GEOTECHNICAL
PROFESSIONALS INC.

April 27, 2023
(Revised, October 1, 2024)

JTC architects, inc.
65 N. First Street, Suite 201
Arcadia, California 91006

Attention: June Quek (jnquek@jtcarch.com)

Subject: Report of Geotechnical Investigation
Proposed Barry J. Nidorf Secure Youth Track Facility
Security and Kitchen Upgrades
16350 Filbert Street, Sylmar, CA 912342
GPI Project No. 3185.1

Dear June:

In accordance with our proposals dated January 13, 2023 and November 29, 2023, this letter report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for four new security fences in Sylmar, California. The location of the site can be found on the attached Site Location Map, Figure 1.

The existing site is currently a juvenile hall. Four fences are planned at the site on the west side, north side, northeast side, and east side of the property. The locations of the proposed fences are shown on our Site Plan, Figure 2.

Proposed grades for the new improvements are not anticipated to change significantly from the existing grades.

SCOPE OF SERVICES

Our scope of work for this investigation consisted of field explorations, laboratory testing, engineering analyses, and the preparation of this report. Our field explorations consisted of eight borings with depths ranging from approximately 10 to 15 feet. We performed four of the borings along the west and north sides of the camp during an initial phase of work in 2023, and we performed four additional borings along the east and north sides during a recent second phase of fieldwork. The locations of the subsurface explorations are shown on Figure 2. Upon completion of the explorations, we backfilled the borings with the soil cuttings. Details of the drilling and Logs of Borings are presented in Appendix A.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determination of in-place moisture content, Atterberg limits, fines content, shear strength, and soil corrosivity. Corrosivity testing was performed

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

by HDR under subcontract to GPI. Laboratory testing procedures and results are summarized in Appendix B.

Engineering evaluations were performed to provide earthwork criteria, foundation and slab design parameters, preliminary pavement sections and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

SURFACE AND SUBSURFACE CONDITIONS

Surface Conditions

The proposed location of the fences is currently a roadway within the limits of the juvenile hall. The road consists of asphalt concrete pavements consisting of 2 to 4 inches of asphalt concrete over 2 to 12 inches of aggregate base. The site is bounded by a commercial development and vacant property on the northeast, Yarnell Street on the southeast, railroad tracks on the southwest, and Filbert Street on the northwest. The site slopes to the south with elevations ranging from +1268 feet to +1320 feet.

Subsurface Soil Conditions

Our field investigation disclosed a subsurface profile consisting of fill soils overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

We encountered fills in our borings to depths ranging from approximately 4 to 7 feet below existing grades. The fill soils encountered in our explorations consisted of silty sands and clayey sands. The fill soils were likely placed during the initial development of the site; however, documentation of the fill was not provided. The undocumented fills present at the site were, in general loose to medium dense and moist to very moist (moisture contents of 7.8 to 13.9 percent).

The natural soils encountered in our borings consisted of sands, silts, clays, and their mixtures. The sandy soils were, in general, loose to very dense while the fine-grained soils were firm to very stiff. The moisture content of the natural soils generally ranged from 8.0 to 19.1 percent with the exception of the clayey samples in B-7 and B-8 showing moisture contents ranging from 20.0 to 27.8 percent.

Groundwater was not encountered in the upper 15 feet explored during our field investigation. This area has been mapped by the state with historical groundwater between 20 and 40 feet (CDMG, 1998).

Caving was not observed in the small-diameter hollow-stem auger used for drilling. Caving of the loose to medium dense sandy soils should be expected.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, it is our opinion that from a geotechnical engineering viewpoint it is feasible to develop the site as proposed.

Furthermore, in accordance with the County of Los Angeles Statement 111, it is our opinion that the project will be safe for its intended use against hazard from landslide, settlement, or slippage and the project will not have adverse effect on the stability of the site or adjoining properties.

The most significant geotechnical issues that will affect the design and construction of the proposed improvements are as follows:

- The proposed fences may be supported on drill pile foundations provided that the subsurface soils are prepared in accordance with the recommendations in this report.
- The contractor should evaluate the potential drilling conditions when planning installation methods for the foundations of the proposed fences. The drilling for the piles may encounter difficult drilling conditions due to caving sandy soils and dense deeper deposits.

SEISMIC

The site is located in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2022 California Building Code (CBC) criteria. A Site Class D may be used for design in accordance with the 2022 CBC. Using the Site Class and the appropriate internet website (SEAOC), the corresponding seismic design parameters from the CBC are as follows:

<u>2022 CBC:</u>

S_S = 2.73g	$S_{MS} = F_a * S_S$ = 2.73g	S_{DS} = 2/3 * S_{MS} = 1.82g
S_1 = 0.88g	$S_{M1} = F_V * S_1$ = 1.49g	S_{D1} = 2/3 * S_{M1} = 1.00g

Note: The Project Structural Engineer should confirm these values prior to their use.

Strong Ground Motion Potential

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website, we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 1.22g (SEAOC) for a mean magnitude of 6.7 earthquake (USGS). This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-

16 (ASCE, 2016) and a site coefficient (F_{PGA}) based on site class. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone (CDMG, 1999). Therefore, ground rupture due to faulting is considered unlikely at this site.

Liquefaction

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like.

Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is located in an area that has been mapped for liquefaction by the State of California. However, soil liquefaction was not analyzed as part of our investigation as there are no planned habitable structures within the scope of this project.

EARTHWORK

The earthwork anticipated at the project site will consist of excavation of undocumented fills, subgrade preparation, and placement and compaction of fill.

Excavations

Remedial excavations are not anticipated for proposed fence structures. Although none are planned at this time, if minor structures such as screen walls or trash enclosures are added to the proposed project the following recommendations may be used.

Remedial excavations are not anticipated for proposed fence structures.

For minor structures we recommend that the removal/replacement of on-site soils extend to a depth of 4 feet below existing grades or 2 feet below footings, whichever is deeper. Where deeper fills are present, we recommend that the removals extend deep enough to remove the undocumented fill and replace it with properly compacted fill. The actual depth of removals should be determined in the field during grading by a representative of GPI.

The removals should extend at laterally outside of the foundation footprint a distance equal to or greater than the depth of the excavation below the foundation.

For new pavements and pedestrian hardscape, removals should extend to a depth of 1-foot below the existing or proposed subgrade, whichever is deeper.

Prior to the placement of fills, the exposed subgrade surface should be scarified, moisture conditioned, and proof rolled as described in "Subgrade Preparation" section of this report.

Subgrade Preparations

After removals are complete and prior to placing fills or construction of the proposed site improvements, the subgrade soils should be scarified, moisture-conditioned and compacted to at least 90 percent of maximum dry density in accordance with ASTM D 1557.

In areas to receive new pavements, the top 12 inches below the pavement base should be scarified and compacted to a minimum of 95 percent (90 percent for silts and clays) of the maximum dry density in accordance with ASTM D 1557.

Material for Fill

The on-site soils are generally suitable for use as compacted fill.

Imported fill material should be predominately granular (well-graded and contain no more than 40 percent fines-portion passing No. 200 sieve), and relatively non-expansive (an Expansion Index of less than 20). GPI should be provided with a sample (at least 50 pounds) and notified at least 72 hours in advance of the location of soils proposed for import. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Soils used for compacted fills should not contain particles greater than 6 inches in size.

Placement and Compaction of Fills

Fill soils consisting of the on-site materials should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-8 inches
Scrapers, heavy loaders, or heavy vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content. Fills consisting of the on-site silts should be placed at a moisture content of 1 to 3 percent over the optimum moisture content in order to achieve the required compaction. The moisture content of the near surface soils can be expected to be moist. The compacted fill should not be allowed to dry out prior to covering or additional processing will be required.

Trench Backfill

Utility trench backfill should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain one sack of cement per cubic yard and have a maximum slump of 5 inches.

Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

FOUNDATIONS

The proposed fences may be supported on drilled piles and any added minor structures may be supported on shallow foundations provided that the subsurface soils are prepared in accordance with the recommendations in this report.

Pile Foundation Design Parameters

We found that the subsurface conditions on the east side of the site are somewhat different from those on the west side (finer grained and weaker) requiring lower pile capacity parameters. Based upon the results of our subsurface explorations and laboratory testing, the drilled pile design parameters shown in the following tables may be used for axial pile design:

****West of Boring B-5, Recommended Design Parameters – Axial**

Depth Below Existing Grade (feet)	Allowable Pile Skin Friction (psf)
0-3	50
3-7	100
7-10	200
>10	250

****East of Boring B-5, Recommended Design Parameters – Axial**

Depth Below Existing Grade (feet)	Allowable Pile Skin Friction (psf)
0-3	40
3-7	100
7-10	150
>10	200

We utilized a factor of safety of 3 to determine the allowable skin friction values. The settlement of the drilled piles designed using the above allowable skin friction values is anticipated to be less than ½-inch. We do not recommend considering the end bearing capacity for the design of the piles as adequate cleaning of the excavations will be difficult.

The uplift capacity of the piles may be taken as one-half of the downward capacities derived from skin friction only.

For design of the piles for the fence structures by the simplified pole formula presented in Section *1807A.3.2.1* of the 2022 CBC, a unit passive pressure of 600 pounds per square foot per foot (up to a maximum value of 6000 psf) for the fence structures west of B-5. For the structures east of B-5 a unit passive pressure of 500 pounds per square foot (up to a maximum value of 5000psf) may be used. These values may be used for the shaft in lieu of presumptive lateral bearing values presented in Table *1806A.2*. This value incorporates the allowable increase stated in the Section *1806A.3.4* of the code for single poles that can tolerate a ½-inch of deflection under short-term loads.

The above pile design parameters for shallow depths assume the ground surface surrounding the piles will be paved with asphalt. If the surface surrounding the pile is not paved, we recommend the upper 1-foot of soil be ignored in developing lateral resistance and skin friction. Otherwise, lateral and skin friction resistance may be utilized beginning at the finished grade.

The contractor should evaluate the potential drilling conditions when planning installation methods for the foundations of the fences. The drilling for the piles may encounter difficult drilling conditions due to caving sandy soils and dense deeper deposits.

Since the drilled piles will be designed to derive resistance from soil friction only, rigorous cleaning of the loose materials from the bottom of the excavation prior to placement of steel and concrete is not considered essential. Every effort should be made to clean the bottom with the drill rig-mounted equipment.

Pile excavations should be filled with concrete on the same day that they are drilled. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling should not be performed within 5 feet of recently excavated or recently poured piles until the concrete has been allowed to set for at least 6 hours. The piles should be poured in a manner that will not result in concrete flowing into adjacent drilled pile excavations and prevent segregation of aggregate. Drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, "Standard Specifications for the Construction of Drilled Piles."

Pile excavations should be observed by a representative of GPI to confirm and document the depth, diameter, and embedment in suitable materials.

Shallow Foundations

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, static allowable net bearing pressures of up to 2,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings if minor structures are added to the scope of the project.

The bearing pressures are for dead-load-plus-live-load, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

The following minimum footing widths and embedment's are recommended for the corresponding allowable bearing pressure.

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
2,500	18	18
2,000	15	18
1,500	15	15

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction.

A minimum footing width of 15 inches should be used even if the actual bearing pressure is less than 1,500 psf.

Under the relatively light static loading conditions anticipated, maximum total settlement of minor structures is expected to be on the order of ¾-inch or less. Maximum differential static settlement between similarly loaded adjacent footings is estimated to be on the order of ¼-inch across a lateral distance of 40 feet. These settlement estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.40 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used for footings.

The allowable lateral bearing pressure values provided are based on the footings being poured tight against compacted fill soils. The friction and lateral bearing values may be used in combination without reduction.

Foundation Concrete

Laboratory testing by HDR (Appendix C) indicates that the near surface soils exhibit a soluble sulfate content of 39 mg/kg. For the 2022 CBC, foundation concrete should conform to the requirements for negligible sulfate exposure (Category S0) as outlined in ACI 318. The chloride content was 4 mg/kg, which is considered to be low since the foundation concrete will be exposed to moisture in the soils (Category C1).

RETAINING STRUCTURES

Retaining walls are not planned at the time this report was prepared although the following recommendations may be used for walls less than 6 feet in height. We recommend that walls be properly drained and backfilled with sandy soils (less than 40 percent passing the No. 200 sieve).

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI $\leq$ 20), a lateral pressure of an equivalent fluid weighing of 35 pounds per cubic foot may be used.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 58 pounds per cubic foot can be used. If the wall backfill is not drained, the combined earth and water pressures will be much higher.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively.

The recommended pressures assume that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel, wrapped in a suitable filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain.

The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

The Structural Engineer should specify the use of select, granular wall backfill on the plans. Wall footings should be designed as discussed in the "Foundations" section.

Additionally, the recommendations above may also be used if temporary shoring is required.

PAVED AREAS

Preliminary pavement design has been calculated using an R-value of 30 based upon the classification of the near-surface soils at the site. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		Asphalt Concrete	**Aggregate Base Course**
Auto Parking	4	3	4
Circulation Drives	5	3	6
Truck Drives	6	3	9
		Portland Cement Concrete	**Aggregate Base Course**
Auto Parking	4	6	---
Circulation Drives	5	6	---
Truck Drives	6	6.5	---

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The pavement base course (as well as the top 12 inches of the subgrade soils) should be compacted to at least 95 percent of the maximum dry density (ASTM D-1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials, excluding processed miscellaneous base.

The above recommendations are based on the assumption that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

DRAINAGE

Positive surface gradients should be provided adjacent to all structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. The introduction of water into the existing fill soils can result in subsidence. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

LIMITATIONS

This letter report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by JTC architects, inc. and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If others perform construction phase services, they must accept full responsibility for geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

James E. Harris V, P.E. — Expires 09-30-26
Project Engineer
(jamesh@gpi-ca.com)

Paul R. Schade, G.E. — Expires 09-30-26
Principal
(pauls@gpi-ca.com)

Enclosures: Figure 1 - Site Location
Figure 2 - Site Plan
Appendix A - Logs of Boring
Appendix B - Laboratory Testing

cc: Emmanuel Perez, JTC architects, Inc. (emperez@jtcarch.com)

REFERENCES

Structural Engineers Association of California, OSHPD, Seismic Design Maps, https://seismicmaps.org/

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

United States Geological Survey, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/.

California Division of Mines and Geology, 1999, Earthquake Zones of Required Investigation, San Fernando Quadrangle, Official Map, Released March 25, 1999.

California Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the San Fernando 7.5-Minute Quadrangle, Report 015, Los Angeles County, California.

BASE PLAN REPRODUCED FROM CALTOPO © 2023

GEOTECHNICAL PROFESSIONALS, INC.

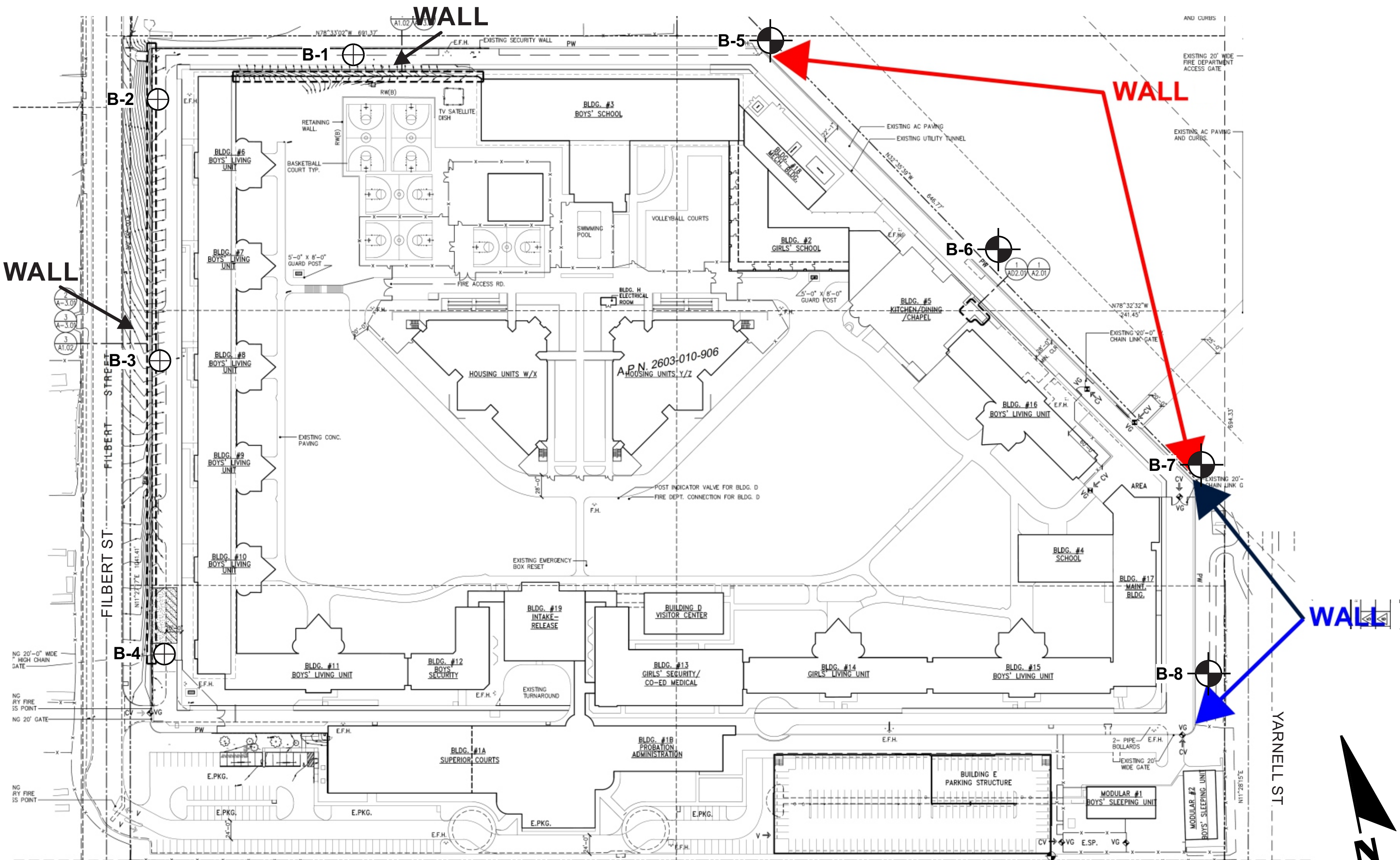
JTC ARCHITECTS BJN SYLMAR

GPI PROJECT NO.: 3185.I

SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-8 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (PHASE 2, 2024)

B-4 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (PHASE 1, 2023)

0 200 400 FEET

BASE PLAN REPRODUCED FROM SITE PLAN AREA OF WORK PROVIDED BY JTC ARCHITECTS DATED: 09-15-2023

GPI GEOTECHNICAL PROFESSIONALS, INC.

Barry J. Nidorf Secure Youth Track Facility Security and Kitchen Upgrades

GPI PROJECT NO.: 3185.I

SCALE: 1" = 200'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling eight exploratory borings The borings were advanced to depths of 10 to 15 feet below the existing ground surface. The locations of the explorations and wells are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-8 in this appendix.

The boring and well locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from Google Earth and should be considered very approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	10.8			B	0	2-Inch AC over 6-Inch Base	
						Fill: **SILTY SAND (SM)** brown, moist, trace gravel	
	13.9			B		@ 2 feet, very moist	
	13.3	111	9	D		@ 4 feet, very loose	1300
					5		
	11.2	109	41	D		Natural: **SILTY SAND (SM)** light brown, moist, medium dense	
							1295
	16.3	127	38	D	10	@ 10 feet, wet	
	19.4	104	50/6"	D		**SANDY SILT (ML)** brown, wet, hard, trace clay, some cementation	1290
					15	Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
4-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
12.4			B	0	3-Inch AC over 2-Inch Base	
					Fill: **SILTY SAND (SM)** brown, moist, trace gravel	
12.7			B		@ 2 feet, with clay	
11.7	118	36	D		**CLAYEY SAND (SC)** brown, very moist, medium dense	1315
10.4	119			5	Natural: **SILTY SAND (SM)** reddish brown, moist, medium dense, with gravel, coarse grained	
19.1	105	43	D		**SANDY SILT (ML)** light brown, wet, very stiff	
					CLAYEY SILT (ML) brown, wet, very stiff	1310
8.0	111	50/6"	D	10	**SILTY SAND (SM)** brown, moist, very dense, some cementation	
7.6	101					
6.7	102	50/6"	D		**CLAYEY SILT (ML)** brown, slightly moist, very stiff, some cementation	1305
				15	**SILTY SAND (SM)** brown, slightly moist, very dense, some cementation	
					Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
4-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
8.7			B	0	3-Inch AC over 2-Inch Base	1295
					Fill: **SILTY SAND (SM)** brown, moist, with gravel	
11.1			B			
15.8	100	27	D	5	Natural: **SANDY SILT (ML)** light brown, very moist, very stiff, trace sand	1290
17.5	106	33	D		**CLAYEY SILT (ML)** brown, very moist, very stiff	
					SILTY SAND (SM) reddish brown, wet, medium dense	
18.0	111	40	D	10	**CLAYEY SILT (ML)** brown, very moist to wet, very stiff	1285
12.5	116				**SILTY SAND (SM)** reddish brown, moist, dense	
12.7	118	50	D		@ 14 feet, some cementation	
				15	Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
4-5-23
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I
JTC BJN SYLMAR

LOG OF BORING NO. B-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
12.9			B	0	4-Inch AC over 4-Inch Base	
					Fill: **SILTY SAND (SM)** brown, moist, trace gravel	
10.2			B			1275
12.0	121	39	D	5	@ 4 feet, medium dense, brick fragments	
10.9	121	54	D		Natural: **SILTY SAND (SM)** brown, moist, dense	1270
12.0	106	40	D	10	@ 10 feet, medium dense	
						1265
11.7	108	30	D	15		
					Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
4-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	3-Inch AC over 4-Inch Base	
9.7			B		Fill: **CLAYEY SAND (SC)** slightly reddish brown, moist	1285
15.2	109	14	D	5	Natural: **CLAYEY SAND (SC)** slightly reddish brown, very moist, loose	
15.1	114	33	D		@ 6 feet, medium dense	1280
12.3	116	35	D	10	@ 9 feet, trace gravel @ 9.5 feet, medium dense, with grey spots	1275
16.8	110	63	D	15		
					Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-8-24

EQUIPMENT USED: 8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-5

FIGURE A-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	4-Inch AC over 6-Inch Base	1280
9.0			B		Fill: **SILTY SAND (SM)** brown, moist, with gravel	
8.9	106	19	D			
				5	Natural: **CLAYEY SAND (SC)** slightly reddish brown, moist, medium dense	1275
9.5	107	23	D			
13.4	107	20	D		@ 9 feet, very moist	
				10	Total Depth 10 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
3-8-24

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-6

FIGURE A-6

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	2-Inch AC over 6-Inch Base	
	9.6			B		Fill: **SILTY SAND (SM)** brown, moist, medium dense, with gravel	1275
	7.8	104	21	D			
					5	@ 4.5 feet, lens of sand	
	15.4	107	18	D		Natural: **CLAYEY SAND (SC)** dark brown, very moist, medium dense	1270
	20.4	100	17	D	10	**SANDY CLAY (CL)** dark brown, very moist, stiff	
							1265
	24.4	98	17	D	15	**CLAY (CL)** brown, very moist, stiff	
						Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
3-8-24

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-7

FIGURE A-7

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	4-Inch AC over 12-Inch Base	
	10.4			B		Fill: **SILTY SAND (SM)** brown, moist, with gravel	1270
	12.5	114	21	D	5	**CLAYEY SAND (SC)** brown, very moist, medium dense, trace gravel	
	20.0	97	12	D		Natural: **SANDY SILT (ML)** brown, wet, firm to stiff	1265
	25.0	90	16	D	10	**CLAY (CL)** brown, wet, stiff	1260
	27.8	92	16	D	15		
						Total Depth 15 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
3-8-24

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3185.I

JTC BJN SYLMAR

LOG OF BORING NO. B-8

FIGURE A-8

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-3	4	Sandy Silt (ML)	51
B-4	10	Silty Sand (SM)	45

ATTERBERG LIMITS

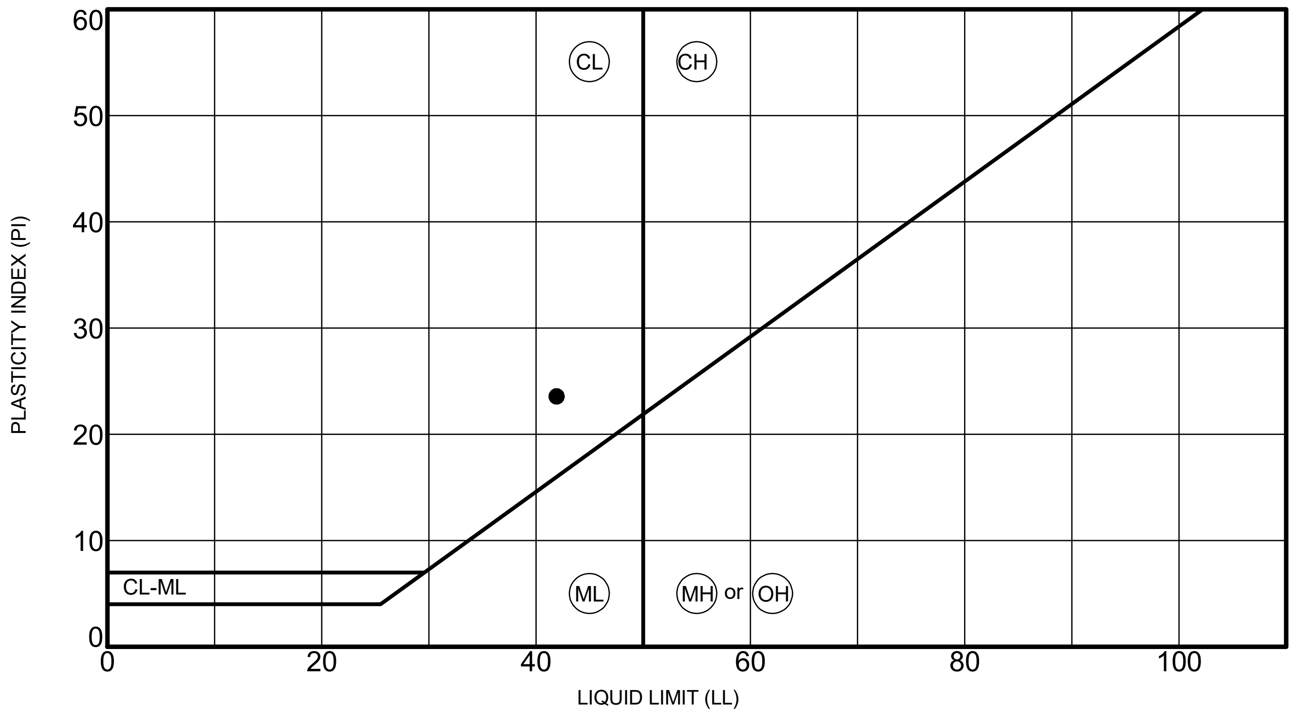
Liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

DIRECT SHEAR

Direct shear tests were performed on undisturbed samples in accordance with ASTM D 3080. The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 and B-3.

CORROSIVITY

Soil corrosivity testing was performed by HDR a soil sample provided by GPI. The test results are summarized in Table 1 of this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-8	9.0	42	18	24		CLAY (CL)

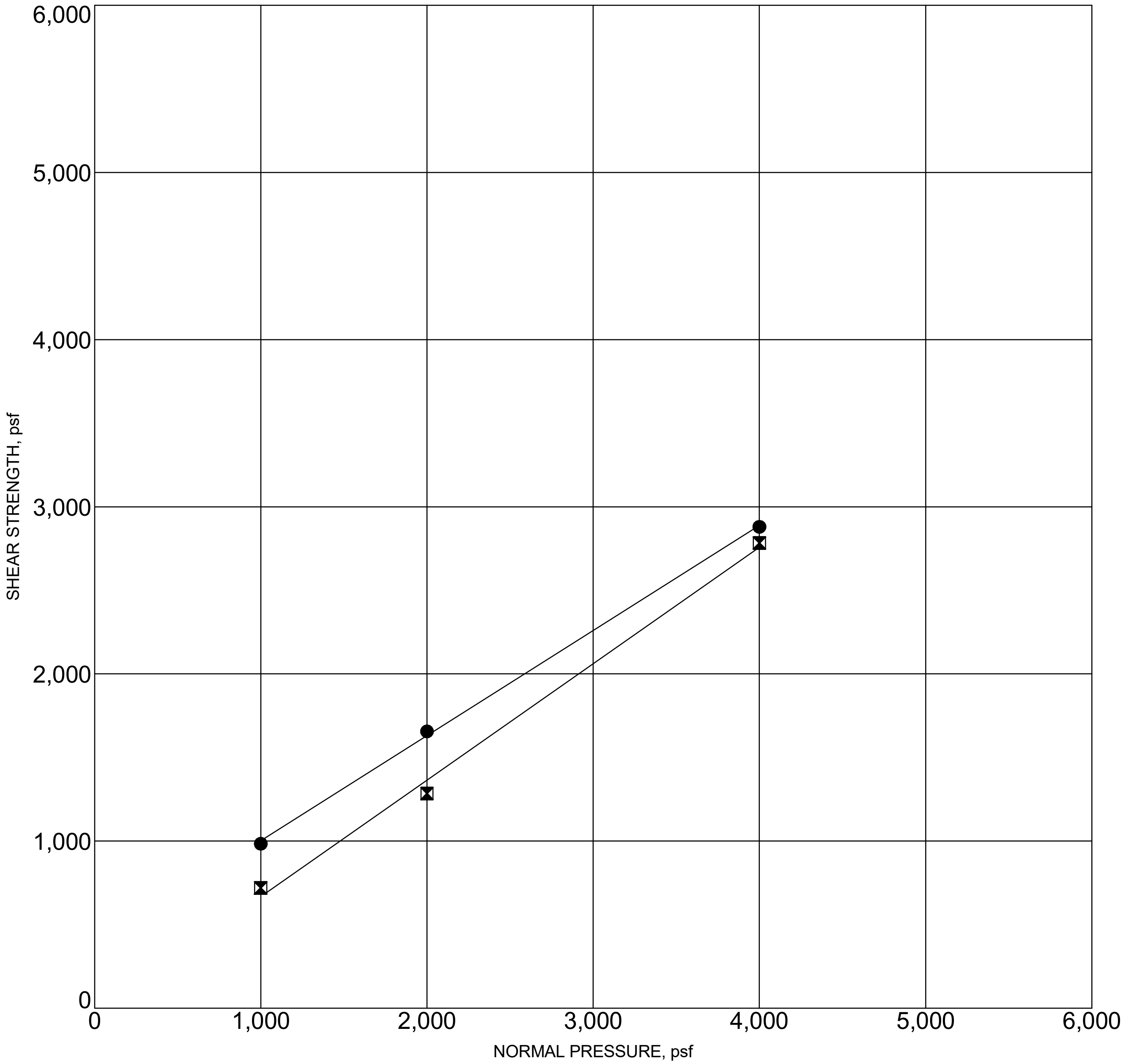
PROJECT: JTC BJN SYLMAR

PROJECT NO. 3185.I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 32 degrees
Cohesion= 372 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 34 degrees
Cohesion= 0 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	4.0	SANDY SILT (ML)	100	15.8

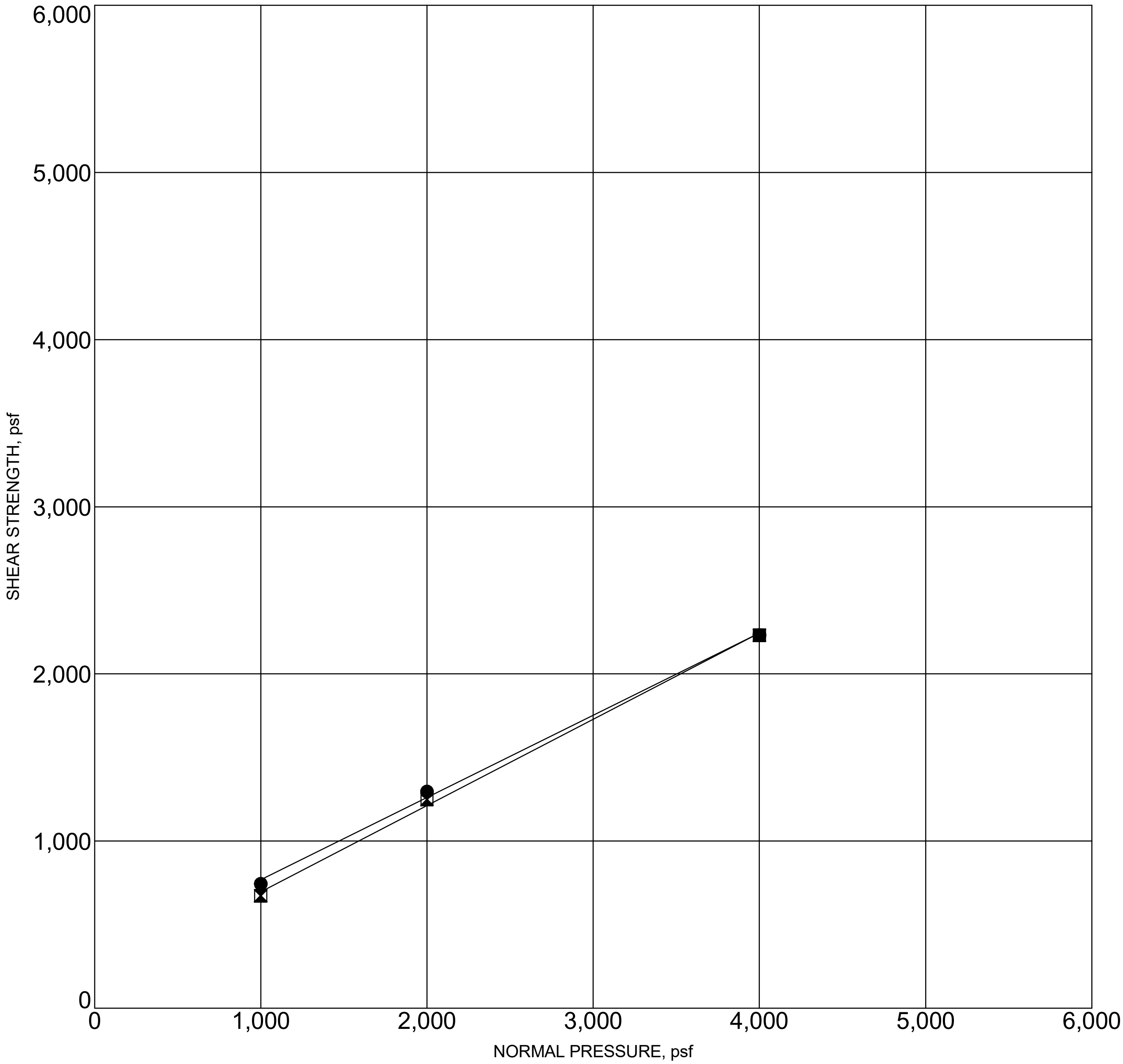
PROJECT: JTC BJN SYLMAR

PROJECT NO.: 3185.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 276 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 27 degrees
Cohesion= 180 psf

Sample Location		Classification	DD,pcf	MC,%
B-8	9.0	CLAY (CL)	90	25.0

PROJECT: JTC BJN SYLMAR

PROJECT NO.: 3185.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-3

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
BJN Sylmar
Your #3185.I, HDR Lab #23-0271LAB
20-Apr-23

Sample ID			
			B-2 @2-4'
Resistivity		**Units**	
as-received		ohm-cm	4,400
saturated		ohm-cm	4,000
pH			7.1
Electrical Conductivity		mS/cm	0.07
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	21
magnesium	Mg^{2+}	mg/kg	5.0
sodium	Na^{1+}	mg/kg	67
potassium	K^{1+}	mg/kg	ND
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	156
fluoride	F^{1-}	mg/kg	6.0
chloride	Cl^{1-}	mg/kg	4.0
sulfate	SO_4^{2-}	mg/kg	39
nitrate	NO_3^{1-}	mg/kg	3.0
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed