changes in final grades can be achieved with retaining walls.

Walls without a drainage system to relieve hydrostatic pressure buildup shall be designed to resist hydrostatic pressures assuming groundwater at the ground surface. For submerged conditions (i.e., groundwater at the surface), permanent walls that are restrained at the top of the wall should be designed to resist an at-rest lateral earth pressure imposed by an equivalent fluid weighing of 30 pcf. Cantilever walls that are not restrained at the top should be designed to resist active earth pressure imposed by an equivalent fluid weighing of 20 pcf. Hydrostatic pressures should be added to these values.

For walls with a drainage system to relieve hydrostatic pressure buildup behind the subterranean walls, hydrostatic pressure can be ignored in the wall design. Permanent basement walls that are restrained at the top of the wall should be designed to resist an at-rest lateral earth pressure imposed by an equivalent fluid weighing 60 pcf. Cantilever walls that are not restrained at the top should be designed to resist active earth pressure imposed by an equivalent fluid weighing 40 pcf.

In addition to the recommended earth pressure, the upper 10 feet of walls below grade and retaining walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet from the top of walls, the traffic surcharge can be neglected. For the basement walls adjacent to the at-grade structures, surcharge pressures can be provided on a case-by-case basis once the estimated loading conditions from these structures and the details of the foundations are provided to us.

Loads from equipment surcharge imposed on adjacent ground may be computed using a coefficient of 0.4 times the uniform load applied.

In addition to the above-mentioned lateral earth pressures, the walls below grade should be designed to support a seismic lateral pressure of 17H (psf) applied uniformly along the wall height H (in feet). This seismic load is a directly calculated value and can be used as-is. When designing for seismic loads, the seismic lateral earth pressure should be combined with the at-rest earth pressures mentioned previously. If designing for static loading condition only, the at-rest lateral earth pressure should be used.

Drainage

Walls below grade and retaining walls should be designed to resist hydrostatic pressures (equivalent fluid pressure of 62.4 pcf) or be provided with positive drainage behind the wall.

A drainage system can be provided by either a 1-ft wide zone of crushed rock protected by filter fabric, or a 4-foot wide strips of Miradrain 6000 (or equivalent) placed at 8 to 10 feet on center. The crushed rock zone or Miradrain (or equivalent) strips may be placed near the base of the wall and should be connected to a perforated discharge pipe at the base of the wall. The drain pipe should consist of a minimum 4-inch-diameter perforated pipe placed with perforations down along the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel separated from the on-site soils by an appropriate filter fabric. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. If Class 2 Permeable Material is not available, ¾ inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric should be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

Waterproofing

We recommend that all retaining walls and walls below grade be waterproofed to prevent seepage of water or moisture due to cracks or water migration. Waterproofing should extend at least 5 feet above the design groundwater level (i.e., to 5 feet below existing ground surface) and that a qualified waterproofing consultant should be retained for recommendations of suitable waterproofing applications behind all walls below grade if necessary.

7.3 Retaining Walls and Miscellaneous Site Structures

We understand that the project may include short (less than 5 ft tall) retaining walls and other miscellaneous light structures such as enclosures, signs, light poles, and fences.

Retaining walls should be designed for lateral earth pressures and other recommendations provided in section 7.2. Footings for retaining walls and enclosure structures should be supported on native alluvial deposits or engineered fill bearing on extending to native alluvial deposits. Isolated and continuous footings at least 3 ft wide and with a minimum 2 ft of embedment may be designed using an allowable vertical bearing capacity of 1,500 and 1,000 psf, respectively. This value is for dead plus live loads and may be increased by one-third to accommodate transient loads that include wind or seismic loads. Based on our evaluation, we estimate the settlement of the footings designed in the manner recommended will be

less than 1 inch. The differential settlement between two adjacent footings could be up to about half of the total settlement. Additionally, up to about 3.5 inch of total seismically induced settlement is estimated in the event of a strong earthquake. Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.25 may be used between the foundations and the underlying native soils. The allowable passive resistance of undisturbed natural soils is recommended to be equal to the pressure developed by a fluid with a density of 100 pcf. The allowable passive resistance should be limited to a maximum value of 1,000 psf. The upper foot of the material should be ignored for calculating this value. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and passive resistance of the soils may be combined without reduction in evaluating the total lateral resistance.

Signs, light poles, and fences should be designed in conformance to Sections 1806 and 1807 of the California Building Code. A lateral bearing pressure of 100 lb/ft should be assumed for design when using approach in Section 1807.3.2 of the Building Code.

7.4 Earthwork and Grading

Earthwork operations at the site will include excavations for the removal of existing basement and pile caps of the existing foundation system. We recommend surveying the top of all the piles after demolition of the pile caps for record keeping purposes to aid future development planning. We also understand that all exterior pavements and other surface features in the project extent will also be removed.

Construction of the proposed Project, including all earthwork, excavations, subgrade preparation, and fill placement, should be performed in accordance with the applicable sections of the LA County Building Code and standards, OSHA safety requirements and the geotechnical recommendations provided in this report. Earthwork should be performed under the observation and testing of a geotechnical engineer. Field technicians should be familiar with the subsurface conditions of the project site and should have a thorough understanding of the assumptions and recommendations presented in this report.

The recommendations for site grades are presented in this section. Recommendations for temporary excavations are presented in the next section.

Final Slopes

Final site slopes should be constructed at slopes no steeper than 2:1 (horizontal to vertical) in native alluvium or engineered fill. Grading observation of slopes should be observed by the geotechnical engineer. Buttress fills or stabilization fills may be needed if fill or weak soil layers are present.

If desired changes in site grades cannot be achieved by slopes, permanent retaining walls should be considered. Recommendations for permanent retaining walls can be provided, if needed.

Drainage

Surface soil erosion due to water run off should be mitigated. We therefore recommend that storm water runoff be discharged in a manner to mitigate slope erosion. We also recommend considering vegetation cover or other erosion mitigation measures on the slopes.

7.5 Temporary Excavations

Temporary excavations up to a height of 4 feet can be cut vertically. Unshored excavations should not extend below a plane drawn at 1½:1 extending downward from adjacent existing footings.

Where space is available, temporary excavations can be made with slopes of 1:1 (horizontal to vertical). Where space is unavailable, shoring is recommended for the proposed excavations adjacent to existing streets and/or buildings.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings or heavy construction equipment, stockpile material etc. so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur.

We recommend that a qualified geotechnical firm observe the excavations and shoring installation, so that necessary modifications based on variations in the soil conditions can be made. Applicable safety requirements and regulations, including OSHA regulations, should be met.

Temporary Shoring Lateral Pressures

Temporary shoring will be required on the south side of the building where it meets the garden area. Excavations up to elevation 76.5ft feet are expected for removal of the deep pile caps and basement level, and a cantilever, braced, or tied-back shoring system can be used to support the sides of the proposed excavations. Given the deep groundwater level encountered, groundwater dewatering and control measures will not be required.

For the design of the shoring system, we recommend the following lateral earth pressures for drained conditions. For cantilever piles we recommend using the triangular lateral pressures with a maximum pressure equal to 40H (psf). For the design of braced or tied-back shoring, we recommend using a trapezoidal pressure distribution with a maximum pressure equal to 24H (psf), where H is the retained height in feet. These recommendations are shown on Figure 6. These pressures are for level ground behind the wall (i.e., no backslope).

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to traffic area should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the face of the shoring, the traffic surcharge may be omitted. In addition, any surcharge (live or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The details of the adjacent structures (elevation of foundation, loads, configuration, etc.) should be provided to us to estimate the pressure on the shoring walls due to surcharge, if applicable.

Design of Soldier Piles

Soldier pile shoring consists of installation of steel soldier piles in drilled holes and backfilling the holes with concrete. Lagging is placed between the soldier piles. For the design of soldier piles spaced at least 2 diameters apart on-center, the allowable lateral resistance value (passive pressure) below the bottom of the proposed excavation may be assumed to be 300 psf per foot of depth, up to a maximum of 3,000 psf for piles founded primarily in natural materials. The passive pressure includes a factor of safety of 1.5.

To develop the full lateral values, firm contact between the soldier piles and the in-situ soils must be achieved. Structural concrete should be used for that portion of the soldier pile below the bottom of the excavation. Lean-mix concrete may be used for that portion of the soldier pile below the bottom of the excavation provided that it will have sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the restrained earth may be taken as 500 psf. This value is based on the assumption that full bearing will be developed between the steel soldier beam and the lean-mix concrete and also between the lean-mix concrete and the retained earth.

Lagging

The soldier piles should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching effects in the soils. Therefore, the lagging can be designed for the

recommended earth pressure but limited to a maximum value of 400 psf. Careful installation of the lagging will be necessary to achieve bearing against the retained earth.

Tie-Back Anchor Design

Tieback friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. These anchors should extend to a minimum of 15 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

For preliminary design purposes, it may be estimated that drilled and grouted friction anchors would develop a soil friction of 750 psf along the anchors in the bonded zone. This value is provided for gravity grouted anchors. For pressure grouted anchors, a soil friction of 2,500 psf may be used along the anchors in the bonded zone. The capacities of the anchors should be determined by testing of the initial anchors as outlined below under the Tie-back Anchor Testing section.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6-feet on center, then no reduction in capacity is necessary. Closer spacing would require evaluation of an appropriate reduction factor.

Tie-Back Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. The anchors should be filled with concrete, placed by pumping from the tip out. The concrete should extend from the tip of the anchor to the active wedge. To minimize caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand. A small amount of cement may be used to allow the sand to be placed by pumping. The sand-cement mixture should fill the portion of the tieback anchor tightly and should be flush with the face of the shoring when finished.

Tie-Back Anchor Testing

The installation of the anchors and the testing of the completed anchors should be observed by a representative of a qualified geotechnical firm. The geotechnical engineer or his representative should select at least four of the initial anchors for 24-hour 200% tests and 10% of the total number of anchors for “quick” 200% tests to verify in the field the friction value assumed in this report. Also, we recommend that the 200% tests be performed at representative locations around the site and not concentrated in a single area.

The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed ¾ inch during the 24-hour period, measured after the 200% test load is

applied. If the anchor movement after the 200% load has been applied for 12 hours is less than ½ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed ¼ inch during the 30-minute period.

All of the production anchors should be pre-tested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked-off load should be verified by rechecking the load on the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by a qualified geotechnical firm.

Raker Bracing

Raker bracing, if used, should be supported by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 1,500 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade and is founded in the native alluvium. To reduce the deflection of the shoring, the rakers should be preloaded to the design load.

Deflection

We recommend the shoring be designed to limit deflections at the top of the shored excavation to less than 1 inch. The design deflections should further be limited to ½ inch where a structure is within a 1:1 plane projected up from the base of the excavation. If greater deflection occurs during construction, additional bracing may be necessary to prevent settlement and loss of support from beneath and adjacent to the shored excavation.

Monitoring

Monitoring of the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles. Initial survey should be taken prior to the first level of excavation so that an accurate baseline may be established.

We recommend that the initial survey and monitoring program also include any adjacent existing structures. Photographs and videos of the existing structures are recommended as part of the documentation process.

Monitoring considerations should be discussed further with the design consultants and the contractor when the design of the shoring system has been finalized.

Removal

Temporary shoring elements should be removed prior to end of construction as per local guidelines and practices.

7.6 Stormwater Infiltration

We understand that after completion of the Project, the site may be left in-place for a period of time till future development, if any, and surface drainage will be required. As part of the investigation, percolation testing was performed at two boring locations (GP-1 and GP-2) shown on Figure 2. Results of these tests show that the surficial (i.e., upper 5 ft) fine-grained soils at the site are not suitable for infiltration, however infiltration may be feasible within deeper sandy zones. Results of the percolation testing can be found in Appendix C.

7.7 Earthwork

General

Earthwork should be performed in accordance with the applicable sections of the LA County Building Code, as well as the recommendations in this report.

Removal of Existing Foundation Elements

Any below-grade pile cap should be removed and backfilled with engineered fill to the surrounding grade. The requirements for fill material and its placement are described later in this section. If removals extend deeper than described above (e.g., complete pile removal), the deeper portion of the removal can be backfilled with concrete slurry with a minimum strength of 150 psi as long as at least 3 ft of engineered fill is placed above the top of the slurry.

Subgrade Preparation and Moisture Conditioning

Areas excavated to receive fill should be cleared and stripped of all debris, deleterious matter, organics and vegetation. Cleared and grubbed material should be disposed of offsite.

After clearing the site of existing debris, the exposed subgrade should be observed for debris, organic material, or other undesirable materials. The exposed subgrade should then be proof-rolled so as to allow placement of any required fill. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Fill Materials and Placement of Fill

The on-site excavated granular materials such as sands and silty sands can be used as engineered fill (except for bedding and shading as described above). The existing fill materials, once debris and vegetation are removed, may be re-used as compacted fill. Oversized material (greater than 6 inches in longest dimension) should be removed from excavated material prior to reuse as engineered fill.

Imported fill material should be granular, non-corrosive, free of organic matter or other deleterious material. The Expansion Index of the fill material should be less than 35 and fill material should have a fines content (passing #200 sieve) less than 40 percent. Oversize material (larger than 6 inches in diameter) should not be used in the fill. All imported fill material should be approved by the geotechnical engineer prior to placement. A sample of proposed fill material(s) should be submitted to the geotechnical engineer for testing at least three business days prior to use at the site.

Fill material should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying elements such as slabs and paving. Backfill should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction. The on-site soils excluding clayey soils may be used in the compacted backfill.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to a building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

The exterior grades should be sloped to drain away from the foundation to prevent ponding of water.

Compaction

The preparation of the subgrade, excavations for the mat foundation and reworking of on-site soils and compaction of any required fills or backfill should be observed and tested by a representative of a qualified geotechnical firm.

The bottom of the excavations should be proof-rolled so as to allow placement of any required. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Fill material should be placed in loose lifts generally no greater than 8 inches thick. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

Any required fill should be compacted to a minimum of 95 percent maximum dry density as determined in accordance with ASTM D1557. Fill with 24 inch below the pavements should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction The field density of fill should be determined in accordance with the Sand Cone Method (ASTM D1556) or the Nuclear Method (ASTM D6938).

7.8 Geotechnical Observation

We recommend that a qualified geotechnical engineer or his representative observe the condition of the final subgrade soils immediately prior to foundation construction, and if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proof-rolling and delineation of areas requiring over-excavation.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

8 GENERAL CONDITIONS

The conclusions and recommendations presented in this report are based upon GeoPentech's understanding of the project and the assumption that the subsurface conditions encountered during construction do not deviate appreciably from those disclosed by the field exploration. If our assumptions differ from the actual design conditions, the differences should be brought to our attention so that we can modify our recommendations.

The information presented in this report is intended to be used based on the assumptions made in this report and the information provided to us. This information is subject to change once the locations, configurations, layout, or features of the proposed buildings are changed. It is the responsibility of the Owner to bring any changes in the proposed structures and any deviations of the subsurface conditions to the attention of GeoPentech.

Professional judgments presented in this report are based on an evaluation of the technical information gathered and GeoPentech's general experience in the field of geotechnical engineering. GeoPentech does not guarantee the performance of the project in any respect, only that the engineering work and judgment rendered meet the standard of care of the geotechnical profession at this time and in this locale.

9 REFERENCES

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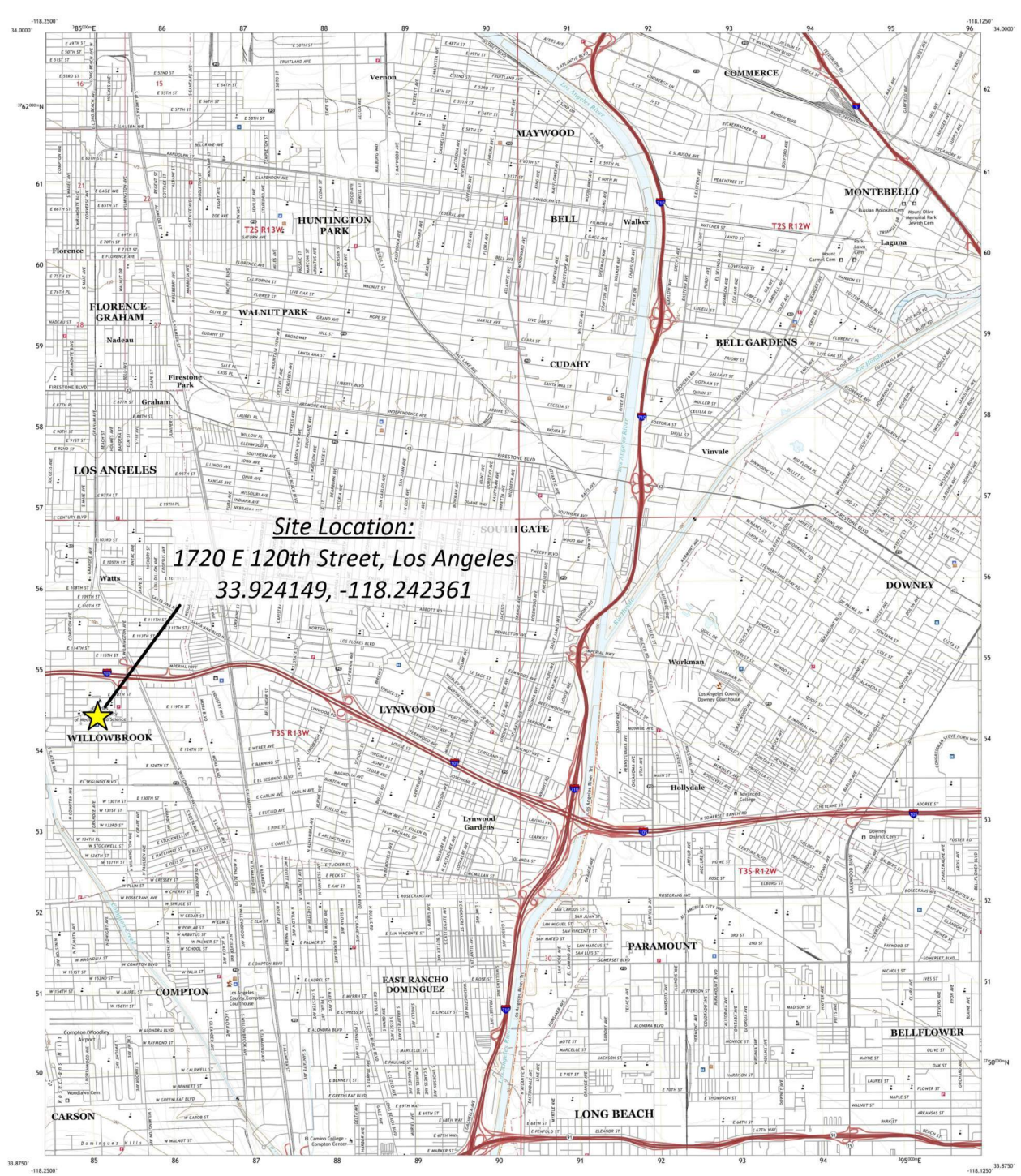
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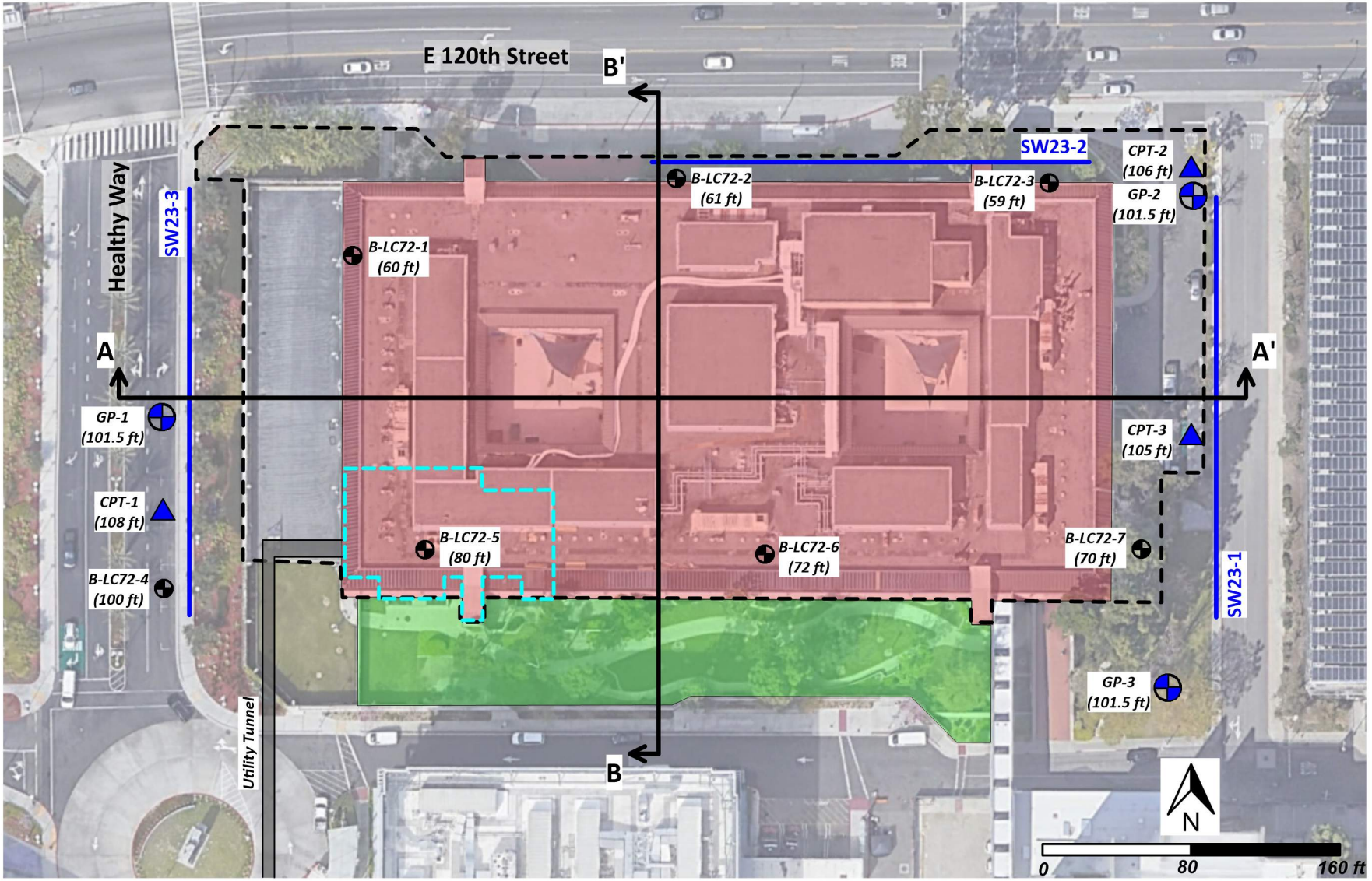


Source: United States Geologic Survey (USGS), topoView, available at [https://ngmdb.usgs.gov/topoview/].

SITE LOCATION MAP

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure 1

GeoPentech

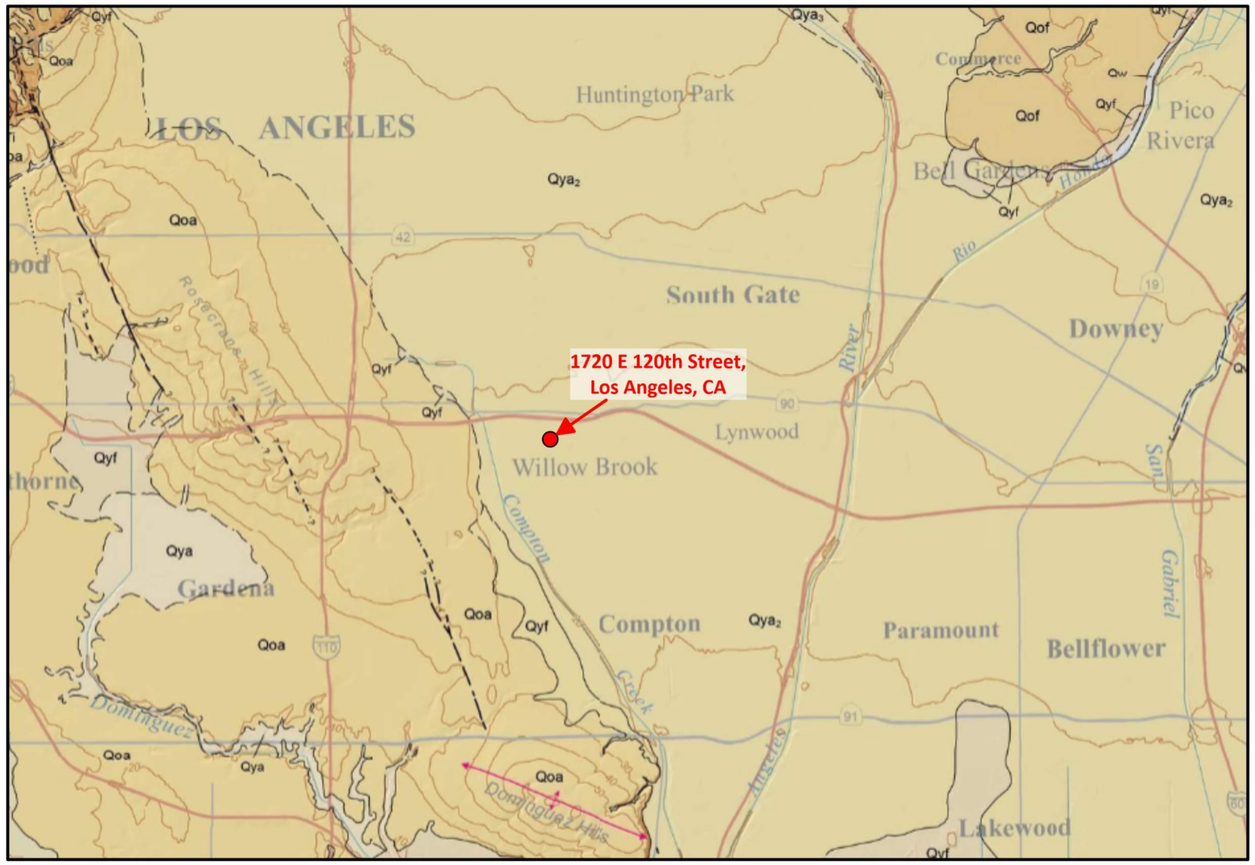


- Approximate Project Extents
- Hawkins Building Extents
- Park Extents
- Approximate Extents of Basement Level
- GP-1 (Depth) — Approximate Location of Borings
- CPT-1 (Depth) — Approximate Location of CPTs
- B-LC72-1 (Depth) — Approximate Location of previous Borings (LeRoy Crandall, 1972)
- SW23-1 — Geophysical Survey Lines (2023)
- Cross Sections

SITE PLAN

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure 2

GeoPentech



Qw **Alluvial Wash Deposits (Late Holocene)** - Unconsolidated deposits of silt, sand, and gravel in active and recently active washes; artificially channelized in part.

Qya **Young alluvium, individed (Holocene and late Pleistocene)** - Poorly consolidated, poorly sorted, permeable flood-plain deposits consisting of soft clay, silt and loose to moderately dense sand and silty sand.

Qya2 **Young alluvium, Unit 2 (Holocene and late Pleistocene)** - poorly consolidated, poorly sorted, permeable flood-plain deposits consisting of soft clay, silt and loose to moderately dense sand and silty sand.

Qyf **Young alluviual fan deposits, undivided (Holocene and late Pleistocene)** - Poorly consolidated and poorly sorted clay, sand, gravel and cobble alluvial fan and valley deposits.

Qoa **Old alluvium, undivided (late to middle Pleistocene)** - Fluvial sediments deposited on canyon floors. COnsists of moderately well consolidated, poorly sorted, permeable, commonly

Qof **Old alluvial fan deposits, undivided (late to middle Pleistocene)** - Moderately to well consolidated, moderately sorted sand, clay , and silt.

Coordinate System:
Universal Transverse Mercator, Zone 11N,
North American Datum 1927.

Base Map:
Onshore topographic contours and shadec relief image derived from USGS 1/3 arc-second National Elevation Dataset (NED). Offshore bathymetry and terrain model from USGS multibeam data (Dartnell and Gardner, 1999) and NOAA. Hydrology from the National Hydrography Dataset (NHD).

Approximate Mean Declination, 2015

N

0 1.75 3.5 miles

GEOLOGIC MAP OF THE LONG BEACH 30' × 60' QUADRANGLE, CALIFORNIA

Version 2.0

Compiled by

George J. Saucedo[1], H. Gary Greene[2], Michael P Kennedy[1], and Stephen P Bezore[1]

2016

Digital preparation by

Carlos I. Gutierrez[1], Janet Tilden[2], Jason D. Little[1], Eric V. Niven[2], Marina T. Mascorro[1], Eric W. Ford[1], and Benjamin Middendorf[1]

1. Department of Conservation, California Geological Survey
2. Moss Landing Marine Laboratories

Geology compiled 2002-2007; modified 2015.

This geologic map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 02HQAG0018.

LOCATION MAP

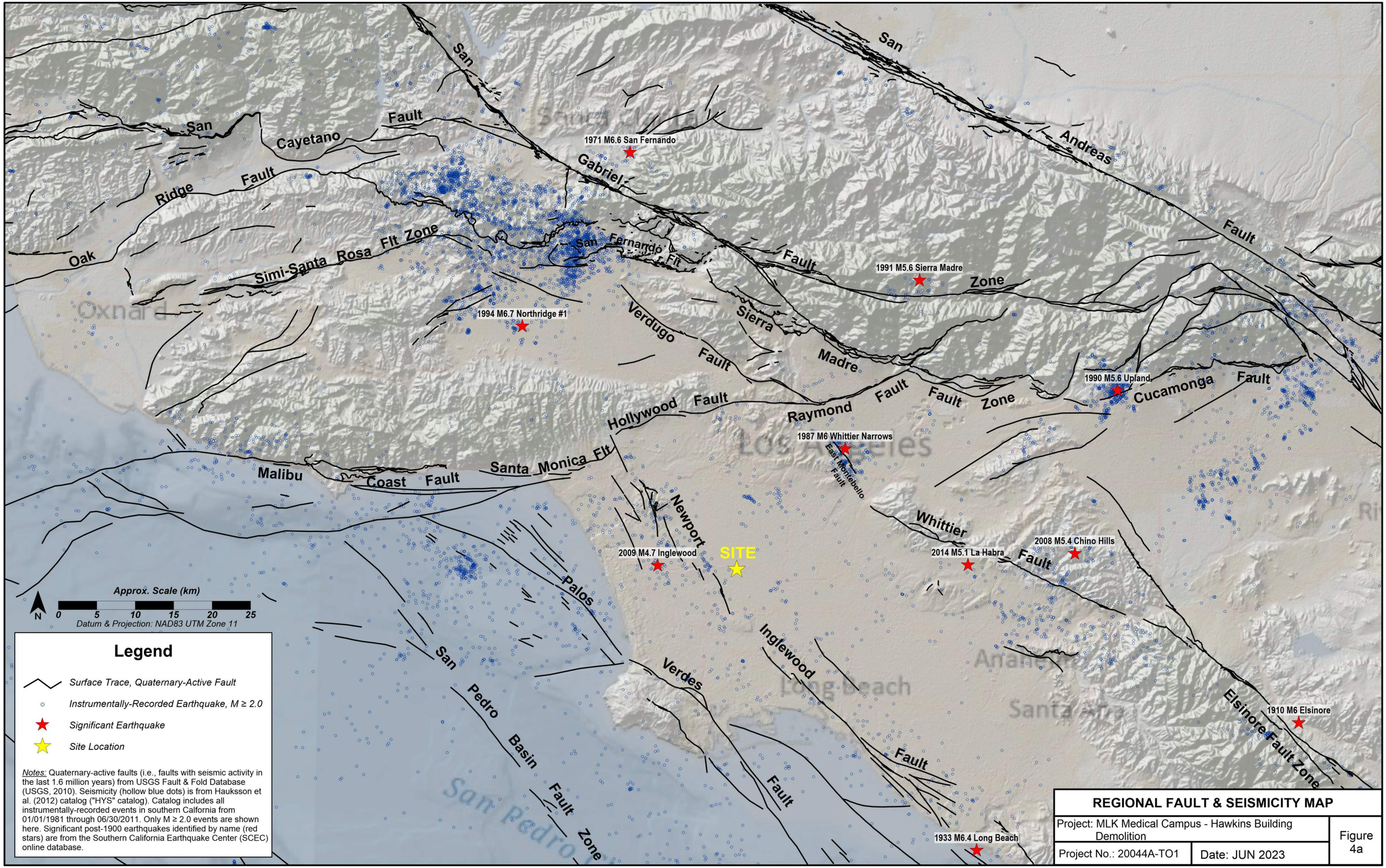
Loong Beach 30x60-Minute Quadrangle

Source: United States Geologic Survey (USGS), MapView, available at [https://ngmdb.usgs.gov/mapview/].

REGIONAL GEOLOGY MAP

Project: MLK Medical Campus - Hawkins Building Demolition		Figure 3
Project No.: 20044A-TO1	Date: JUN 2023	

GeoPentech



Legend

- *Surface Trace, Quaternary-Active Fault*
- *Instrumentally-Recorded Earthquake, M ≥ 2.0*
- *Significant Earthquake*
- *Site Location*

Notes: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Seismicity (hollow blue dots) is from Hauksson et al. (2012) catalog ("HYS" catalog). Catalog includes all instrumentally-recorded events in southern Calfornia from 01/01/1981 through 06/30/2011. Only M ≥ 2.0 events are shown here. Significant post-1900 earthquakes identified by name (red stars) are from the Southern California Earthquake Center (SCEC) online database.

REGIONAL FAULT & SEISMICITY MAP

Project: MLK Medical Campus - Hawkins Building Demolition		Figure 4a
Project No.: 20044A-TO1	Date: JUN 2023	

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Ventura-Pitas Point
Oak Ridge (Onshore)
Northridge
Sim-Santa Rosa
Elysian Park
Puente Hills (LA)
Puente Hills
Puente Hills (Santa Fe Springs)
Puente Hills (Coyote Hills)
Anaheim
Peralta Hills
Anacapa-Dune
Compton
Compton
SITE

Approx. Scale (km)
0 5 10 15 20 25
Datum & Projection: NAD83 UTM Zone 11

Legend

- Surface Trace, Quaternary-Active Fault
- Blind Trace, Top of Fault (Both Fault Models)
- Blind Thrust Footprint (Both Fault Models)
- Blind Trace, Top of Fault (Fault Model 1)
- Blind Thrust Footprint (Fault Model 1)
- Blind Trace, Top of Fault (Fault Model 2)
- Blind Thrust Footprint (Fault Model 2)
- Site

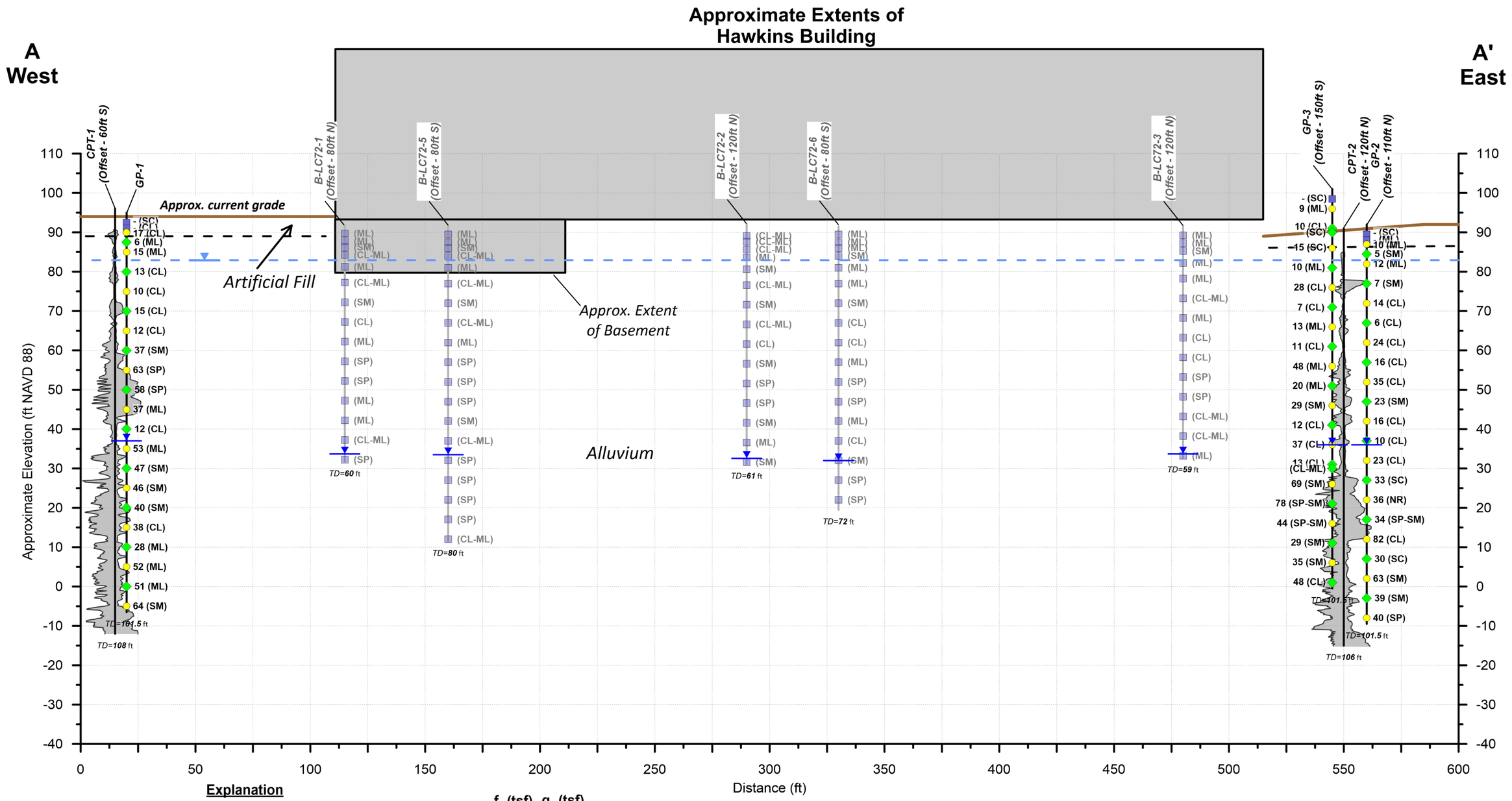
Notes: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Blind thrust fault traces, footprints, and alternative models based on UCERF3 (WGCEP, 2013) and Hernandez and Treiman (2014).

REGIONAL FAULT MAP - BLIND THRUST FAULTS

Project: MLK Medical Campus - Hawkins Building Demolition		Figure 4b
Project No.: 20044A-TO1	Date: JUN 2023	

GeoPentech

Geologic Cross-Section A-A'



Explanation

- (ML): Grab Sample (USCS)
- 50/6" (ML): Measured Cal. Mod. Blowcount per foot (USCS)
- 50/6" (ML): Measured SPT Blowcount per foot (USCS)
- Encountered Groundwater
- Historic High Groundwater

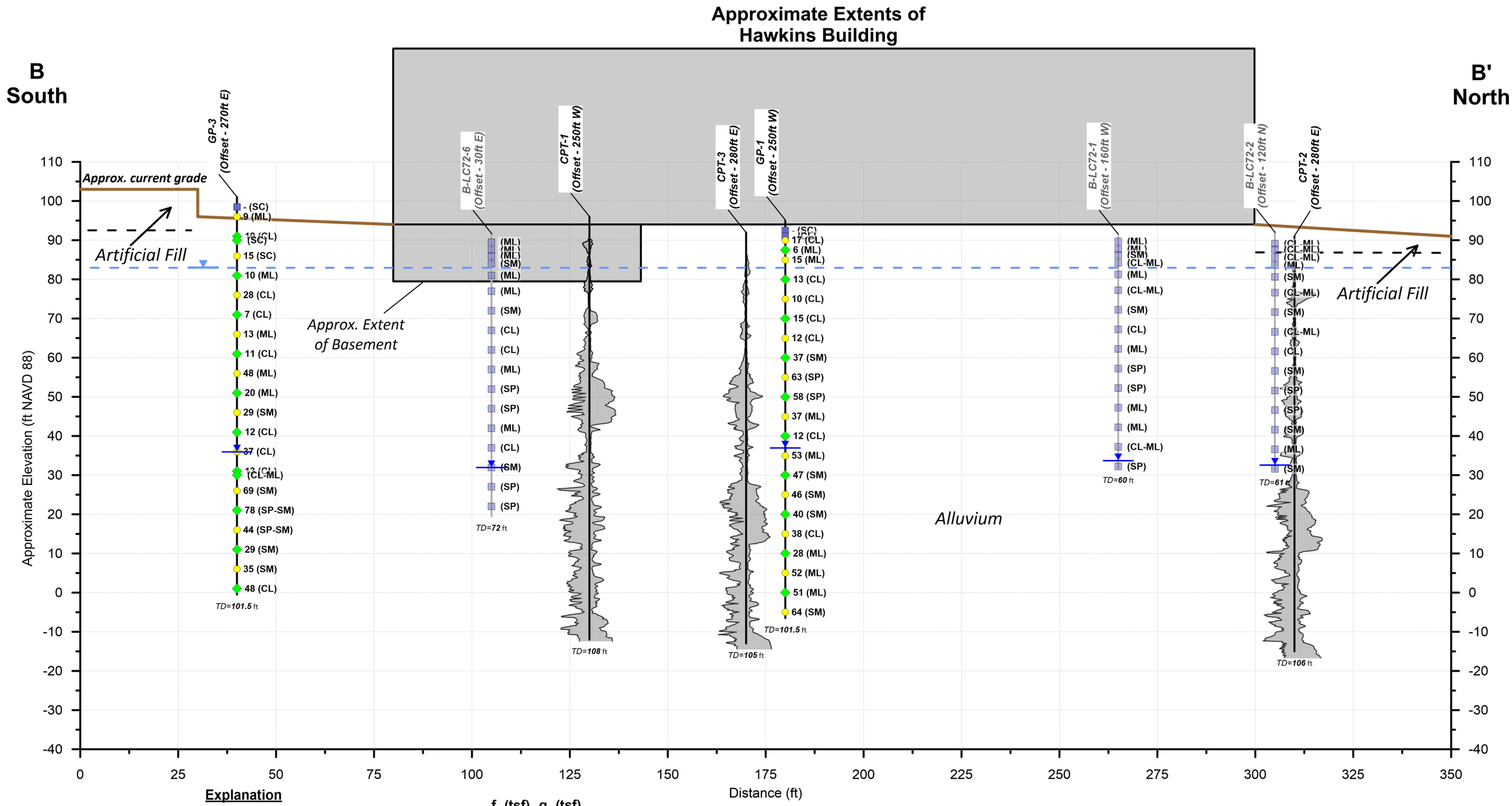
f_s (tsf) q_c (tsf)

10 5 0 400 800

Geologic Cross-Section A-A'		
Project: MLK Medical Campus - Hawkins Building Demolition		Figure 5a
Project No.: 20044A-TO1	Date: JUN 2023	

GeoPentech

Geologic Cross-Section B-B'



Explanation

- (ML): Grab Sample (USCS)
- 50/6" (ML): Measured Cal. Mod. Blowcount per foot (USCS)
- 50/6" (ML): Measured SPT Blowcount per foot (USCS)
- Encountered Groundwater
- Historic High Groundwater

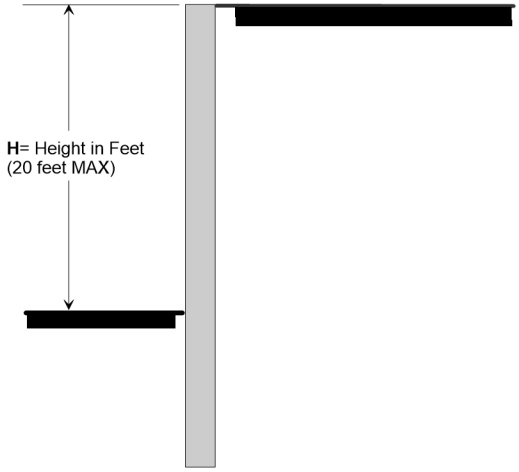
f_s (tsf) q_c (tsf)

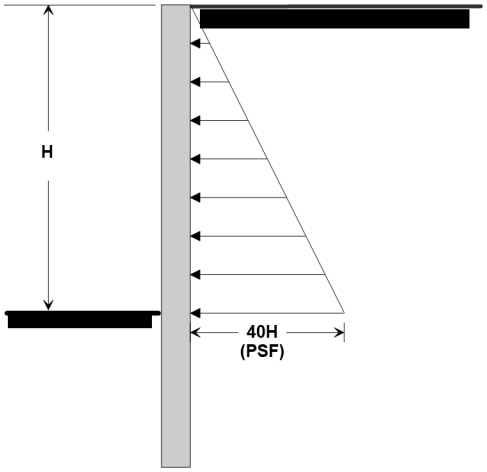
10 5 0 400 800

Geologic Cross-Section B-B'		
Project: MLK Medical Campus - Hawkins Building Demolition		Figure 5b
Project No.: 20044A-TO1	Date: JUN 2023	

GeoPentech

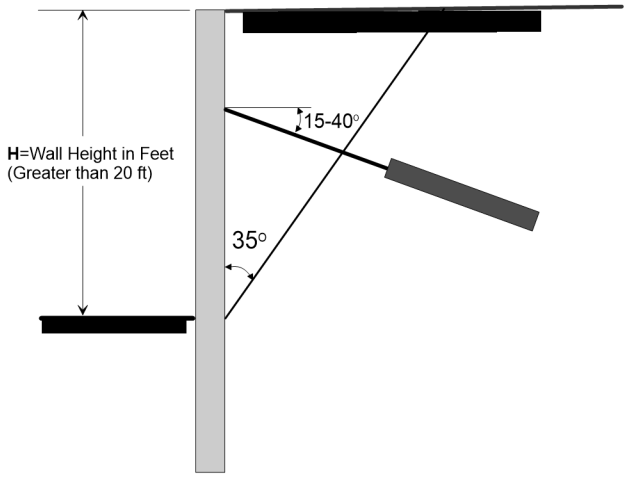
Cantilever Shoring

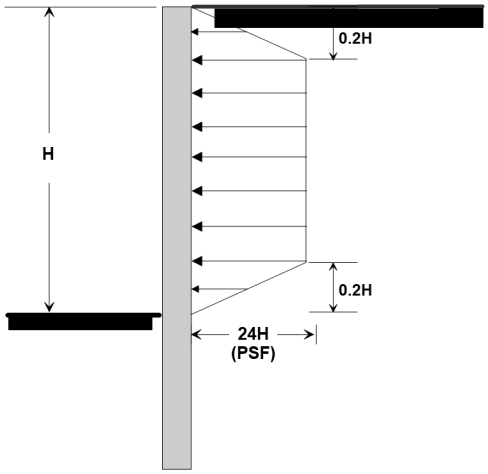




1. Lateral Earth Pressure is 40H with a triangular distribution.
2. The lateral pressure above assumes no build up of hydrostatic pressure behind the shoring.
3. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

Tiedback Shoring





1. Maximum Lateral Earth Pressure is 24H with a trapezoidal distribution.
2. The lateral pressure above assumes no build up of hydrostatic pressure behind the shoring.
3. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

TEMPORARY SHORING DETAILS

Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Date: JUN 2023	Figure 6

APPENDIX A

EXISTING FOUNDATION ELEMENTS

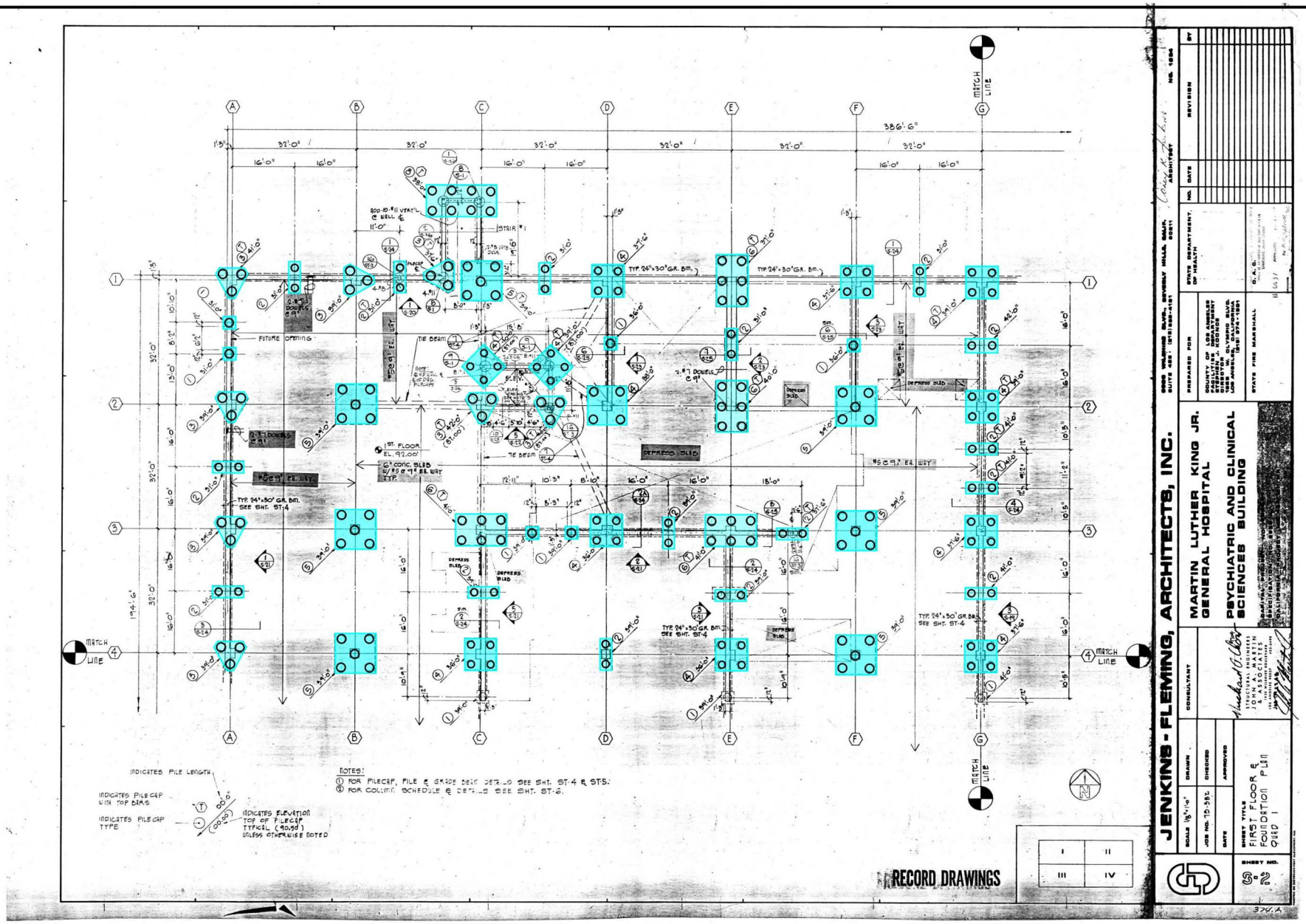
A.1 EXISTING FOUNDATION ELEMENTS

GeoPentech was provided with 6 sheets of drawings that contain foundation plans and foundation details for the project building (Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheets S-1 through S-5, ST-4, and ST-5). These sheets are shown in Figures A-1 through A-7 with annotations highlighting the pile cap locations. Figure A-5 also highlights the 9 ft wide utility tunnel that is planned to be capped. Across the 196 pile caps, depths of pile caps range from 3 to 5.5 feet, and the depth of piles range from 31 to 42 feet. Table A-1 below summarizes details on the pile caps planned to be removed during the Project. We understand that the current civil plans (RBB Architects, dated March 21st, 2023) reference an elevation of 93.84 ft (NAVD 88) for the first floor, while the previous foundation plans by Jenkins - Fleming, Architects, Inc., (1976) reference an elevation of 92 ft (NGVD 29). The Basement level elevation is referenced as 79.35 ft in the civil plans. Table 1 below summarizes the top and bottom elevations of the pile caps for both NGVD 29 and NAVD 88, where a conversion factor of +1.84 ft was used to estimate the NAVD 88 elevations.

Table A-1 – Pile Cap Details

Floor	Pile Cap Type[1]	Amount	Depth (ft)	Pile Cap Elevations (ft)			
				NGVD 29		NAVD 88	
				Top	Bottom	Top	Bottom
1	1	26	3	90.5-86.5	87.5-83.5	92.3-88.3	89.3-85.3
	2	48	3.5	90.5-82.5	87-79	92.3-84.3	88.8-80.8
	3	32	3.5	90.5-81.5	87-78	92.3-83.3	88.8-79.8
	4, 4A	17	3.5, 3	90.5-86.5	87.5-83	92.3-88.3	89.3-84.8
	5	26	3.5	90.5-83.5	87-80	92.3-85.3	88.8-81.8
	6	11	4.25	90.5-84.8	86.3-80.5	92.3-86.6	88.1-82.3
	8	4	4.25	90.5-87	86.3-82.8	92.3-88.8	88.1-84.6
Basement	1	17	3	76.5	73.5	78.3	75.3
	2	4	3.5	76.5-75.5	73-72	78.3-77.3	74.8-73.8
	3	1	3.5	75.5	72	77.3	73.8
	4	5	3.5	76.5-75.5	73-72	78.3-77.3	74.8-73.8
	5	2	3.5	76.5-75.5	73-72	78.3-77.3	74.8-73.8
	6, 6A	1	4.25	76.5	72.3	78.3	74.1
	8	1	4.25	76.5	72.3	78.3	74.1
	16	1	5.5	76.5	71	78.3	72.8

[1] Based on typical pile cap plans by Jenkins - Fleming, Architects, Inc., (1976)

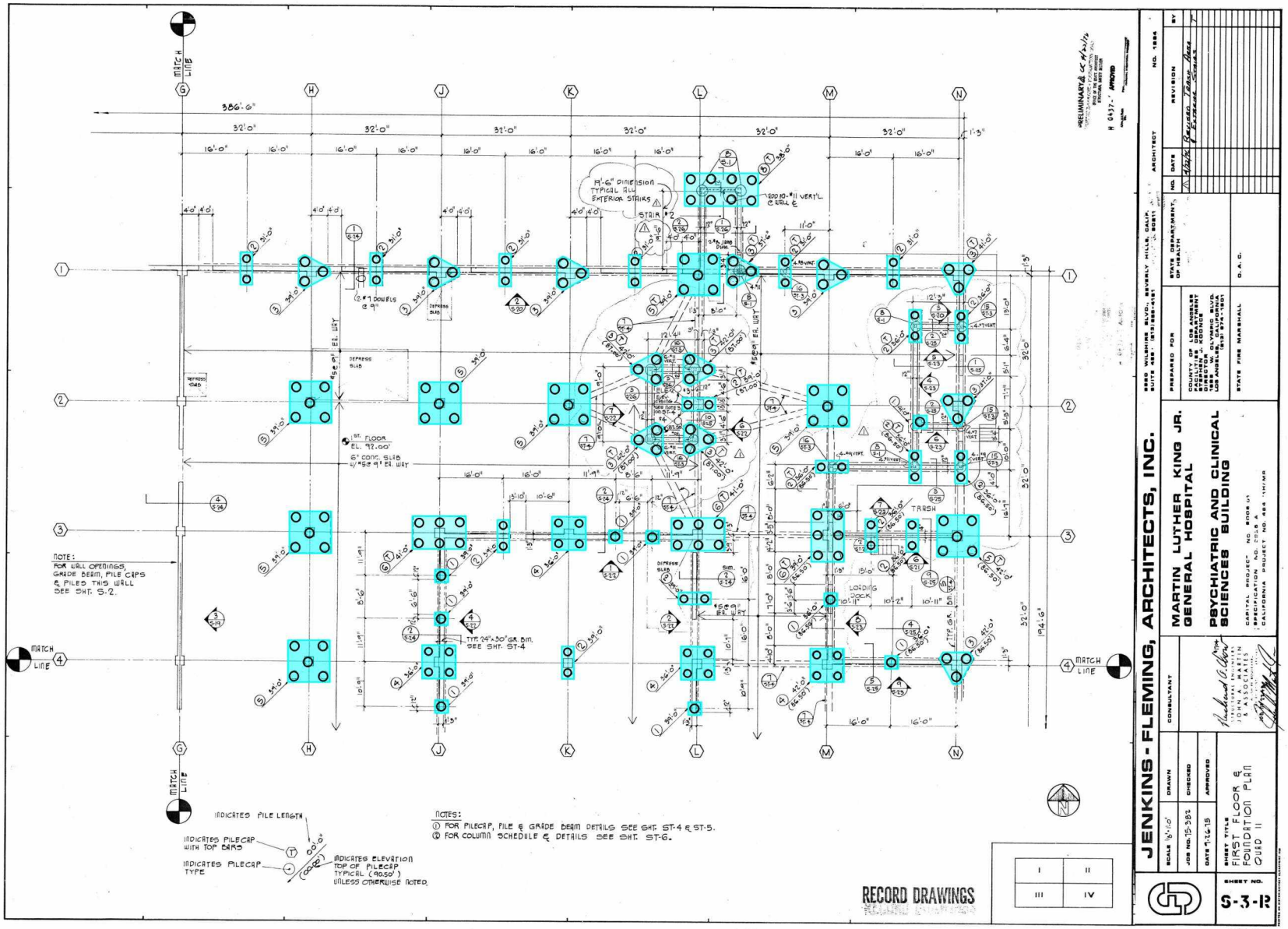


Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet S-2

EXISTING FOUNDATIONS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-1

GeoPentech



Pile cap — Piles

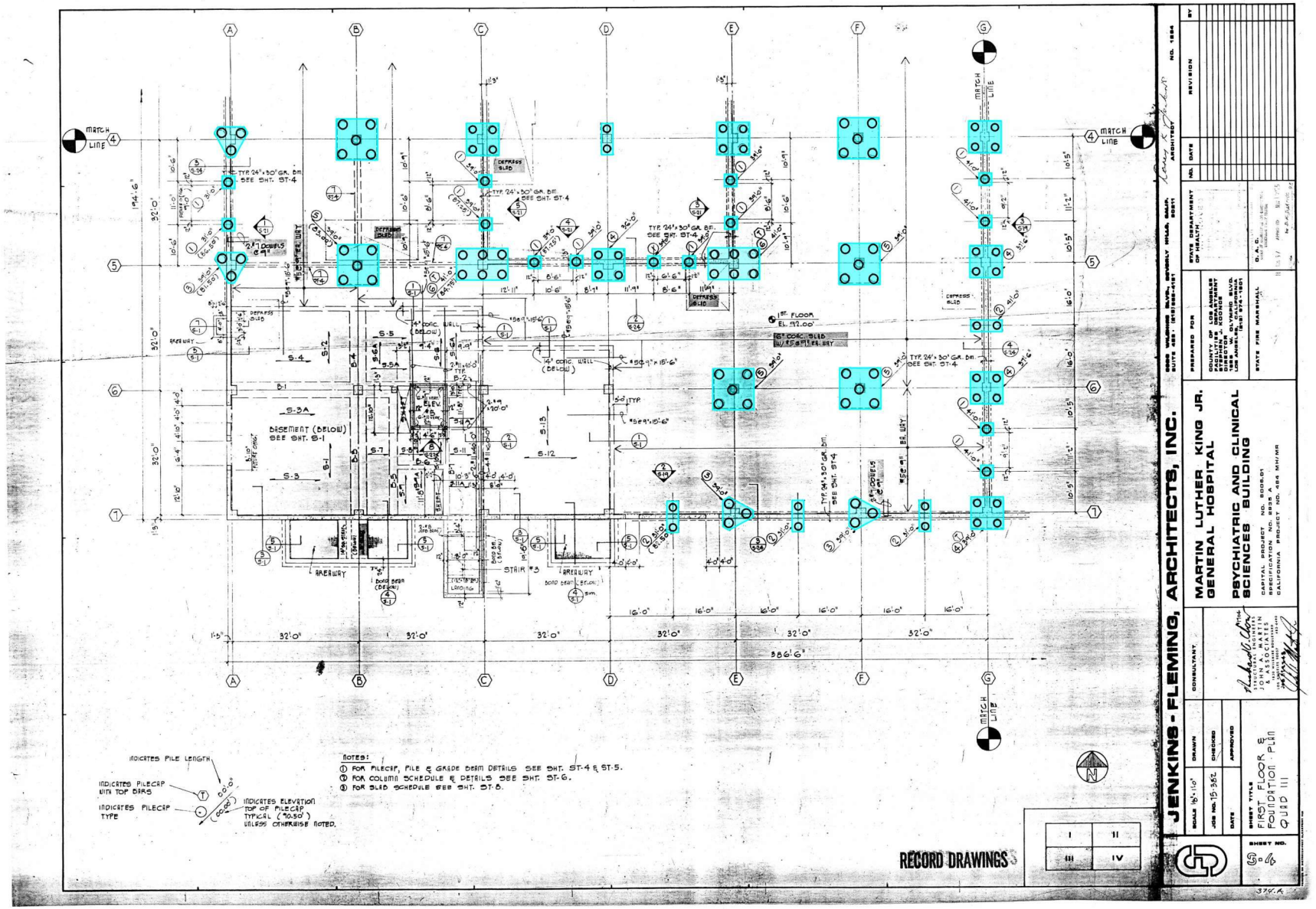
Existing Foundation

Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet S-3-R

EXISTING FOUNDATIONS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-2

GeoPentech

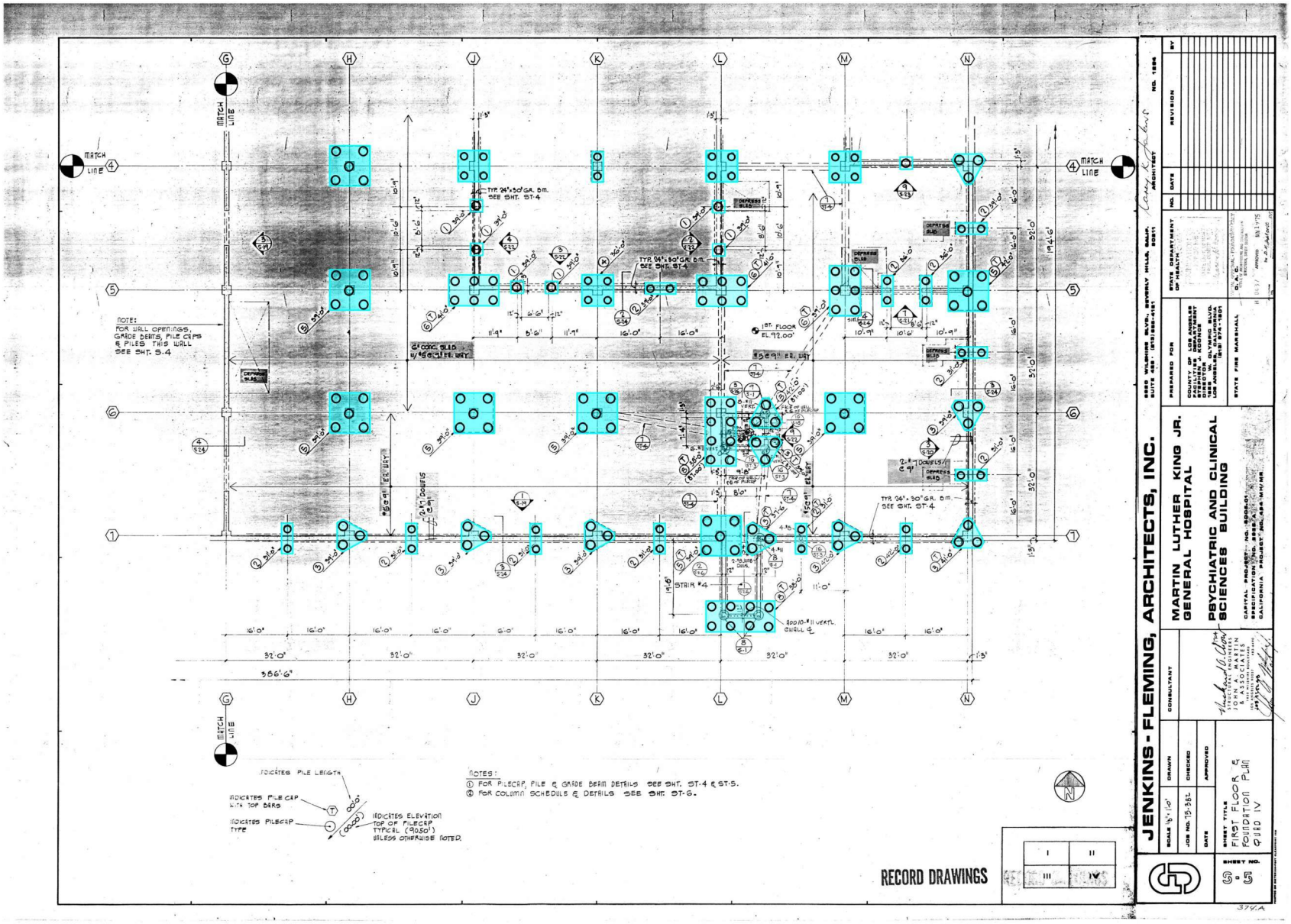


Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet S-4

EXISTING FOUNDATIONS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-3

GeoPentech



Pile cap — Piles

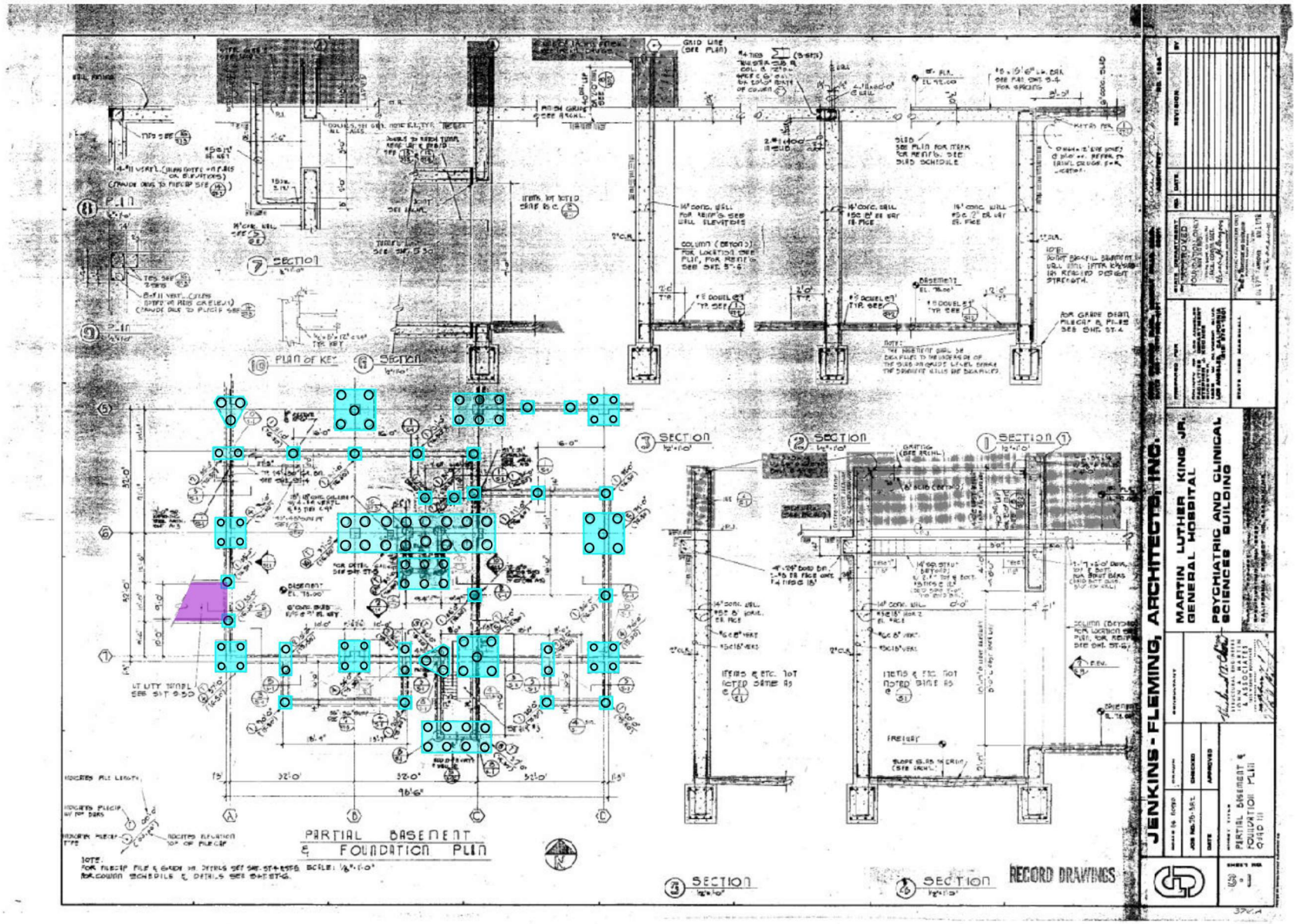
Existing Foundation

Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet S-5

EXISTING FOUNDATIONS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-4

GeoPentech



Pile cap — Piles

Existing Foundation

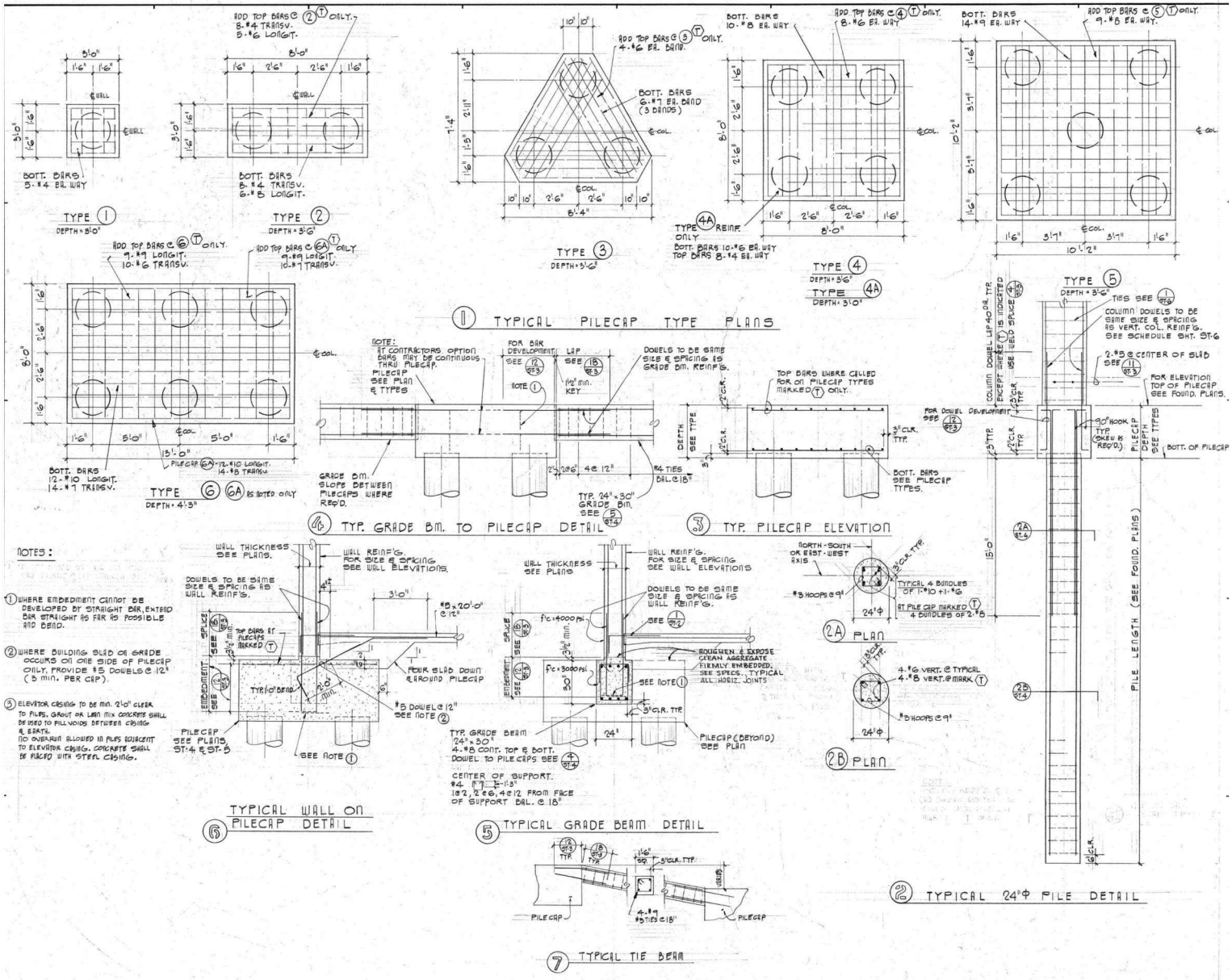
Existing Utility Tunnel

Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet S-2

EXISTING FOUNDATIONS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-5

GeoPentech

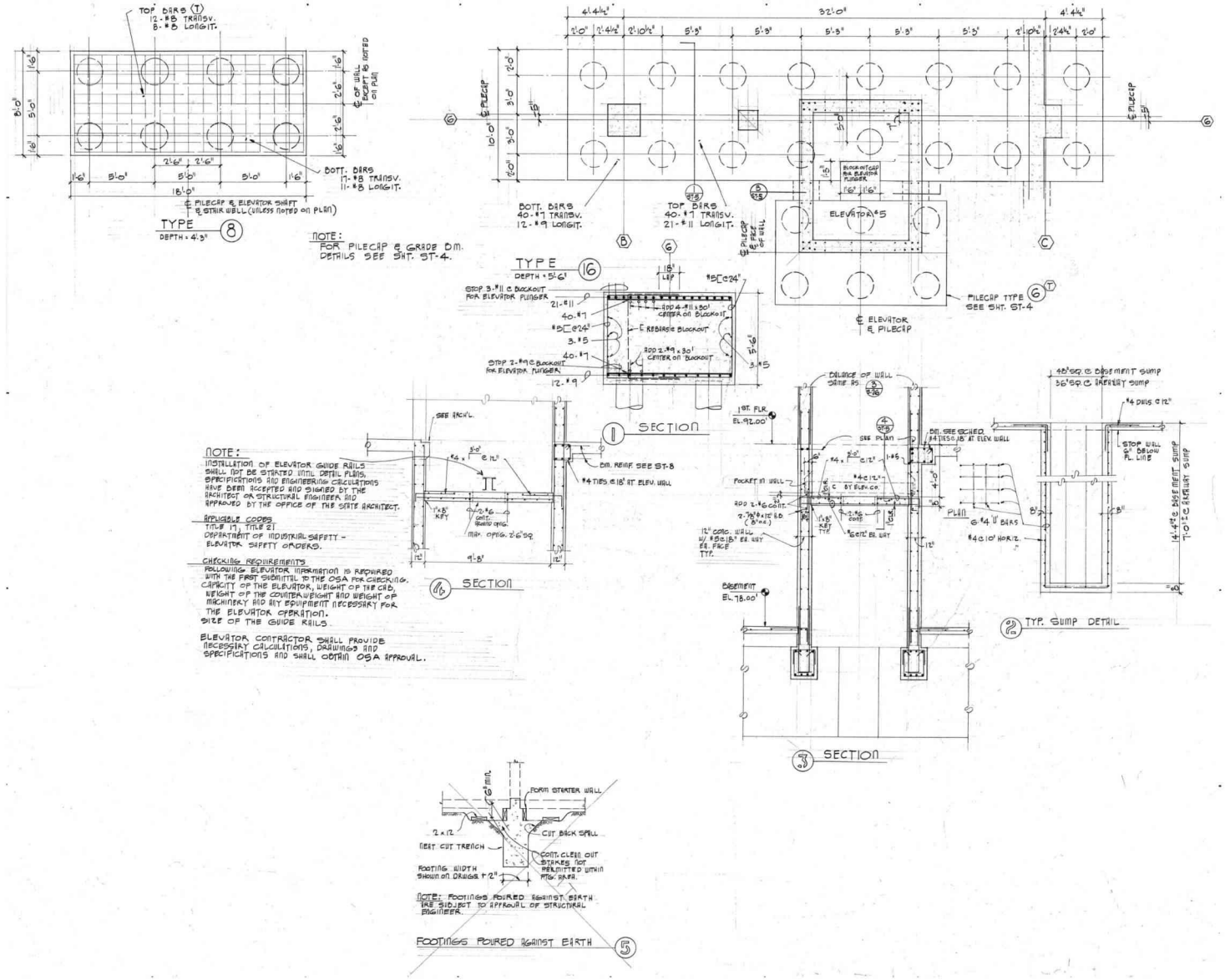


Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet ST-4

FOUNDATION DETAILS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-6

GeoPentech



Source: Jenkins - Fleming, Architects, Inc., (1976), "Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building", Sheet ST-5

FOUNDATION DETAILS

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure A-7

GeoPentech

APPENDIX B

PREVIOUS FIELD INVESTIGATIONS

B.1 PREVIOUS FIELD INVESTIGATIONS

This Appendix includes the nearby subsurface data gathered previously by LeRoy Crandall and Associates (1973). The attached logs include soil descriptions, moisture content, and density values for seven (7) bucket auger borings.

B.2 REFERENCES

Jenkins - Fleming, Architects, Inc., 1976, “Martin Luther King Jr. General Hospital, Psychiatric and Clinical Sciences Building”, Sheets A-1 and A-2, July 30, 1976.

BORING 3 (CONTINUED)
DATE DRILLED February 3, 1972
EQUIPMENT USED 24"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	
85		7.9	109		SP SAND - fine, light brown
30	40	7.5	97		Layer of Silty Sand
35	45	8.7	104		SM SILTY SAND - fine, light greyish-brown
40	50	28.9	94		ML SANDY SILT - brown
35	55	29.3	94		Grey
30	60	18.9	112		SM SILTY SAND - fine, grey

NOTE: Water encountered at 59'. No caving

BORING 2
DATE DRILLED February 3, 1972
EQUIPMENT USED 24"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	
90					ELEVATION 90.1
		19.6	80		ML FILL - SANDY SILT - pieces of wood and glass, brown
85	5	14.4	103		ML CLAYEY SILT - greyish-brown Sandy
		12.7	103		
80	10	26.5	95		ML SANDY SILT - clayey, grey
		18.0	98		SM SILTY SAND - fine, light brown
75	15	34.1	86		ML CLAYEY SILT - dark greyish-brown
70	20	26.4	93		SM SILTY SAND - fine, grey
65	25	19.9	110		ML CLAYEY SILT - some organic matter, dark grey
					SM SILTY SAND - fine, light grey
60	30	18.4	111		CL SANDY CLAY - some silt, light grey Few cemented lumps
					SM SILTY SAND - fine, light brown

BORING 1 (CONTINUED)
DATE DRILLED February 3, 1972
EQUIPMENT USED 24"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	
50	40	4.3	102		No Silt
					SM SILTY SAND - fine, brown
45	45	7.5	108		SM SILTY SAND - light brown
					SP SAND - fine, light brown
40	50	28.7	94		ML SANDY SILT - light greyish-brown
					ML CLAYEY SILT - some organic matter, grey
35	55	34.1	86		
30	60	16.9	114		SP SAND - fine, grey

NOTE: Water encountered at 59'. Caving from 32' to 40' and 47' to 60' (to 24" in diameter).

BORING 1
DATE DRILLED February 3, 1972
EQUIPMENT USED 24"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	
90					ELEVATION 89.7±
		16.2	95		ML SANDY SILT - dark brown
85	5	19.7	101		
		18.5	98		SM SILTY SAND - fine, brown
80	10	30.9	89		ML CLAYEY SILT - sandy, dark greyish-brown
		19.7	103		ML SANDY SILT - light brown
75	15	23.7	92		ML CLAYEY SILT - dark brown
70	20	17.2	97		SM SILTY SAND - fine, light greyish-brown Thin layers of Sand
					ML SANDY SILT - grey
65	25	17.2	113		CL SANDY CLAY - some silt, light brown
60	30	19.2	108		ML SANDY SILT - some silt, light brown
					SP SAND - fine, light brown
55	35	7.3	107		Thin layers of Silt

E 10 000 E 10 200 E 10 400 E 10 600 E 10 800 E 11 000 E 11 200 E 11 400 E 11 600 E 11 800 E 12 000 E 12 200

N 11 000 N 10 800 N 10 600 N 10 400 N 10 200 N 10 000

PHASE II DEMOLITION AREA:
EXISTING PAVING, LIGHT STANDARDS, WALKS, CURBS, CURB CUTS, WALLS, ETC. TO BE DEMOLISHED FOR PHASE II LANDSCAPING.

EXISTING CONSTRUCTION BARRICADE
120TH STREET
CONTRACTOR'S ENTRY F.H.
SERVICE ROAD F.H.
EMERGENCY ROAD
PARKING ENTR.
MAIN ENTRY
N.89° 54' 51"W. 2189.20
OAK WATER VAULT
L.A. CO. NO 69122
LOADING DOCK
CONTRACT LIMITS
EXISTING BARRICADE
EMPLOYEE PARKING
PSYCHIATRIC AND CLINICAL SCIENCES BUILDING
EMERGENCY ENTRY RAMP
CONTRACTOR'S YARD
NEW BARRICADE FOR PHASE II CONSTRUCTION
THESE LIMITS OF CONTRACTOR'S YARD ARE APPROXIMATE (CONTRACTOR TO VERIFY EXACT LOCATION)
CONTRACTOR'S ENTRY
PARKING
CONTRACT LIMITS
NEW BRIDGE TO CONNECT TO EXISTING ACUTE UNIT
F.H. (EXISTING TO REMAIN)
ACUTE EMERGENCY ENTRY
VISITORS PARKING
N. 0° 05' 09" E. 907.70'
EXISTING UTILITY TUNNEL TO BE CONSTRUCTED IN PHASE I
NEW PHASE II BARRICADE FOR BRIDGE CONSTRUCTION
CONTINUOUS FLOW OF TRAFFIC ACROSS BRIDGE AREA TO BE MAINTAINED DURING COURSE OF CONSTRUCTION
EXISTING UTILITY TUNNEL
ENTRY
ENTRY
COMPTON AVENUE
N. 1° 10' 25" W. 585.79'
WILMINGTON AVENUE
FIND OUT LOCATION OF CONST BENCHMARK & MOUNT FOR PHASE 4
INTERNS & PHYSICIANS' BLDG. L.A.CO.BLDG.NO. 5581
SERVICE AND SUPPLY BUILDING
COOLING TOWER
MAINTENANCE BUILDING
CENTRAL HEATING & COOLING PLANT
LAUNDRY BUILDING
NEW BARRICADE FOR TUNNEL EXTENSION
ACUTE UNIT BUILDING L.A. CO. BLDG. NO. 5035
AUDITORIUM
PARKING
SERVICE BUILDING
SERVICE ROAD
N. 89° 54' 51" W. 2176.37'

PLOT PLAN

SCALE: 1"=80'-0"

KEY

- 2 ● BORINGS FOR CURRENT INVESTIGATION (A-9206)
- 4 ○ BORINGS FOR CONCURRENT INVESTIGATION (A-7202B)
- ⊖ BORINGS FOR PREVIOUS INVESTIGATION (A-66270-B, A-70259)
- BORING NUMBER

NOTES:

ALL ELEMENTS OF THE PLAN OUTSIDE LIMITS OF CONTRACT ENCLOSURE ARE EXISTING; NO CHANGE UNLESS NOTED OTHERWISE.

BRIDGE AREA TO BE BARRICADED SO AS TO ALLOW CONTINUOUS TRAFFIC FLOW DURING COURSE OF BRIDGE CONSTRUCTION. INTERRUPTION OF THE FUNCTIONAL OPERATION OF EXISTING FACILITIES CANNOT OCCUR.

FIRE FLOW REQUIREMENT FOR THIS BUILDING, AS DETERMINED BY THE LOS ANGELES COUNTY FIRE PREVENTION BUREAU IS 2500 GALLONS PER MINUTE.

CONSTRUCTION BENCHMARK TO BE SET BY COUNTY SURVEYOR AT THE DIRECTION OF THE CONTRACTOR.

LACo SB 1953 – D&M Project No. 22701-014-004
KING / DREW – Hawkins (AD-03)
Spec No – 2935B Psychiatric. **ARCHITECTURAL**

JENKINS - FLEMING, ARCHITECTS, INC.
9350 WILSHIRE BLVD., BEVERLY HILLS, CALIF. 90211
SUITE 428 · (213) 550-4151

MARTIN LUTHER KING JR. GENERAL HOSPITAL
PSYCHIATRIC AND CLINICAL SCIENCES BUILDING
CAPITAL PROJECT NO. 6008.01
SPECIFICATION NO. 2935B

NO.	DATE	REVISION	BY

PREPARED FOR
COUNTY OF LOS ANGELES
FACILITIES DEPARTMENT
STEPHEN J. KOONCE
DIRECTOR
1526 W. OLYMPIC BLVD.
LOS ANGELES, CALIFORNIA
(213) 974-1601

STATE FIRE MARSHALL
APPROVED
JUN 25 1977
STATE FIRE MARSHAL
SOUTHERN REGION

STATE DEPARTMENT OF HEALTH APPROVED
MAY 12 1977
STATE DEPT. OF HEALTH
Fault Consistr Sect.
Supervising Architect

O. A. C.
OFFICE OF THE STATE ARCHITECT
SOUTHERN SURVEY SECTION
APPROVED APR 25 1977
H0437
Per D. Hawkins

SCALE 1"=80'-0" | DRAWN CTL | CONSULTANT
JOB NO. 75-382 | CHECKED
DATE 7-30-76 | APPROVED
SHEET TITLE PLOT PLAN

SEP 13 1977

SHEET NO. A1-1 OF A-105 SHTS.

374

BORING 7

DATE DRILLED: February 4, 1972
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 88.3

ELEV (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE	
				ML	SANDY SILT - brown
		16.7	84		
85		11.9	100		
	5			SM	SILTY SAND - fine, grown
		10.5	101		
80		31.4	90	ML	CLAYEY SILT - dark brown
	10			ML	SANDY SILT - light brown
		22.0	106	SM	SILTY SAND - fine, light brown
75				ML	CLAYEY SILT - some alkali, greyish-brown
	15	28.9	93		
70				SM	SILTY SAND - fine, grey
	20	24.8	93		
				ML	SANDY SILT - brown
65				ML	CLAYEY SILT - some organic matter, dark grey
	25	26.0	101		
60				SC	CLAYEY SAND - fine, light grey
	30	17.5	71	CL	SANDY CLAY - some alkali, light grey
55				ML	SANDY SILT - light brown
	35	21.6	103		
50				SM	SILTY SAND - fine, light brown
	40	7.1	99		
45				ML	SANDY SILT - light brown
	45	22.6	98	SM	SILTY SAND - fine, light brown
				ML	SANDY SILT - greyish-brown
40	50	21.6	102		
35				ML	CLAYEY SILT -light grey
	55	26.7	95		
30				SM	SILTY SAND - fine, grey
	60	16.8	115		
25				SP	SAND - fine, brown
	65	23.3	107		
20	70				(BORING TERMINATED DUE TO HEAVY SLOUGHING)

NOTE: Water encountered at 54½'; water level at 55' at completion of drilling. Water level at 54' 10 minutes after completion of drilling. Heavy sloughing below 65'.

BORING 6

DATE DRILLED: February 7, 1972
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 89.5

ELEV (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE	
				ML	SANDY SILT - light brown
		14.6	74		
		8.9	87		
85	5			SM	SILTY SAND - fine, greyish-brown
		13.9	81		
		11.4	91	ML	SANDY SILT - brown
80	10				Clayey
		21.1	104		
75	15	21.7	104		
				ML	CLAYEY SILT - some alkali, dark grey
70	20	27.9	98	SM	SILTY SAND - fine, light greyish-brown
				CL	SILTY CLAY - dark grey
65	25	22.5	105		
				CL	SANDY CLAY - some alkali, light grey
60	30	18.9	111		
				ML	SANDY SILT - light brown
55	35	26.1	96	SM	SILTY SAND - fine, light brown
				SP	SAND - fine, light brown
50	40	7.8	97		
45	45	3.7	103		
				ML	CLAYEY SILT - grey
40	50	30.3	93	ML	SANDY SILT - grey
				CL	SILTY CLAY - light grey
35	55	26.8	98		
				ML	SANDY SILT - grey
30	60	17.3	117	SM	SILTY SAND - fine, grey
				SP	SAND - fine, light brown
25	65	19.7	109		
20	70	18.7	114		(BORING TERMINATED DUE TO HEAVY SLOUGHING)
15	75				

NOTE: Water encountered at 59'; water level at 59' at completion of drilling. Water level at 56' 15 minutes after completion of drilling. Heavy sloughing below 62'.

BORING 5

DATE DRILLED: February 4, 1972
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 89.5

ELEV (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE	
				ML	SANDY SILT - dark brown
		13.5	89		
		17.1	96		
85	5			SM	SILTY SAND - fine, light brown
		13.7	105		
		26.2	96	ML	CLAYEY SILT - greyish-brown
80	10			ML	SANDY SILT - light brown
		20.3	96		
75	15	33.5	87	ML	CLAYEY SILT - dark brown
				SM	SILTY SAND - fine, light brown
70	20	27.4	96		
				ML	CLAYEY SILT - dark brown
65	25	13.9	121		
				ML	SANDY SILT - light brown
60	30	16.8	113		
				SM	SILTY SAND - fine, light brown
55	35	13.8	101	SP	SAND - fine, light brown
50	40	5.8	96		
				SM	SILTY SAND - fine, light brown
45	45	5.9	98	SP	SAND - fine, light brown
				ML	SANDY SILT - grey
40	50	33.0	88	SM	SILTY SAND - fine, light brown
				ML	CLAYEY SILT - grey
35	55	32.9	91	ML	SANDY SILT - grey
				SP	SAND - fine, grey
30	60	18.4	109		Light brown
25	65	20.2	108		
20	70	24.4	103		
15	75	22.9	103		
10	80	29.9	96	ML	CLAYEY SILT - grey

NOTE: Water encountered at 58'; water level at 56½' at completion of drilling. Water level at 55½' 10 minutes after completion of drilling. Heavy sloughing from 58' to 78'.

BORING 4

DATE DRILLED: February 8, 1972
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 89.6

ELEV (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE	
				ML	SANDY SILT - brown
		14.2	80		
		9.2	95		
85	5	6.4	93	SM	SILTY SAND - fine, light brown
		19.7	95	ML	CLAYEY SILT - brown
80	10			ML	SANDY SILT - light brown
		12.2	102		
75	15	25.9	99	ML	CLAYEY SILT - some alkali, dark brown
				SM	SILTY SAND - fine, light brown
				SP	SAND - fine, light brown
70	20	7.5	86	ML	CLAYEY SILT - dark grey
65	25	22.8	103	CL	SILTY CLAY - greyish-brown
				ML	SANDY SILT - light brown
60	30	29.4	96	SM	SILTY SAND - fine, light brown
55	35	4.1	102	SP	SAND - fine, light brown
50	40	9.6	116		Layers of Silt
45	45	3.0	102		
40	50	30.2	92	ML	CLAYEY SILT - grey
35	55	28.3	96	ML	SANDY SILT - grey
30	60	36.5	86		Some alkali
25	65	19.6	110	SM	SILTY SAND - fine, grey
				ML	SANDY SILT - grey
20	70	29.5	96		
15	75	20.6	108		
10	80	20.6	109		Layer of Clay
5	85	25.8	100		
0	90	26.1	98	SM	SILTY SAND - fine, grey
-5	95	26.8	98		Layer of Clayey Silt
-10	100	20.9	109		
-15	105				

NOTE: Water encountered at 59½'; water level at 57½' at completion of drilling. Water level at 56' 15 minutes after completion of drilling. No caving.

BORING 3

DATE DRILLED: February 7, 1972
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 89.7

ELEV (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE	
				ML	SANDY SILT - dark brown
		19.6	81		
		10.1	108		
85	5	8.4	105	SM	SILTY SAND - fine, light brown
		20.9	105	ML	SANDY SILT - dark brown
80	10				
		30.6	89		
75	15				
		32.4	88	ML	CLAYEY SILT - some alkali, greyish-brown
70	20			ML	SANDY SILT - grey
		32.6	91	SM	SILTY SAND - fine, grey
65	25			CL	SILTY CLAY - dark brown
		21.8	107		
60	30			CL	SANDY CLAY - some alkali, light grey
					Few cemented lumps
		15.1	118	CL	SILTY CLAY - some alkali, brown
55	35			ML	SANDY SILT - light brown
		14.7	110	SP	SAND - fine, light brown
50	40				
		22.7	104		Layer of Sandy Silt
45	45				
		32.6	91	ML	CLAYEY SILT - grey
40	50				Layer of Silty Sand
		30.2	93		
35	55				
		26.6	99	ML	SANDY SILT - grey
30	60				

NOTE: Water encountered at 58'. No caving.

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

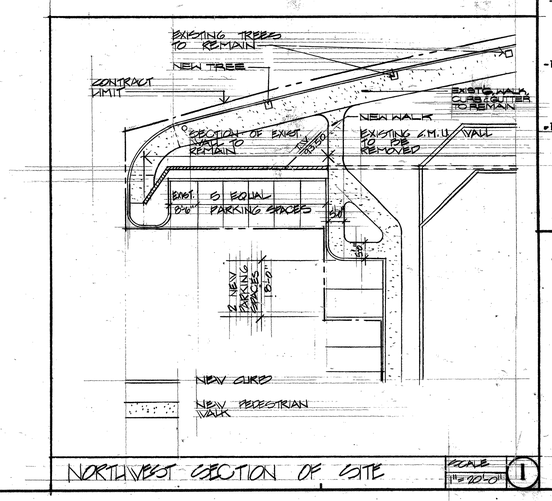
PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
NO. 200	NO. 40	NO. 10	NO. 4	¾ in.	3 in. (12 in.)		

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953. (Revised April, 1960)



NORTHWEST SECTION OF SITE — SCALE 1" = 20'-0" — 1

JENKINS - FLEMING, ARCHITECTS, INC.
8980 WILSHIRE BLVD., BEVERLY HILLS, CALIF. 90211
SUITE 488 · (213) 655-4781

MARTIN LUTHER KING JR. GENERAL HOSPITAL

PSYCHIATRIC AND CLINICAL SCIENCES BUILDING

CAPITAL PROJECT NO. 8008.01
SPECIFICATION NO. 2935 B

NO.	DATE	REVISION	BY
	4-12-76	REVISED WALKS & TREES	

PREPARED FOR
COUNTY OF LOS ANGELES FACILITIES DEPARTMENT
STEPHEN J. KOONCE
DIRECTOR OF FACILITIES
1955 WORKMAN MILL ROAD
LOS ANGELES, CALIFORNIA
(213) 974-1801

STATE DEPARTMENT OF HEALTH APPROVED MAY 13 1977 STATE DEPT. OF HEALTH FACILITIES CONSTRUCTION SECT. — Clarence M. Gonyon

STATE FIRE MARSHALL APPROVED JAN 25 1977 — STATE FIRE MARSHAL

O. Angeles County Dept. of Health Services APPROVED APR 25 '77 — For D. K. Maddock, M.D.

APPLICATION NO. 04,37

CONSULTANT

DRAWN CNL | CHECKED | APPROVED

SCALE NONE | JOB NO. 75-382 | DATE 7-30-76

SHEET TITLE: BORING LOGS

SHEET NO. A-2 OF A-10 SHTS.

BORINGS ON SEPARATE POST CLEARANCE 1983

MAR 13 1977

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APPENDIX C

BORING LOGS AND PERCOLATION TESTING

C.1 BORING LOGS

The borings were performed by GeoPentech over the course of three days. The explorations consisted of advancing 3 borings to depths of about 101.5 ft. The hollow stem auger drilling (GP-1 through GP-3) took place on April 19th through April 20th, 2023. The approximate locations of the borings are indicated on Figure 2 in the main report. The borings were drilled using an 8-inch diameter hollow stem auger bit, and drilling mud was poured in the auger when groundwater was encountered. The work was performed under the supervision of an engineer or a geologist who monitored the drilling operations and prepared a field record of soils observed and drilling conditions. The drilling was subcontracted to Gregg Drilling Inc. Gregg provided all equipment, crew, and supplies for their respective work.

During drilling, soil samples were obtained at approximate intervals ranging between 2.5 and 5-foot using a Standard Penetration Test (SPT) sampler, or a Modified California (MC) sampler SPT and MC samples were taken by driving a sampler approximately 18 inches into the soil at the bottom of the boring using a 140-pound hammer falling approximately 30 inches. The truck mounted CME 75 rig used by BC2 Environmental utilized an automatic-trip hammer. The SPT sampler cutting shoe and barrel have nominal inside diameters of 1.375 and 1.50 inches, respectively, and a nominal outside diameter of 2.00 inches. The barrel had no space for internal liners which were not used. The SPT samples were placed in plastic bags, labeled, and sealed. The MC sampler cutting shoe and barrel have nominal inside diameters of 2.38 and 2.50 inches, respectively, and a nominal outside diameter of 3 inches. Nominal 6-inch long, 2.4-inch diameter brass tubes or alternatively assemblies of 1-inch long, 2.4-inch diameter brass rings combined to fill the sampler were used to line the barrel. Plastic end caps were placed on the MC tubes to help preserve the moisture content of the samples. Bulk soil samples were also obtained at certain depths in selected boreholes. Upon completion of drilling, logging, and sampling, all borings were backfilled with neat cement slurry and patched at the surface with concrete.

After recovering the sample, the engineer or geologist noted the depth interval, recorded a description of the recovered material onto a field log and sealed and labeled the sample for transport to the laboratory. The soil descriptions noted on the field logs were visually classified in accordance with the Unified Soil Classification System and updated based on lab test results. The results of the borehole drilling and logging effort are provided on the borehole logs and on a key to the logs of boreholes attached at the end of this appendix. Laboratory testing run on collected samples are summarized on the logs and shown in Appendix E.

C.2 PERCOLATION TESTING

Percolation testing was initially performed in the upper 5 feet of GP-1 and GP-2. A percolation rate was not measured in either hole due to the surrounding clay layers stopping water from flowing. A second hole was drilled by GP-2 to 8.5 feet below ground surface to reach a sandy layer for percolation testing. A 2-inch diameter slotted (0.02-inch) Schedule 40 PVC pipe was installed, and a sand pack was poured through the auger to fill the annular space of the boring outside the pipe in order to surround the pipe up to the ground surface. Water was then gravity fed into the pipe and measured with a manual water level meter. After testing was complete, the pvc pipe and sand were removed and drilling resumed.

Before starting the percolation testing, each hole was presoaked with water to the ground surface to saturate the surrounding soil. After presoaking, each hole was tested for two cycles to ensure a steady percolation rate was measured. Manual water level readings were recorded every 30-60 seconds.

Recorded water level data was converted to the height of the water column in the boring and plotted versus time for each boring. The changes in water level with time were used to compute percolation rates for the measured test cycles. The calculated percolation rates for each borehole are reported on Figure F-1 and at the end of this section.

Note that because changes in water level reflect percolation both into the borehole walls and through the bottom of the borehole, the percolation rate for each test cycle was corrected for non-vertical flow and reported as an adjusted percolation value. The Reduction Factor (Rf) applied to adjust the raw field measurements is based on guidance from LACDPW (2014) and resulted in adjustments ranging from about a factor of 14 to 17. The formula used is given below:

$$R_f = 1 + \frac{(2WL_0 - \Delta H)}{DIAM}$$

Where:

WL_0 = Initial water level (depth) in inches.

ΔH = Water level drop in inches.

DIAM = Diameter of the borehole in inches.

Based on the recorded data, the adjusted percolation rates mainly ranged from 15 in/hr to 17 in/hr except for the tests done in clay.

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGENDS
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO. 4. SIEVE	CLEAN GRAVELS <5% FINES [1]	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES [1]	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% [2]	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES [2]	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT <50	INORGANIC	PI >7 AND PLOTS ABOVE "A" LINE	CL	LEAN CLAY
			PI <4 OR PLOTS BELOW "A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried) <0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT >50	INORGANIC	PI PLOTS ABOVE "A" LINE	CH	FAT CLAY
			PI PLOTS BELOW "A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried) <0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

1. Gravels with 5 to 12% fines require dual symbols: GW-GM (well-graded gravel with silt), GW-GC (well-graded gravel with clay), GP-GM (poorly graded gravel with silt), GP-GC (poorly graded gravel with clay)
2. Sands with 5 to 12% fines require dual symbols: SW-SM (well-graded sand with silt), SW-SC (well-graded sand with clay), SP-SM (poorly graded sand with silt), SP-SC (poorly graded sand with clay)

PENETRATION RESISTANCE

SAND & GRAVEL			SILT & CLAY			
RELATIVE DENSITY	SPT BLOWS/FOOT [1]	MOD CAL BLOWS/FOOT [2]	CONSISTENCY	SPT BLOWS/FOOT [1]	MOD CAL BLOWS/FOOT [2]	UNC. COMP. STRENGTH (KSF)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 1	0 - 2	0 – ½
LOOSE	5 - 10	7 - 16	SOFT	2 - 4	3 - 6	½ - 1
MEDIUM DENSE	11 - 30	17 - 46	MEDIUM STIFF	5 - 8	7 - 13	1 - 2
DENSE	31 - 50	47 - 76	STIFF	9 - 15	14 - 23	2 - 4
VERY DENSE	>50	>76	VERY STIFF	16 - 30	24 - 46	4 - 8
			HARD	>30	>46	>8

1. Number of blows of a 140 lbs hammer falling 30 inches to drive a nominal 2-inch O.D. (1.375-inch I.D.) split-barrel sampler the last 12 inches of an 18-inch drive (ASTM-1586 Standard Penetration Tests)
2. Number of blows of a 140 lbs hammer falling 30 inches to drive a nominal 3-inch O.D. (2.4-inch I.D.) split-barrel sampler the last 12 inches of an 18-inch drive (ASTM-3550 Standard Penetration Tests) Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils (a multiplier of 0.65 was used to adjust densities for the larger surface area of the modified California sampler).

SOIL PLASTICITY DESRIPTIONS

Plasticity Index Range	Plasticity Adjective	Adjective for Soil Type, Texture, and Plasticity Chart Location		
		ML & MH (SILT)	CL & CH (CLAY)	OL & OH (ORGANIC SILT or CLAY)
0	nonplastic	-	-	ORGANIC SILT
1 – 10	low plasticity	-	Silty	ORGANIC SILT
>10-20	medium plasticity	clayey	Silty to no adj.	ORGANIC clayey SILT
>20 – 40	high plasticity	clayey	-	ORGANIC silty CLAY
>40	very plastic	clayey	-	ORGANIC CLAY

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Key to Log of Boring

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows per 6"	SAMPLES: Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to mean sea level (MSL).

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Sampling Resistance:** Number of blows required to advance driven sampler 6 inches, or distance noted, using the drive weight listed in hammer data. Hydraulic down-pressure may be recorded for pushed samplers.

6 **Sample Recovery:** Amount of sample recovered from sampling interval; given as inches of sample recovered or ratio of sample length to drive length (expressed as a percentage, %)

7 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

8 **Material Description:** Description of material encountered; may include density/consistency (from field assessments), moisture, color (Munsell code), and grain size.

9 **Water Content:** Water content of sample, as percentage of dry weight of soil, measured in lab according to ASTM D2216.

10 **Dry Unit Weight:** The weight of soil solids per cubic foot of total volume of soil mass, measured according to ASTM D2937.

11 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Other lab tests are indicated using abbreviations explained below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

- Clayey SAND (SC)
- Lean CLAY (CL)
- Silty SAND (SM)
- SILT (ML)
- SAND with Silt (SW-SM)
- SAND (SP)
- Clayey SILT (ML-CL)
- Sandy CLAY (CL)
- SAND with Silt (SP-SM)
- SILT with Clay (ML)

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bulk Sample
- Grab Sample
- Modified California Sampler
- SPT Sampler

OTHER LABORATORY TEST ABBREVIATIONS

G/S/F=	Gravel/Sand/Fines Percentage
DS	Direct Shear Test Performed
LL/PI	Liquid Limit/Plasticity Index Values
FC	Fines Content <#200 (%)
EI	Expansion Index
CONSOL	Consolidation Testing
CORR	Corrosion Testing

OTHER GRAPHIC SYMBOLS

____ Contact between strata

_ _ _ Inferred contact between strata or gradational change

Change within material properties within a stratum

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times. Datum used is WGS84.

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-1

Sheet 1 of 4

Date(s) Drilled	**4/19/2023**	Logged By	**W. Erickson**	Checked By	**S. Duke**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**101.5 feet**
Drill Rig Type	**CME-75**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**95 ft NAVD 88**
Groundwater Level(s)	**Encountered at 58.3'**	Sampling Method	**Bulk, Grab, SPT, Mod Cal**	Hammer Data	**Automatic hammer 140 lbs/30" drop**
Borehole Location	**33.924040° N, -118.243334° E**	Borehole Completion	**Backfilled with cement-grout via tremie**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per 6"	SAMPLES Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
95	0		Bulk-1				**[1' Asphalt]**			Bulk-1: FC=36.2% CORR, EI
							[Artificial Fill (Af)] **Clayey SAND (SC)**, moist, medium grained, slightly plastic			
			1a							
			1b				**CLAY (CL)**, brown, med plastic, little sand in lower .5'			
90	5		2	4 8 9			**[Alluvium (Qa)]** **Sandy CLAY (CL)**,stiff, light brown, medium plastic, fine sand, no HCl reaction, bricks and roots	25	100.1	LL=36 PI=17
			3	2 2 4			**Sandy SILT (ML)**, loose, dark brown, nonplastic, medium grained, no HCl reaction			FC=51.7%
85	10		4	3 8 7			becomes stiff, brown, nonplastic, fine grainde, no HCl reaction	22	102.5	LL=NP PI=NP DS
80	15		5	3 6 7			**Sandy CLAY (CL)**, stiff, moist, light brown, slightly plastic, fine grained, no HCl reaction			FC=75.5%
75	20		6	2 5 5			becomes medium stiff, highly plastic, some oxide staining	25	100.0	LL=38 PI=19 CONSOL
70	25		7	3 7 8			becomes stiff, medium plastic, black bottom 3"			
65	30									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-1

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
65	30		8	4 6 6			becomes medium stiff, gray, little SAND, sand in auger shoe	26	98.6	LL=36 PI=21
60	35		9	12 18 19			**Silty SAND (SM)**, dense, dry, light brown, medium grained, no HCl reaction			G=0% S=87% F=13%
55	40		10	16 25 38			**SAND (SP)**, dense, dry, slightly orange, medium grained, no HCl reaction	4	108.5	
50	45		11	11 26 32			becomes very dense, coarser grain			
45	50		12	11 14 23			**SILT (ML)**, stiff, moist, gray, highly plastic	27	96.9	LL=34 PI=6 DS
40	55		13	4 4 8			**CLAY (CL)**, stiff, moist, gray, highly plastic			LL=42 PI=23 Water at 58.3'
35	60		14	8 18 35			**Sandy SILT (ML)**, hard, dark gray, medium plastic, medium grained	15	122.1	LL=NP PI=NP CONSOL
30	65									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-1

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
30	65		15	18 20 27			**Silty SAND (SM)**, dense, wet, dark gray, fine grain, no HCl reaction			G=1% S=80% F=19%
25	70		16	6 11 35			becomes medium dense	21	108.0	DS
20	75		17	3 19 21			becomes dense			
15	80		18	8 17 21			**CLAY (CL)**, very stiff, slightly plastic, some fine SAND	30	97.0	FC=92.5% LL=41 PI=20 CONSOL
10	85		19	5 10 18			**SILT (ML)**, very stiff, olive brown, slightly plastic, some fine SAND			
5	90		20	6 22 30			becomes more stratified	29	96.8	
0	95		21	12 18 33			becomes hard, dry, medium plastic, no HCl reaction			LL=37 PI=11
-5	100									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

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Log of Boring GP-1

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per 6"	SAMPLES Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
-5	100		22	8 23 41			**Silty SAND (SM)**, dense, wet, fine grained, non plastic, no HCl reaction	26	99.5	
							Borehole terminated @ 101.5'			
-10	105									
-15	110									
-20	115									
-25	120									
-30	125									
-35	130									
-40	135									

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-2

Sheet 1 of 4

Date(s) Drilled	**4/20/2023**	Logged By	**W. Erickson**	Checked By	**S. Duke**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**101.5 feet**
Drill Rig Type	**CME-75**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**92 ft NAVD 88**
Groundwater Level(s)	**Encountered at 56'**	Sampling Method	**Bulk, Grab, SPT, Mod Cal**	Hammer Data	**Automatic hammer 140 lbs/30" drop**
Borehole Location	**33.924399° N, -118.241556° E**	Borehole Completion	**Backfilled with cement-grout via tremie**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
92	0		Bulk-1				**[4" Asphalt]**			Bulk-1: FC=48.4% CORR, EI
							[Artificial Fill (Af)] **Clayey SAND (SC)**, moist, dark brown, medium plastic, fine SAND, slight HCl reaction, some gravel			
90			1a							
			1b				**Sandy SILT (ML)**, moist, slightly plastic			
	5		2	3 4 6			**[Alluvium (Qa)]** **Sandy SILT (ML)**, medium stiff, dry, nonplastic, fine grained, strong HCl reaction	18	102.8	LL=NP PI=NP
85			3	3 3 2			**Silty SAND (SM)**, loose, dry, fine grained, no HCl reaction			G=0% S=63% F=37%
	10		4	3 5 7			**SILT (ML)**, medium stiff, moist, slightly plastic, little fine grained SAND	26	95.9	
80										
	15		5	2 3 4			**Silty SAND (SM)**, loose, moist, light brown, fine grained, nonplastic, no HCl reaction			FC=40.7%
75										
	20		6	2 5 9			**CLAY (CL)**, stiff, moist, olive brown, highly plastic, no HCl reaction	28	95.4	LL=45 PI=22 DS, CONSOL
70										
	25		7	3 3 3			becomes medium stiff, dark brown, little SAND			
65										
	30									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

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Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
	30		8	4 9 15			**CLAY (CL)**, stiff, moist, black, highly plastic, highly reactive white spots, (roots?)	20	109.4	
60										
	35		9	4 8 8			becomes very stiff, dry, light brown, slightly plastic, no HCl reaction, fine SAND			FC=87.2%
55										
	40		10	6 10 25			becomes medium plastic	20	103.2	DS
50										
	45		11	3 9 14			**Silty SAND (SM)**, medium dense, moist, fine to medium grained, nonplastic, no HCl reaction			FC=38.9%
45										
	50		12	4 5 11			**CLAY (CL)**, stiff, moist, medium plastic, strong HCl reaction	36	86.7	LL=45 PI=17
40										
	55		13	3 4 6			becomes highly plastic ▽			LL=46 PI=26 water at 56'
35										
	60		14	5 10 13			becomes medium dense, gray, fine gained, medium plastic	20	110.9	FC=57.1% LL=28 PI=9 DS CONSOL
30										
	65									

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
65										
			15	6 11 22			**Clayey SAND (SC)**, dense, light brown, fine gained, slightly plastic			
25										
70			16	3 17 19	NR					
20										
75			17	2 15 19			**SAND with Silt (SP-SM)**, dense, wet, medium grained, no HCl reaction			G=0% S=93% F=7%
15										
80			18	10 35 47			**CLAY with silt (CL)**, hard, olive brown, medium plastic	20	108.4	FC=86.3% LL=32 PI=9 DS
10										
85			19	6 12 18			**Clayey SAND (SC)**, medium dense, wet, fine-medium grained, slightly plastic, highly reactive white spots, some staining			
5										
90			20	10 24 39			**Silty SAND (SM)**, dense, wet, fine grained, nonplastic, no HCl reaction	24	101.6	
0										
95			21	10 20 19			same as above with clay lense at 96'			
-5										
100										

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

Project: MLK Medical Campus - Hawkins Building Demolition
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Log of Boring GP-2

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per 6"	SAMPLES Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
100			22	6 15 25			**SAND (SP)**, dense, wet, medium grained, no HCl reaction	19	106.9	
							Borehole terminated @ 101.5'			
-10	105									
-15	110									
-20	115									
-25	120									
-30	125									
-35	130									
-40	135									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

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Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-3

Sheet 1 of 4

Date(s) Drilled	**4/21/2023**	Logged By	**W. Erickson**	Checked By	**S. Duke**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**101.5 feet**
Drill Rig Type	**CME-75**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**101 ft NAVD 88**
Groundwater Level(s)	**Encountered at 65.3'**	Sampling Method	**Bulk, Grab, SPT, Mod Cal**	Hammer Data	**Automatic hammer 140 lbs/30" drop**
Borehole Location	**33.923675° N, -118.241603° E**	Borehole Completion	**Backfilled with cement-grout via tremie**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
100	0		Bulk-1				**[Artificial Fill (Af)]** **Clayey SAND (SC)**, moist, dark brown, medium plastic, fine-coarse grained, some gravels to 1.5", strong HCl reaction			FC=41.1% some gravel
			1							
95	5		2	3 4 5			**Sandy SILT (ML)**, medium stiff, moist, brown, slightly plastic, fine to medium grained	24	96.6	FC=69.9% LL=NP PI=NP
90	10		3a 3b	4 5 5			**Sandy CLAY (CL)**, stiff, moist, slightly plastic, fine grained **[Alluvium (Qa)]** **Clayey SAND (SC)**, becomes more SAND, coarser			G=26% S=54% F=20% LL23 PI=8
85	15		4	4 7 8			becomes stiff, moist, slightly plastic, fine SAND, trace gravel to 1"	17	111.7	@14' concrete, move hole LL=27 PI=9 DS, CONSOL
80	20		5	3 6 4			**Sandy SILT (ML)**, stiff, dry, dark brown, nonplastic, fine SAND, 3" clay lense at bottom of sample			LL=NP PI=NP
75	25		6	7 15 13			**Sandy CLAY (CL)**, very stiff, dry, light brown, slightly plastic, fine SAND, strong HCl reaction, roots	8	102.8	roots
	30									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-3

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
70	30		7	2 3 4			becomes medium stiff, moist, highly plastic, strong HCl reaction on lighter spots			FC=83.4% LL=44 PI=26
65	35		8	3 5 8			**SILT (ML)**, medium stiff, moist, dark gray, slightly plastic, fine grained, no HCl reaction	31	92.8	
60	40		9	3 4 7			**CLAY (CL)**, stiff, moist, highly plastic, strong HCl reaction, little sandier last 3"			
55	45		10	8 19 29			**Sandy SILT (ML)**, hard, dry, light brown, fine grained, medium plastic, strong HCl reaction	17	107.7	G=17% S=16% F=67% LL=34 PI=9 DS
50	50		11	6 9 11			same as above, lense with more clay			
45	55		12	4 10 19			**Silty SAND (SM)**, medium dense, dry, fine grained, nonplastic, no HCl reaction	18	110.3	
40	60		13	2 4 8			**CLAY (CL)**, stiff, moist, brown/dark brown/orange, medium plastic, mottled color			
	65									

Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-3

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
35	65		14	8 16 21			becomes very stiff, wet, olive brown, highly plastic ▽	25	101.3	FC=90.2% LL=48 PI=28 CONSOL Water at 65.3
30	70		15a 15b	4 5 8			becomes medium plastic **Silty CLAY (CL-ML)**, medium dense, gray, fine grained, slightly plastic, no HCl reaction			FC=59.2% LL=25 PI=7
25	75		16	15 23 47			**SIlty SAND (SM)**, dense, wet, light brown, fine-medium grained, nonplastic, no reaction	18	112.8	
20	80		17	6 28 50			**Poorly graded SAND with Silt (SP-SM)**, very dense, wet, medium grained			G=0% S=88% F=12%
15	85		18	6 19 25			becomes medium dense	21	107.7	DS
10	90		19	8 14 15			**Silty SAND (SM)**, medium dense, wet, medium grained, nonplastic			
5	95		20	6 16 19			same as above	19	109.0	G=0% S=81% F=19%
	100									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

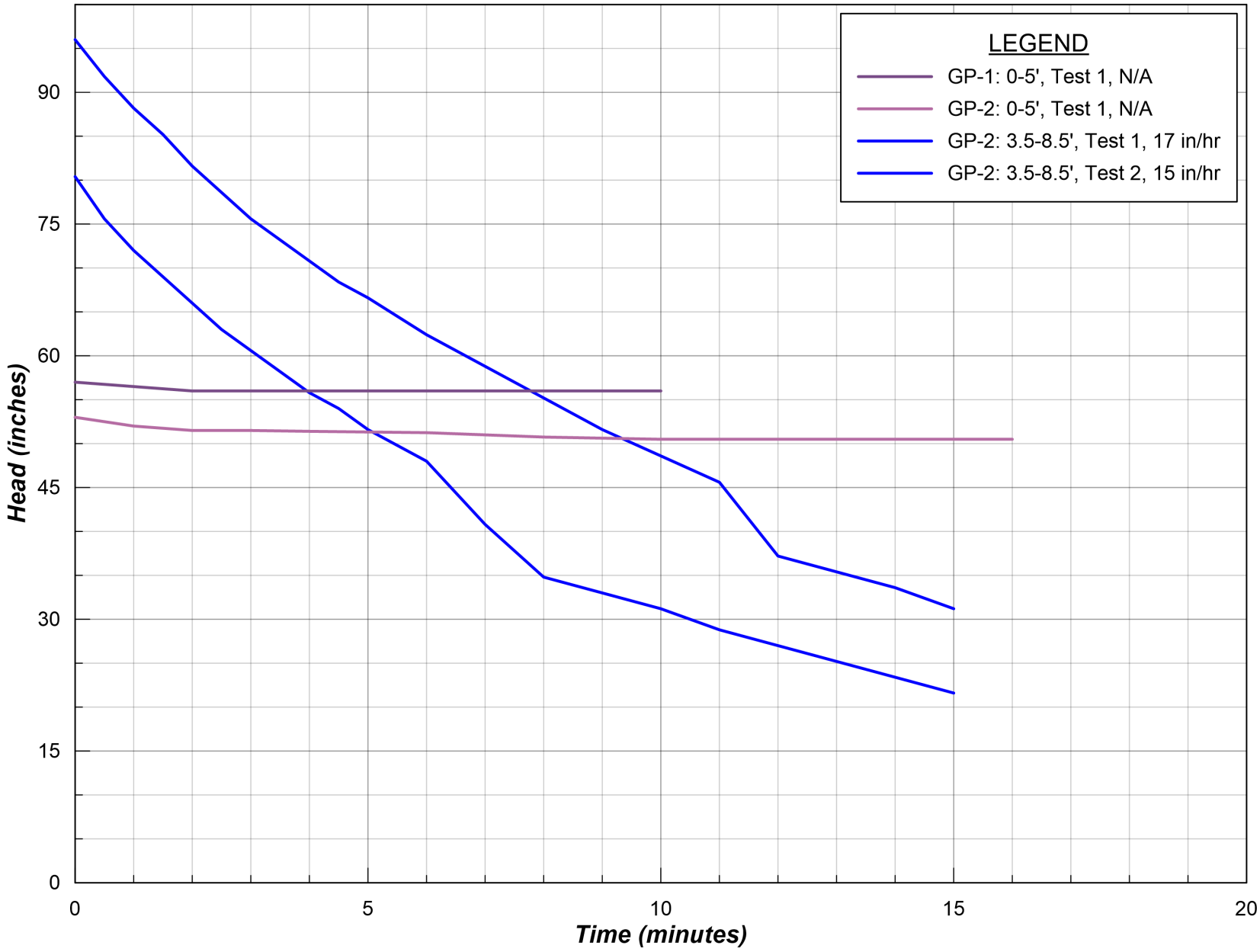
Project: MLK Medical Campus - Hawkins Building Demolition
Project Location: Los Angeles, CA
Project Number: 20044A-TO1

Log of Boring GP-3

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per 6"	SAMPLES Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
0	100		21	10 17 31			**CLAY (CL)**, hard, medium plastic clay, nonplastic silt, little SAND			
							Borehole terminated @ 101.5			
-5	105									
-10	110									
-15	115									
-20	120									
-25	125									
-30	130									
	135									

Report: GP SOIL BA LOG; File: 20044A - MLK HOSPITAL.GPJ; 6/19/2023

Percolation Data



PERCOLATION TEST DATA FOR LOCATIONS GP-1 AND GP-2

Date: MAY 2023	Project No.: 20044A-TO1	Project: MLK MEDICAL CAMPUS - HAWKINS BUILDING DEMOLITION	Figure C-1

GeoPentech

APPENDIX D

LABORATORY TESTING

D.1 LABORATORY TESTING

The laboratory testing program performed by GeoPentech for the proposed project site included the following tests: moisture content, dry density, sieve analysis, wash analysis, direct shear, corrosion, consolidation, and expansion index. The geotechnical testing was conducted at the laboratory facilities of AP Engineers in Pomona, California. The tests were performed in general accordance with applicable procedures of ASTM and the State of California Department of Transportation, Standard Test Methods (DOT CA). The results of the laboratory testing are included in this Appendix and are summarized in Table D-1 and on the boring logs in Appendix C. GeoPentech has reviewed the results of the laboratory testing and finds them acceptable. Brief descriptions of each test are presented in the following sections.

D.1.1 Moisture Content and Dry Density

For selected Modified California samples, the dry unit weight (in units of pounds-per-cubic-foot) and field moisture content (%) were measured in general accordance with ASTM D2937 and ASTM D2216, respectively, or with ASTM D7263.

D.1.2 Sieve Analysis, Wash Analysis, and Hydrometer

For sieve analysis, the particle-size distribution was determined in general accordance with ASTM D6913. Sieve sizes ranged from ¾ in to 75 µm (No. 200).

For wash analysis, the percentage of fines (material passing the No. 200 sieve) was measured in accordance with ASTM D1140.

For hydrometer, the percentage of silts and clays was measured in accordance with ASTM D7928.

D.1.3 Atterberg Limits

The Atterberg limits test is a classification test that is performed on cohesive soils (i.e., silty and clayey soils) to measure the soil plastic limit (PL) and liquid limit (LL), from which the plasticity index (PI) is calculated. The measured values can be plotted on a plasticity chart, which is used as an aid in classifying the soil material and behavior. These tests were performed in accordance with ASTM D4318.

D.1.4 Direct Shear

Direct shear tests were performed on selected Modified California samples in accordance with ASTM D3080 to measure peak and ultimate strength parameters. Shear stress and sample deformation were monitored throughout the tests.

D.1.5 Corrosion

Soil samples were tested for electrical resistivity, pH, sulfate content, and chloride content. These tests were performed in general accordance with DOT CA test methods 643 (electrical resistivity and pH), 417 (sulfate content), and 422 (chloride content). The test results were used to evaluate the corrosivity potential of the soil on underground improvements associated with the proposed structure.

D.1.6 Consolidation

Tests for one-dimensional consolidation properties of soils using incremental loading were performed on relatively undisturbed soil samples according to ASTM D2435. The test determines the magnitude and rate of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. The test results provide clayey soil settlement parameters under different loading conditions.

D.1.7 Expansion Index

Expansion Index testing was performed on selected samples in accordance with ASTM D4829 to measure the swelling potential of the soil. A specimen is compacted into a mold and a confining load is applied. The sample is then immersed in water and volumetric swell is recorded.

Table D-1
Summary of Laboratory Testing

	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
Boring Number	Sample No.	Depth (ft)	USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	TEST TYPE
GP-1	Bulk-1	0-5	SC							36.2	1,925	217	32	9			EI=21
GP-1	1a	2.5	SC														
GP-1	1b	4.5	CL														
GP-1	2	5	CL	25.2	100.1	36	17										
GP-1	3	7.5	ML							51.7							
GP-1	4	10	ML	21.8	102.5	NP	NP								34	200	
GP-1	5	15	CL							75.5							
GP-1	6	20	CL	24.6	100.0	38	19										CONSOL
GP-1	7	25	CL														
GP-1	8	30	CL	26.3	98.6	36	21										
GP-1	9	35	SM					0	87	13							
GP-1	10	40	SP	3.5	108.5												
GP-1	11	45	SP														
GP-1	12	50	ML	27.2	96.9	34	6								30	300	
GP-1	13	55	CL			42	23										
GP-1	14	60	ML	15.0	122.1	NP	NP										CONSOL
GP-1	15	65	SM					1	80	19							
GP-1	16	70	SM	20.8	108.0										36	150	
GP-1	17	75	SM														
GP-1	18	80	CL	29.7	97.0	41	20			92.5							CONSOL
GP-1	19	85	ML														
GP-1	20	90	ML	29.1	96.8												
GP-1	21	95	ML			37	11										
GP-1	22	100	SM	25.7	99.5												
GP-2	Bulk-1	0-5	SC							48.4	1825	240	30	8.6			EI=31
GP-2	1a	2.5	SC														
GP-2	1b	4.5	ML														
GP-2	2	5	ML	18.1	102.8	NP	NP										
GP-2	3	7.5	SM					0	63	37							
GP-2	4	10	ML	25.8	95.9												
GP-2	5	15	SM							40.7							
GP-2	6	20	CL	28.2	95.4	45.0	22.0								29	300	CONSOL

Table D-1
Summary of Laboratory Testing

	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
Boring Number	Sample No.	Depth (ft)	USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	TEST TYPE
GP-2	7	25	CL														
GP-2	8	30	CL	19.8	109.4												
GP-2	9	35	CL							87.2							
GP-2	10	40	CL	20.2	103.2										30	600	
GP-2	11	45	SM							38.9							
GP-2	12	50	CL	35.8	86.7	45.0	17.0										
GP-2	13	55	CL			46.0	26.0										
GP-2	14	60	CL	19.5	110.9	28.0	9.0			57.1					28	600	CONSOL
GP-2	15	65	SC														
GP-2	16	70	NR														
GP-2	17	75	SP-SM					0	93	7							
GP-2	18	80	CL	20.4	108.4	32.0	9.0			86.3					28	750	
GP-2	19	85	SC														
GP-2	20	90	SM	24.0	101.6												
GP-2	21	95	SM														
GP-2	22	100	SP	19.0	106.9												
GP-3	Bulk-1	0-5	SC							41.1							
GP-3	1a	2.5	SC														
GP-3	2	5	ML	23.7	96.6	NP	NP			69.9							
GP-3	3a	10	CL														
GP-3	3b	10	SC			23	8	26	54	20							
GP-3	4	15	SC	16.6	111.7	27	9								31	500	CONSOL
GP-3	5	20	ML			NP	NP										
GP-3	6	25	CL	7.7	102.8												
GP-3	7	30	CL			44	26			83.4							
GP-3	8	35	ML	30.8	92.8												
GP-3	9	40	CL														
GP-3	10	45	ML	17.2	107.7	34	9	17	16	67					31	400	
GP-3	11	50	ML														
GP-3	12	55	SM	18.3	110.3												
GP-3	13	60	CL														
GP-3	14	65	CL	24.7	101.3	48	28			90.2							CONSOL

Table D-1
Summary of Laboratory Testing

	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
Boring Number	Sample No.	Depth (ft)	USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	TEST TYPE
GP-3	15a	70	CL														
GP-3	15b	70	CL-ML			25	7			59.2							
GP-3	16	75	SM	18.3	112.8												
GP-3	17	80	SP-SM					0	88	12							
GP-3	18	85	SP-SM	20.9	107.7										34	350	
GP-3	19	90	SM														
GP-3	20	95	SM	18.5	109.0			0	81	19							
GP-3	21	100	CL														

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Lab No.: 23-0456
Test Date: 05/01/23

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-1	2	5	25.2	100.1
GP-1	4	10	21.8	102.5
GP-1	6	20	24.6	100.0
GP-1	8	30	26.3	98.6
GP-1	10	40	3.5	108.5
GP-1	12	50	27.2	96.9
GP-1	14	60	15.0	122.1
GP-1	16	70	20.8	108.0
GP-1	18	80	29.7	97.0
GP-1	20	90	29.1	96.8
GP-1	22	100	25.7	99.5

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Lab No.: 23-0456
Test Date: 05/01/23

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-2	2	5	18.1	102.8
GP-2	4	10	25.8	95.9
GP-2	6	20	28.2	95.4
GP-2	8	30	19.8	109.4
GP-2	10	40	20.2	103.2
GP-2	12	50	35.8	86.7
GP-2	14	60	19.5	110.9
GP-2	16	70	NO RECOVERY	
GP-2	18	80	20.4	108.4
GP-2	20	90	24.0	101.6
GP-2	22	100	19.0	106.9

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Lab No.: 23-0456
Test Date: 05/01/23

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-3	2	5	23.7	96.6
GP-3	4	15	16.6	111.7
GP-3	6	25	7.7	102.8
GP-3	8	35	30.8	92.8
GP-3	10	45	17.2	107.7
GP-3	12	55	18.3	110.3
GP-3	14	65	24.7	101.3
GP-3	16	75	18.3	112.8
GP-3	18	85	20.9	107.7
GP-3	20	95	18.5	109.0

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: GeoPentech
Project Name: MLK Hospital
Project Number: 20044A
AP Lab No.: 23-0456
Test Date: 05/10/23

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-1	Bulk-1	0-5	36.2
GP-1	3	7.5	51.7
GP-1	5	15	75.5
GP-1	18	80	92.5

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client:	GeoPentech	AP Lab No.:	23-0456
Project Name:	MLK Hospital	Test Date:	05/10/23
Project Number:	20044A		

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-2	Bulk-1	0-5	48.4
GP-2	5	15	40.7
GP-2	9	35	87.2
GP-2	11	45	38.9
GP-2	14	60	57.1
GP-2	18	80	86.3

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PERCENT PASSING NO. 200 SIEVE
ASTM D1140

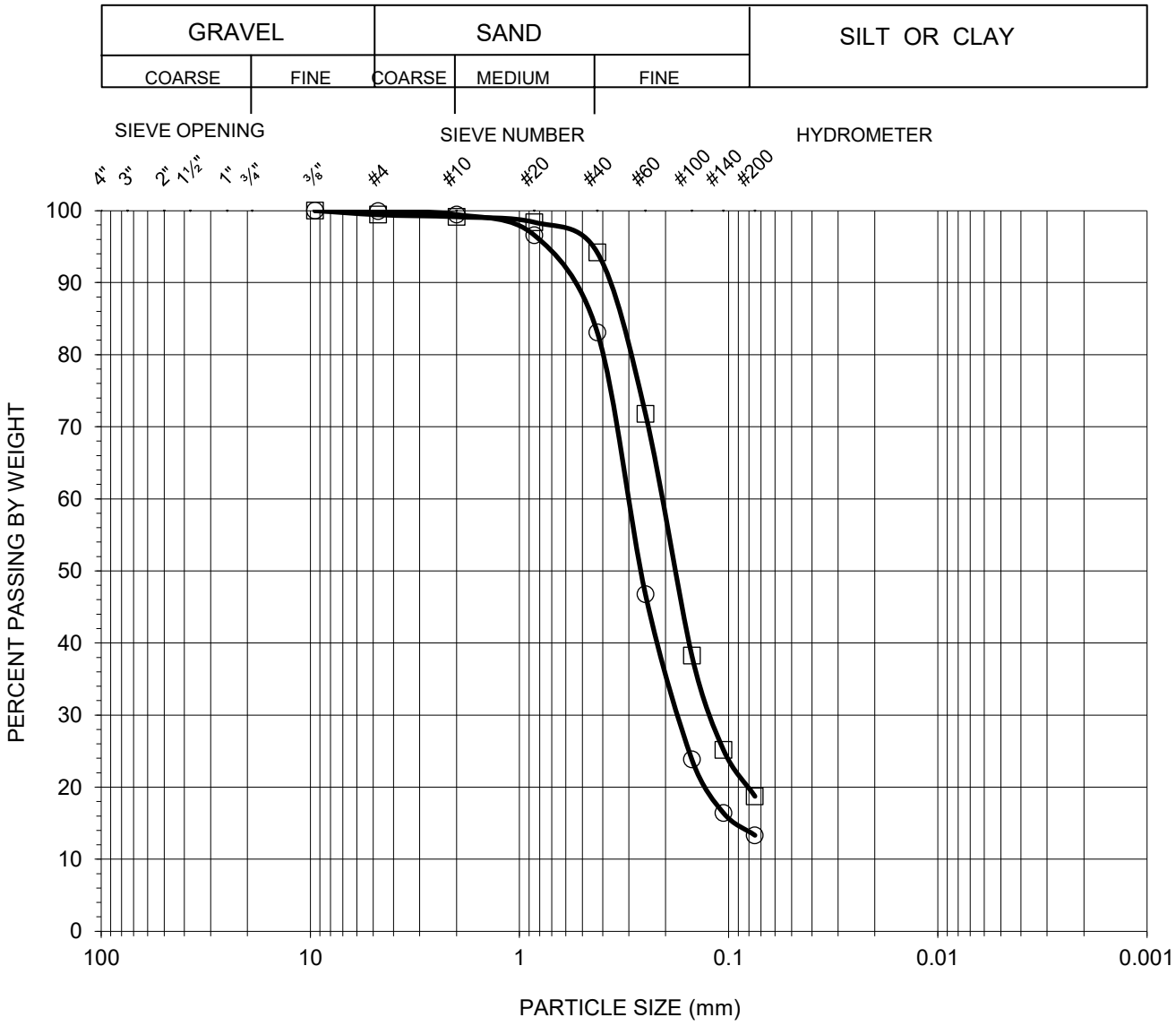
Client: GeoPentech
Project Name: MLK Hospital
Project Number: 20044A
AP Lab No.: 23-0456
Test Date: 05/10/23

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-3	Bulk-1	0-5	41.1
GP-3	2	5	69.9
GP-3	7	30	83.4
GP-3	14	65	90.2
GP-3	15b	70	59.2

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	GeoPentech	Tested by:	TV	Date:	05/11/23
Project Name:	MLK Hospital	Computed by:	NR	Date:	05/11/23
Project No.:	20044A	Checked by:	AP	Date:	05/15/23

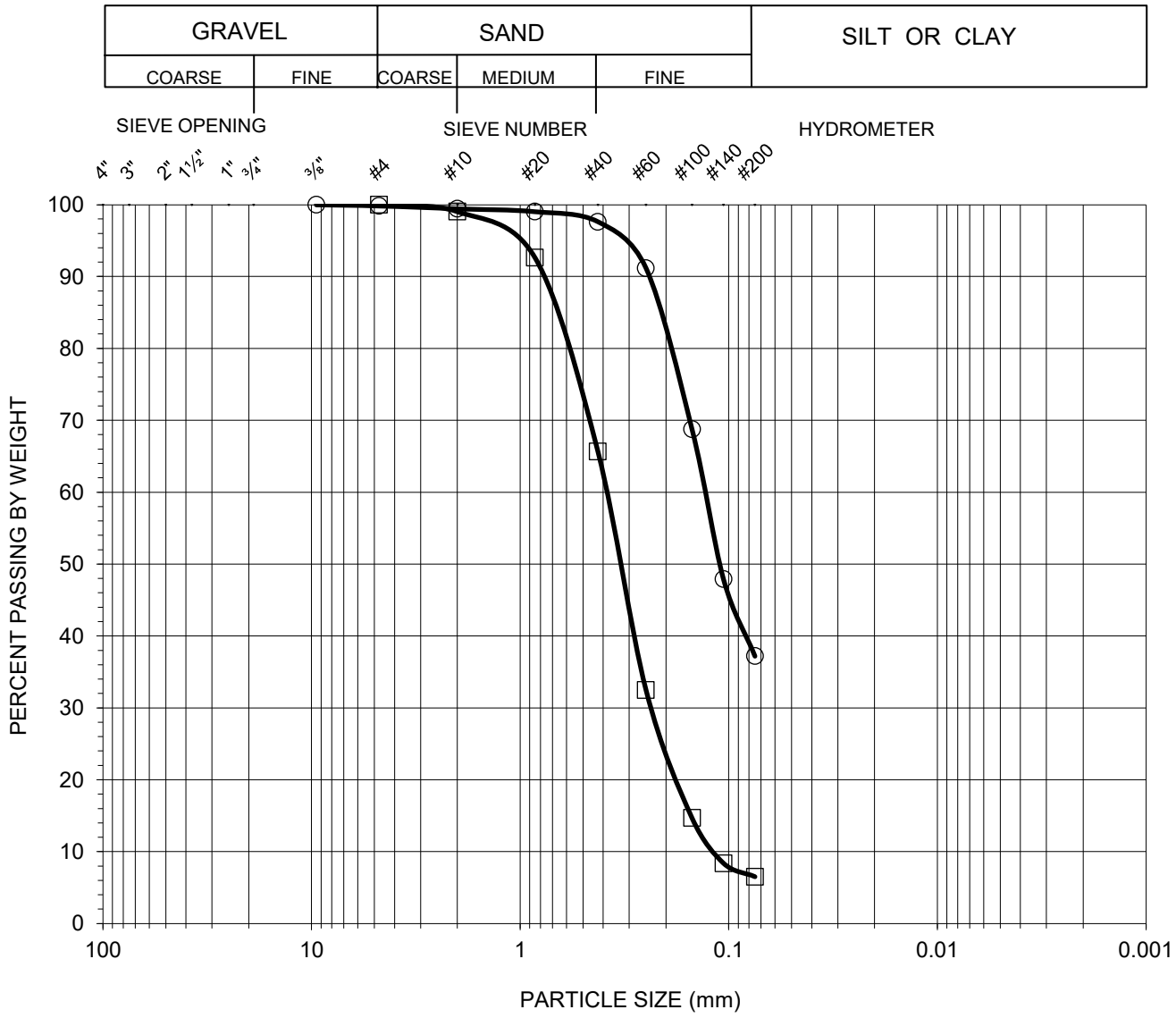


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-1	9	35	0	87	13	N/A	SM
□	GP-1	15	65	1	80	19	N/A	SM

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: GeoPentech — Tested by: TV — Date: 05/11/23
Project Name: MLK Hospital — Computed by: NR — Date: 05/11/23
Project No.: 20044A — Checked by: AP — Date: 05/15/23

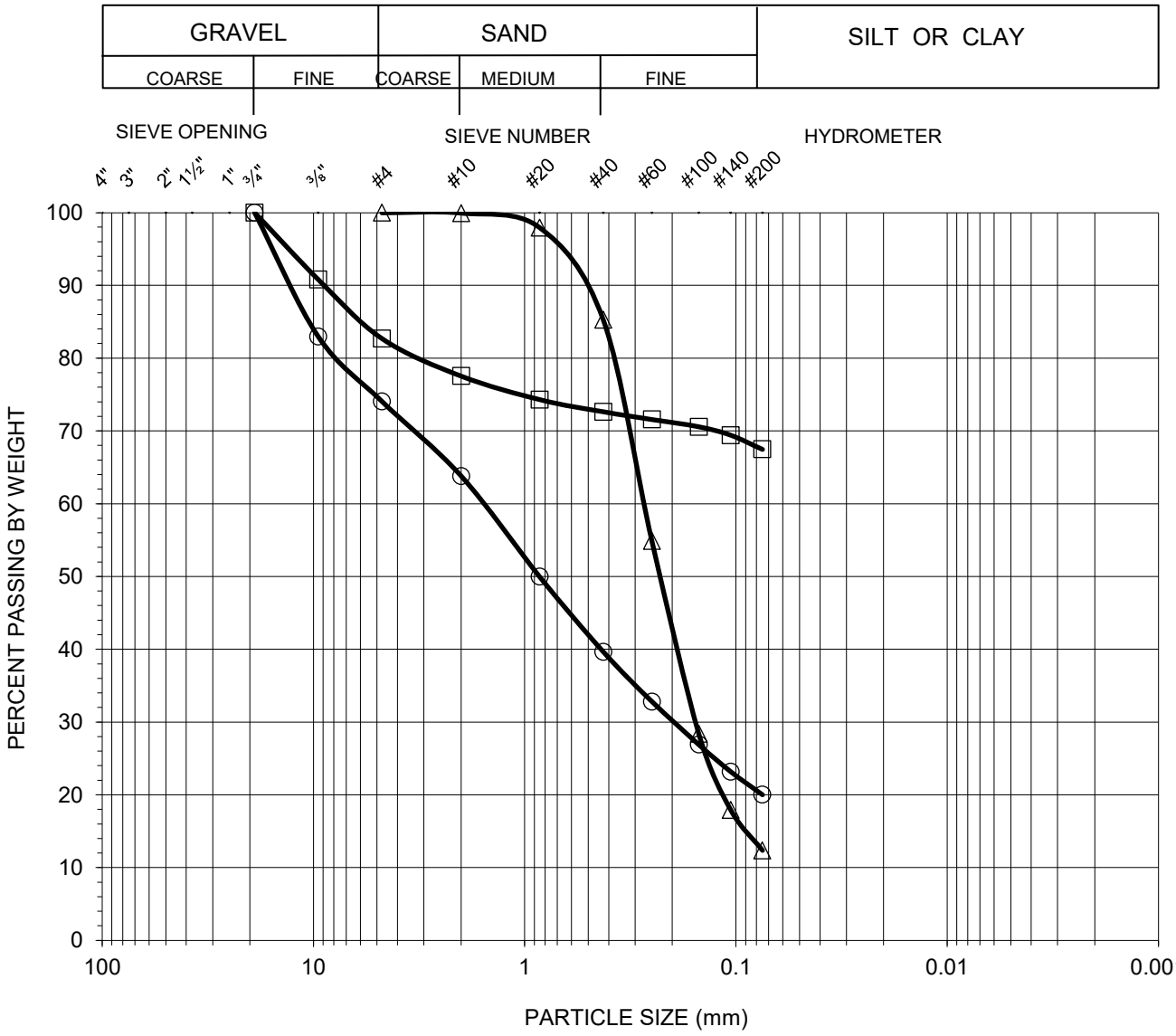


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-2	3	7.5	0	63	37	N/A	SM
□	GP-2	17	75	0	93	7	N/A	SP-SM

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	GeoPentech	Tested by:	JT	Date:	05/11/23
Project Name:	MLK Hospital	Computed by:	JP	Date:	05/12/23
Project No.:	20044A	Checked by:	AP	Date:	05/15/23

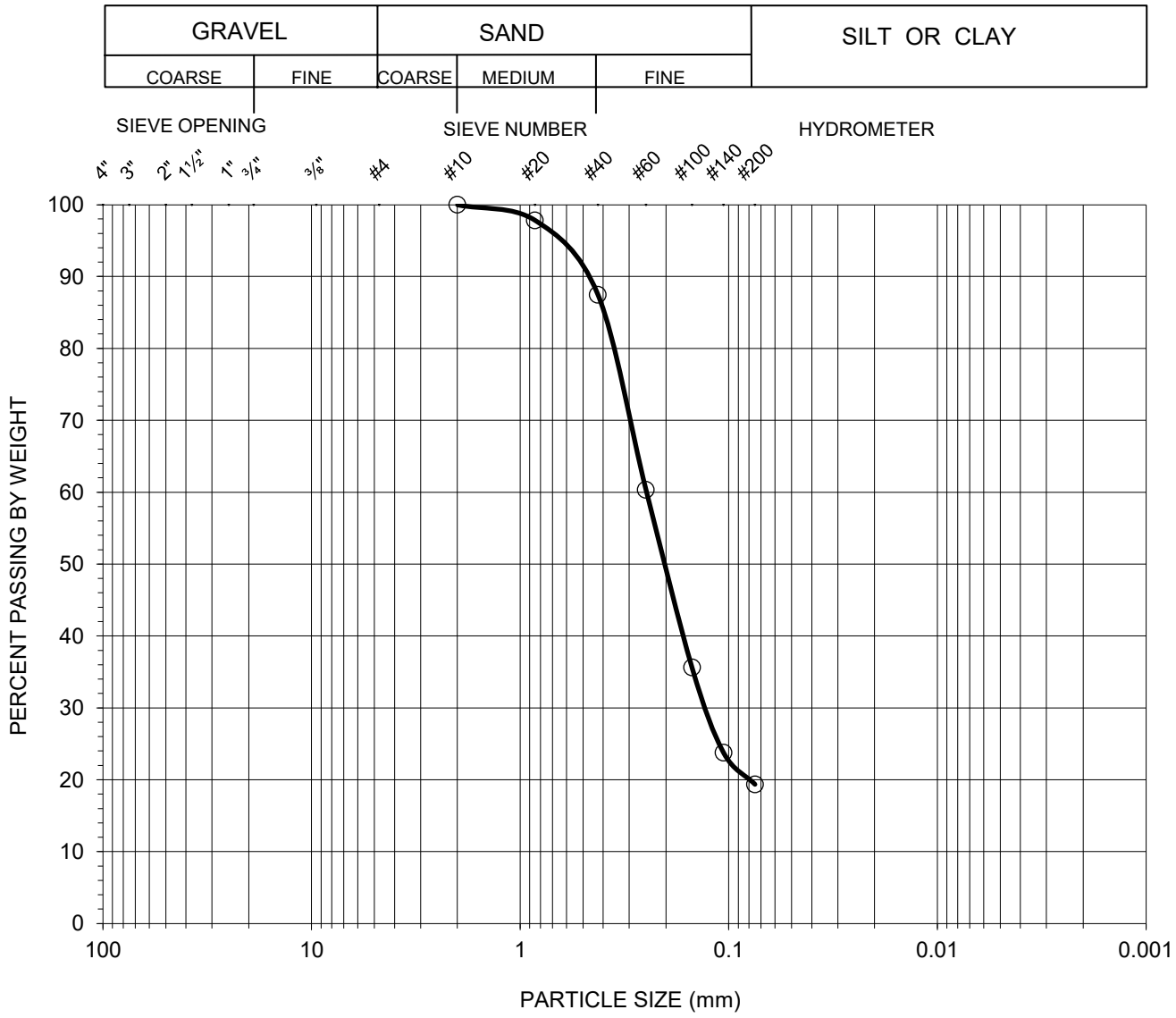


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-3	3b	10	26	54	20	23:15:8	SC
□	GP-3	10	45	17	16	67	34:25:9	ML
△	GP-3	17	80	0	88	12	N/A	SP-SM

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	GeoPentech	Tested by:	JT	Date:	05/11/23
Project Name:	MLK Hospital	Computed by:	JP	Date:	05/12/23
Project No.:	20044A	Checked by:	AP	Date:	05/15/23

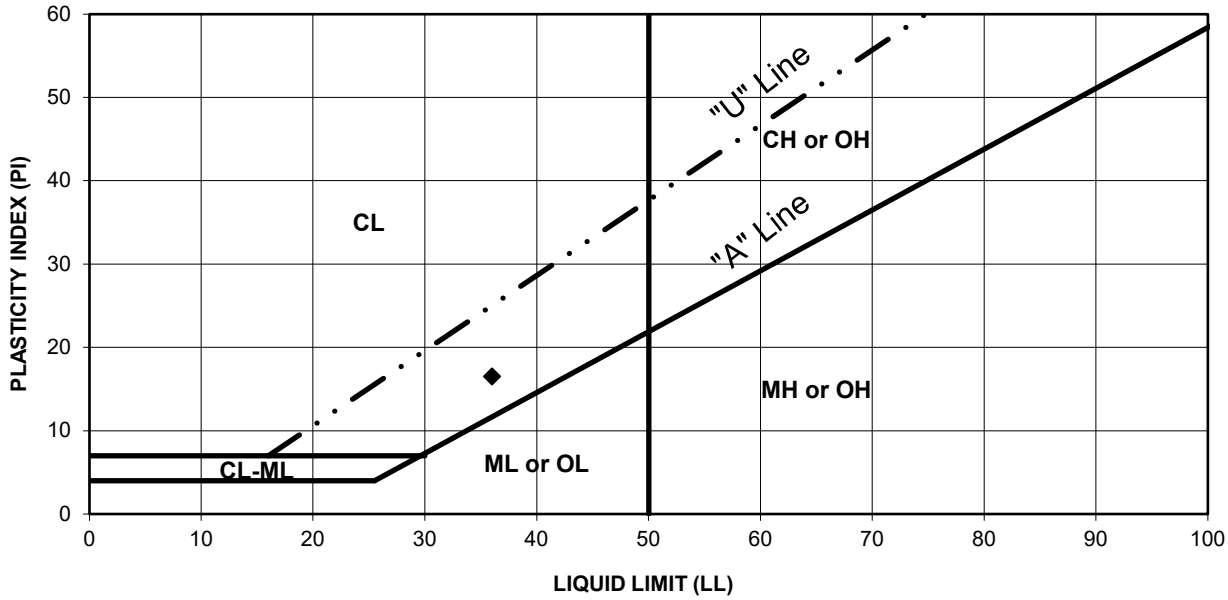


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-3	20	95	0	81	19	N/A	SM

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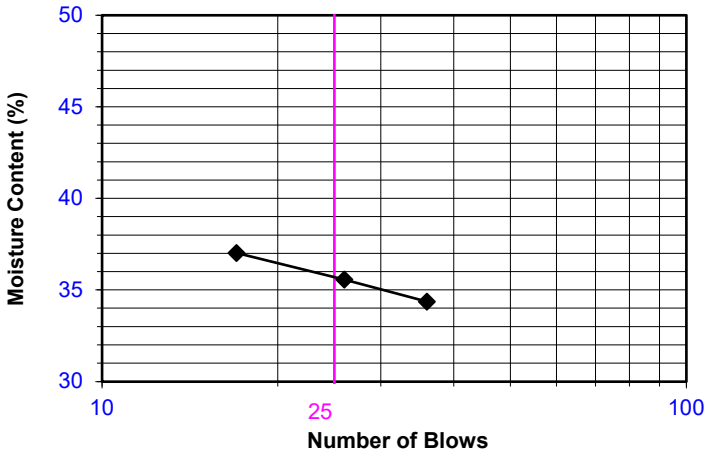
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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



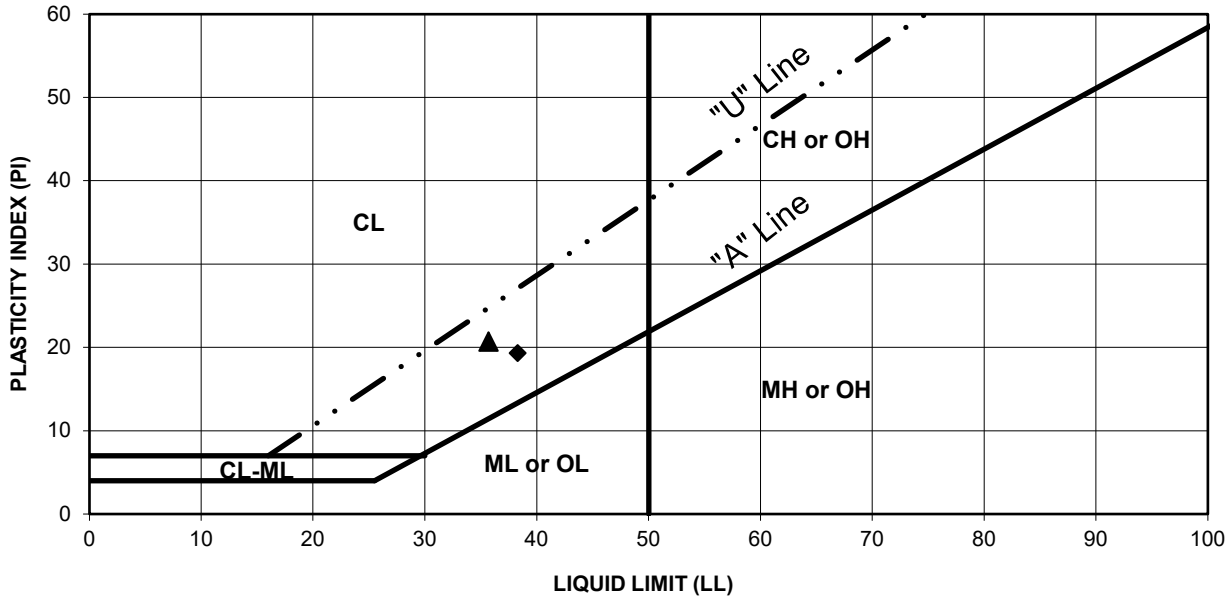
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	2	5	36	19	17	CL
	GP-1	4	10	NP	NP	NP	

* NP denotes "non-plastic"

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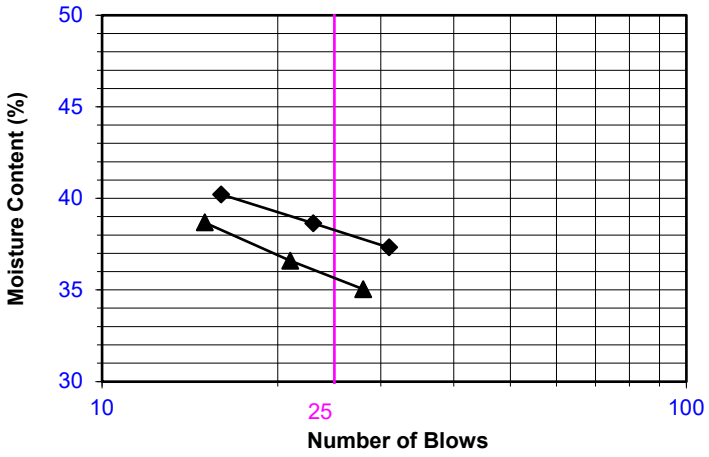
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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

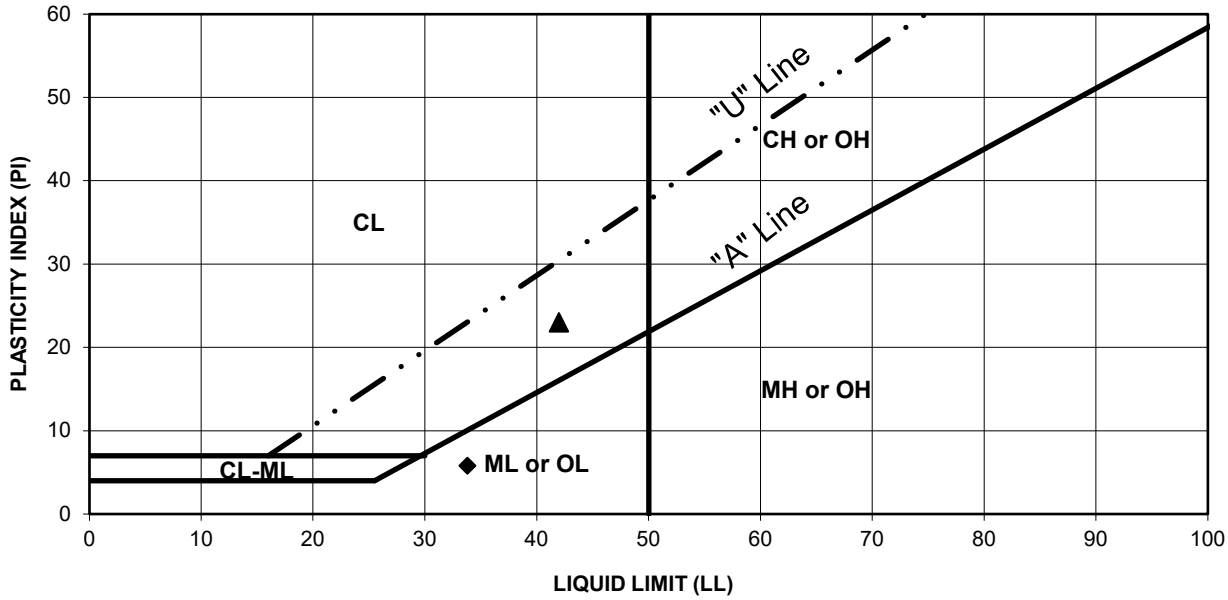


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	6	20	38	19	19	CL
▲	GP-1	8	30	36	15	21	CL

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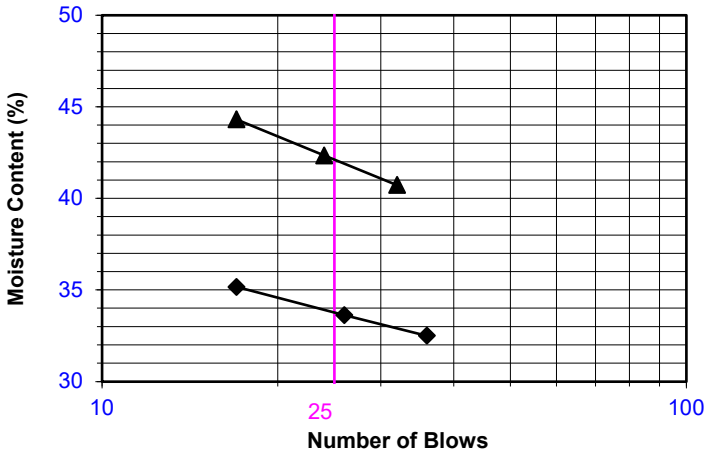
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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

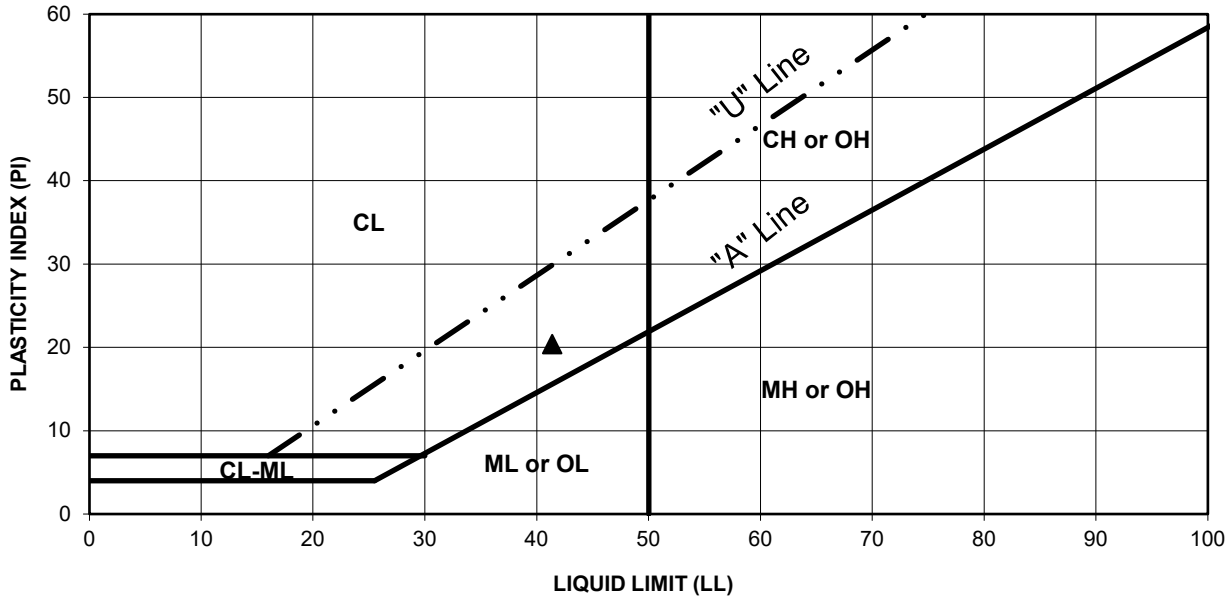


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	12	50	34	28	6	ML
▲	GP-1	13	55	42	19	23	CL

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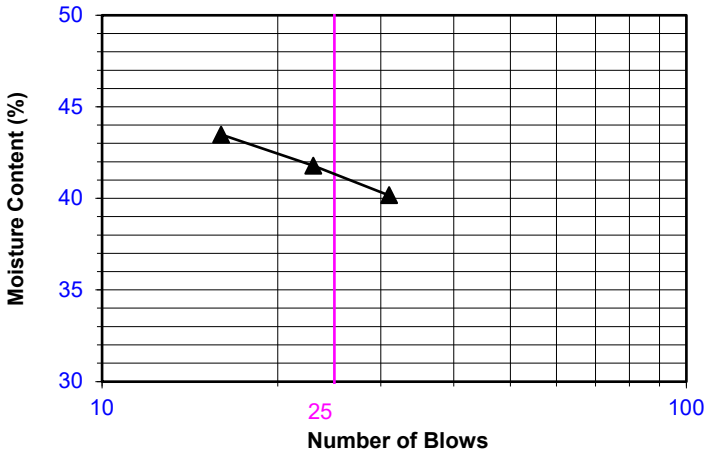
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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



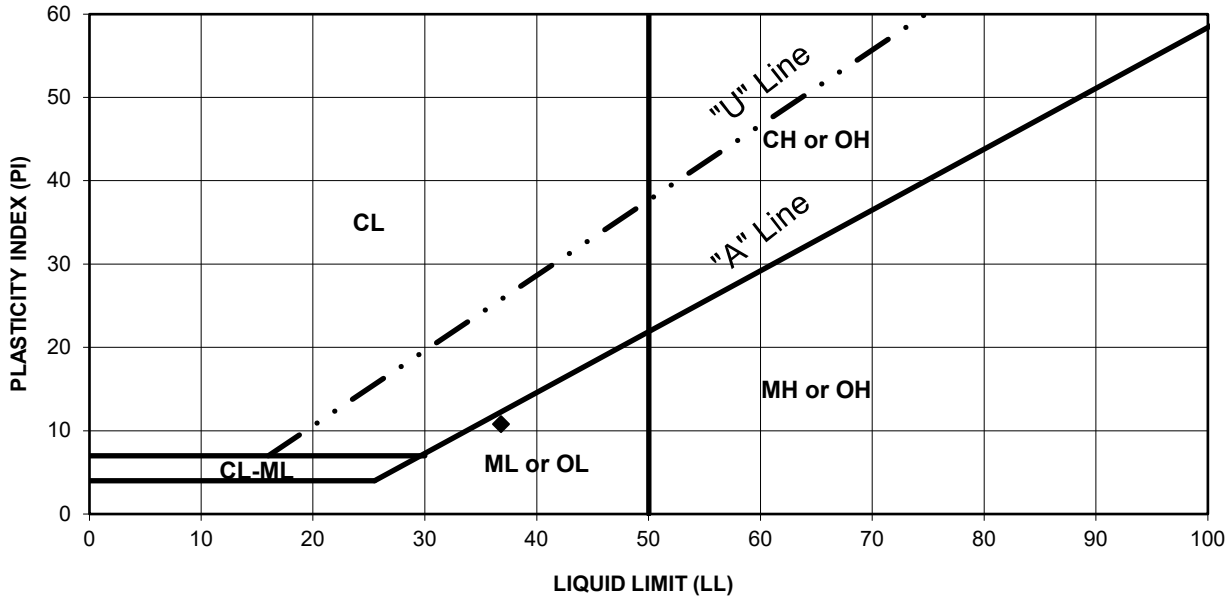
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	GP-1	14	60	NP	NP	NP	
▲	GP-1	18	80	41	21	20	CL

* NP denotes "non-plastic"

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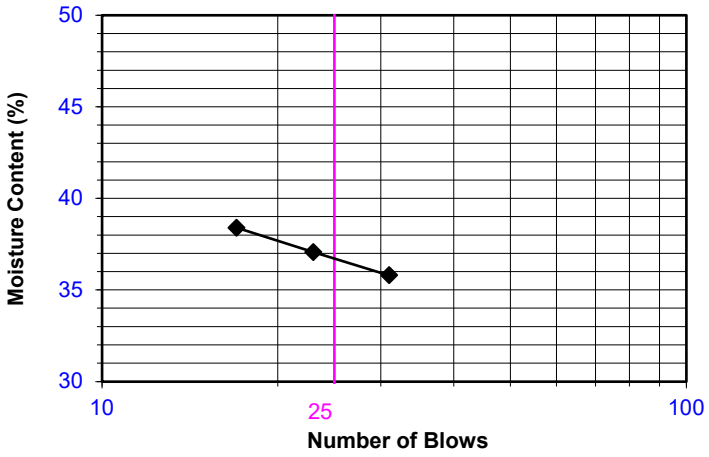
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

- ☐ Wet Preparation
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- ☐ Procedure B One-point Test

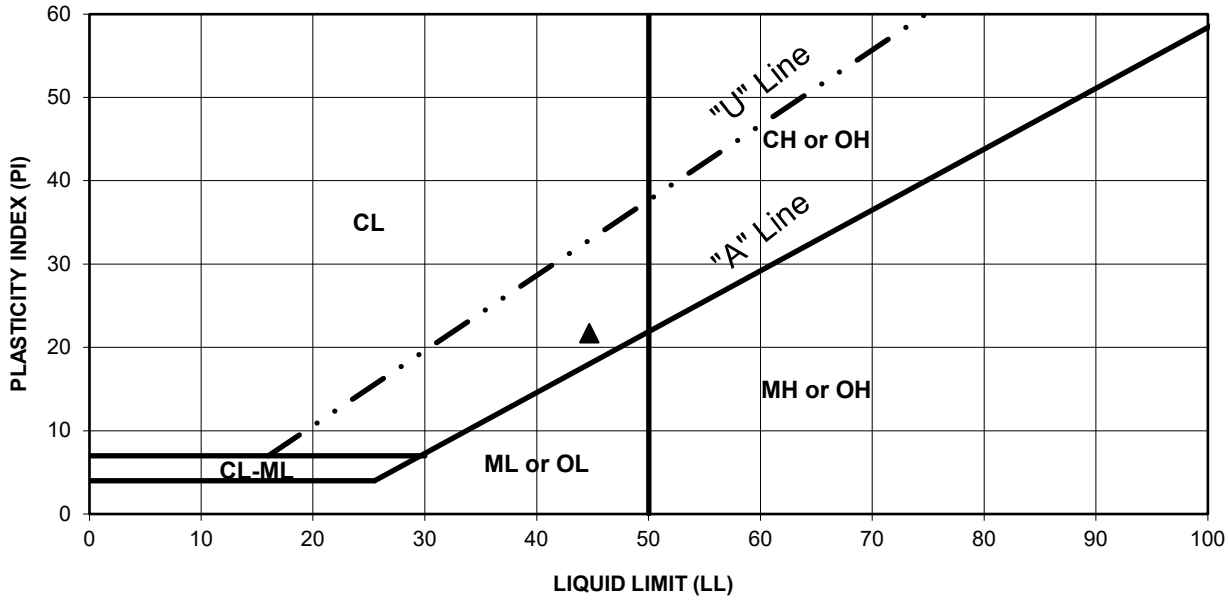


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	21	95	37	26	11	ML

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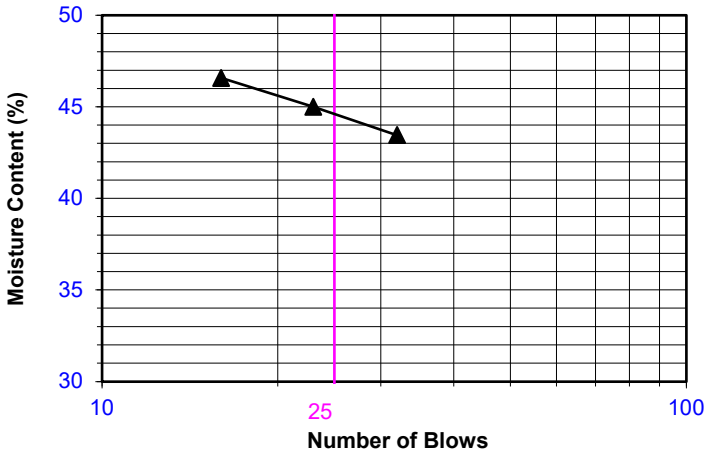
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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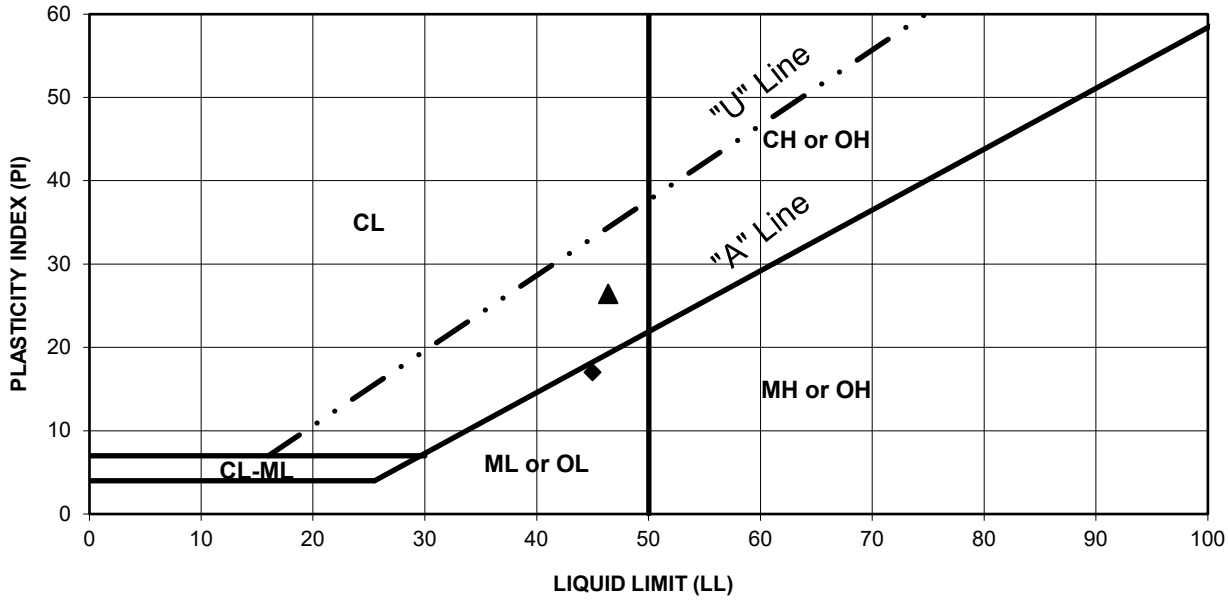
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	GP-2	2	5	NP	NP	NP	
▲	GP-2	6	20	45	23	22	CL

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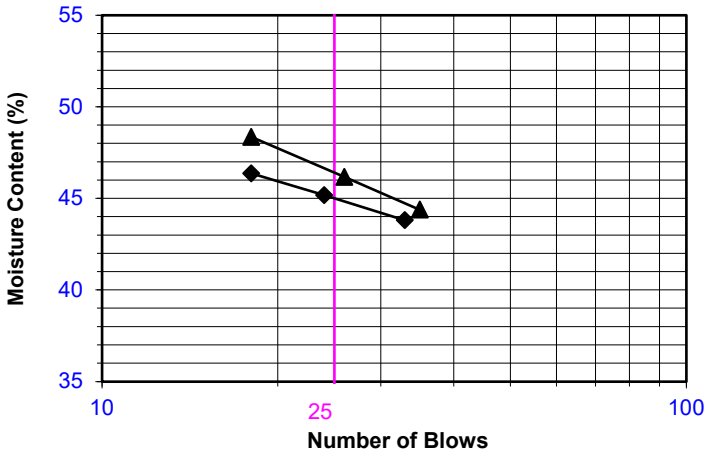
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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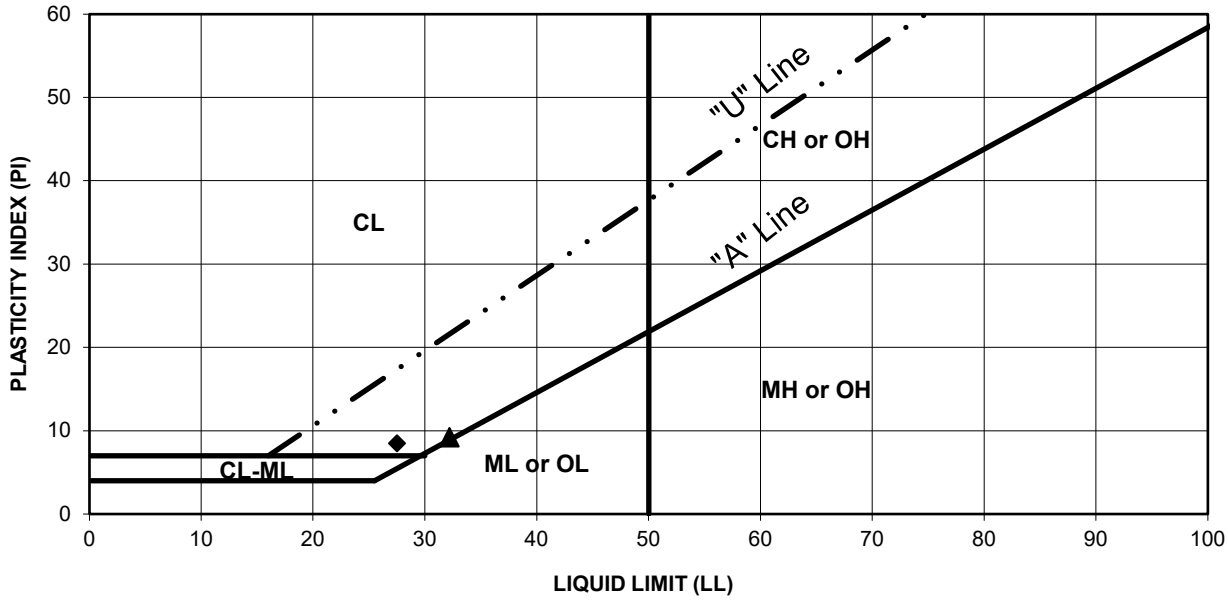


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	12	50	45	28	17	ML
▲	GP-2	13	55	46	20	26	CL

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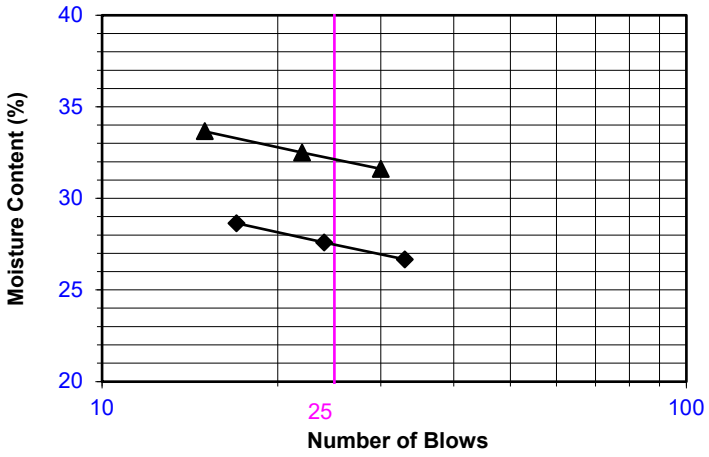
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	05/09/23
Project Name:	MLK Hospital	Computed By:	NR	Date:	05/11/23
Project No.:	20044A	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

- ☐ Wet Preparation
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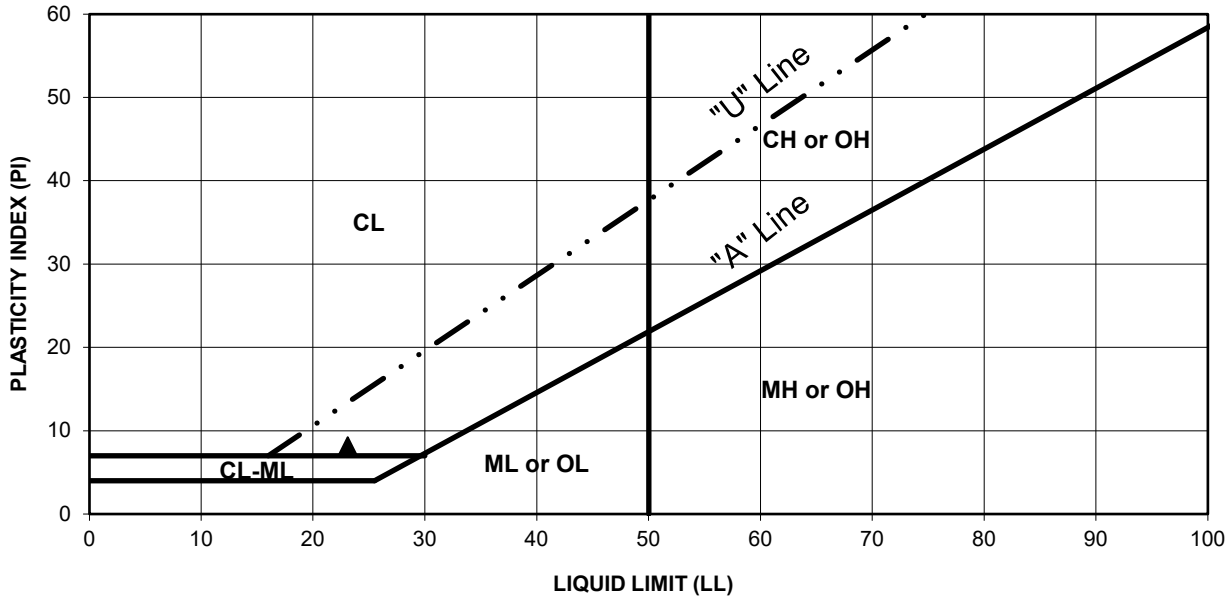


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	14	60	28	19	9	CL
▲	GP-2	18	80	32	23	9	CL

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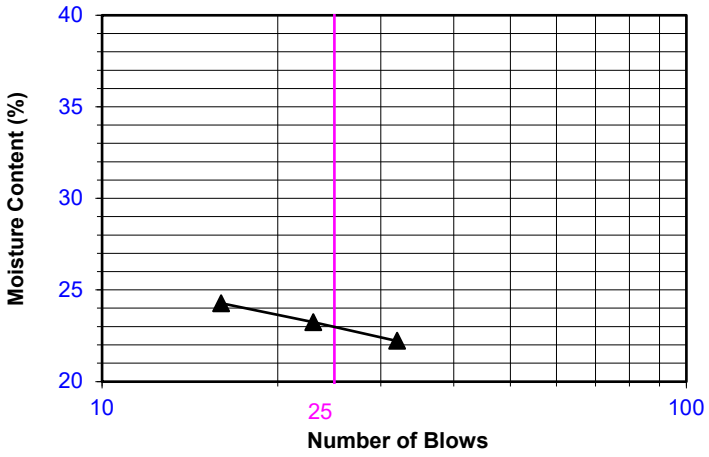
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Kleinfelder	Tested By:	DK	Date:	05/30/13
Project Name:	Terminal Island Water Reclamation Plant	Computed By:	JP	Date:	06/03/13
Project No.:	133978	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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



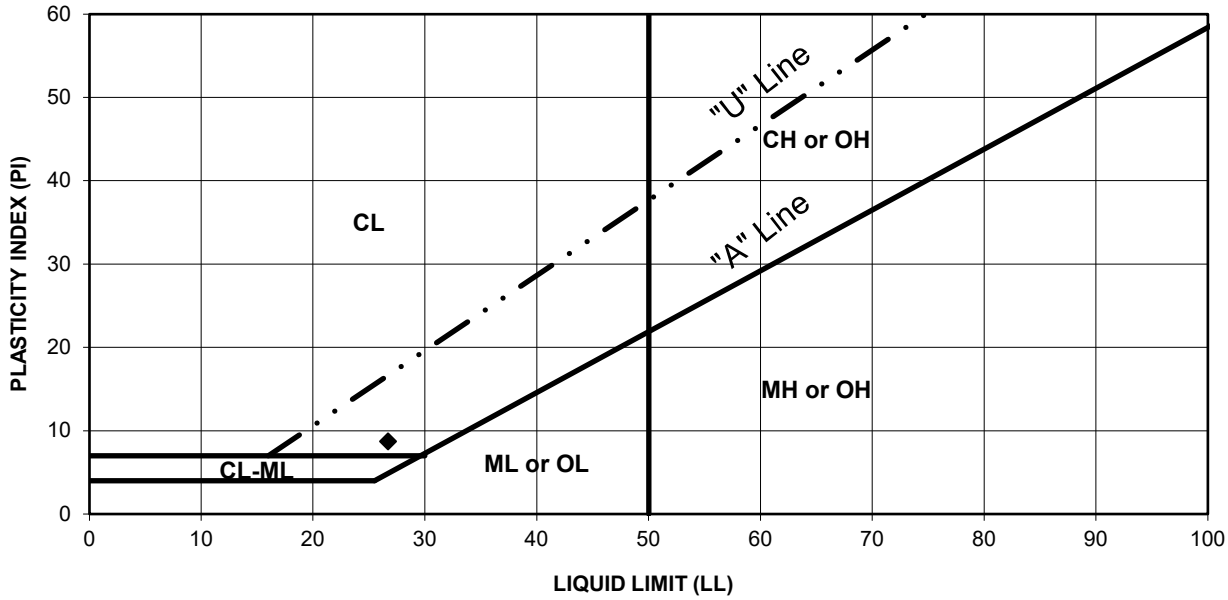
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	GP-3	2	5	NP	NP	NP	
▲	GP-3	3b	10	23	15	8	CL

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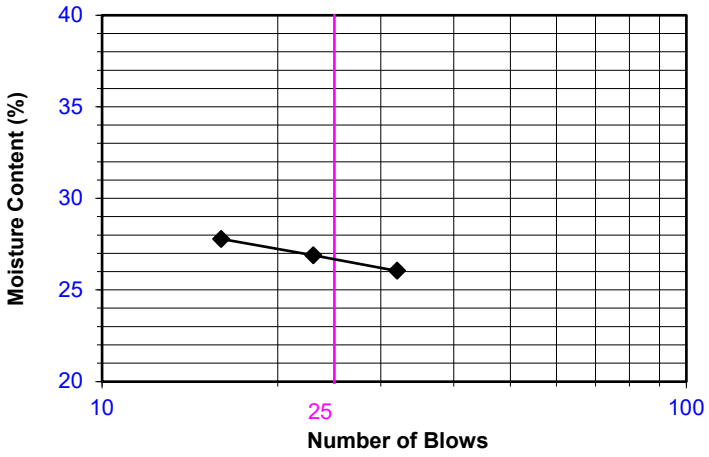
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Kleinfelder	Tested By:	DK	Date:	05/30/13
Project Name:	Terminal Island Water Reclamation Plant	Computed By:	JP	Date:	06/03/13
Project No.:	133978	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
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- ☐ Procedure B One-point Test



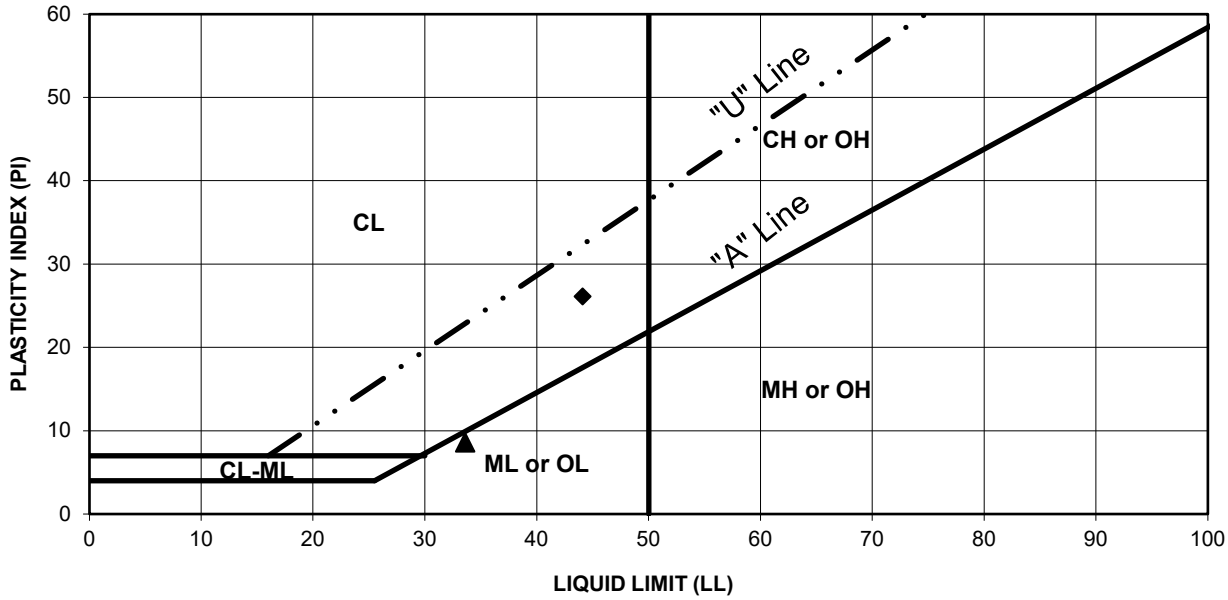
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	4	15	27	18	9	CL
	GP-3	5	20	NP	NP	NP	

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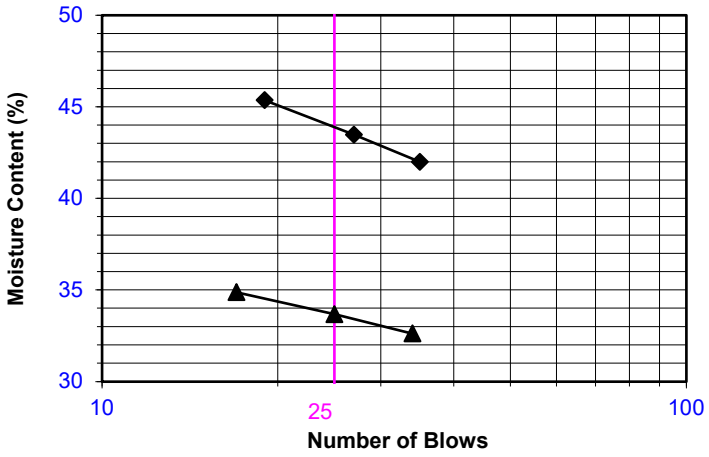
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Kleinfelder	Tested By:	DK	Date:	05/30/13
Project Name:	Terminal Island Water Reclamation Plant	Computed By:	JP	Date:	06/03/13
Project No.:	133978	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

- ☐ Wet Preparation
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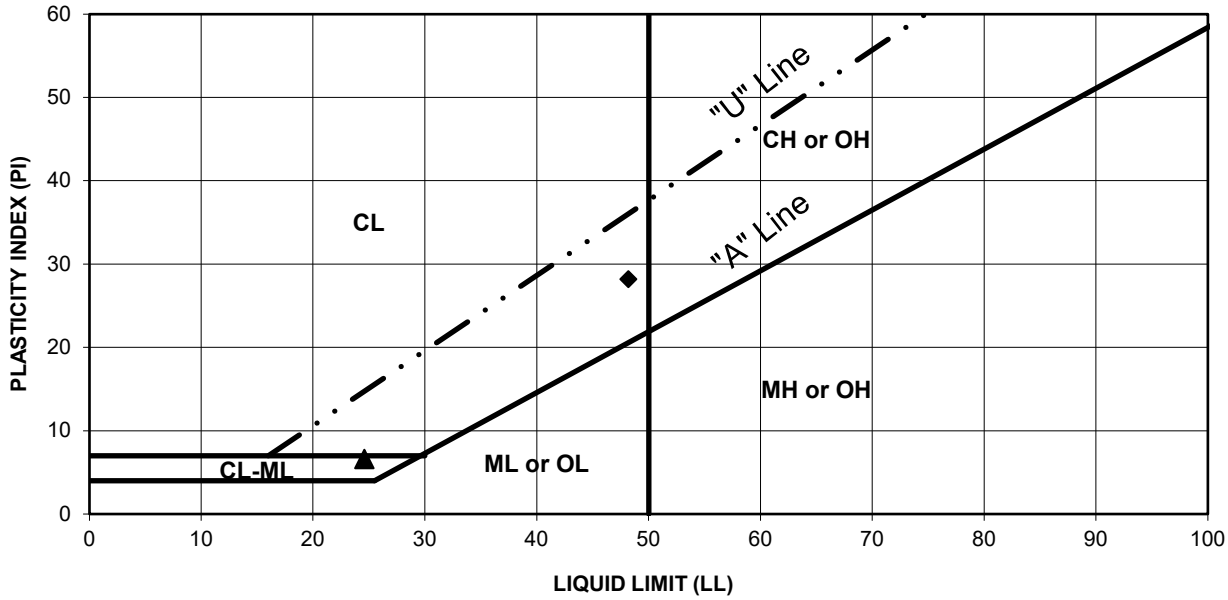


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	7	30	44	18	26	CL
▲	GP-3	10	45	34	25	9	ML

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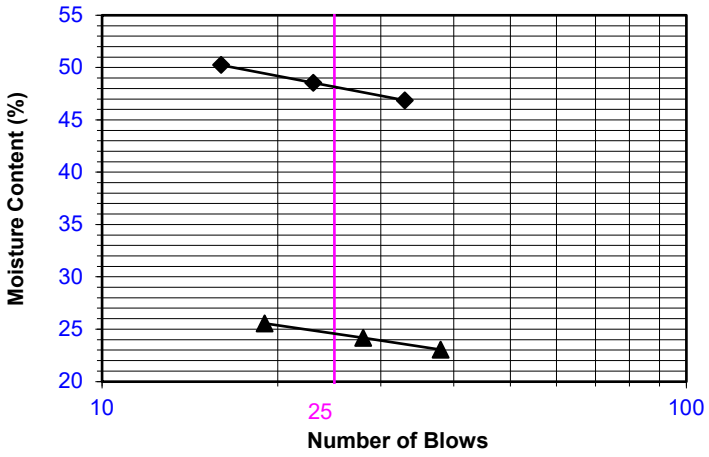
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Kleinfelder	Tested By:	DK	Date:	05/30/13
Project Name:	Terminal Island Water Reclamation Plant	Computed By:	JP	Date:	06/03/13
Project No.:	133978	Checked By:	AP	Date:	05/15/23



PROCEDURE USED

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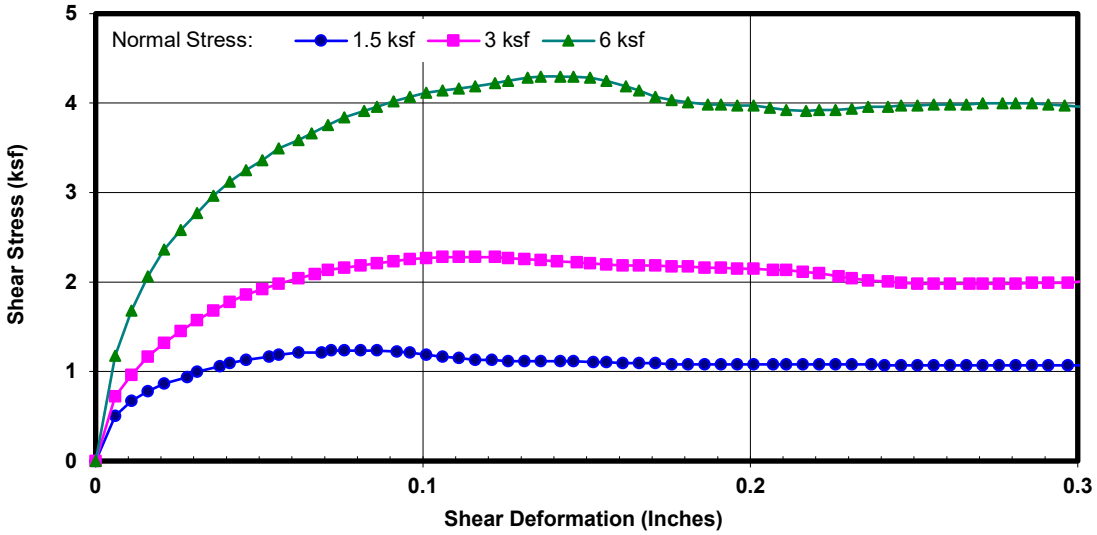
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	14	65	48	20	28	CL
▲	GP-3	15b	70	25	18	7	CL-ML

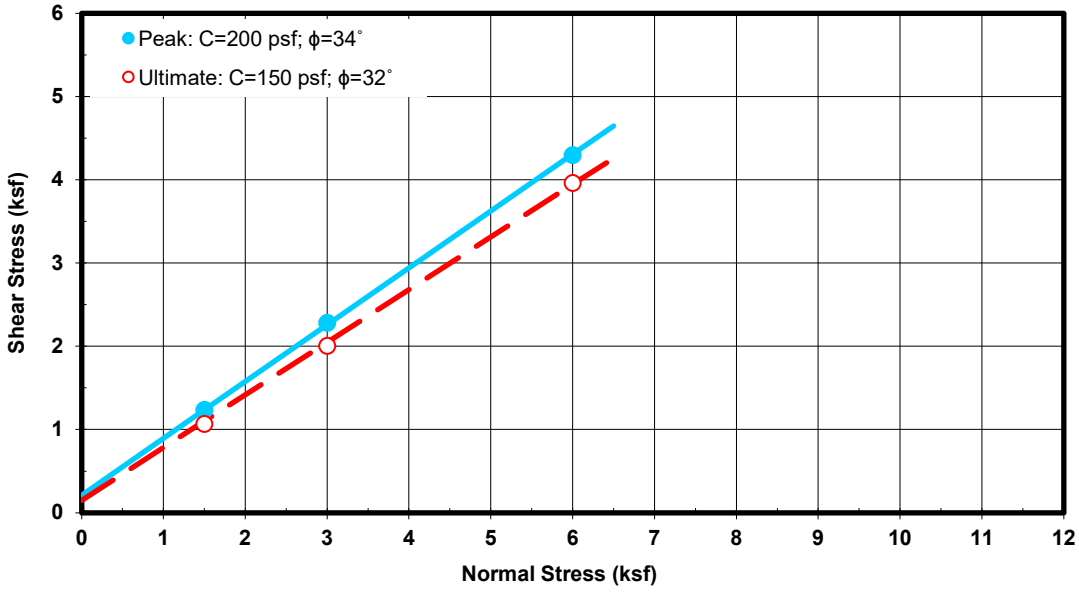
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	ST	**Date:** 05/03/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/11/23
Boring No.:	GP-1		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	4	**Depth (ft):** 10			
Sample Type:	Mod. Cal.				
Soil Description:	Sandy Silt				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.4	102.9	21.8	23.7	92	100	1.5	1.236	1.068
						3	2.280	2.004
						6	4.296	3.960



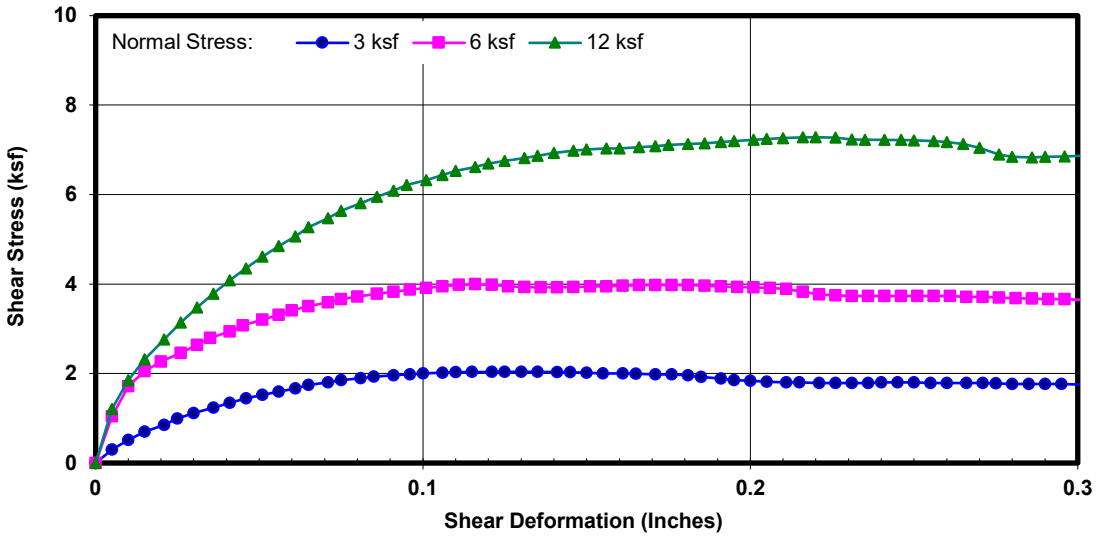


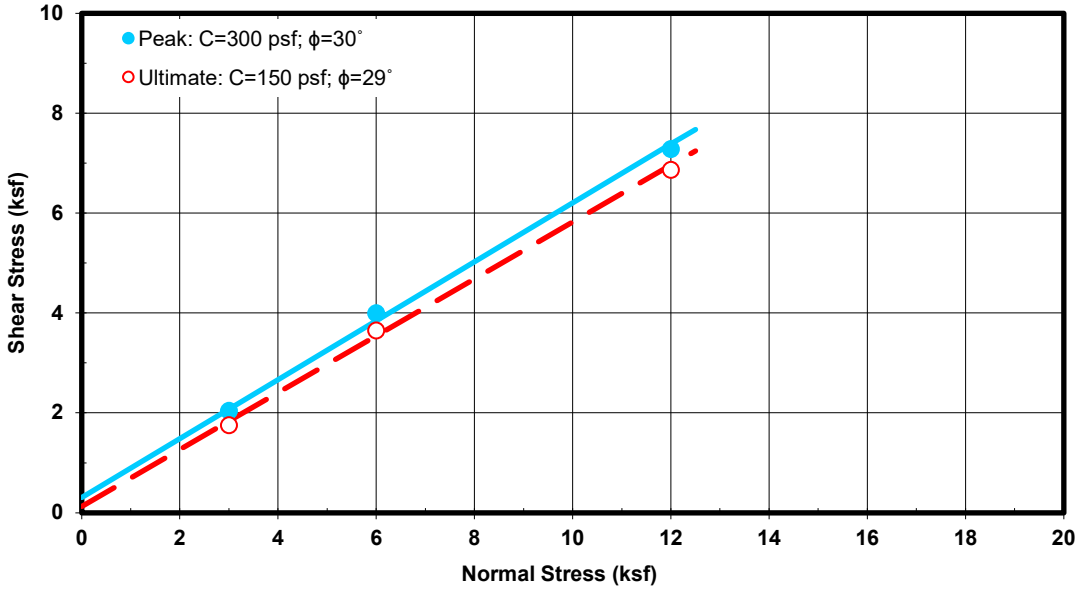
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	ST	**Date:** 05/11/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/15/23
Boring No.:	GP-1		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	12	**Depth (ft):** 50			
Sample Type:	Mod. Cal.				
Soil Description:	Silt w/sand				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
123.6	97.2	27.2	27.3	100	100	3	2.040	1.752
						6	3.996	3.648
						12	7.283	6.867



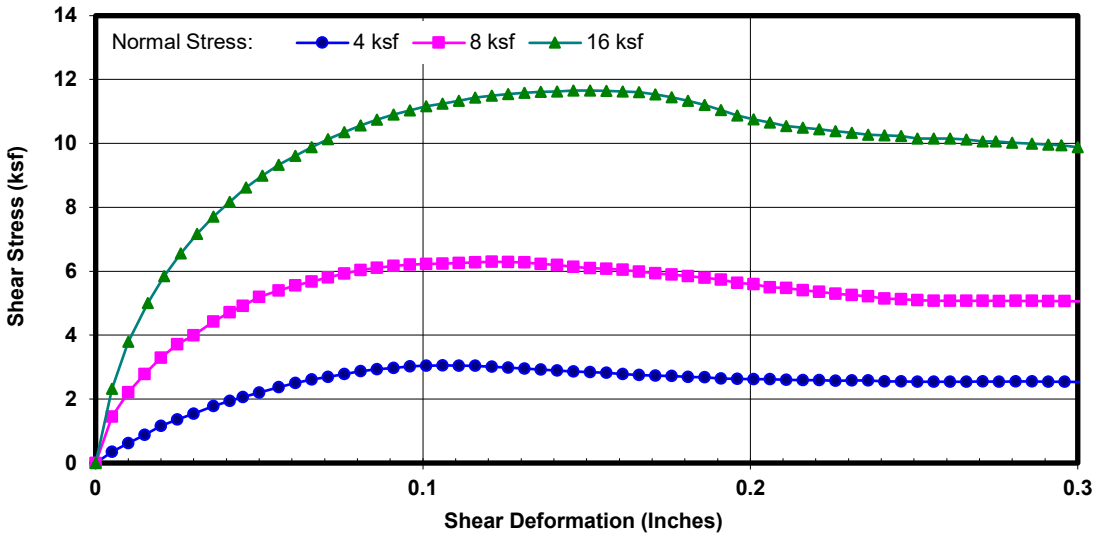


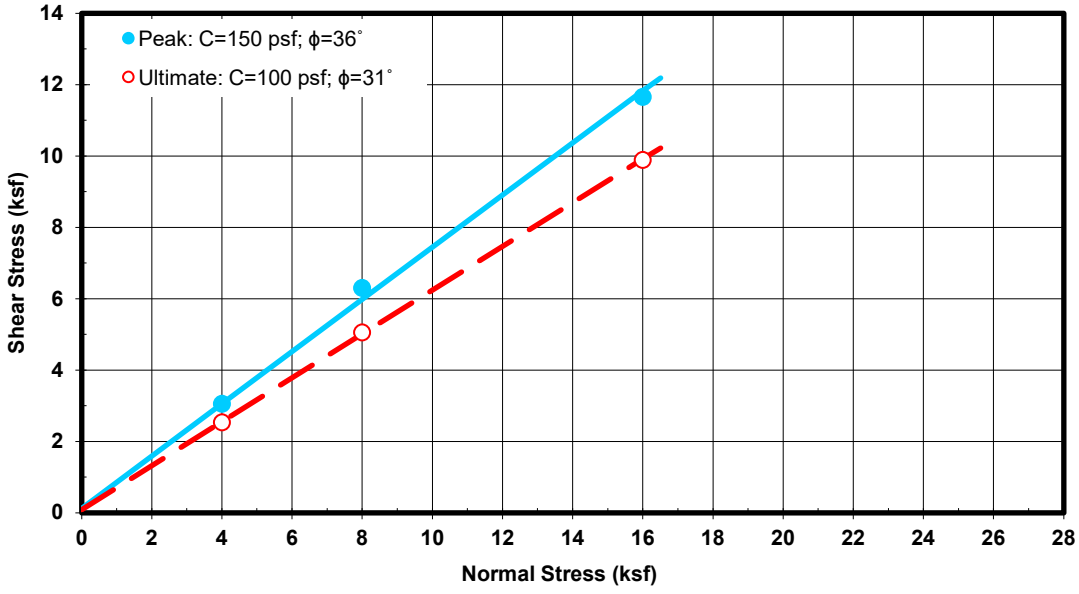
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	ST	**Date:** 05/07/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/11/23
Boring No.:	GP-1		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	16	**Depth (ft):** 70			
Sample Type:	Mod. Cal.				
Soil Description:	Sand w/silt				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
128.4	106.3	20.8	21.8	96	100	4	3.054	2.532
						8	6.300	5.052
						16	11.654	9.886



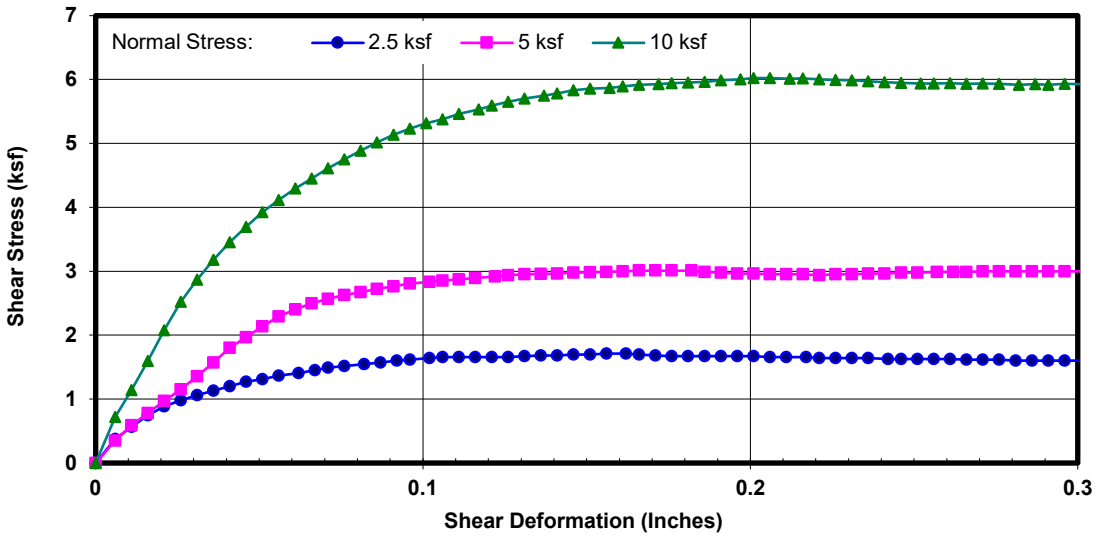


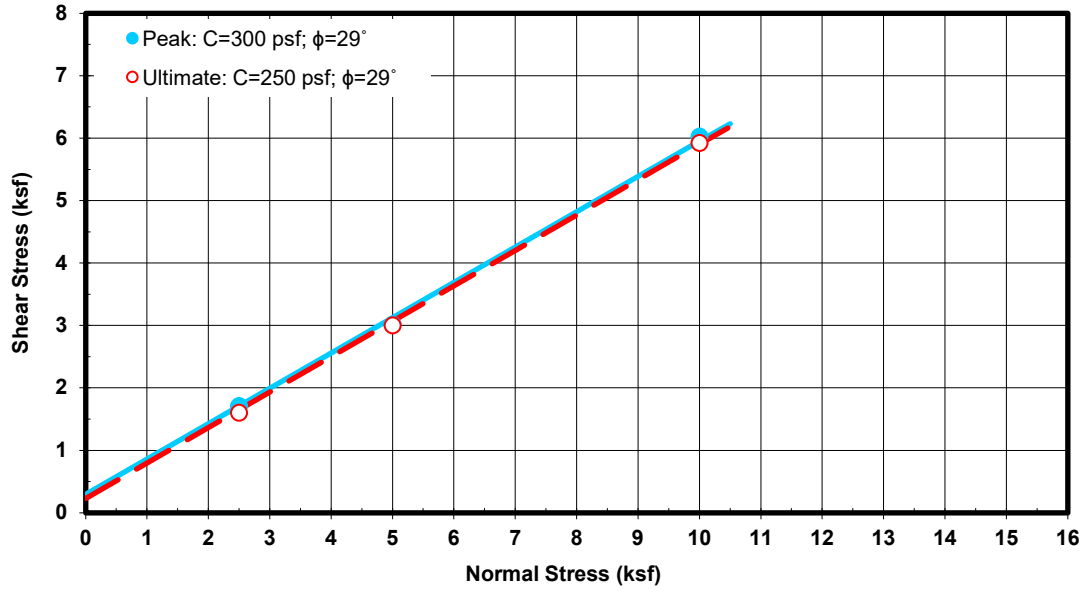
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	AP	**Date:** 05/10/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/12/23
Boring No.:	GP-2		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	6	**Depth (ft):** 20			
Sample Type:	Mod. Cal.				
Soil Description:	Lean Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
121.8	95.0	28.2	28.8	98	100	2.5	1.711	1.601
						5	3.012	3.000
						10	6.024	5.925



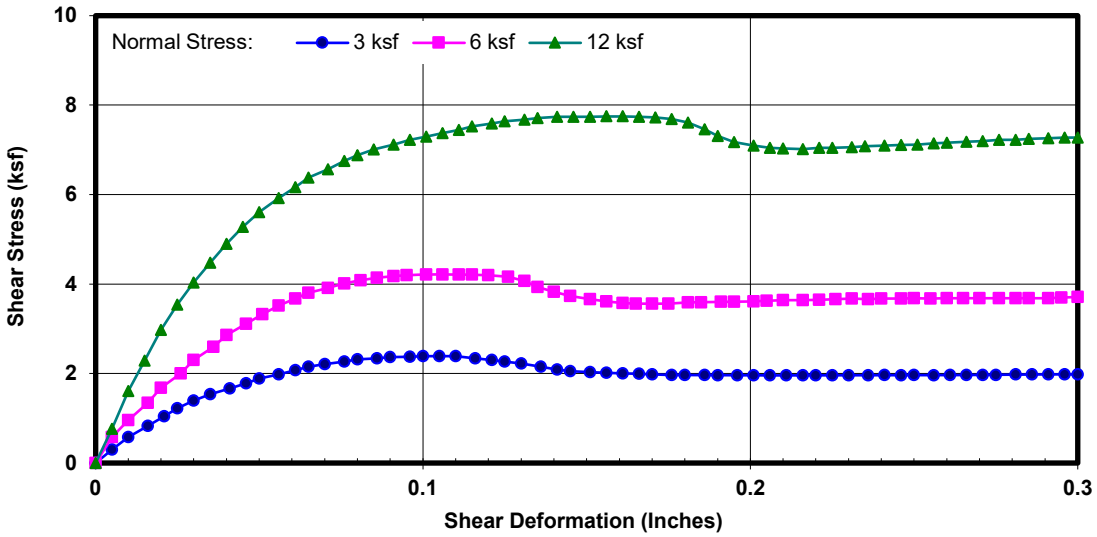


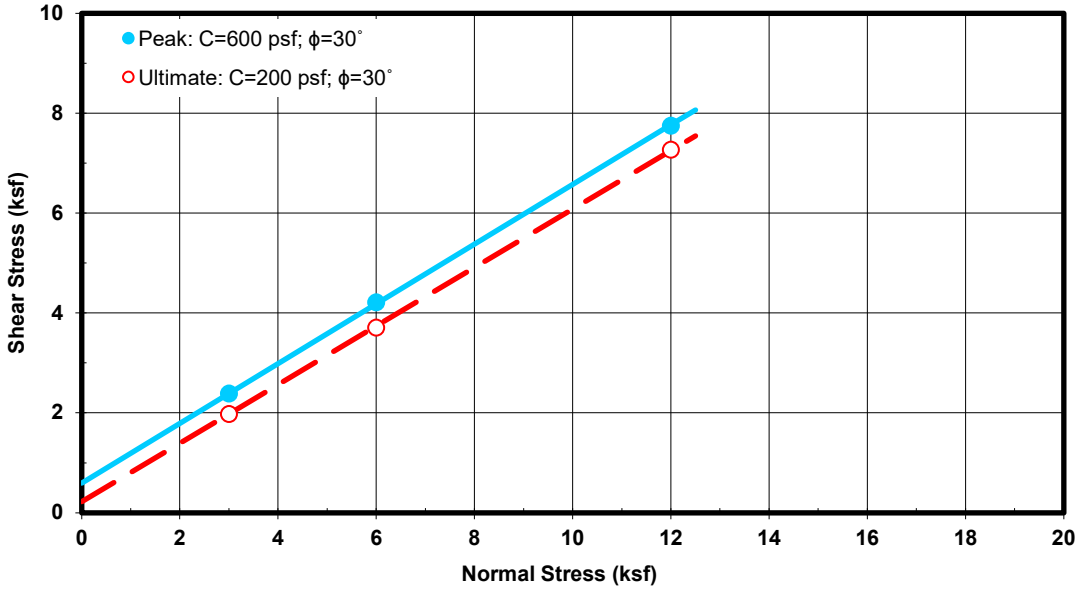
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	AP	**Date:** 05/08/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/11/23
Boring No.:	GP-2		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	10	**Depth (ft):** 40			
Sample Type:	Mod. Cal.				
Soil Description:	Sandy silt				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
120.5	100.2	20.2	23.1	80	92	3	2.388	1.980
						6	4.212	3.708
						12	7.749	7.270



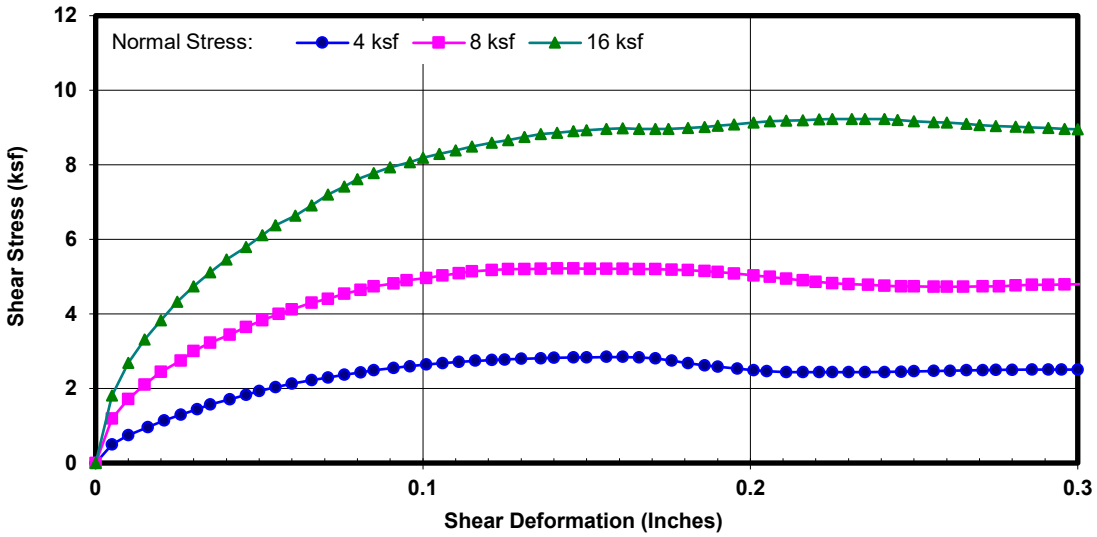


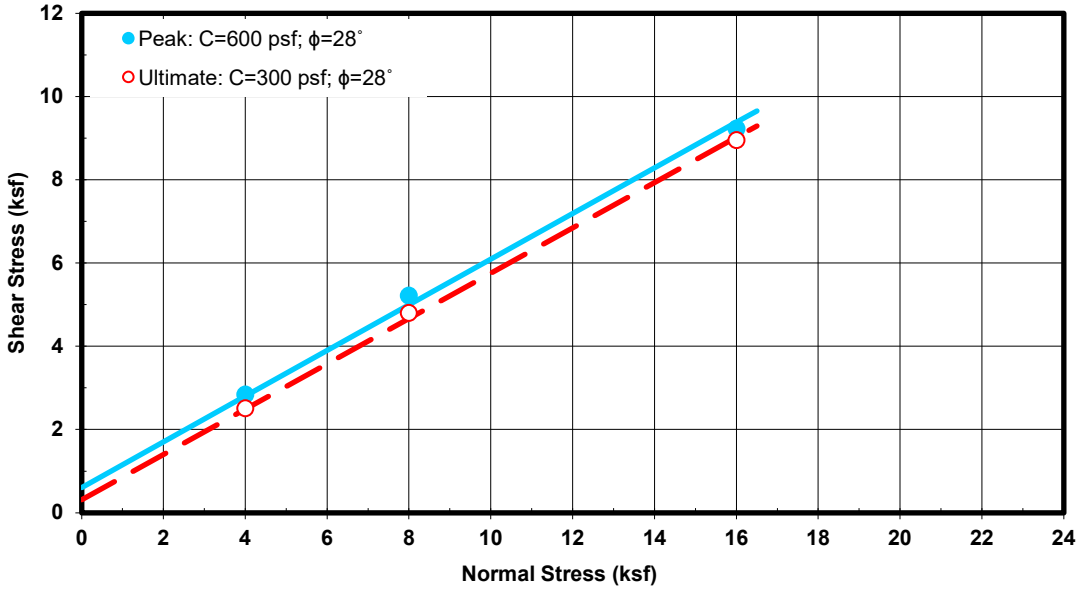
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	AP	**Date:** 05/09/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/11/23
Boring No.:	GP-2		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	14	**Depth (ft):** 60			
Sample Type:	Mod. Cal.				
Soil Description:	Sandy Lean Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
131.9	110.3	19.5	19.6	100	100	4	2.844	2.508
						8	5.220	4.800
						16	9.227	8.951



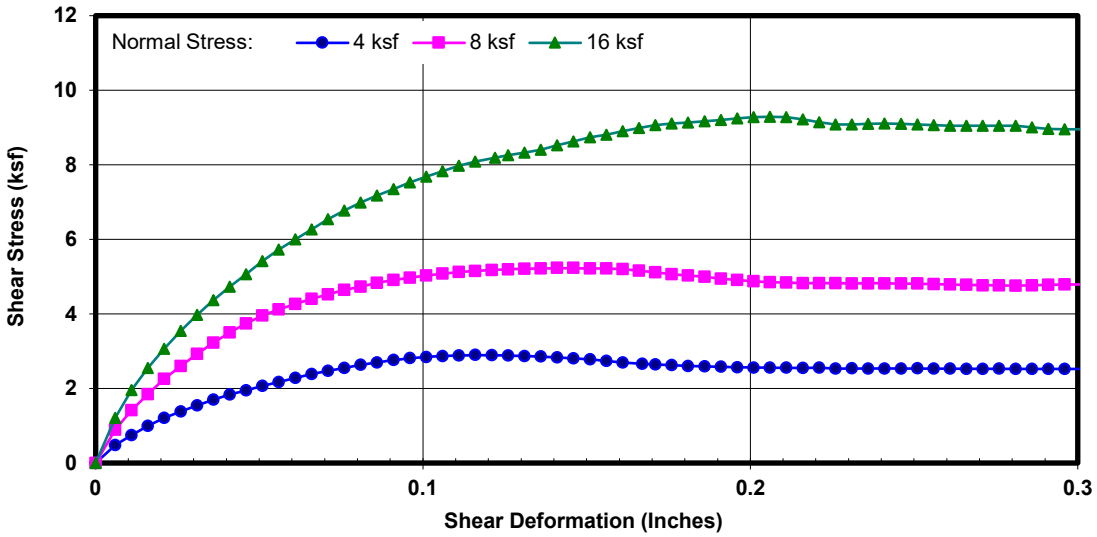


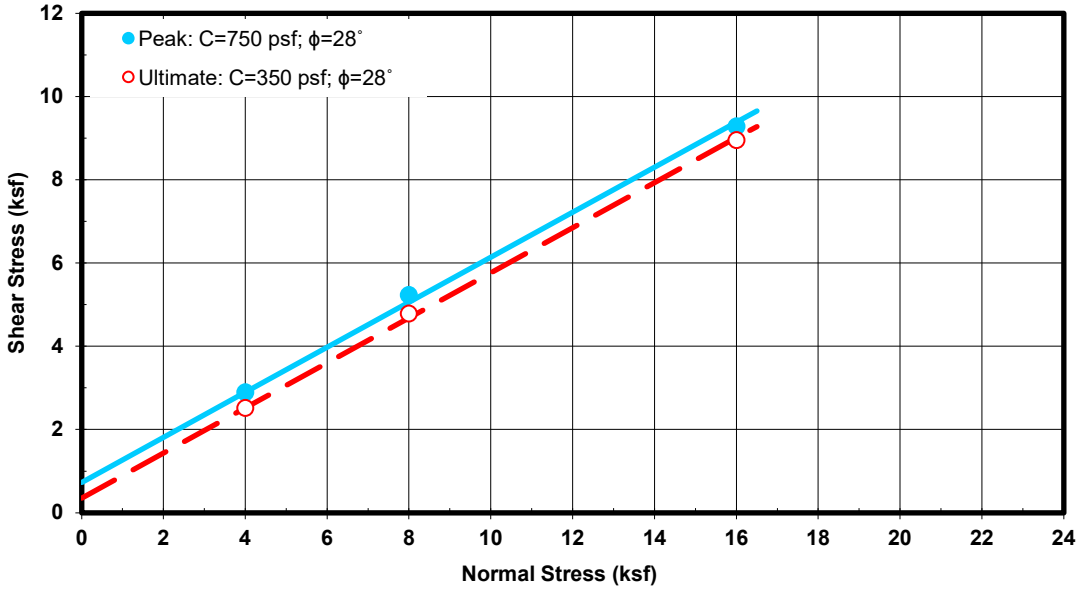
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	AP	**Date:** 05/09/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/11/23
Boring No.:	GP-2		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	18	**Depth (ft):** 80			
Sample Type:	Mod. Cal.				
Soil Description:	Lean Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.4	107.5	20.4	20.9	97	99	4	2.895	2.518
						8	5.232	4.788
						16	9.287	8.951





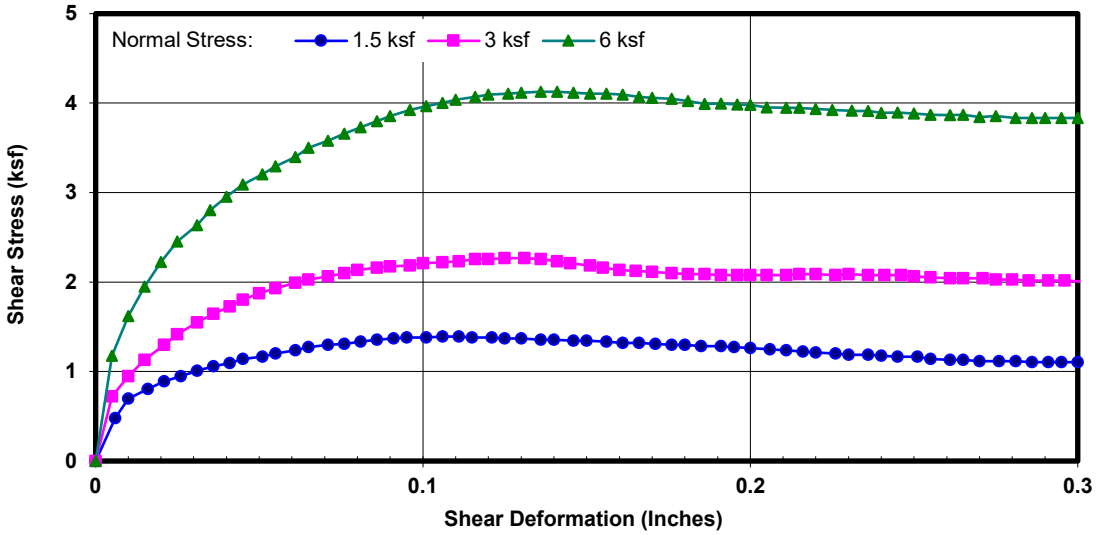
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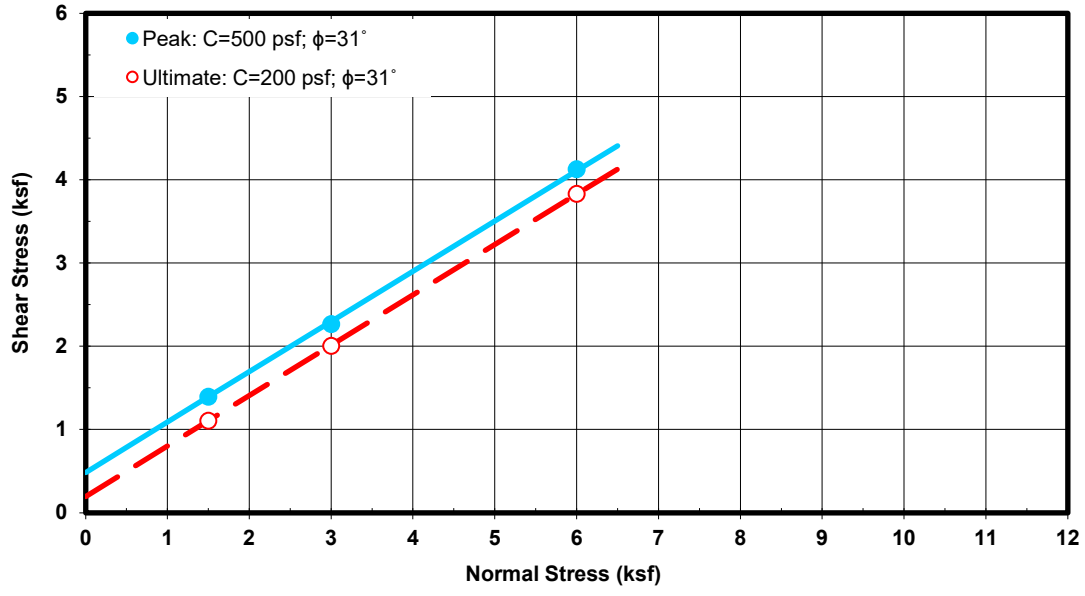
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: MLK Hospital
Project No.: 20044A
Boring No.: GP-3
Sample No.: 4 **Depth (ft):** 15
Sample Type: Mod. Cal.
Soil Description: Sandy Lean Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 05/02/23
Computed By: NR **Date:** 05/11/23
Checked by: AP **Date:** 05/15/23

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.6	111.1	16.6	17.8	87	93	1.5	1.392	1.104
						3	2.268	2.004
						6	4.127	3.832



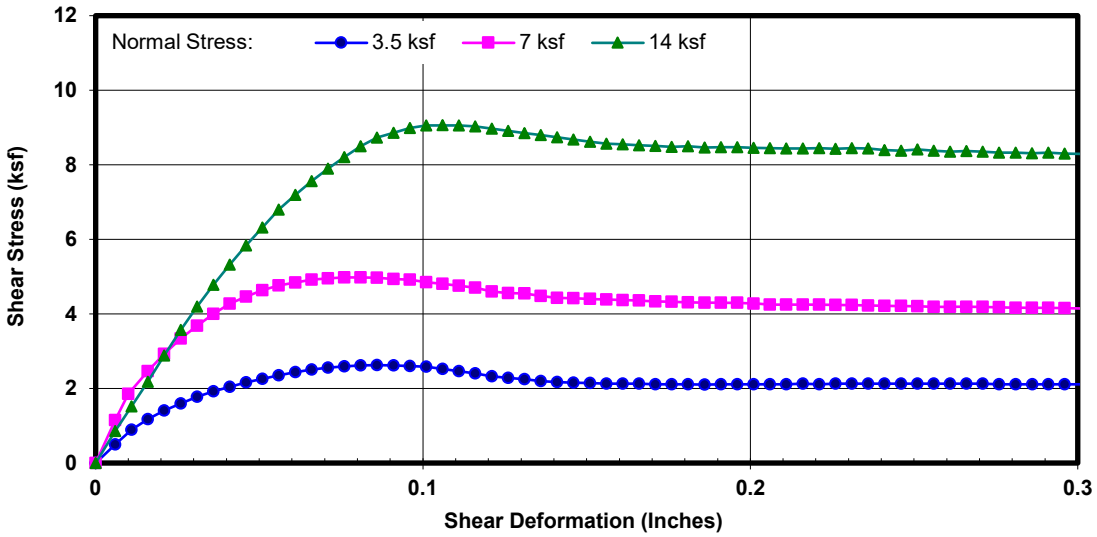


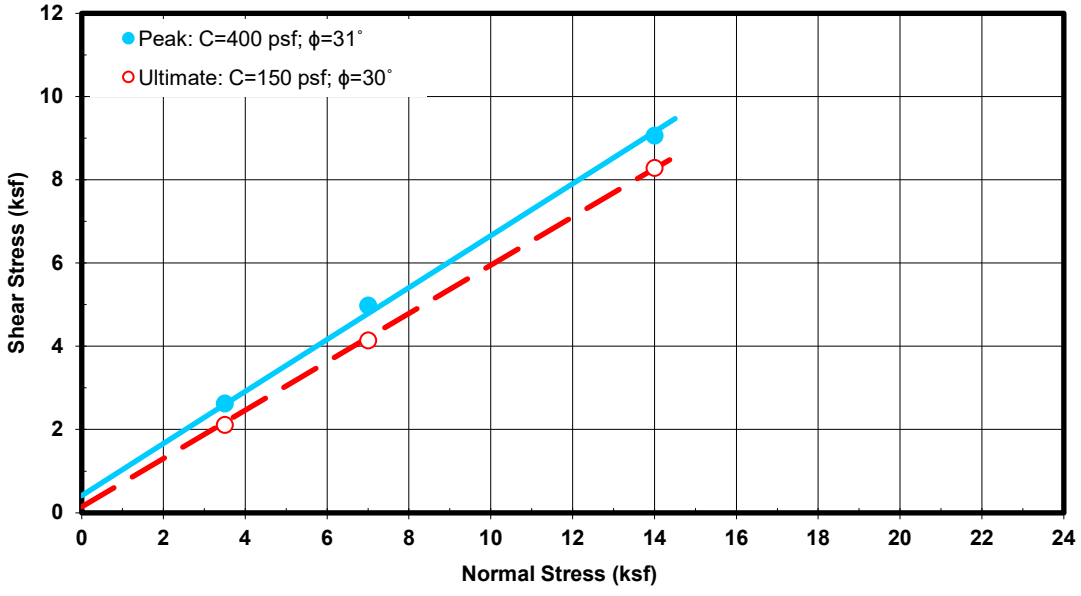
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	MLK Hospital		**Tested By:**	ST	**Date:** 05/11/23
Project No.:	20044A		**Computed By:**	JP	**Date:** 05/15/23
Boring No.:	GP-3		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	10	**Depth (ft):** 45			
Sample Type:	Mod. Cal.				
Soil Description:	Sandy Silt w/gravel				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.2	106.8	17.2	21.3	80	99	3.5	2.628	2.112
						7	4.978	4.140
						14	9.062	8.287



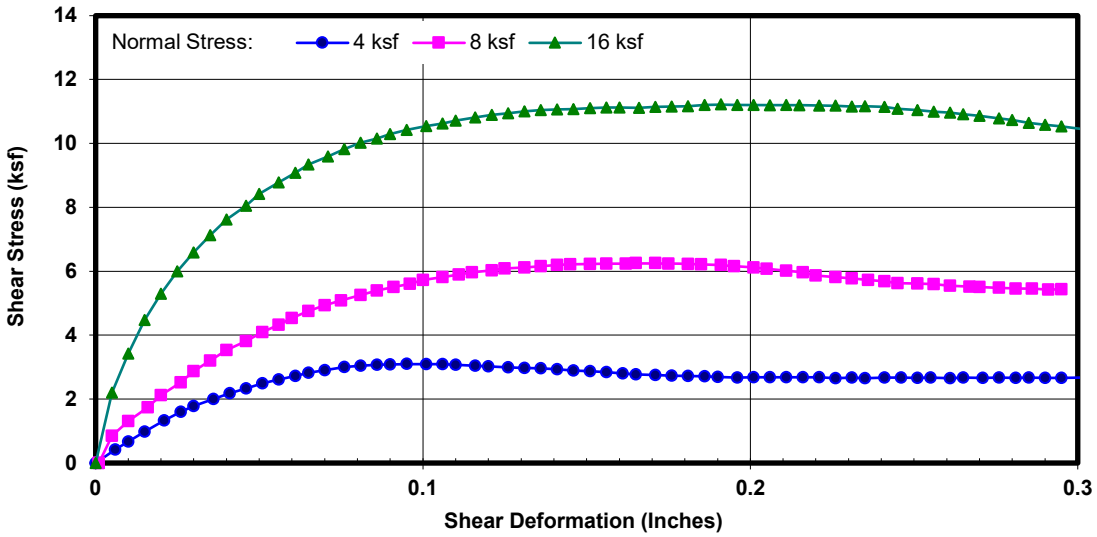


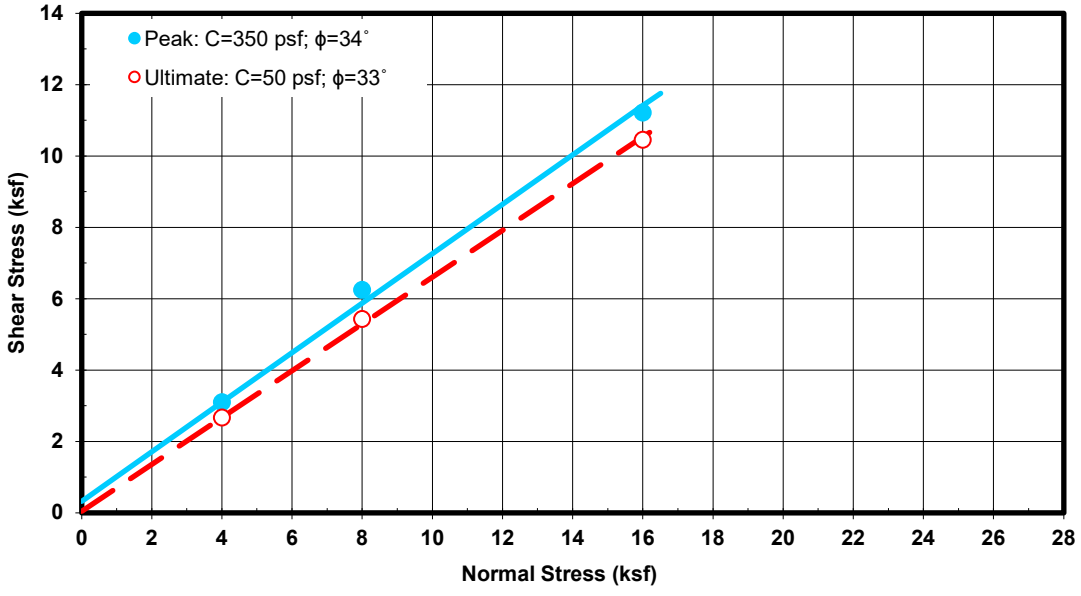
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

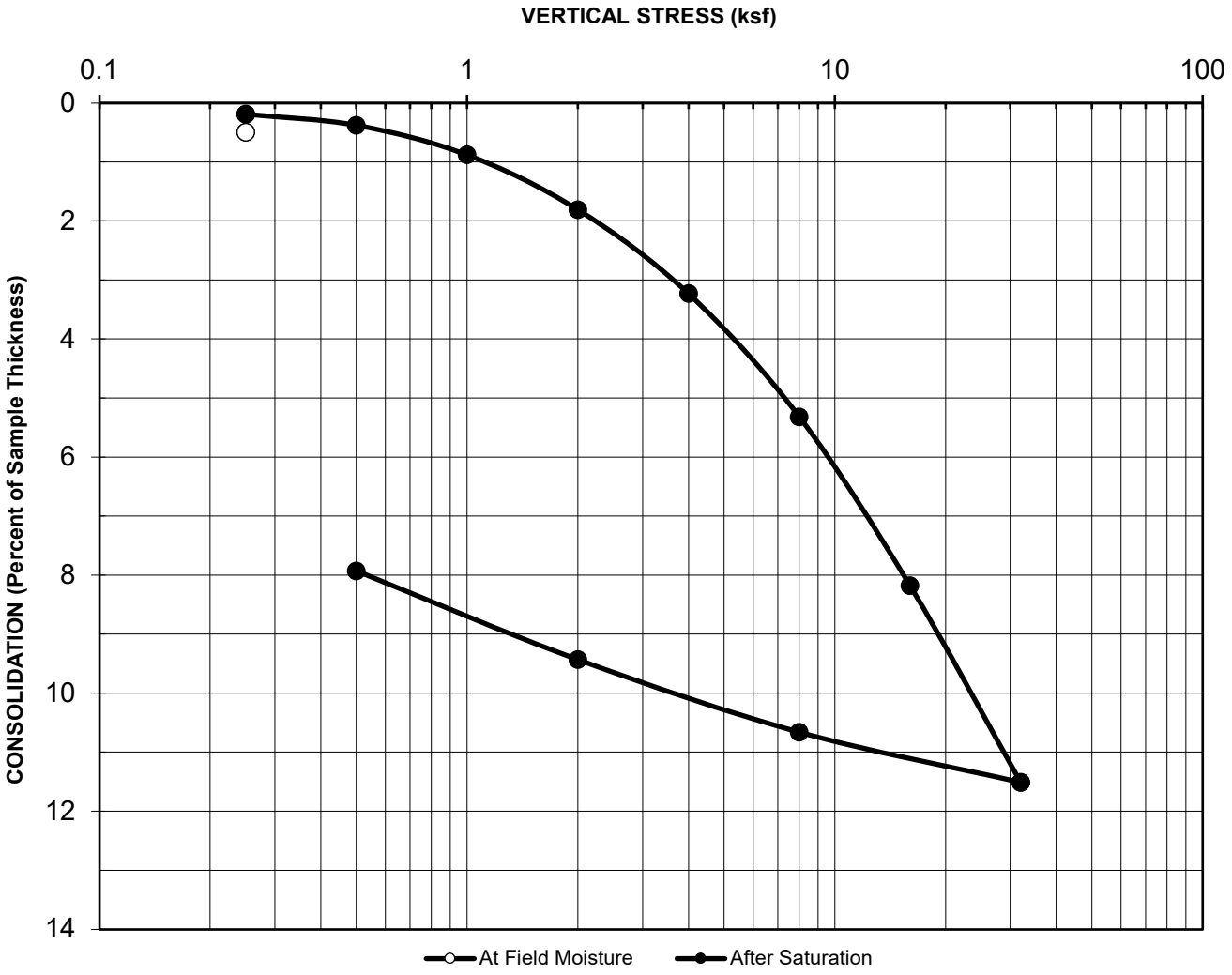
Project Name:	MLK Hospital		**Tested By:**	AP	**Date:** 05/10/23
Project No.:	20044A		**Computed By:**	NR	**Date:** 05/12/23
Boring No.:	GP-3		**Checked by:**	AP	**Date:** 05/15/23
Sample No.:	18	**Depth (ft):** 85			
Sample Type:	Mod. Cal.				
Soil Description:	Sand w/silt				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.1	106.8	20.9	21.3	97	99	4	3.099	2.670
						8	6.252	5.436
						16	11.219	10.453





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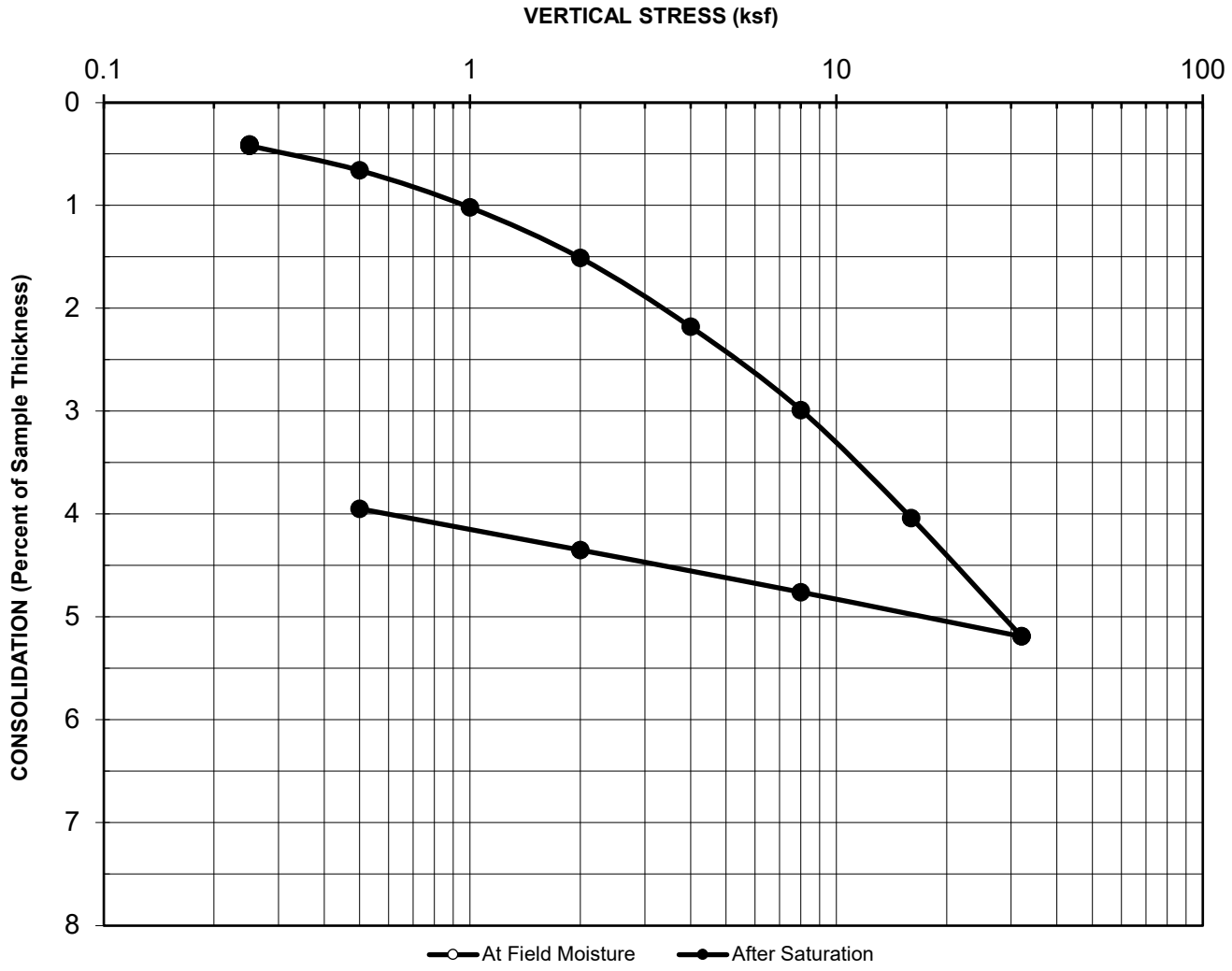


Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	101.5
Sample No.:	6	Initial Moisture Content (%):	24.6
Depth (feet):	20	Final Moisture Content (%):	24.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay	Initial Void Ratio:	0.66
Remarks:	Swell= 0.31% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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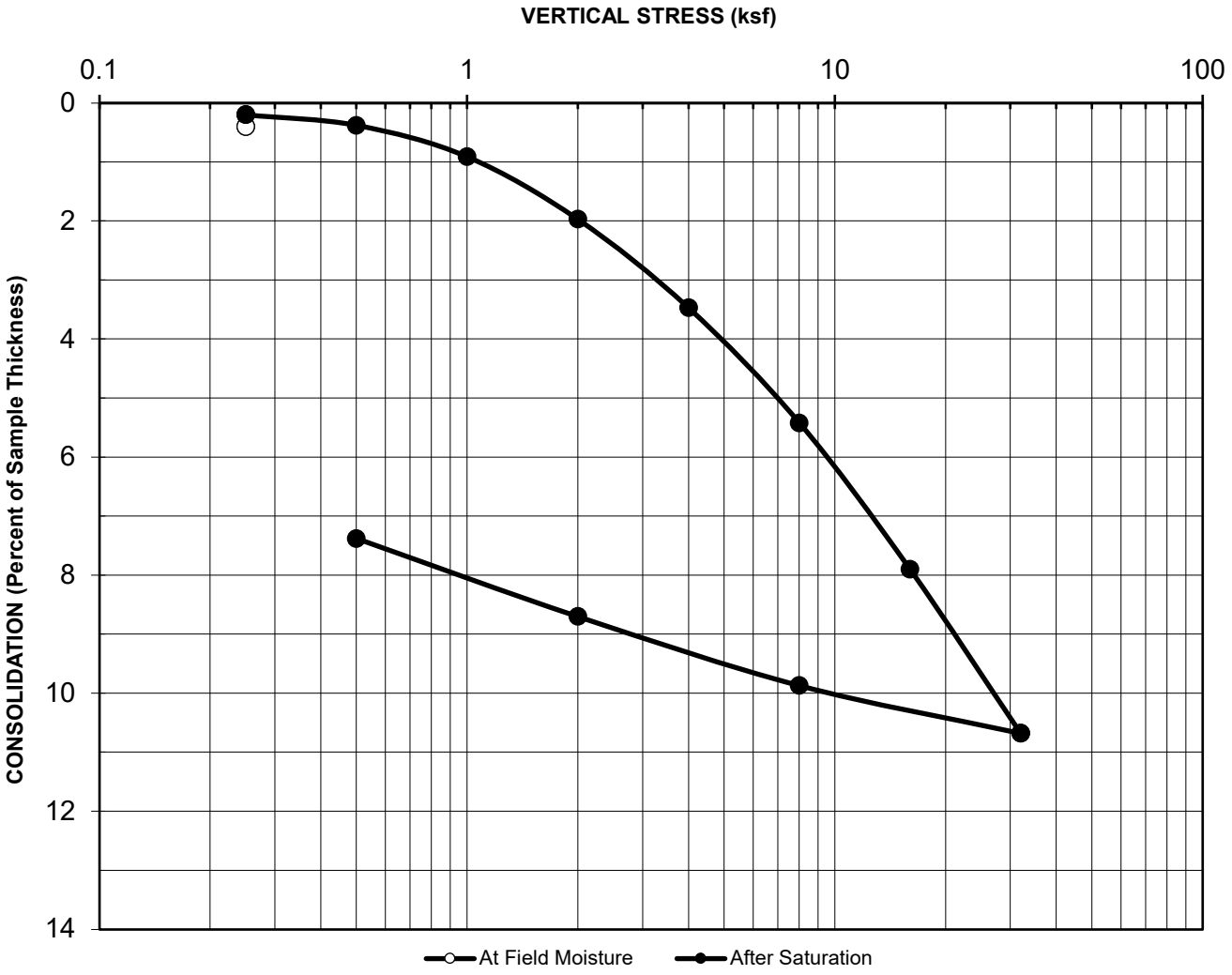


Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	122.9
Sample No.:	14	Initial Moisture Content (%):	14.9
Depth (feet):	60	Final Moisture Content (%):	14.8
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Silty Sand	Initial Void Ratio:	0.37
Remarks:	Collapse= 0.01% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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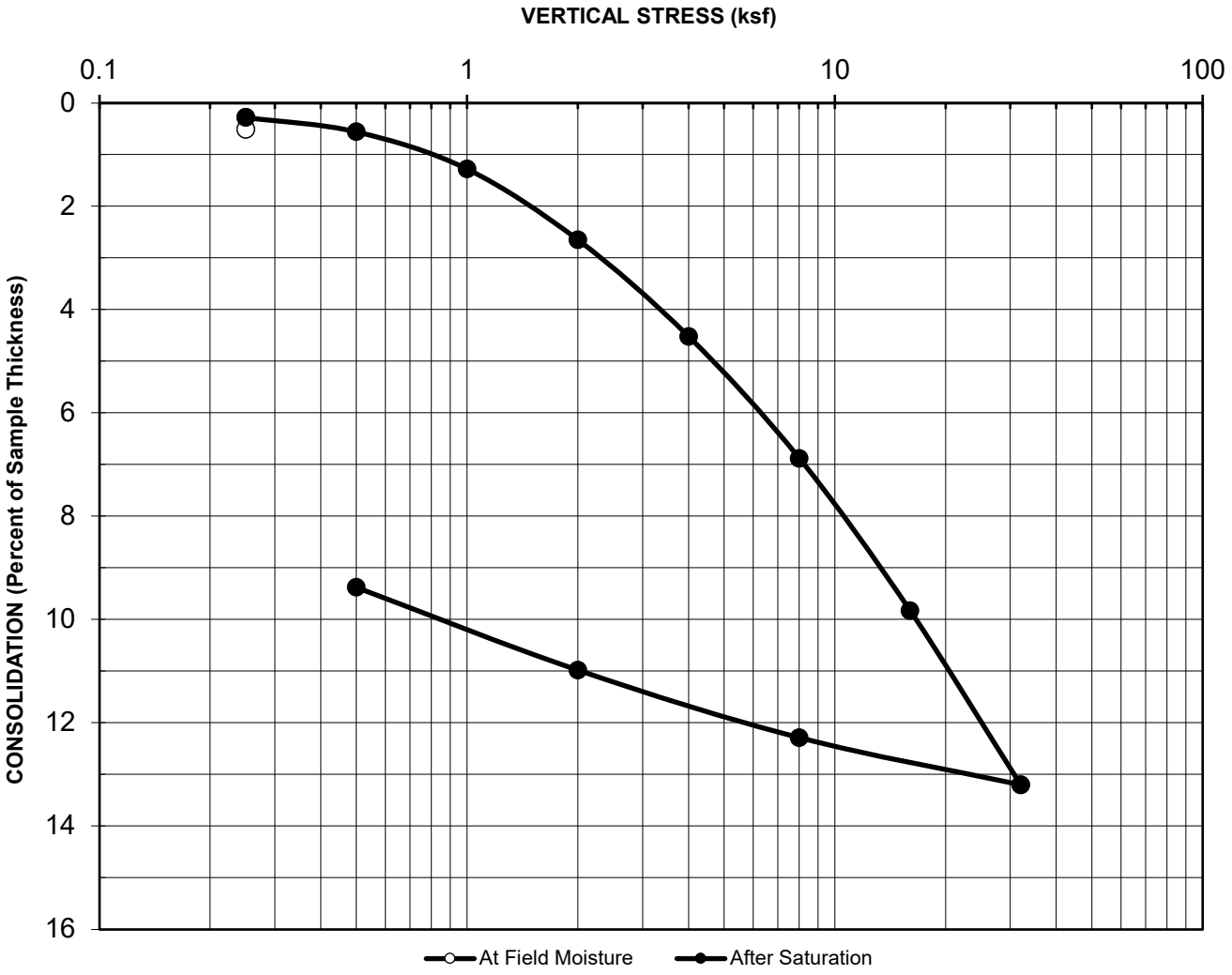


Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	94.9
Sample No.:	18	Initial Moisture Content (%):	29.7
Depth (feet):	80	Final Moisture Content (%):	29.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay	Initial Void Ratio:	0.77
Remarks:	Swell= 0.20% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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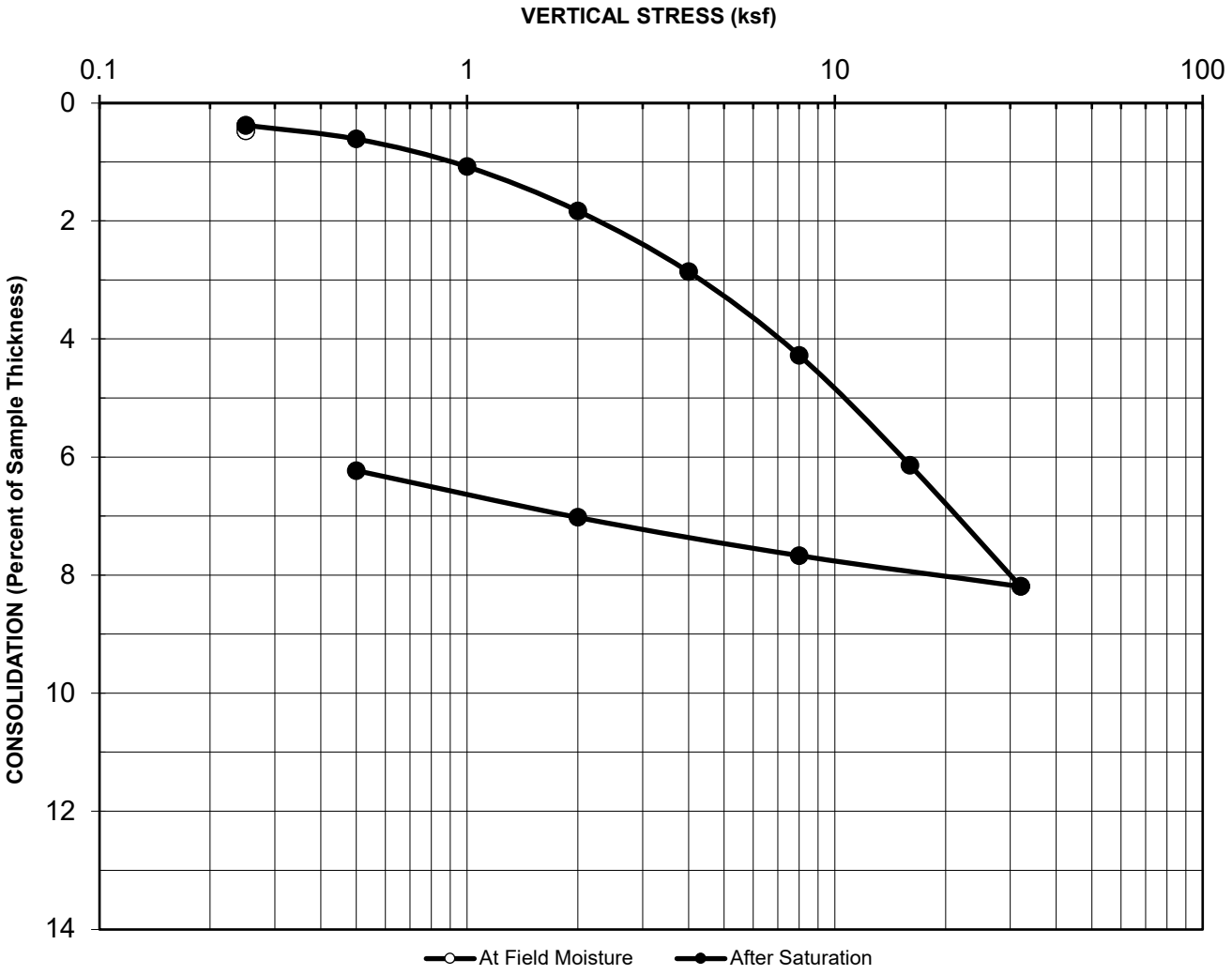


Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	93.0
Sample No.:	6	Initial Moisture Content (%):	28.2
Depth (feet):	20	Final Moisture Content (%):	28.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay	Initial Void Ratio:	0.81
Remarks:	Swell= 0.23% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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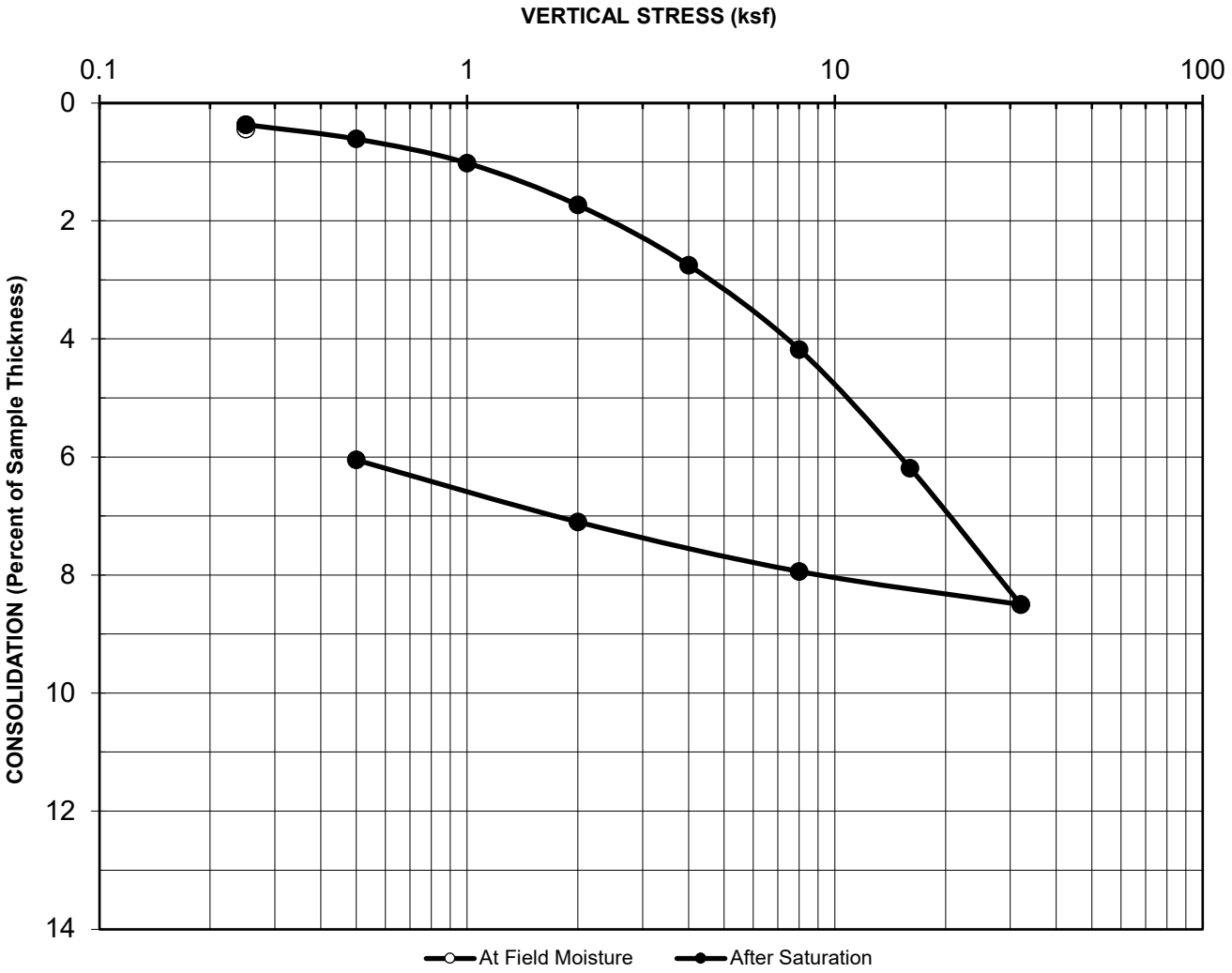


Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	108.5
Sample No.:	14	Initial Moisture Content (%):	20.4
Depth (feet):	60	Final Moisture Content (%):	20.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Lean Clay	Initial Void Ratio:	0.55
Remarks:	Swell= 0.09% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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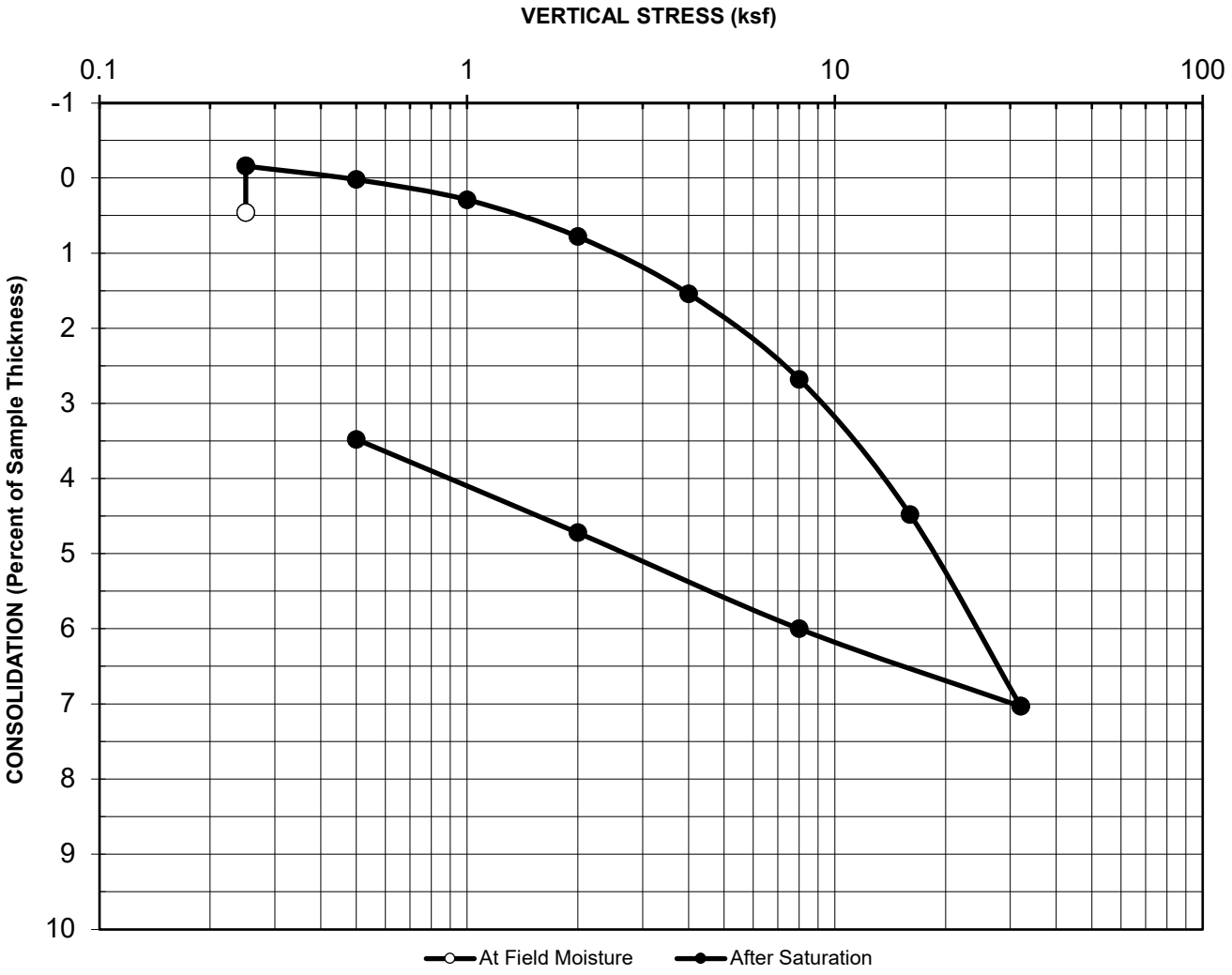


Boring No. :	GP-3	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	4	Initial Moisture Content (%):	16.6
Depth (feet):	15	Final Moisture Content (%):	16.9
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Lean Clay	Initial Void Ratio:	0.53
Remarks:	Swell= 0.07% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: MLK Hospital
Project No.: 20044A
Date: 5/1/2023
AP No: 23-0456 **Sheet No:** 1

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Boring No. :	GP-3	Initial Dry Unit Weight (pcf):	100.7
Sample No.:	14	Initial Moisture Content (%):	24.7
Depth (feet):	65	Final Moisture Content (%):	24.6
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay	Initial Void Ratio:	0.67
Remarks:	Swell= 0.62% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	MLK Hospital
Project No.:	20044A
Date:	5/1/2023
AP No:	23-0456 **Sheet No:** 1

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CORROSION TEST RESULTS

Client Name: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Job No.: 23-0456
Date: 05/10/23

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
GP-1	Bulk-1	0-5	Clayey Sand	1,925	9.0	217	32

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

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CORROSION TEST RESULTS

Client Name: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Job No.: 23-0456
Date: 05/10/23

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
GP-2	Bulk-1	0-5	Clayey Sand	1,825	8.6	240	30

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Job No.: 23-0456
Date: 05/10/23

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
GP-1	Bulk-1	0-5	Clayey Sand	112.6	9.5	51.4	20	21

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: GeoPentech
Project Name: MLK Hospital
Project No.: 20044A

AP Job No.: 23-0456
Date: 05/10/23

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
GP-2	Bulk-1	0-5	Clayey Sand	109.2	9.9	49.3	31	31

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

APPENDIX E

CONE PENETRATION TESTING

E.1 CONE PENETRATION TESTING

The Cone Penetration Testing (CPT) was performed by GeoPentech on April 17th, 2023. The explorations consisted of advancing three (3) CPTs to depths of approximately 105 ft below the ground surface. The approximate locations of the CPTs are indicated on Figure 2 in the main report. The work was subcontracted to Gregg Drilling Inc., who provided all equipment, crew, and supplies. The CPT data recordings and pore pressure dissipation tests are presented in this appendix.

CONE PENETRATION TESTING (CPT) REPORT

Gregg Drilling LLC

Prepared for: GeoPentech
Project D1235019
April 18, 2023

Prepared by: Eleni Pateras
epateras@greggdrilling.com

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Ground Water Sampling 10
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APPENDIX A: Cone Penetration Test Plots

APPENDIX B: Pore Pressure Dissipation Test Plots

April 18, 2023

GeoPentech
Attn: William Erickson

Subject: CPT Site Investigation
MLK Hospital
Willowbrook, CA
GREGG Project Number: D1235019

Dear William:

The following report presents the results of Gregg Drilling's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Groundwater Samples	(GWS)	☐
5	Soil Samples	(SS)	☐
6	Vapor Samples	(VS)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 562-427-6899.

Sincerely,

CPT Reports Team
Gregg Drilling, LLC.

Cone Penetration Testing (CPT) Procedure

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, Figure CPT.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (PPDT). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

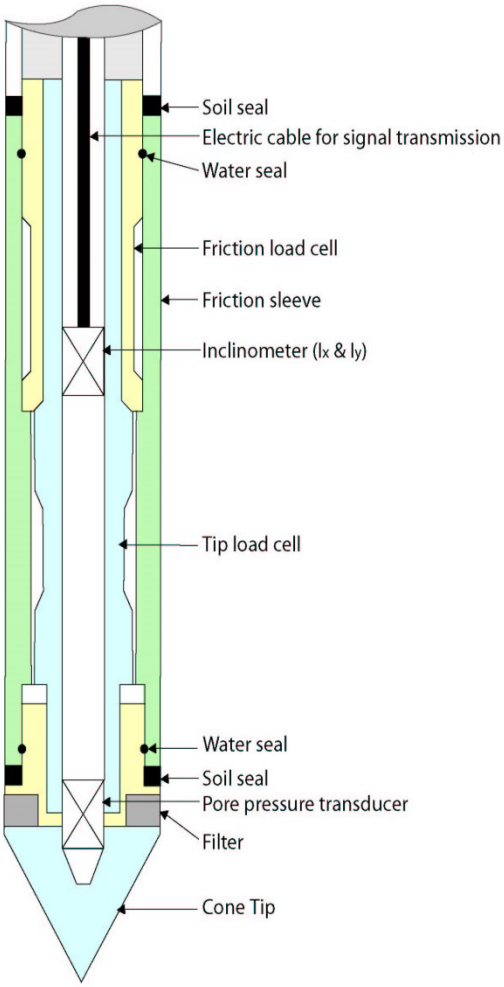


Figure CPT

15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.85
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

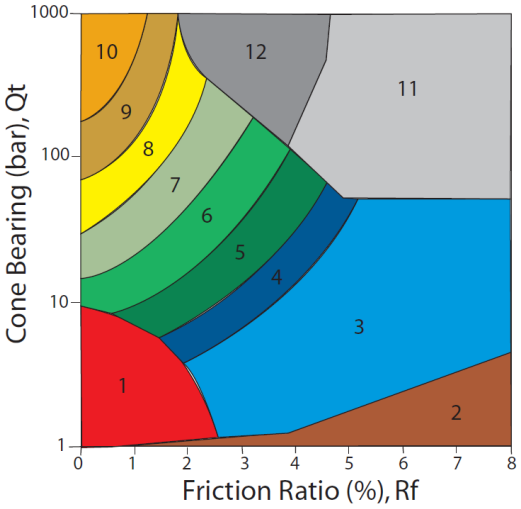
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1990) or normalized data (2010 and 2016). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2010 and 2016) which can be displayed as SBTn (Figure SBTn). The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling LLC does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

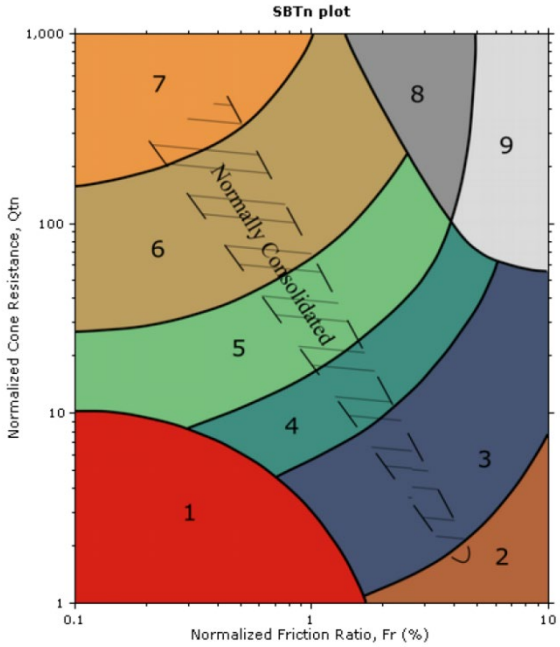
Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

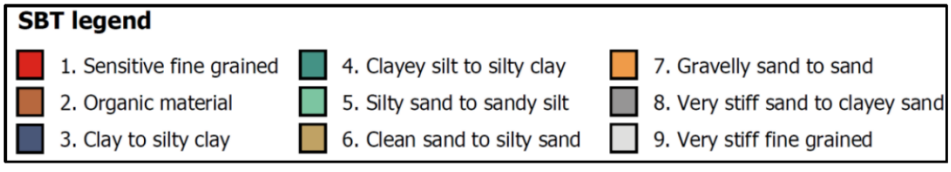


ZONE	Qt/N	SBT
1	2	Sensitive, fine grained
2	1	Organic materials
3	1	Clay
4	1.5	Silty clay to clay
5	2	Clayey silt to silty clay
6	2.5	Sandy silt to clayey silt
7	3	Silty sand to sandy silt
8	4	Sand to silty sand
9	5	Sand
10	6	Gravelly sand to sand
11	1	Very stiff fine grained*
12	2	Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1990) – Note: Colors may vary slightly compared to plots





Pore Pressure Dissipation Tests (PPDTs)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u)* with time is measured behind the tip of the cone and recorded.

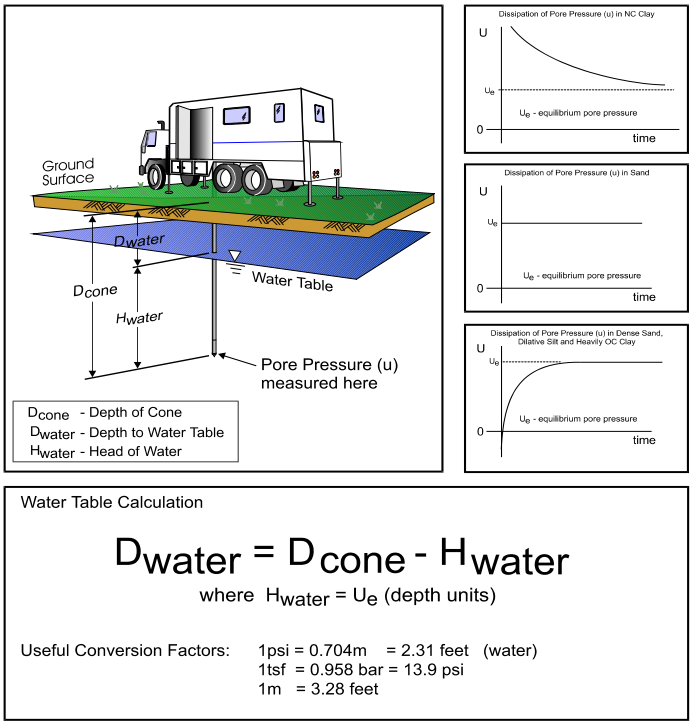
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

To correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests completed for this project is included in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Tests (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg Drilling's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: v = Δd/Δt

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary of the shear wave velocities, arrival times and wave traces are provided with the report.

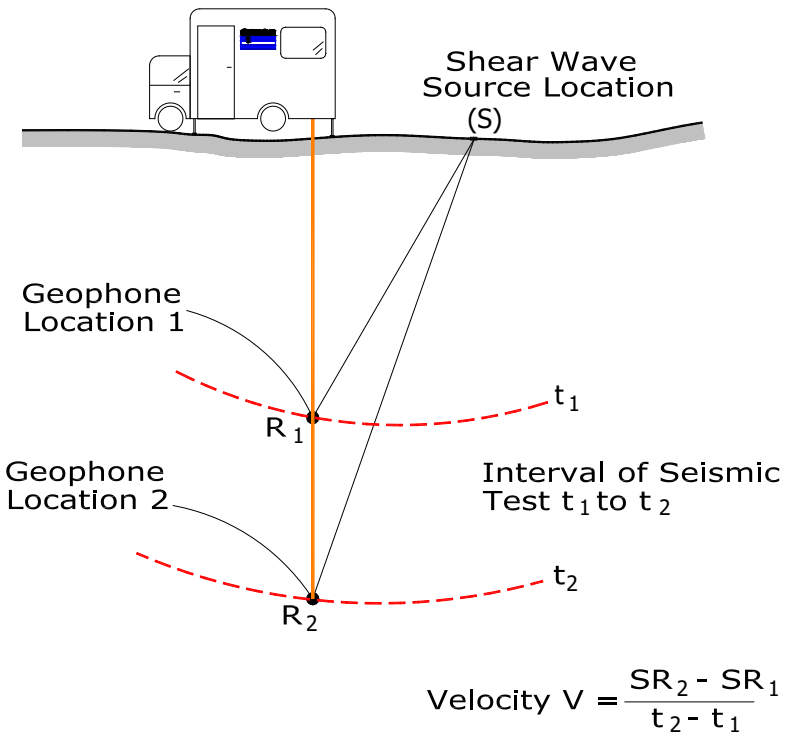


Figure SCPT

Soil Sampling

Gregg Drilling uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

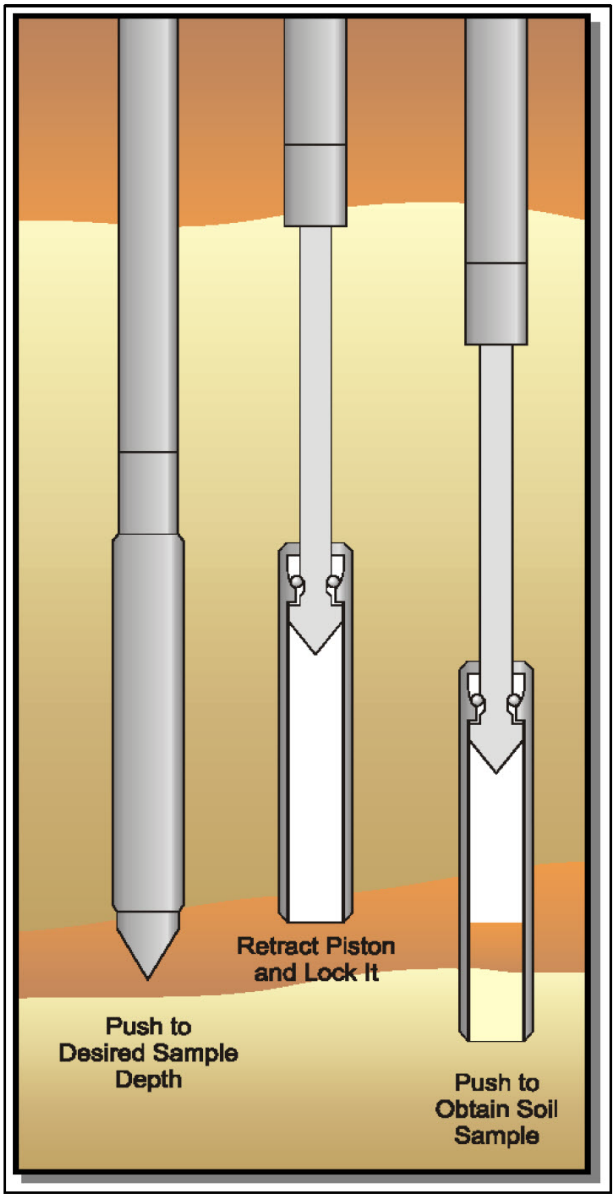


Figure SS

Ground Water Sampling

Gregg Drilling conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

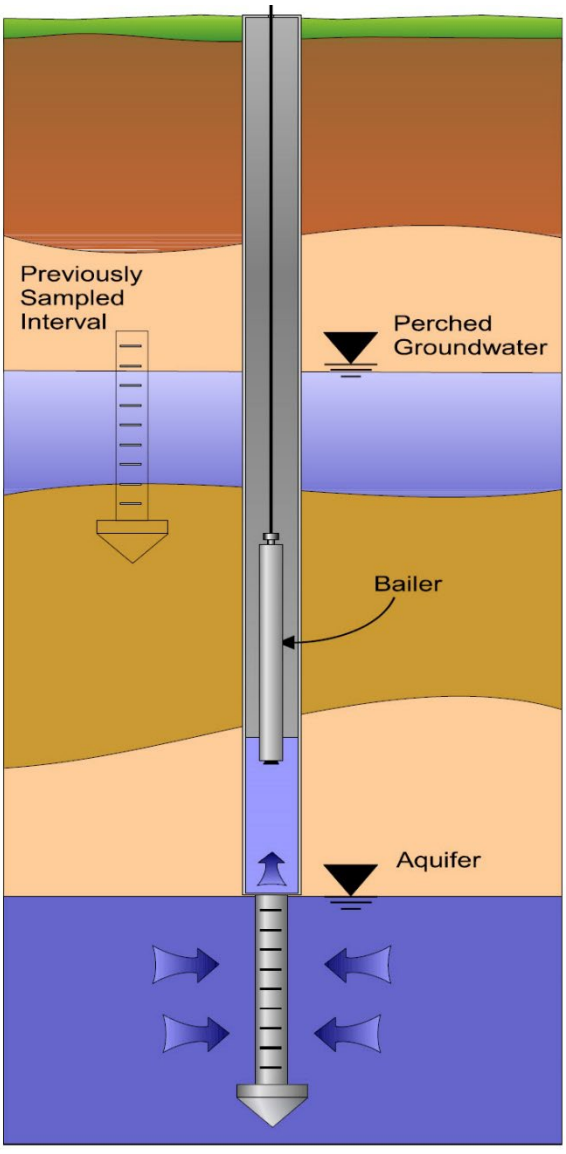


Figure GWS

References

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

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Robertson, P.K., 2010, "Soil Behavior type from the CPT: an update", 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, Vol.2. pp 575-583

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Robertson, P.K. and Cabal, K.L., "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition, 2015, 145 p. Free online, http://www.greggdrilling.com/technical-guides.

Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, "Geo-Environmental Application of Penetration Testing", Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Copies of ASTM Standards are available through www.astm.org

Table 1: Cone Penetration Testing Summary

CPT Sounding Identification	Date	Termination Depth (ft)	Depth of Soil Samples (ft)	Depth of Groundwater Samples (ft)	Depth of Pore Pressure Dissipation Tests (ft)
CPT-01	4/17/2023	108.1	-	-	76.28, 107.12
CPT-02	4/17/2023	105.64	-	-	63.81
CPT-03	4/17/2023	104.99	-	-	53.97, 64.30, 104.00

APPENDIX A:

CPT PLOTS

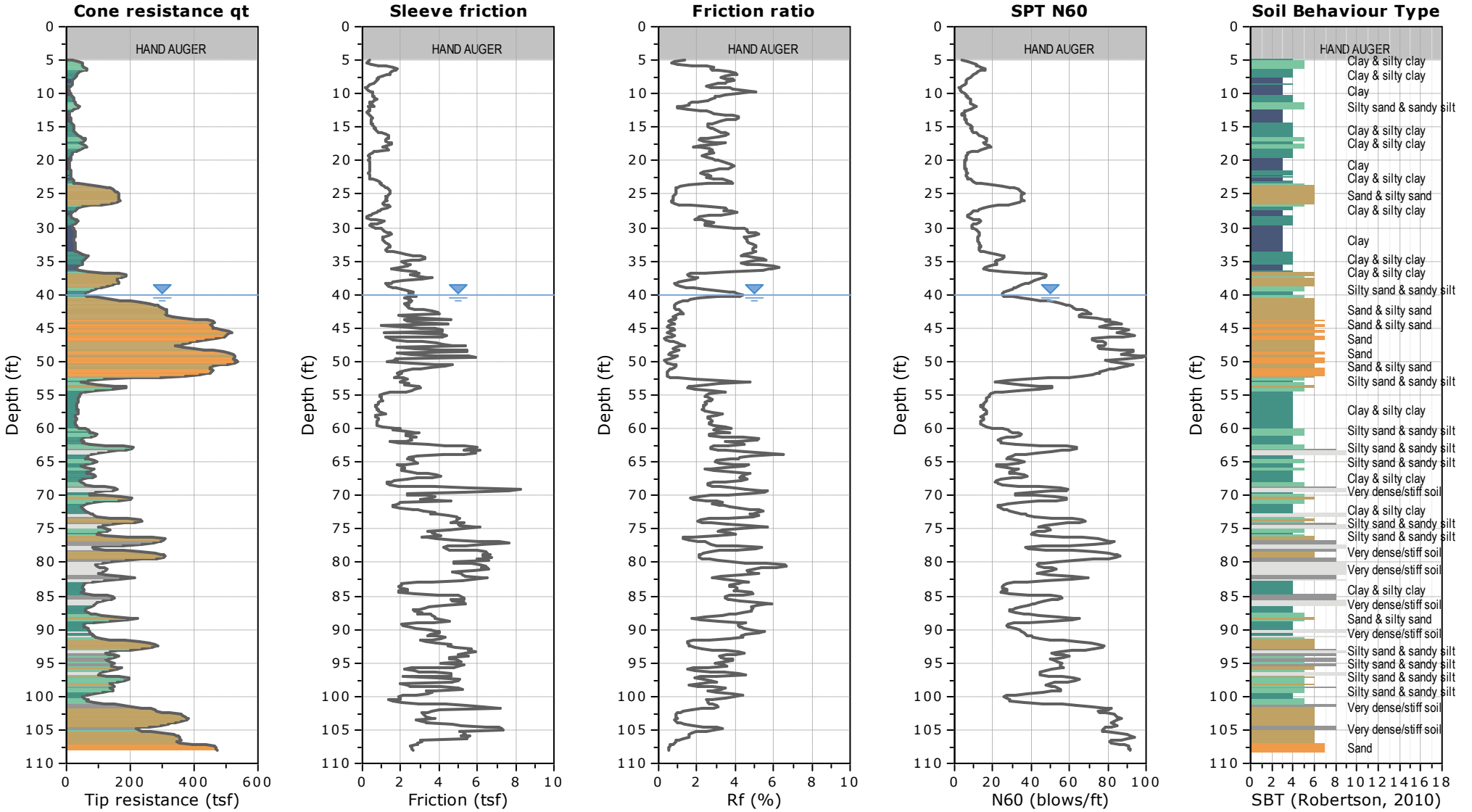
GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-01

CLIENT: GEOPENTECH
SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 108.10 ft, Date: 04/17/2023



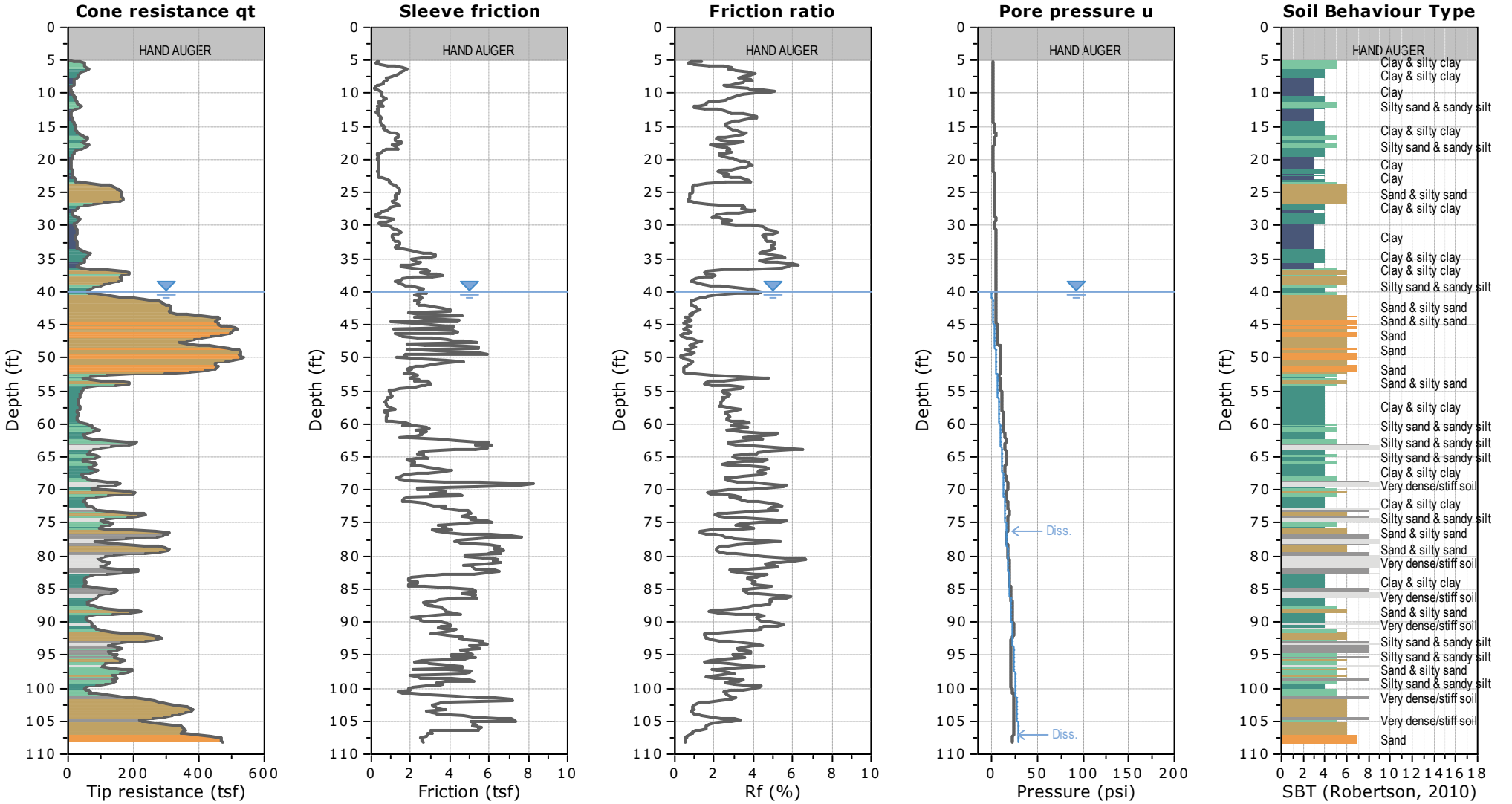
CPT: CPT-01

CLIENT: GEOPENTECH

SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 108.10 ft, Date: 04/17/2023



WATER TABLE FOR ESTIMATING PURPOSES ONLY

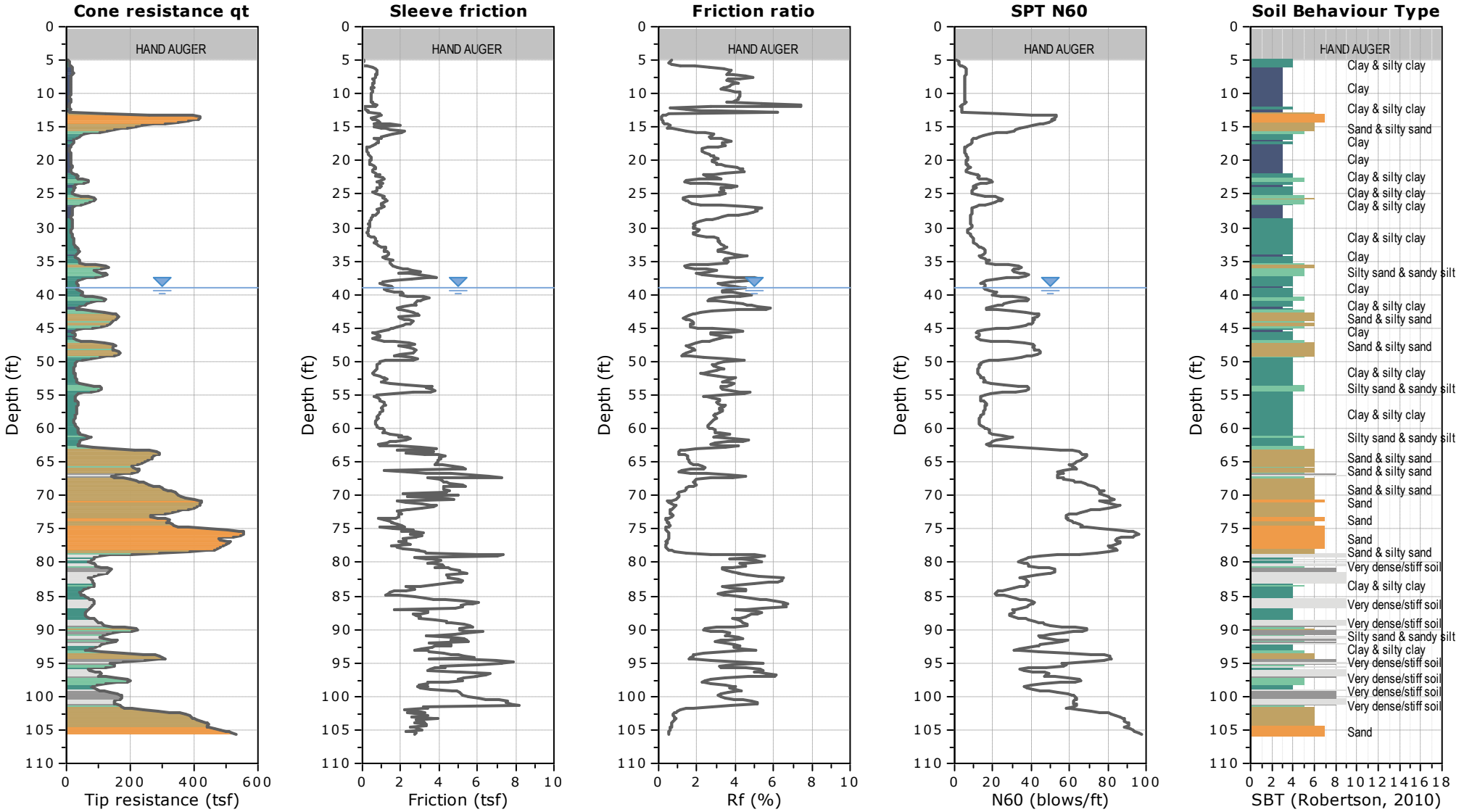
CPT: CPT-02

CLIENT: GEOPENTECH

SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 105.64 ft, Date: 04/17/2023



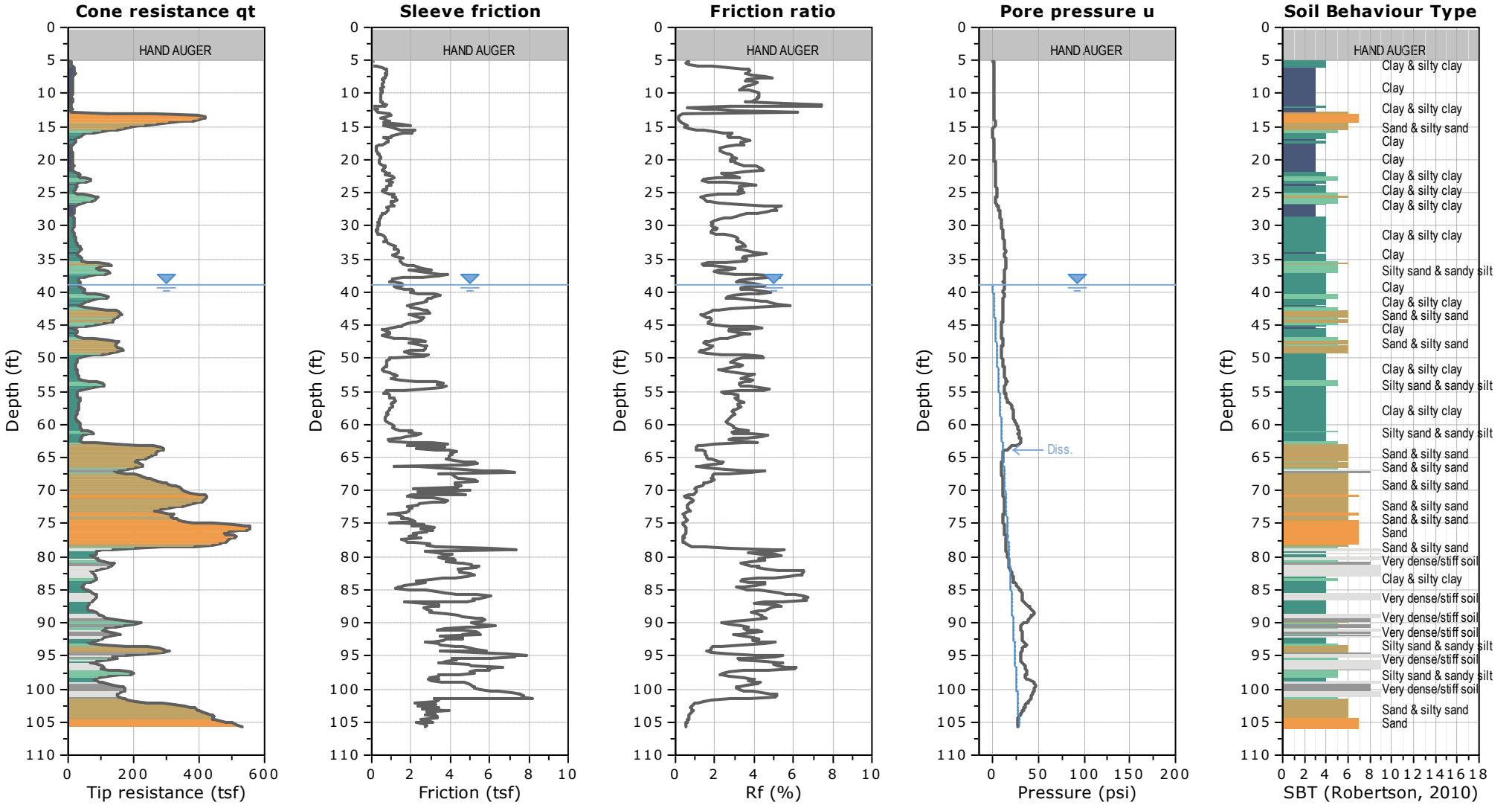
CPT: CPT-02

CLIENT: GEOPENTECH

SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 105.64 ft, Date: 04/17/2023



WATER TABLE FOR ESTIMATING PURPOSES ONLY

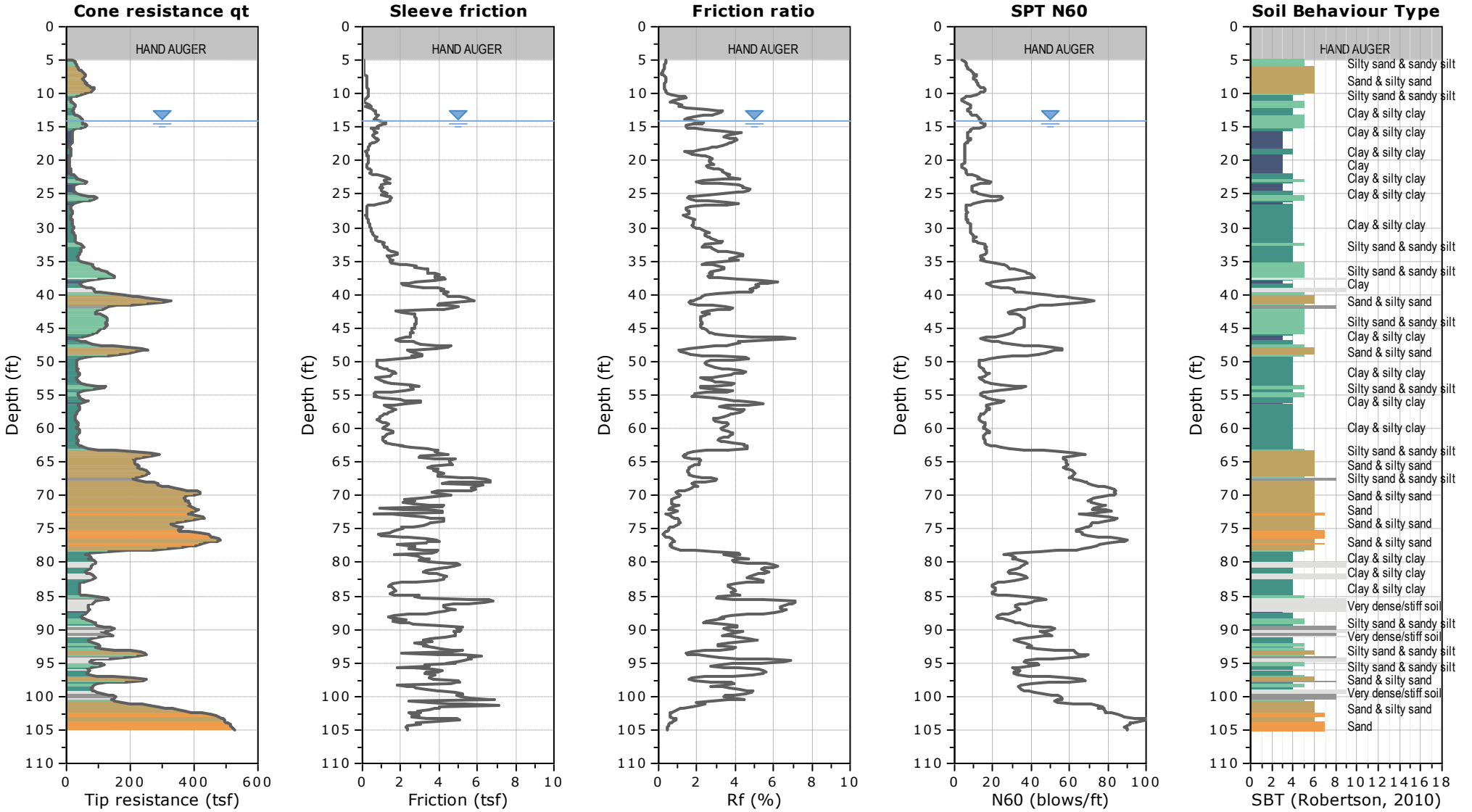
CPT: CPT-03

CLIENT: GEOPENTECH

SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 104.99 ft, Date: 04/17/2023



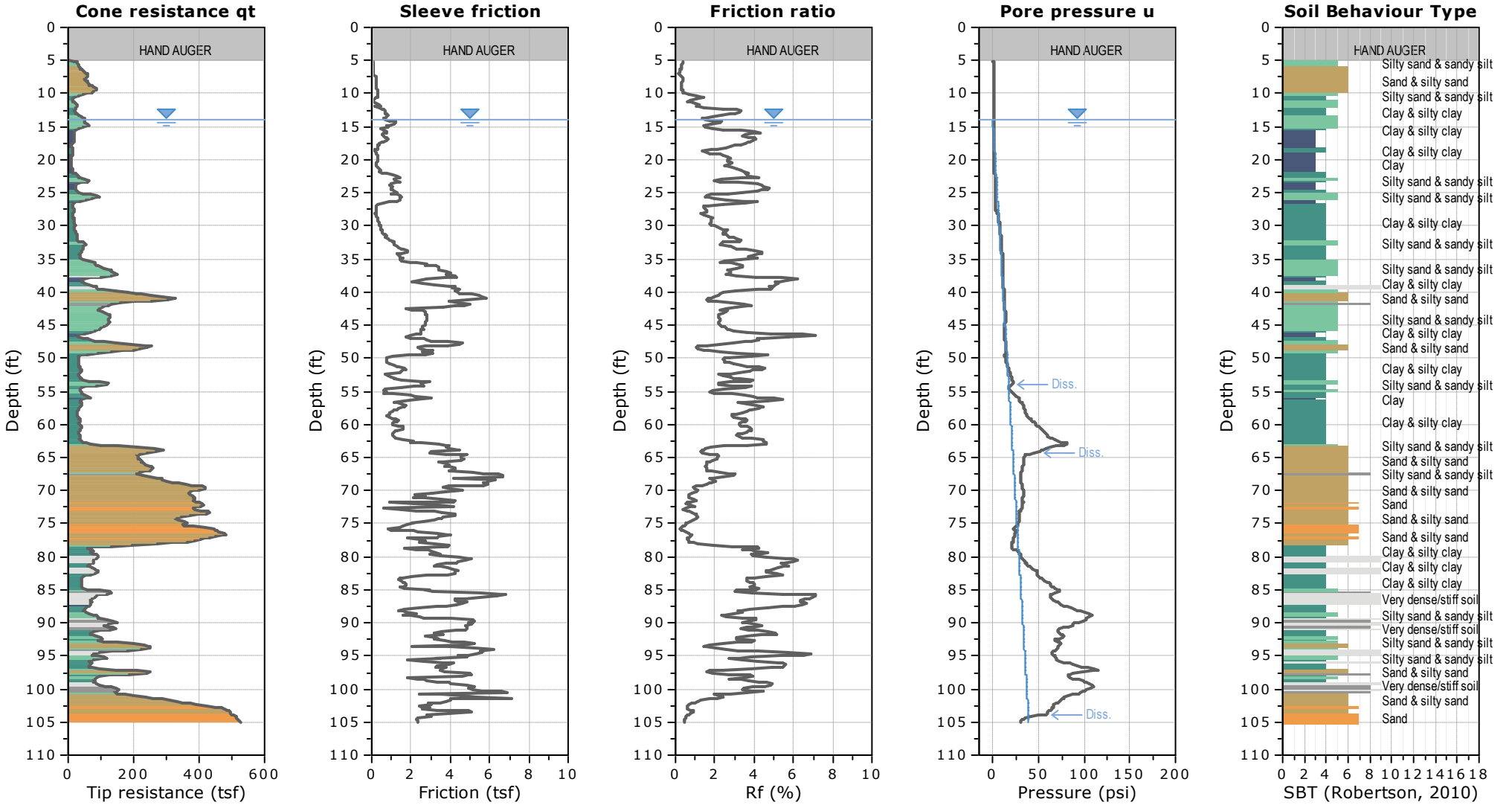
CPT: CPT-03

CLIENT: GEOPENTECH

SITE: MLK HOSPITAL, WILLOWBROOK, CA

FIELD REP: WILLIAM E.
Cone ID: GDC-92

Total depth: 104.99 ft, Date: 04/17/2023



WATER TABLE FOR ESTIMATING PURPOSES ONLY

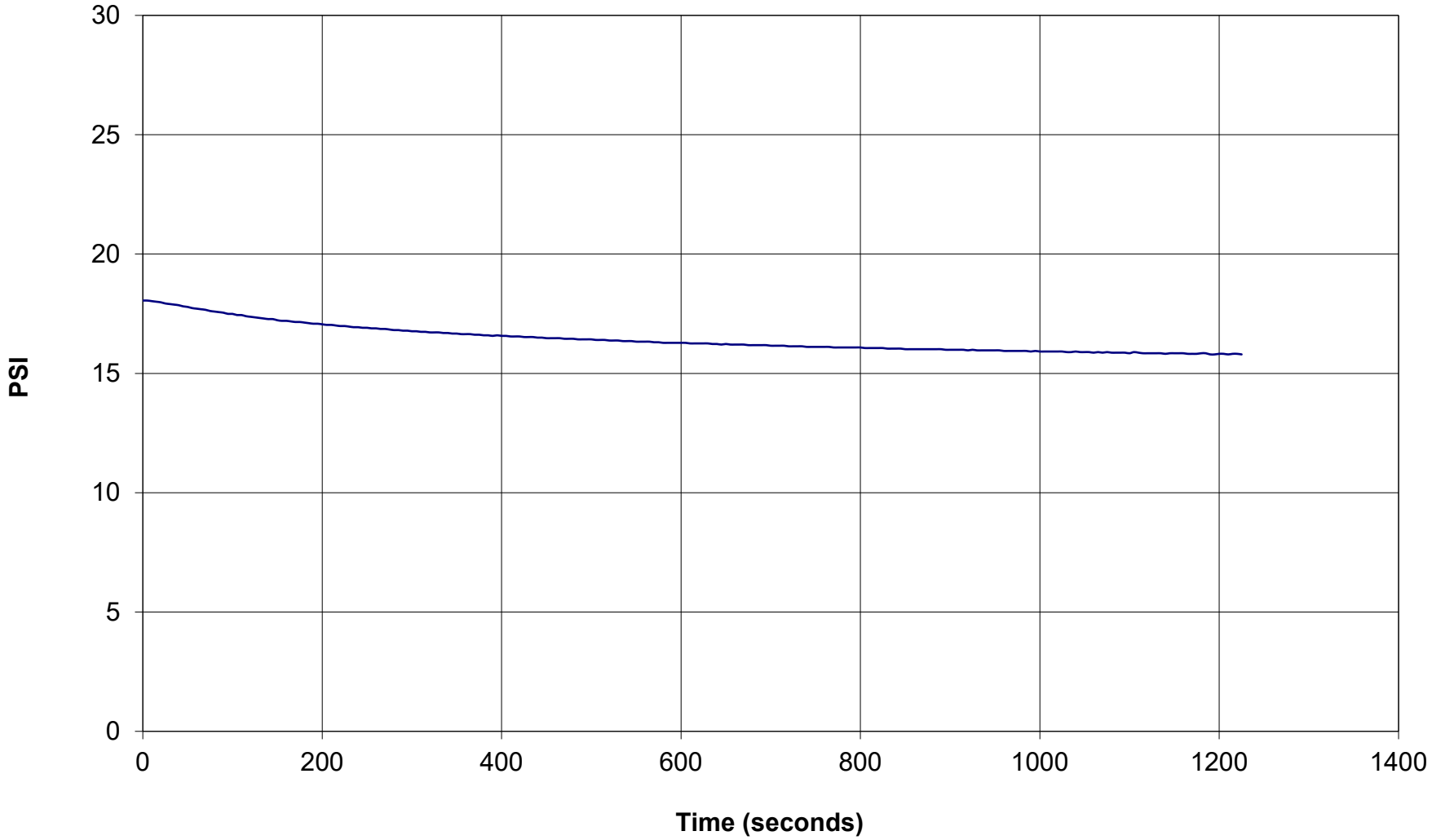
APPENDIX B:

PORE PRESSURE DISSIPATION TEST PLOTS

GREGG DRILLING, LLC

Pore Pressure Dissipation Test

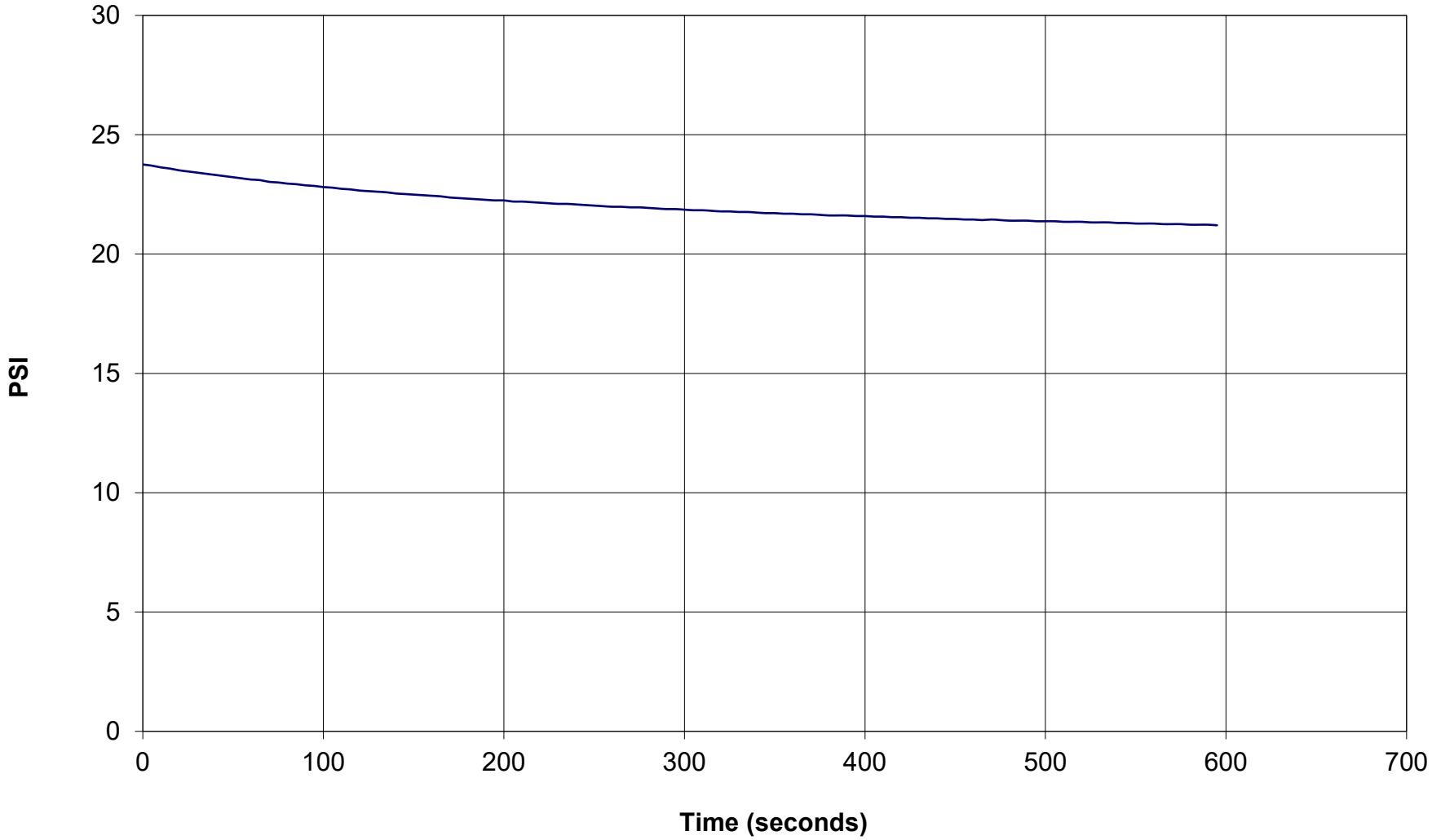
Sounding: CPT-01
Depth (ft): 76.28
Site: MLK HOSPITAL
Engineer: WILLIAM E.



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

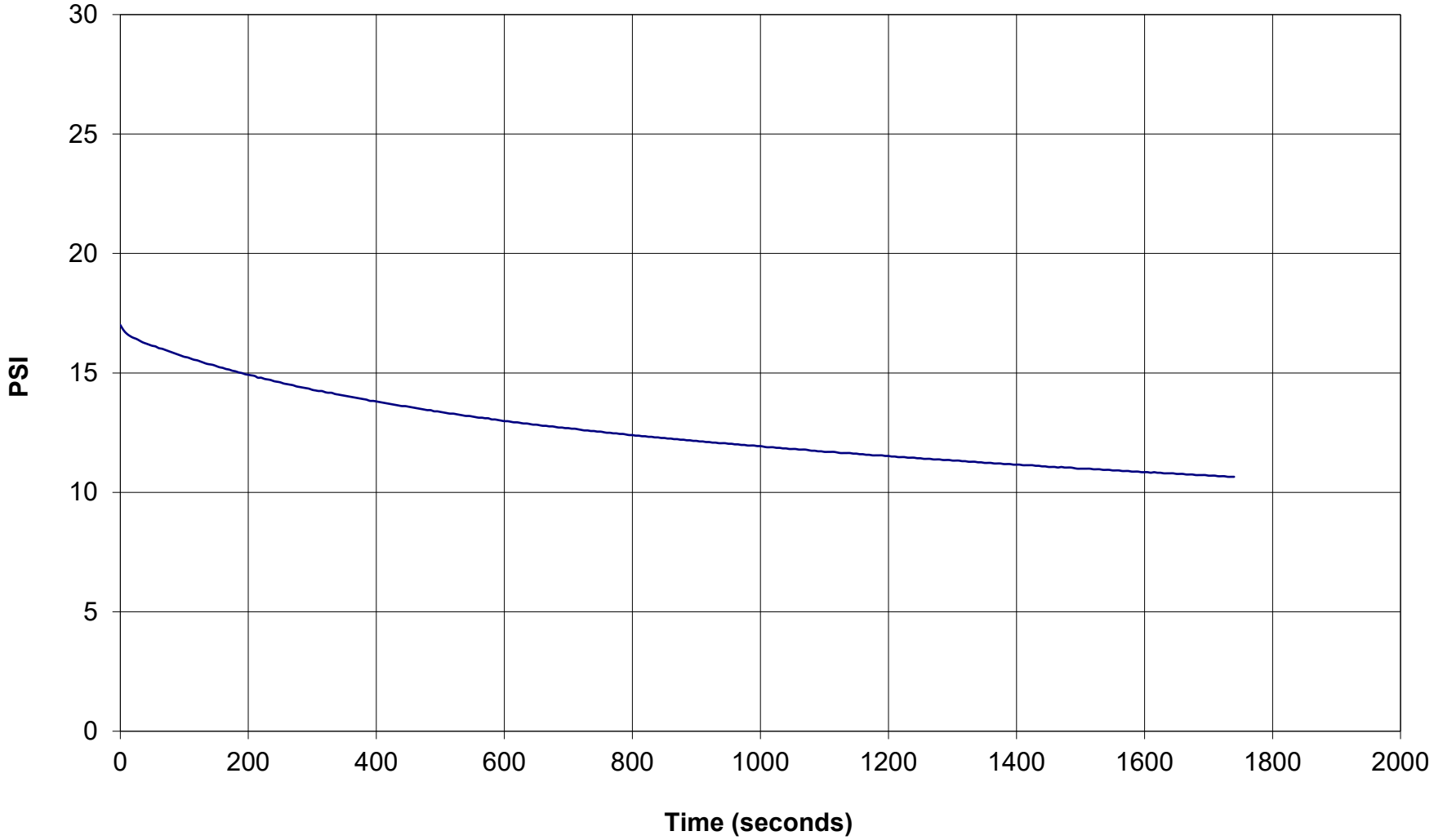
Sounding: CPT-01
Depth (ft): 107.12
Site: MLK HOSPITAL
Engineer: WILLIAM E.



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

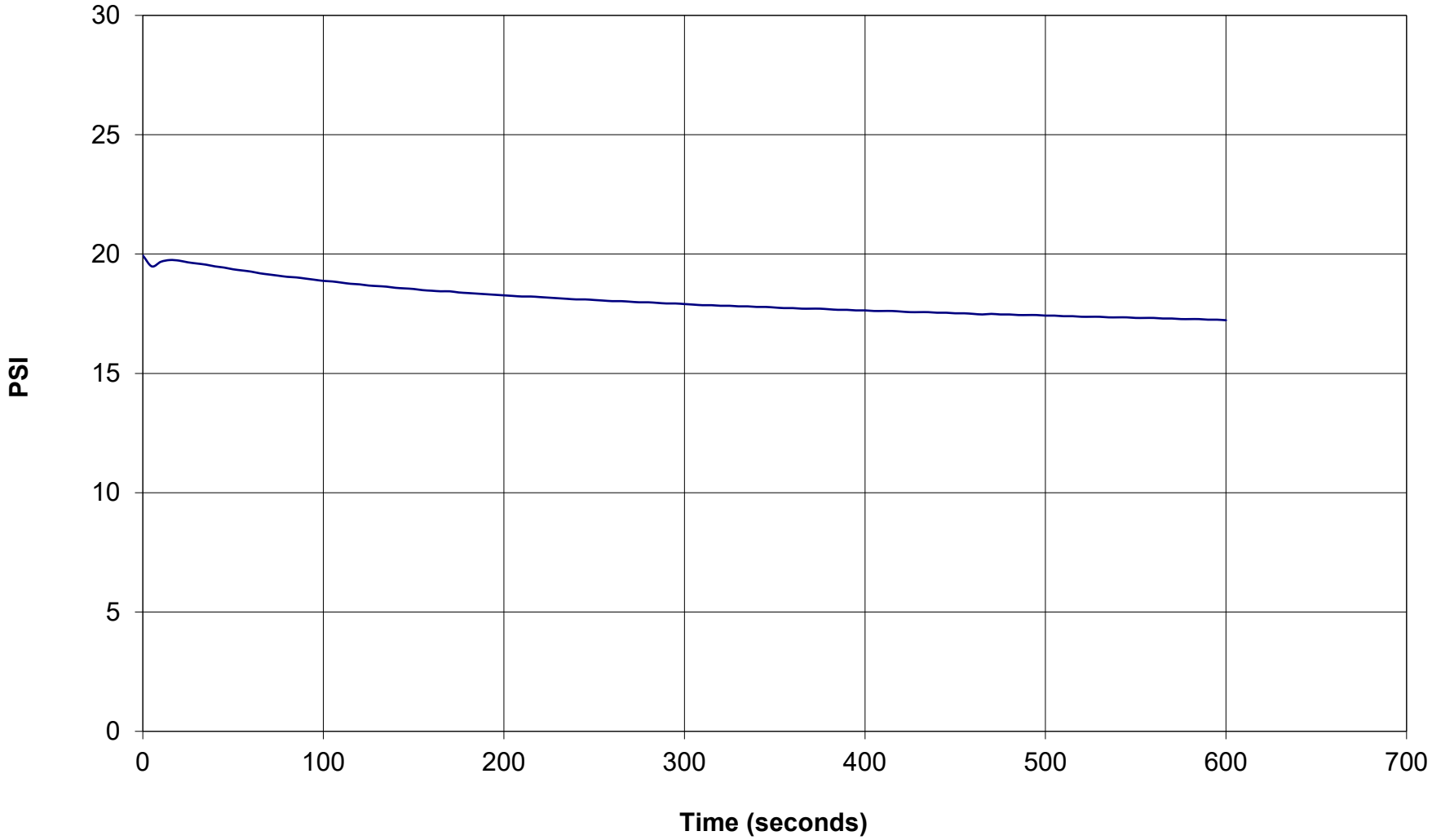
Sounding: CPT-02
Depth (ft): 63.81
Site: MLK HOSPITAL
Engineer: WILLIAM E.



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

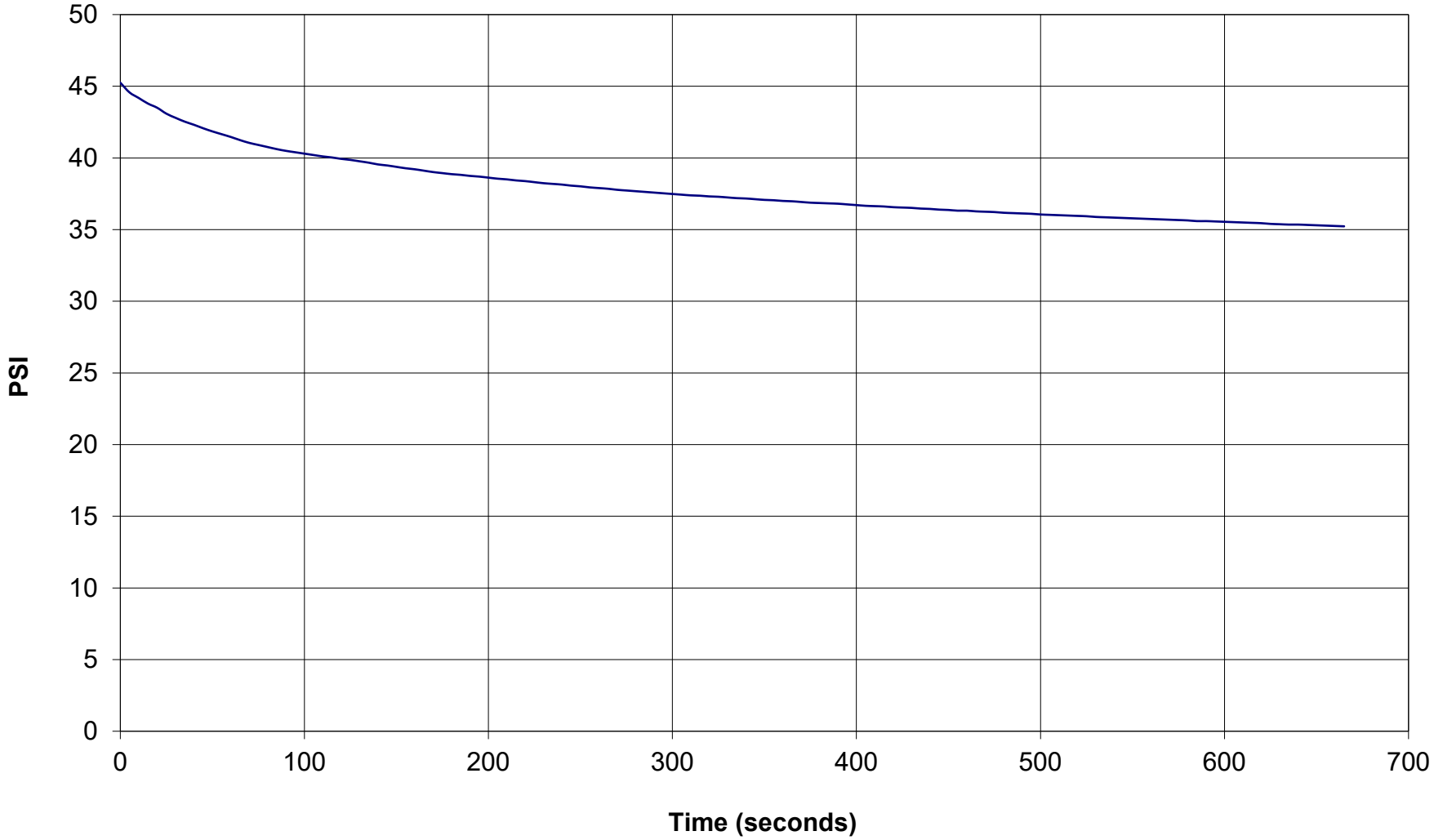
Sounding: CPT-03
Depth (ft): 53.97
Site: MLK HOSPITAL
Engineer: WILLIAM E.



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

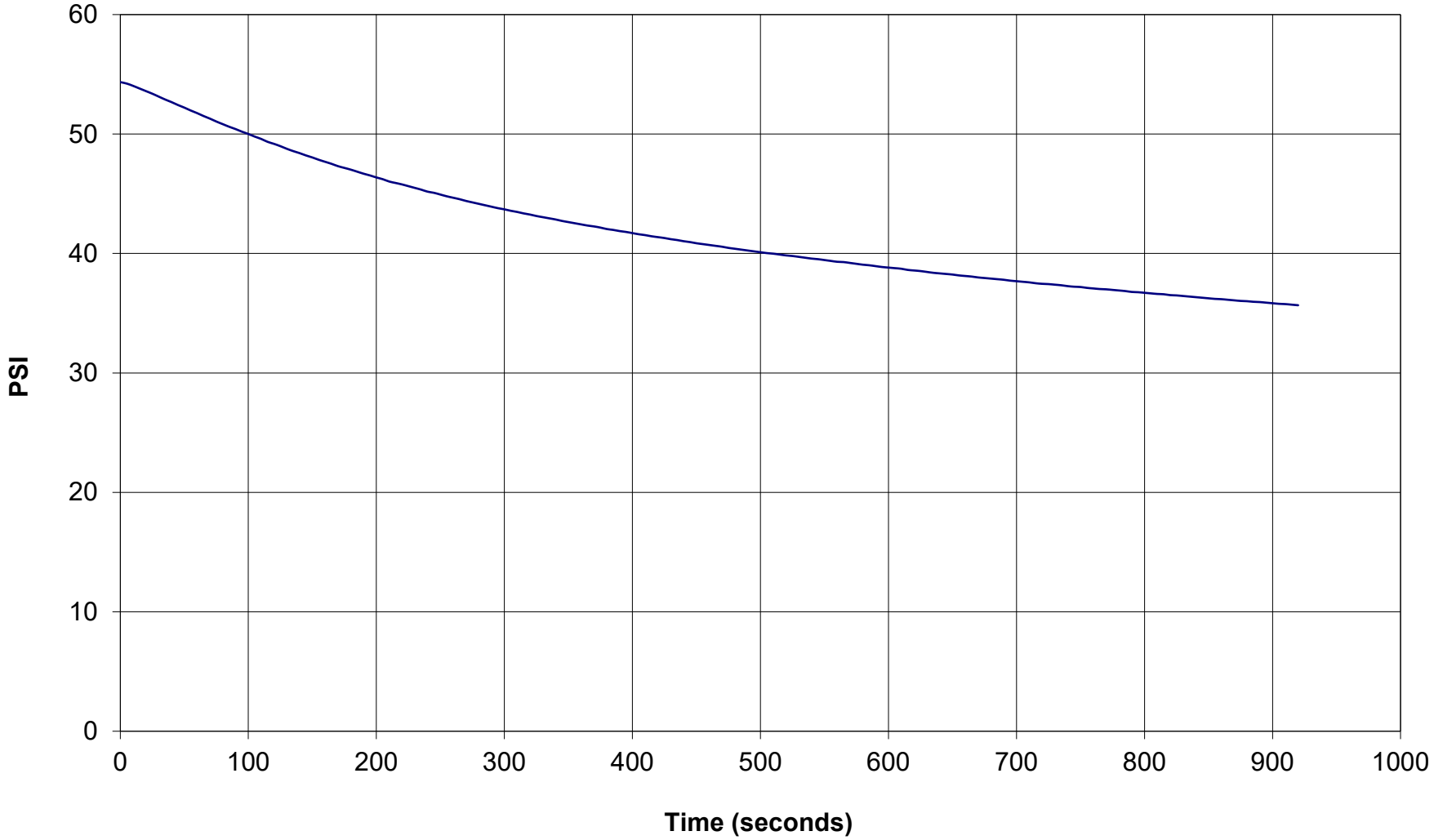
Sounding: CPT-03
Depth (ft): 64.30
Site: MLK HOSPITAL
Engineer: WILLIAM E.



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

Sounding: CPT-03
Depth (ft): 104.00
Site: MLK HOSPITAL
Engineer: WILLIAM E.



APPENDIX F

GEOPHYSICAL SURVEYS

F.1 INTRODUCTION

This appendix presents the results of the surface wave geophysical investigation performed in support of soil site class characterization and geotechnical investigation of the current Hawkins Mental Health Center at the MLK Community Hospital campus in Los Angeles, California. The purpose of the geophysical investigation was to evaluate foundation properties (i.e. VS30) at the site.

GeoPentech performed surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and microtremor array methods from three survey lines (SW23-1 through SW23-3) to measure seismic shear-wave (S-wave) velocities at a range of depths. The locations of these lines are shown on Figure F-1. Additionally, seismic refraction data were collected from line SW23-2 to measure compression wave (P-wave) velocities to provide constraint for the S-wave modeling. The geophysical data were collected and processed by a California-licensed Professional Engineer under the supervision of a California-licensed Professional Geophysicist.

F.2 SURFACE WAVE GEOPHYSICAL METHODS

Both active and passive surface wave surveys were performed at the site. The active surface wave surveys were performed using MASW methods, and the passive surveys were performed using microtremor array methods. A detailed description of MASW is provided in Park et al. (1999) and Foti et al. (2018). Microtremor array methods are described in Foti et al. (2018).

In general, the surface wave method records Rayleigh waves generated either with (1) an active source (e.g. sledgehammer) for the MASW method or (2) a passive (ambient) source (e.g. vehicular traffic) for microtremor array methods. In a layered medium, Rayleigh surface waves of different frequencies (or wavelengths) propagate at different velocities, referred to as phase velocity. This phase velocity primarily depends on the material stiffness properties (e.g. S-wave velocity) over a depth approximately equal to one wavelength. Consequently, lower frequency, longer wavelength surface wave energy will provide samples to greater survey depths than higher frequency, shorter wavelength energy. Because surface waves of different frequencies (wavelengths) sample different depths, they travel at different velocities (dispersion) in a layered medium. Surface wave geophysical surveys measure the dispersive nature of the geologic medium and produce dispersion curves, which show the variation of Rayleigh wave phase velocity as a function of frequency (or wavelength). Due to the generally lower frequency nature of passive surface wave energy, passive surface wave techniques (i.e. microtremor array) have the potential to supplement active surface wave data to achieve deeper investigation depths. For this reason, it is advantageous to perform both types of measurement along the same lines as was done for this project.

After the dispersion curve is generated, the dispersion curve picks are then iteratively fitted to a horizontally layered, laterally continuous, homogeneous-isotropic, S-wave velocity model that would account for the measured surface wave velocity dispersion. The results provide a representative average estimate of the one-dimensional S-wave velocity profile under the array (velocity vs. depth).

F.3 SEISMIC REFRACTION GEOPHYSICAL METHODS

Seismic refraction surveying involves measuring the time required for a seismic compression wave (P-wave) to travel from a shot point to a series of co-linear geophones. To resolve a subsurface layer boundary using the seismic refraction method, a density or velocity change must occur at that layer boundary. P-wave arrival times are plotted on time-distance graphs and interpreted using the generalized reciprocal method or by seismic tomography. The results provide a two-dimensional picture of subsurface conditions and show the interpreted distribution of P-wave velocity as a function of depth along the profile.

The interpretation of seismic refraction data depends on several assumptions including:

- The subsurface is composed of a series of discrete homogeneous layers, which may vary laterally in seismic velocity;
- The boundaries between these layers consist of a significant seismic velocity contrast and are laterally continuous to be individually resolved;
- The thicknesses of these layers are great enough to critically refract seismic source energy to produce a detectable refracted wave arrival at the surface;
- The seismic velocity of successive layers increases with depth.

All the assumptions above are unlikely to be completely fulfilled, and the extent to which each assumption is valid varies from site to site. Consequently, the interpreted seismic refraction profiles typically show a simplified bulk average subsurface P-wave velocity distribution. The degree of simplification depends on the extent the above assumptions are violated at the site.

F.4 GEOPHYSICAL DATA COLLECTION AND PROCESSING PROCEDURES

The surface wave and seismic refraction data were collected at the site on April 17, 2023. As shown on Figure F-1, the survey lines were performed on the sidewalks to the west, north, and east of the subject building. The survey measurements were collected using a Geometrics Geode seismograph with a linear array of 24 4.5-Hz geophones. The geophones were spaced at 10-foot intervals (230-foot line length) for lines SW23-1 and SW23-2 and at 8-foot intervals (184-foot line length) for line SW23-3.

The active seismic source for the MASW and seismic refraction measurements consisted of a sledgehammer blow to a ground plate. Six shot locations were used beginning 10 to 50 feet behind the first geophone and moving toward the survey line in increments equal to the geophone interval (10 feet for SW23-1 and SW23-2, 8 feet for SW23-3). Additional shots were performed within the line and at the opposite side of the line at SW23-2 to collect seismic refraction data. Shot locations for each line are shown below in Table F-1. At each shot location the sledgehammer was hit once for

seismic refraction data collection and then two more times to create a stacked waveform for both seismic refraction and surface wave data collection. A 1.2-second-long record with 0.5 millisecond sample interval was recorded at each shot location.

TABLE F-1
SURVEY LINE DIMENSIONS

Survey Line	Line Purpose	Geophone Interval / Total Line Length	Shot Locations (ft from first geophone)*
SW23-1	Surface Wave	10 ft / 230 ft	-50, -40, -30, -20, -10, 0
SW23-2	Surface Wave / Seismic Refraction	10 ft / 230 ft	-10, 0, 10, 20, 30, 40, 55, 115, 175, 230
SW23-3	Surface Wave	8 ft / 184 ft	-48, -40, -32, -24, -16, -8

* Geophones at and behind the shot location were removed during surface wave data processing.

The recorded MASW data were processed using the program SurfSeis by Kansas Geological Survey. This program uses a modified F-K filter (type of 2-dimensional Fourier transform) to convert the raw seismic data from time and distance to wave frequency and velocity. The highest amplitude energies along the frequency and phase velocity plot for each shot location were then selected to create a dispersion curve.

Because of the typical lower frequency nature of passive surface wave energy, passive microtremor measurements were performed to supplement the MASW measurements at deeper investigation depths. A total of ten 32-second-long ReMi records (2 millisecond sample interval) were collected from each survey line. The primary source of ambient surface wave energy was vehicular traffic near the site. The recorded microtremor data were also processed using the Kansas Geological Survey's SurfSeis program. Because microtremor records are generally created using lower-energy vibration sources, the microtremor records were stacked together to create one record with sufficient energy. Prior to stacking, the microtremor records were analyzed individually to ensure sufficient energy was captured in the record, and poor-quality records were removed prior to stacking. Wavefield transformation was then performed on the stacked microtremor records in a similar manner to the MASW processing to create a frequency/phase velocity plot. An overall microtremor dispersion curve was then created from this plot.

For each line, the microtremor dispersion curve picks were combined with the dispersion curve picks generated from MASW to create an overall seismic dispersion curve. The degree of fit of the overlapping microtremor and MASW dispersion picks provided confidence in the results. Additionally, as noted above, the microtremor and MASW data complement each other by generally sampling different frequency ranges of surface wave data. After the data were combined, a best fit polynomial dispersion curve was calculated for modeling. The best fit dispersion curve was then iteratively fitted to a one-dimensional S-wave velocity model using the SurfSeis software. The results of the modeling

provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the extent of the survey line.

Seismic refraction travel times were plotted on time-distance graphs and interpreted using seismic tomography methods with OYO Corporation's SeisImager/2D software. The profile results were then imported into Golden Software's Grapher program to extract a one-dimensional profile representing the average P-wave velocity from ground surface to the bottom of the seismic refraction investigation depth (approximately 80 feet). The two-dimensional and average one-dimensional P-wave profiles from line SW23-2 are shown in Figure F-2. The average vertical P-wave velocity profile was input into SurfSeis prior to inversion of the S-wave velocity models to constrain the initial S-wave model. Borings at the site indicated a depth to water of approximately 58 feet. Consequently, for depths below the extent of P-wave measurement, a Poisson ratio of 0.48 was assumed, which is consistent with saturated sediment.

F.5 SURFACE WAVE GEOPHYSICAL RESULTS

The results of the combined MASW and microtremor array surface wave measurements are presented in Figures A-3 through A-5 for lines SW23-1 through SW23-3, respectively. These figures present the MASW, microtremor array, and best fit surface wave dispersion curves and the corresponding representative S-wave velocity models. As seen in these figures, the MASW and microtremor array dispersion curves are generally in good agreement in the regions that overlap.

Figure F-6 summarizes the surface wave measurement results for the site. This figure shows (1) the S-wave velocity models for lines SW23-1 through SW23-3 plotted as a function of depth below ground surface and (2) the site average S-wave velocity for all the measurements calculated at 1-foot increments.

Based on the results shown on Figure F-6, the VS30 was calculated based on the procedures outlined in the National Earthquake Hazards Reduction Program (NEHRP) and UBC. The VS30 was calculated from the following equation from these references:

$$v_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n
v_{si} = shear wave velocity in feet per second of layer i
d_i = thickness of any layer within the 100-foot interval
$\sum_{i=1}^{n} d_i$ = 100 feet

Based on this procedure, the site average VS30 was calculated from ground surface to 100 feet below ground surface. The VS30 below ground surface was calculated as 936 ft/s (285 m/s), which corresponds with ASCE 7-16 Site Class D, stiff soil (600 < VS30 ≤ 1,200 ft/s), and ASCE 7-22 Site Class

D, medium dense sand or stiff clay (700 < VS30 ≤ 1,000 ft/s). VS30 values for depth intervals beginning below ground surface are also shown on Figure F-6 and in Table F-2 below.

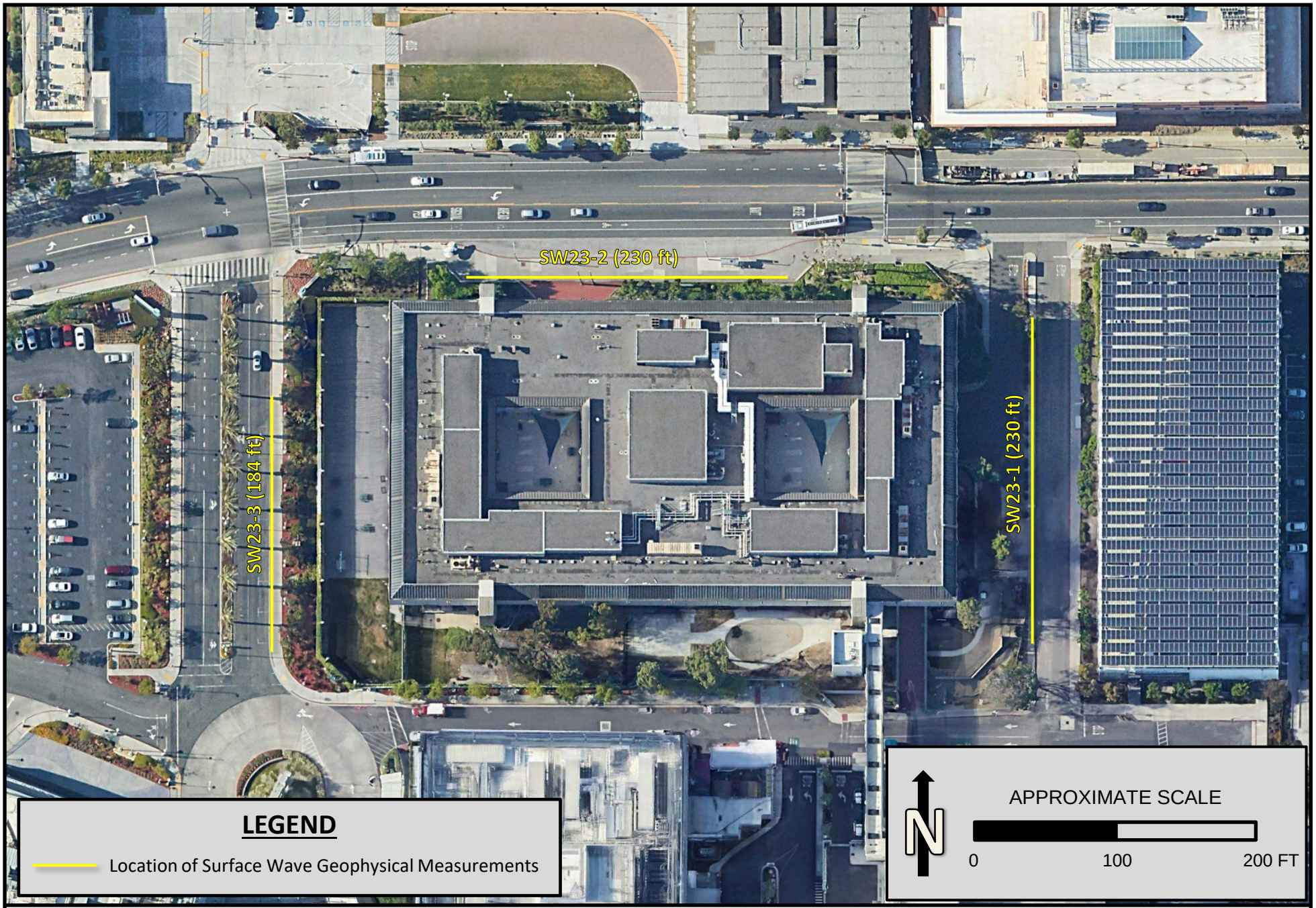
TABLE F-2
CALCULATED SITE V_{S30}

Depth Range (ft, below planned ground surface)	V_{s30} (ft/sec)	Seismic Site Class ASCE 7-16 (2017)	Seismic Site Class ASCE 7-22 (2022)
0 to 100	936	D	D
10 to 110	1,029	D	CD
20 to 120	1,123	D	CD
30 to 130	1,213	C	CD
40 to 140	1,288	C	CD

F.6 REFERENCES

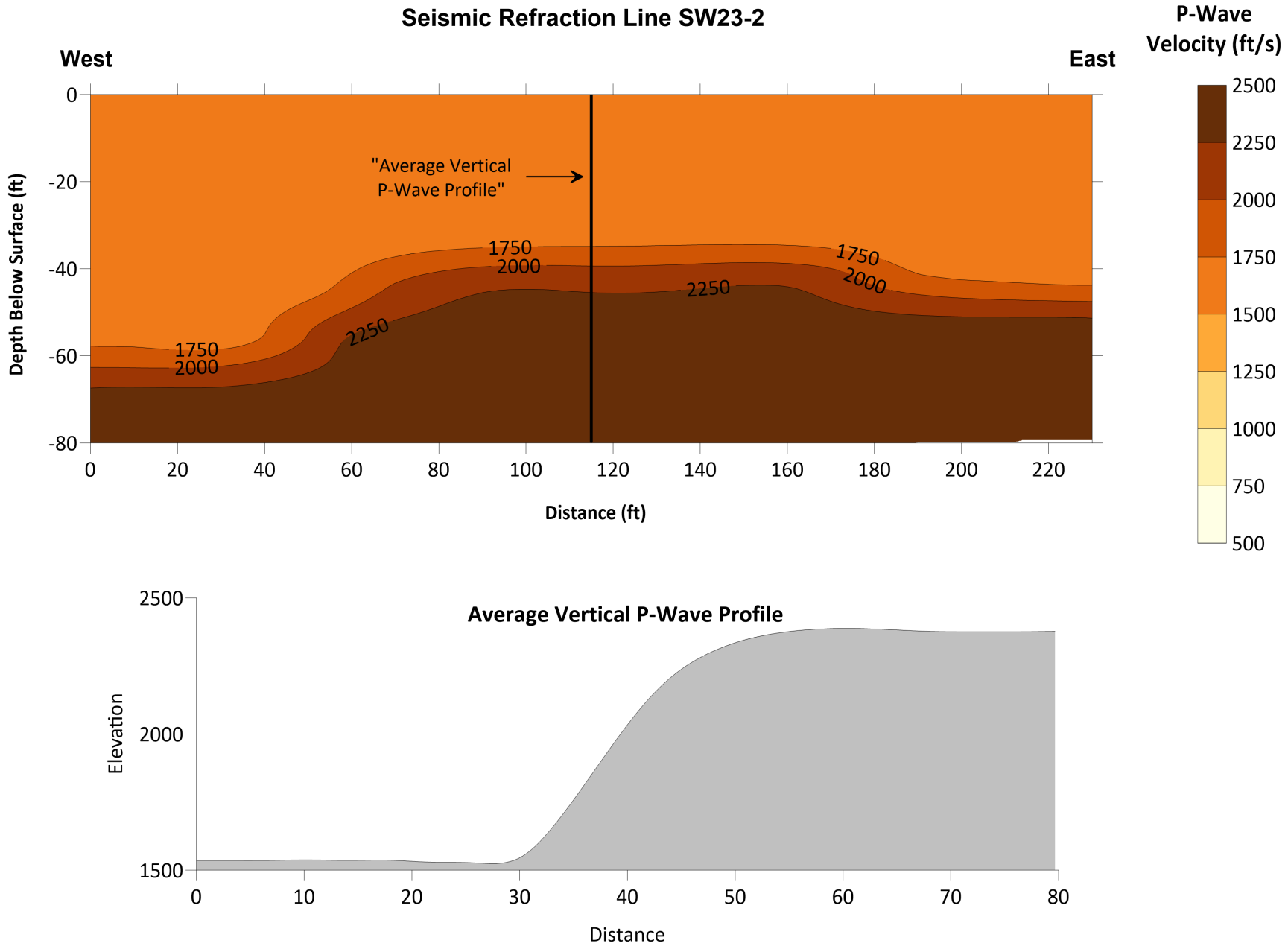
Foti, S., Hollender, F., Garofalo, F., Albarello, D., Asten, M., Bard, P., Comina, C., Cornou, C., Cox, B., Di Giulio, G., Forbriger, T., Hayashi, K., Lunedei, E., Martin, A., Mercerat, D., Ohrnberger, M., Poggi, V., Renalier, F., Sicilia, D., and Socco, V. (2018). Guidelines for the good practice of surface wave analysis: a product of the InterPACIFIC project. Bull Earthquake Eng, v. 16, pp. 2367-2420.

Park, C.B., Miller, R.D., and Xia, J. (1999). Multichannel analysis of surface waves. Geophysics, v. 64, no. 3, pp. 800-808.



GEOPHYSICAL LINE LOCATIONS

GeoPentech	Project No.: **20044A TO-1**	Project: **MLK Medical Campus – Hawkins Building Demolition**	Date: **APR 2023**	Figure: **F-1**

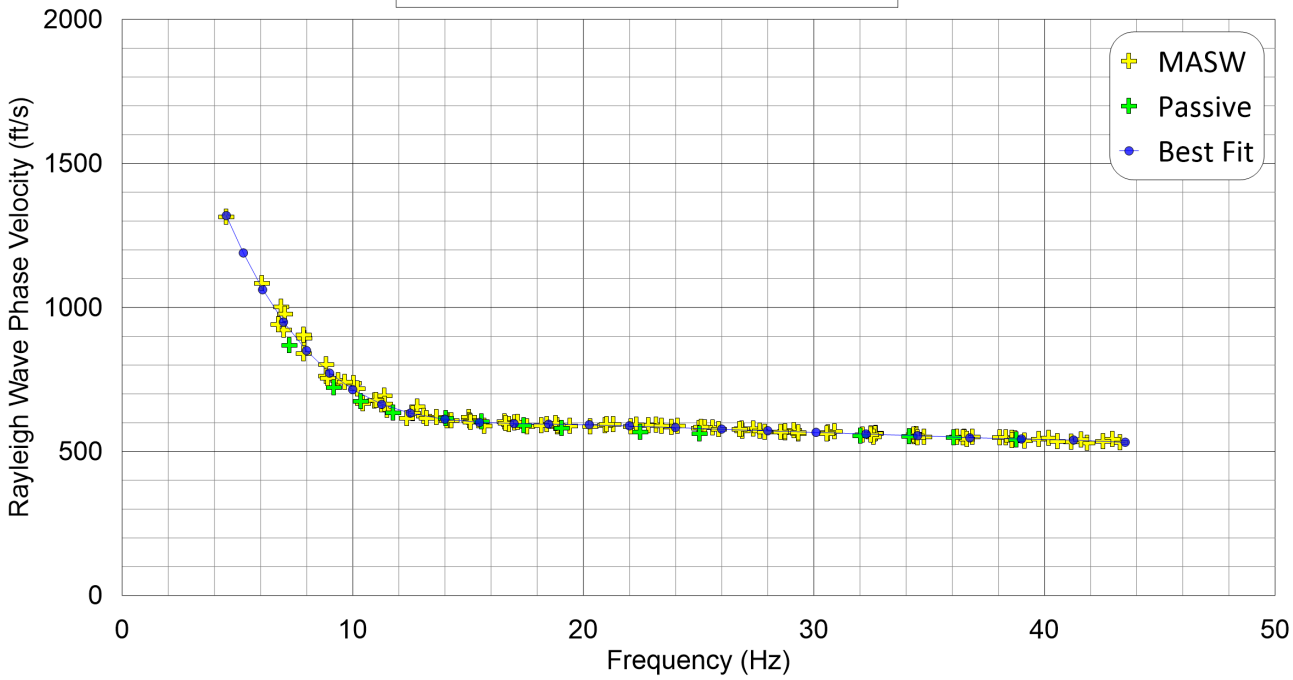


LINE SW23-2 P-WAVE VELOCITY PROFILE

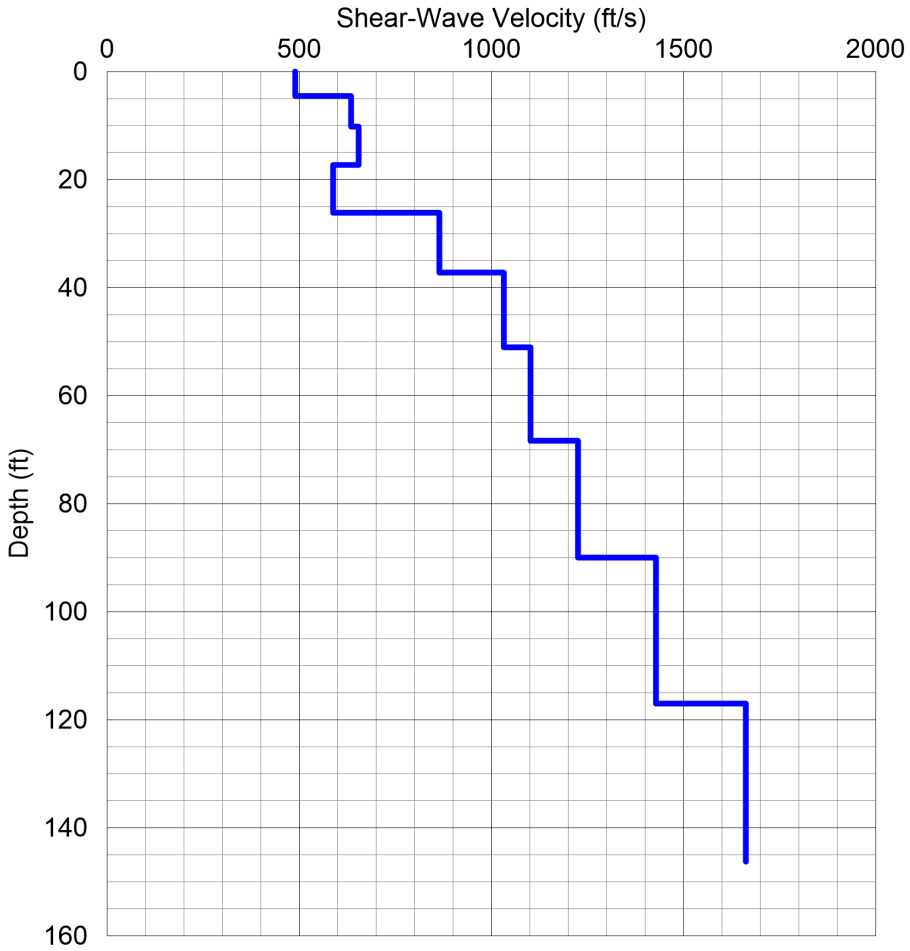
Date: APR 2023	Project No.: 20044A TO-1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure F-2

GeoPentech

SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

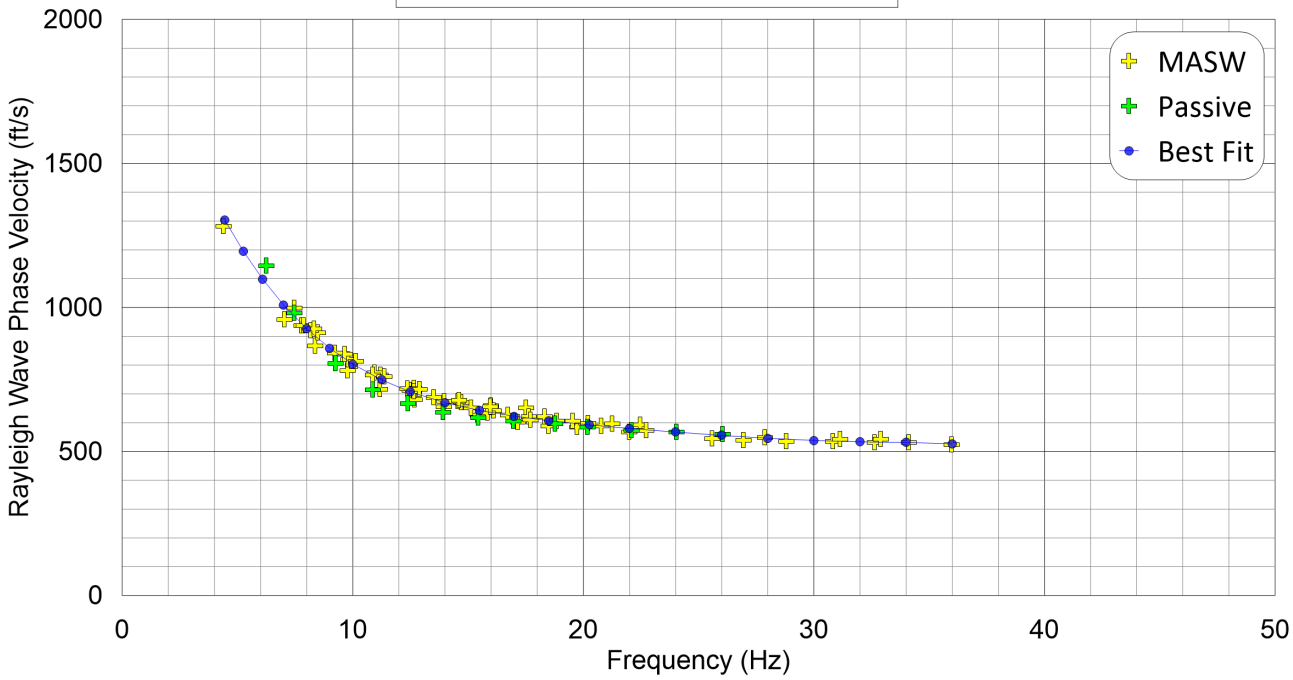


SW23-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

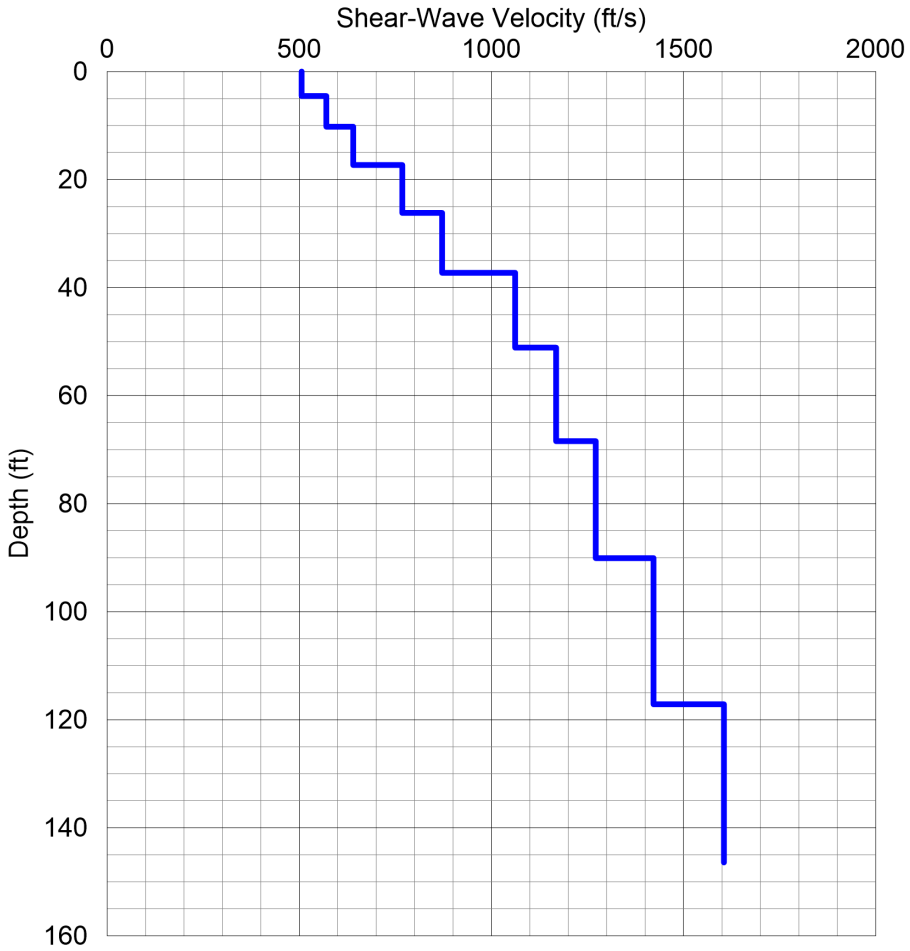
Project No.: 20044A TO-1	Project: MLK Medical Campus - Hawkins Building Demolition	Date: APR 2023	Figure: F-3

GeoPentech

SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

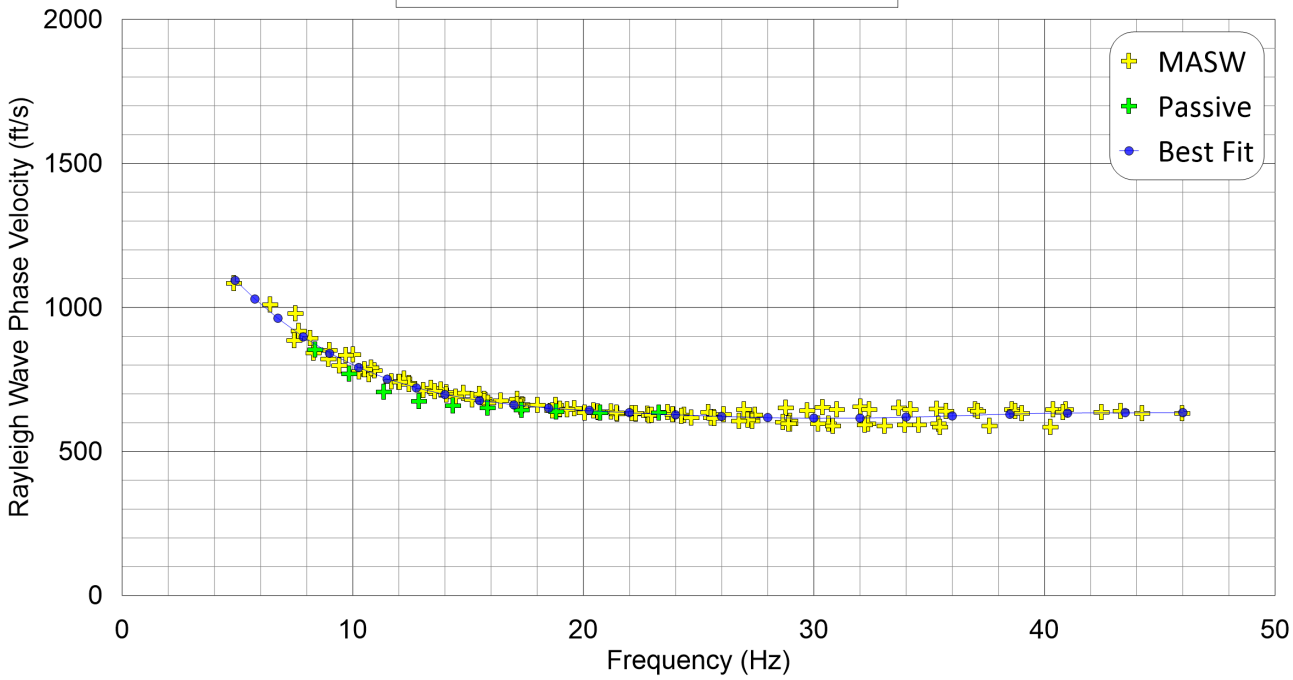


SW23-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

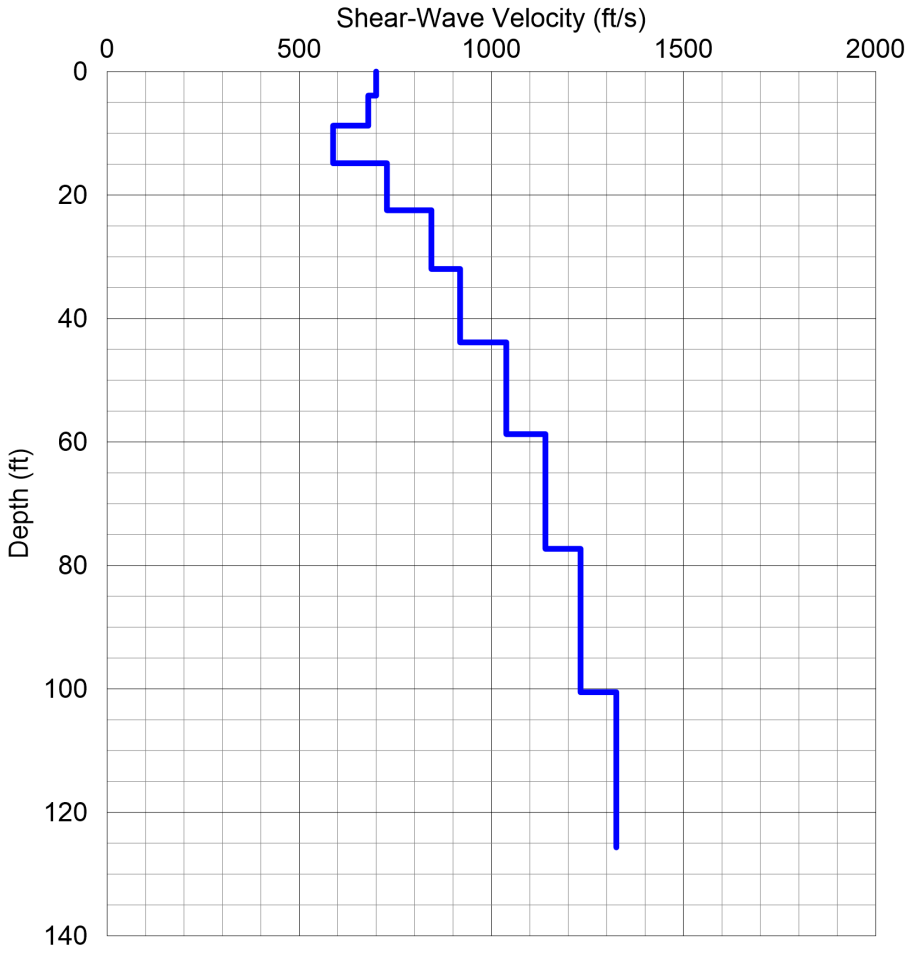
Project No.: 20044A TO-1	Project: MLK Medical Campus - Hawkins Building Demolition	Date: APR 2023	Figure: F-4

GeoPentech

SURFACE WAVE DISPERSION CURVE



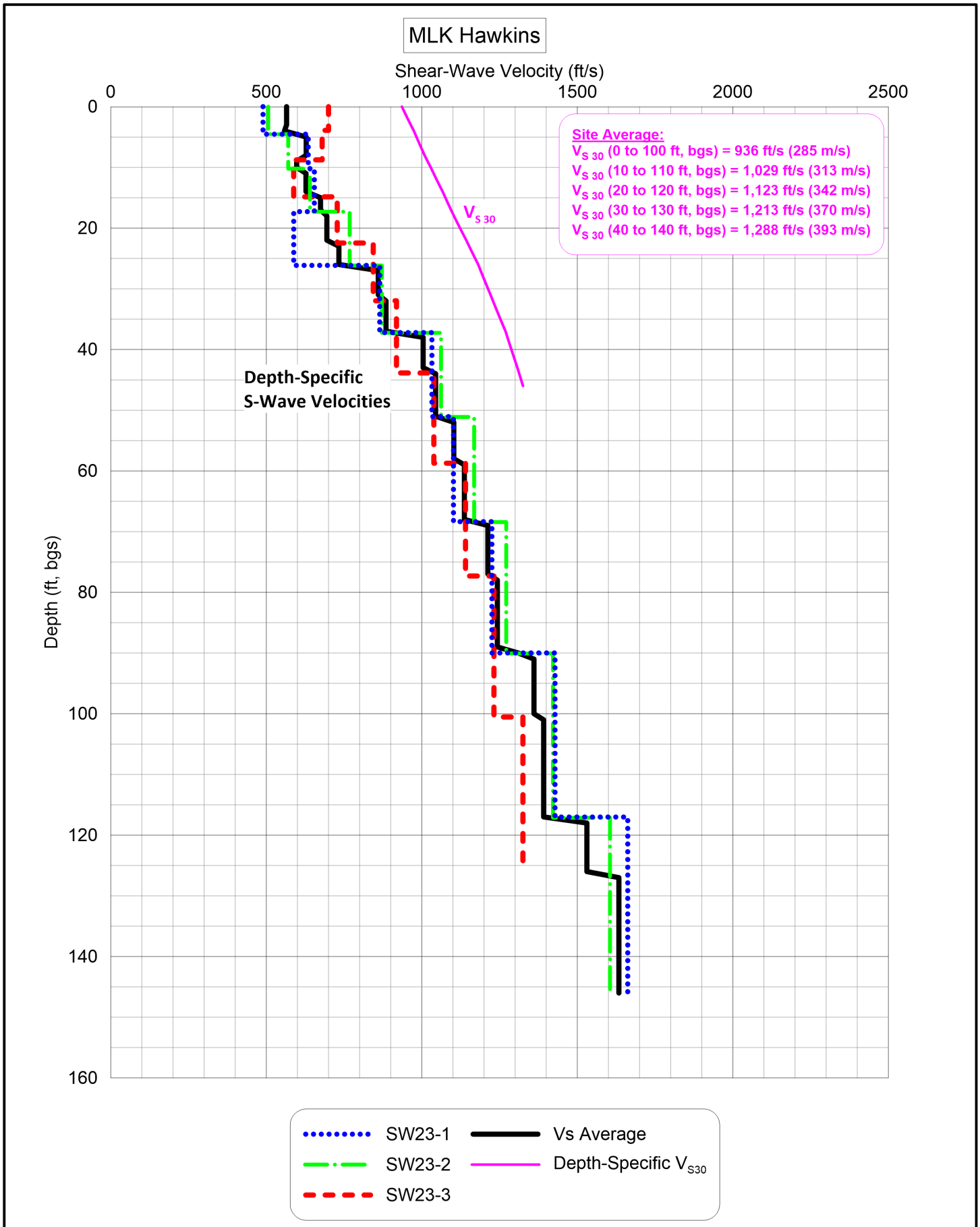
REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL



SW23-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 20044A TO-1	Project: MLK Medical Campus - Hawkins Building Demolition	Date: APR 2023	Figure: F-5

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SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 20044A TO-1	Project: MLK Medical Campus - Hawkins Building Demolition	Date: APR 2023	Figure F-6

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APPENDIX G

LIQUEFACTION ANALYSIS

G.1 LIQUEFACTION ANALYSIS

Our recent field investigations at the project site, shown in Figure 2 of the main report, encountered sandy soils potentially susceptible to liquefaction and seismic-induced dry sand settlement during a strong shaking seismic event. The objective of the analysis documented in this appendix is to evaluate liquefaction triggering and estimate the potential earthquake-induced settlements. The evaluation is based on the current unimproved site conditions as a baseline input for the project design team to consider during potential future development planning.

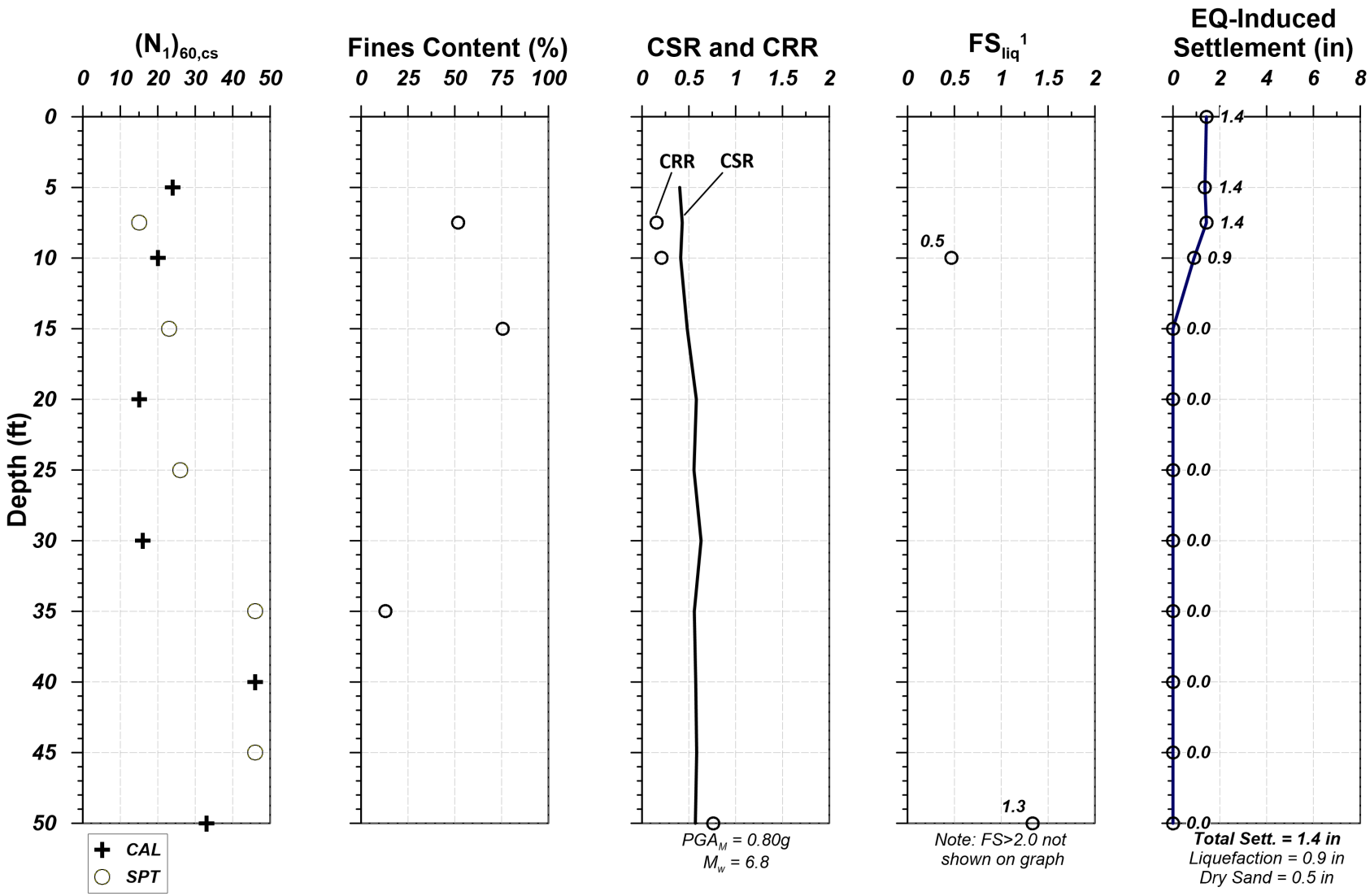
The soil liquefaction potential, liquefaction-induced settlements, and dry sand settlements at the site were evaluated with the methods described by Idriss & Boulanger (2014) and Tokimatsu and Seed (1987) for SPT and Modified California measurements within the site limits assuming the historic high groundwater depth of 10 ft bgs. Using the deaggregation USGS Unified Hazard Tool, the design input ground shaking for a 2475-year return period event consisted of PGA = 0.80g and M_W = 6.80. Blowcounts were corrected for overburden stresses, hammer energy, rod length, sampler type, borehole diameter, and fines content. The normalized clean-sand SPT blowcounts, $(N_1)_{60cs}$, and clean-sand cone tip resistance, q_{c1ncs} shown in Figures G-1 through G-6 (for borings GP- 1 through GP-3 and CPT-1 through CPT-3), were used to determine the cyclic resistance to liquefaction, and liquefaction triggering is considered when a Factor of Safety less than 1.3 was calculated. Figures G-1 through G-6 also show the fines content, Factor of Safety against liquefaction triggering, and the calculated liquefaction-induced settlements. Dry sand settlements are calculated and noted on these figures for layers above the groundwater table.

G.2 RESULTS

As shown on Figures G-1 through G-6, analysis procedures identified soil layers susceptible to liquefaction settlement in borings GP-1 through GP-3 and CPT-1 through CPT-3. These layers result in about 1 to 2.5 inches of liquefaction-induced settlement if strong shaking were to occur.

Analysis procedures also identified soil layers susceptible to dry sand settlement in all borings and CPTs. These layers could result in about 0.3 to 1 inch of seismic-induced settlement during a strong shaking event. CPT-3 shows a potential for up to 4 inches of dry sand settlement. Further testing would be required to verify whether this finding is spread-out through the site.

Based on the analysis, it is expected that the site could experience a total of approximately 3½ inches of seismic settlement.

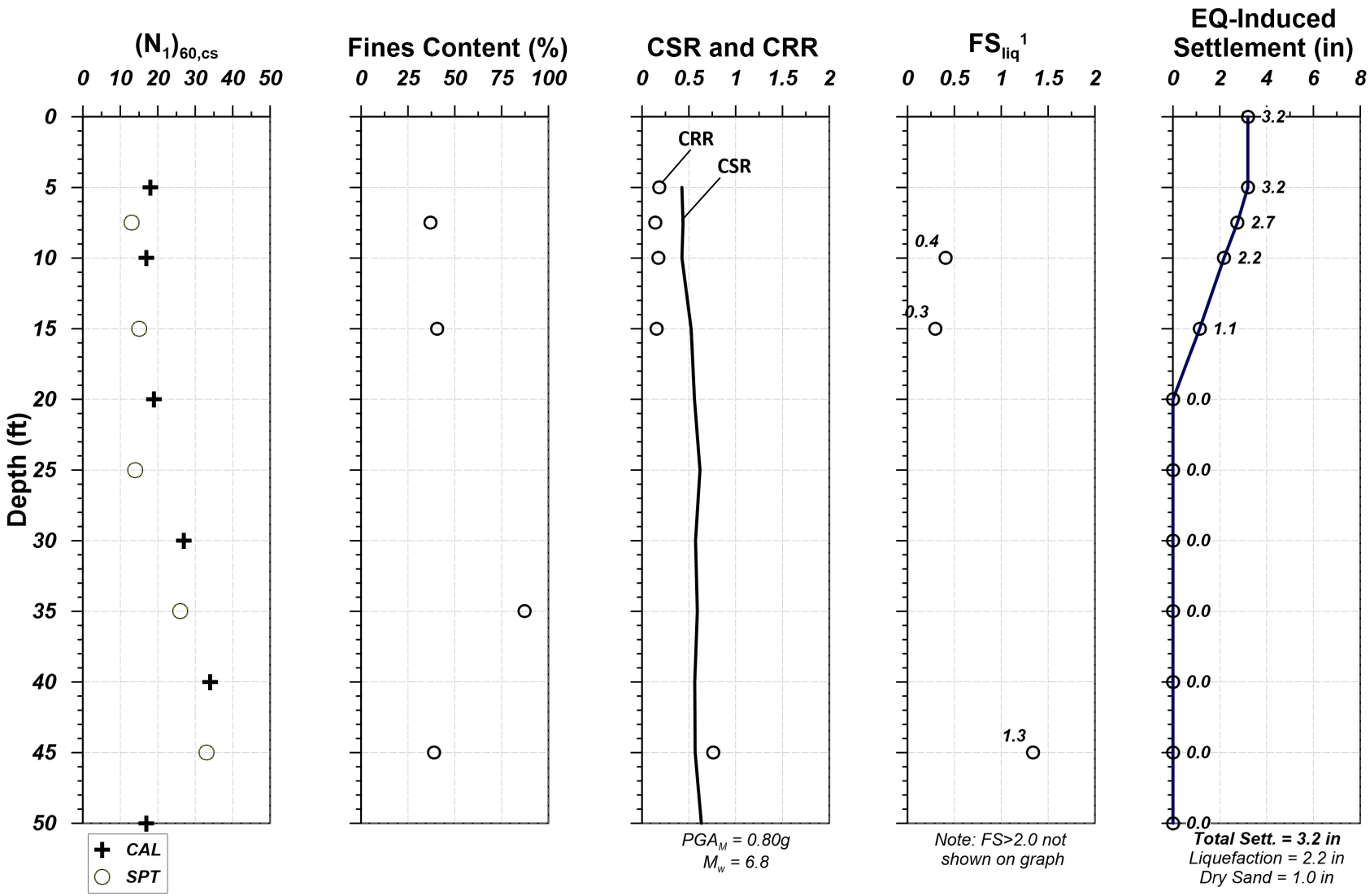


1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

GP-1 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-1

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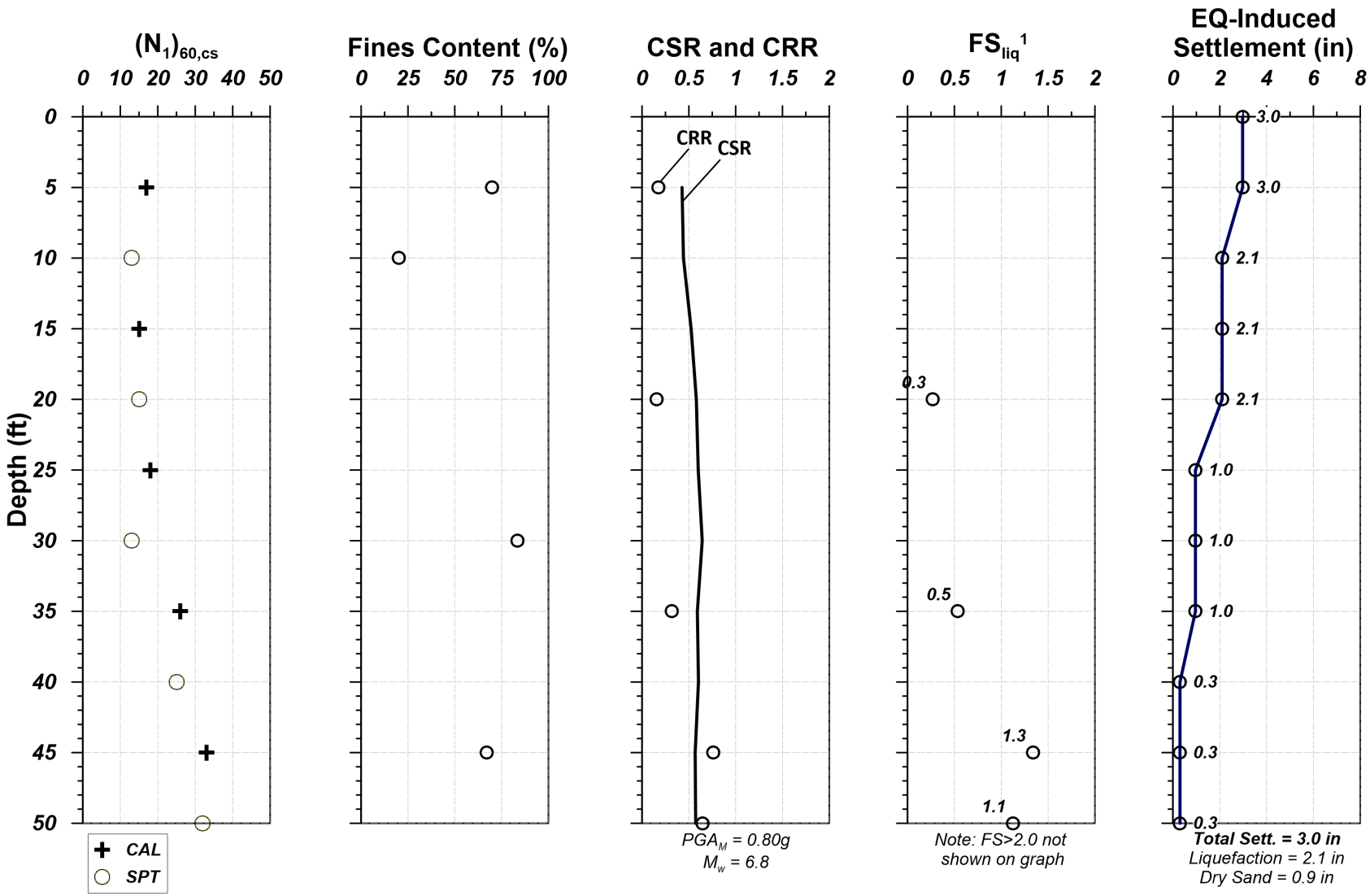


1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

GP-2 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-2

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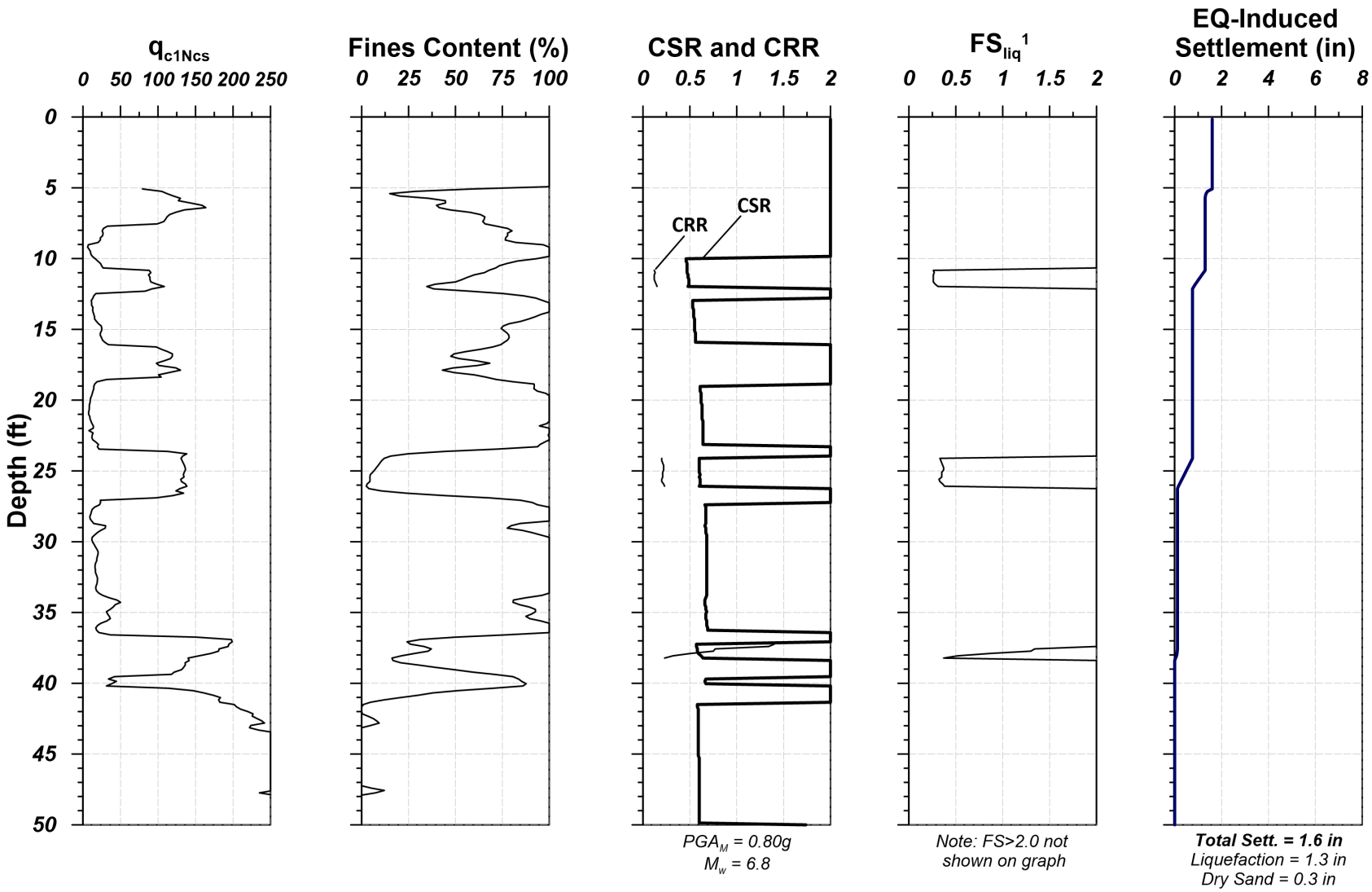


1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

GP-3 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-3

GeoPentech

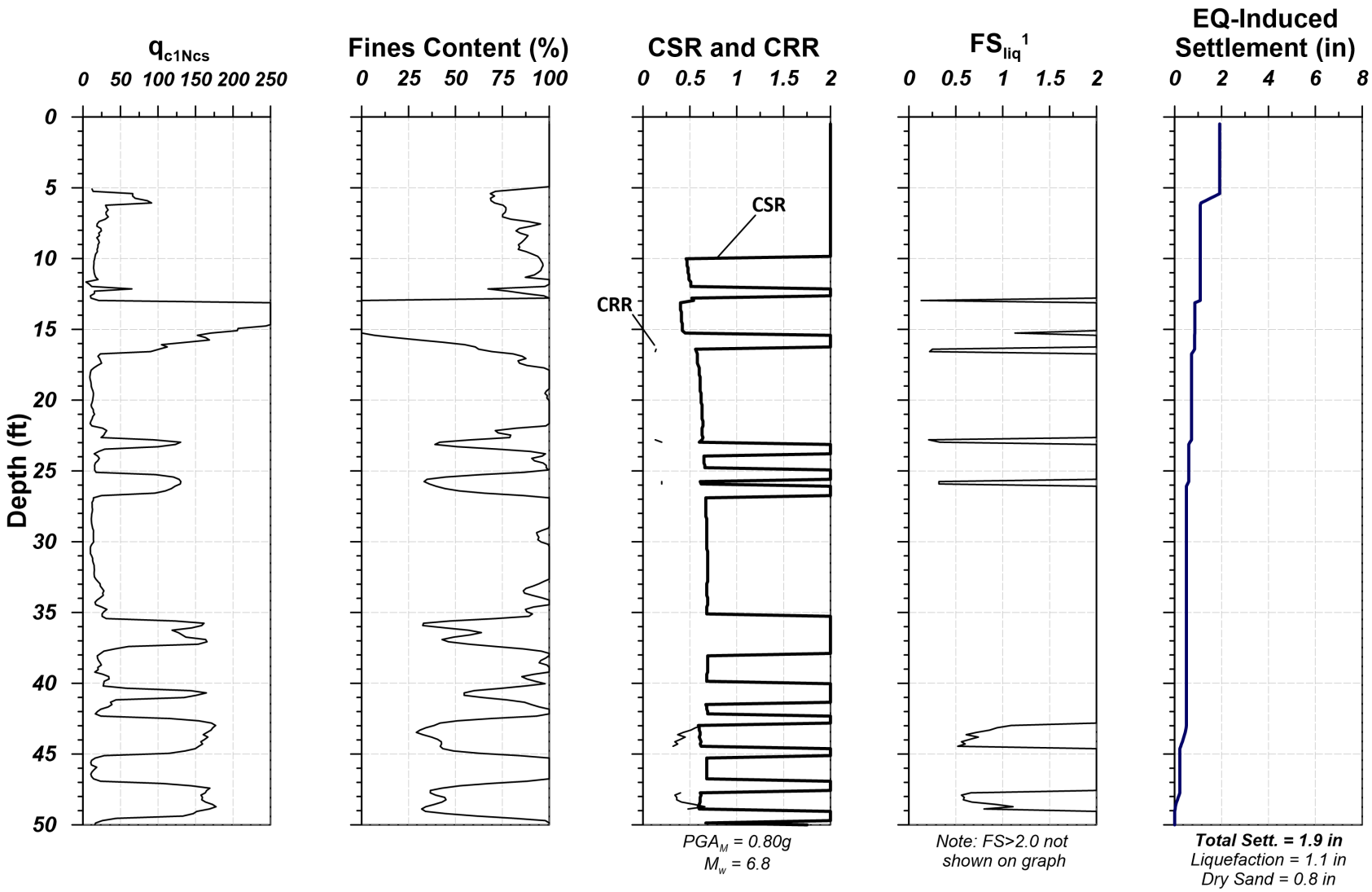


1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

CPT-1 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-4

GeoPentech

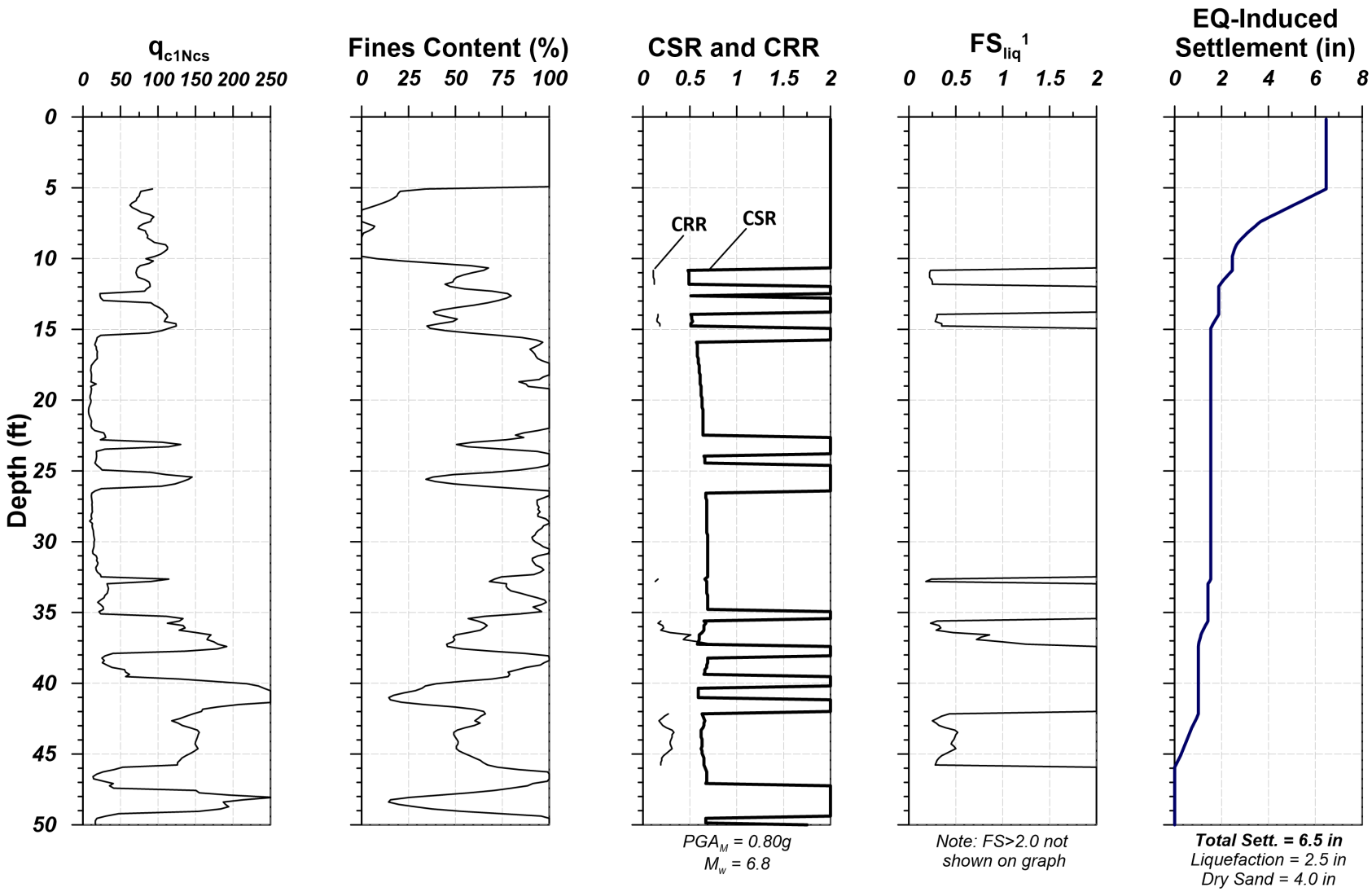


1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

CPT-2 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-5

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1. No FOS calculated for material considered non-susceptible to liquefaction triggering.

CPT-3 Liquefaction Triggering

Date: JUN 2023	Project No.: 20044A-TO1	Project: MLK Medical Campus - Hawkins Building Demolition	Figure: G-6

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