GEOTECHNICAL
PROFESSIONALS INC.

November 3, 2022

Blake/Griggs Properties, LLC
550 Hartz Avenue, Suite 200
Danville, CA 94526

Attention: Mr. Scott Griggs

Subject: Geotechnical Design-Level Investigation
Mixed-Use Apartment Development
701 S. Myrtle Avenue
Monrovia, California
GPI Project No. 3160.I

Dear Mr. Griggs:

In accordance with your request, this letter presents the results of our geotechnical feasibility investigation for the subject project. The site location is shown on the Site Location Map, Figure 1.

We understand that our evaluation of the site is desired prior to the purchase of the property by Blake/Griggs Properties, LLC. The primary purpose of our investigation is to determine whether significant geotechnical conditions ("fatal flaws") are present at the site that may impact the development of a mixed-use building. This letter is not intended to be a design level document and should not be submitted to the regulatory agency.

We provided geotechnical services for the projects at 700 and 825 S. Myrtle Avenue. The conditions at the subject site are similar to these nearby sites.

PROJECT DESCRIPTION

Based on information you provided, we understand that a podium-style apartment/retail complex is proposed that will contain approximately 200 residential units with 350 parking stalls. The structure will be 4 stories of wood framed apartments over 2 levels of above grade concrete podium. Below grade, the structure includes an additional 1 to 2 levels of the concrete podium with the lowest level being a half level. The proposed site plan and sections of the proposed development are provided in Figure 2.

Based on our experience with similar projects, we have assumed maximum wall loads for the building will be on the order of 8 to 12 kips per lineal foot and maximum column loads of approximately 500 to 750 kips. Finish grades at the street level are expected to correspond approximately to existing grades. The finished floor of the subterranean parking is expected to be approximately 10 to 20 feet below existing site grades.

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The entire site covers a footprint of 1.61 acres. The site is bounded by W. Olive Avenue to the north, S. Myrtle Avenue to the east, an alley and single-story warehouse building to the south, and a single-story office building and parking to the west.

SCOPE OF WORK

Our scope of work included limited subsurface exploration, review of prior nearby geotechnical data, engineering evaluations, and preparation of this feasibility-level geotechnical report.

We performed three cone penetration tests (CPT's) to evaluate subsurface conditions at the site. The CPTs were extended to depths of 40, 50, and 71 feet below existing grades. Two CPT's refused at depths of 40 and 71 feet below existing grade in very dense sandy soils. Logs of the CPT's are presented in the Appendix. At one CPT (C-3), we performed seismic shear wave measurements at depth intervals of 10 feet. The approximate locations of the CPT's are shown on the Site Plans, Figures 2 and 3.

SUBSURFACE CONDITIONS

The subsurface conditions encountered in our CPT's were interpreted to consist of predominately interbedded layers of sands, silty sands, and sandy silts. In CPT C-1, we encountered a layer of clayey silt extending from about 30 to 35 feet below existing grades. While difficult to distinguish in CPT's, man-made fills appear to extend approximately 2 to 4 feet below existing grades. The interpreted sandy soils range from medium dense to dense within the upper 25 to 40 feet, with the soils becoming dense to very dense with depth. The limited fine-grained soils are typically stiff to hard.

We observed variability between the CPT's performed in the northern portion of the site (CPT C-1 and C-2) and the southern portion of the site (CPT C-3). In general, the depth of medium dense sands at CPT C-3 extended to depths of approximately 25 feet. In general, the depth of medium dense sands was on the order of 5 to 7 feet below existing grade at the other CPT's.

Detailed descriptions of the materials as interpreted from the CPT's are shown on the Logs of CPT's in the Appendix.

Seismic shear wave velocity measurements performed within CPT C-3 indicate an average shear wave velocity of 1256 ft/sec within the upper 100 feet of the soil profile.

Groundwater was not encountered during our investigation. Based on the Seismic Hazard Zone Report (Reference 1), groundwater is anticipated to be deeper than 100 feet.

OBSERVATIONS AND FINDINGS

Based on available documents and our field investigation, we offer the following:

General

- The site currently contains a single-story office building, asphalt pavements, a concrete drive entry, flatwork, and landscape areas. The pavements are in poor to fair condition with significant cracking.

- Based on a review of historical aerial photos (Reference 2), the site has been occupied by the single-story office since 1979. Based on historic aerials from 1952 to 1977, the site appears to have been occupied by a single-family home, garage, and large yard.

- The single-story building to the west of the site extends to the property line of the site. The single-story warehouse building to the south of the site is approximately 20 feet from the property line.

Seismic

- The site is not located in a Special Studies Fault Zone. There are no known active faults crossing or trending towards the site. The site is located approximately one mile from a Special Studies Fault Zone (Reference 3). The Special Studies Fault Zone includes the Sierra Madre Fault Zone.

- The site is not located in a Seismic Hazard Zone for liquefaction according to the Seismic Hazard Zone Map (Reference 3).

Subsurface Conditions

- Based on the existing and former structures at the site, foundation elements and undocumented fills associated with the buildings may exist below the footprint of the structures.

- The upper on-site soils are predominantly medium dense sands and silty sands. As such, the soils are susceptible to caving in open cuts and excavations.

- Corrosivity testing at nearby sites (Reference 4) indicated negligible levels of sulfate exposure and mild corrosivity to buried metals.

CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

Based on our observations and findings, subsurface investigation, and experience in the area, we conclude the following:

Earthwork

- Undocumented fills or debris buried from the current commercial structure that is present and encountered during demolition will need to be removed within the building footprint. However, these undocumented fills and debris will likely be removed with excavations to the subterranean parking levels.

- Based on the variability of the depth to dense soils in the CPT's, our preliminary recommendation is that removals within the building footprint should extend to the bottom of the footings. Our preliminary recommendation is to compact the bottom of the excavation with heavy vibratory equipment in order to densify soils directly beneath the footing bottoms.

- Our preliminary recommendations for removals may be modified when more field and laboratory data is obtained at the site or in the field based on conditions observed in the bottom of the excavations.

- Based on the preliminary explorations, the on-site soils in the upper 25 feet of the soil profile are suitable for use as compacted fill including as fill below footings and as non-expansive fill directly beneath slab-on-grade floors/hardscape or behind retaining walls.

- Granular soils under the building footprint should be compacted to at least 95 percent of maximum dry density in accordance with ASTM D-1557.

Seismic

- We assume seismic design of the proposed development will be in accordance with the 2022 California Building Code (CBC). Based on the results of the shear wave measurements performed during our subsurface explorations, a Site Class C should be used for seismic design.

- There is a potential that the proposed site development will be subjected to strong ground shaking due to earthquakes on nearby faults. Based on published information (References 5 and 6) and the Site Class provided above, the site could be subjected to a peak horizontal ground acceleration of (PGAM) 0.98g and a mean magnitude 7.1 earthquake. This acceleration has a 2 percent chance of being exceeded in 50 years.

- Based on the results of our CPT's and deep groundwater, we do not anticipate liquefaction induced settlement to negatively impact the site. However, based on our analyses, we estimate a potential for dry seismic settlement of about 1-inch or less assuming removals of the existing soils will extend to at least 10 to 15 feet below existing grades.

Foundations and Slab-On-Grade Floors

- Spread footings and a slab-on-grade floor can be used to support the proposed apartment building (4 stories of wood framed apartments over 2 levels of above-grade concrete podium over 1 to 2 subterranean concrete levels).

- The footings can likely be supported on natural soils or a layer of engineered fill derived from natural soils. We anticipate the footings can be designed for an allowable net bearing capacity on the order of 4,000 to 5,000 pounds per square foot (psf). These preliminary recommendations will limit the total settlement (static and seismic) to approximately 1½ inches.

- Soil resistance to lateral loads can 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 preliminary design of footings, a coefficient of friction of 0.35 may be used for frictional resistance. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used for the preliminary design of footings.

- The slab-on-grade floors can be supported on engineered fill derived from natural soils. For concrete floors for the subterranean parking level, a vapor retarder is not necessary from a geotechnical standpoint except where moisture sensitive flooring such as at lobby areas near the elevators.

Shoring

- Since there is not sufficient space for sloped embankments along the property limits, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete. Driven or vibrated soldier piles may be a feasible and more economical alternative to drilled holes, however vibrating piles adjacent to the existing building at the property line to the west should not be permitted.

- The shoring contractor should evaluate the potential drilling conditions when planning the installation methods and should note that caving of drilled holes is likely due to the presence of medium dense clean sands.

- Due to the proximity to the adjacent streets, and buildings, an additional surcharge load will likely need to be added to the recommended earth pressure to account for structure and traffic surcharge loading.

- Excavations deeper than approximately 15 to 17 feet will likely require tied-back earth anchors or rakers to support the temporary shoring.

- Due to anticipated caving soil conditions, the voids between the lagging and retained earth should be backfilled with lean-mix sand-cement slurry prior to continuing the excavation deeper.

Stormwater Infiltration

- Our interpretation of the CPT data suggests that the site consists of predominately sandy soils. These soil types suggest that stormwater infiltration on-site is feasible.

- Current plans show the proposed building to extend to the property limits on all sides making conventional stormwater detention systems not feasible. As such, drywells would likely need to extend to depths of at least 25 to 30 feet below the proposed foundations (approximately 40 to 55 feet below existing grades) to provide stormwater infiltration at the site.

- The drywells should be grouted to depths of at least 25 to 30 feet below the proposed foundations to mitigate impact of water infiltrating within the influence zone of the footings.

FURTHER STUDIES

A design-level geotechnical investigation will be required to provide specific design and construction recommendations for the proposed residential development. The design-level investigation will need to be reviewed and approved by the appropriate regulatory agency.

As part of a design-level geotechnical investigation, we recommend at least 3 exploratory borings be performed with the footprint of the apartment building. The exploratory borings will allow us to obtain soil samples to perform consolidation (collapse) testing, index testing, and corrosivity testing. These tests will allow us to determine the collapse potential of medium dense sands under foundation level of the structures and to refine our seismic settlement analysis with correlation of the fines in the soil layers.

Infiltration testing will need to be performed prior to the design of a system to determine specific infiltration rates.

LIMITATIONS

The geotechnical investigation reported herein was performed for the exclusive use by Blake/Griggs Properties, LLC and their consultants, in evaluating the feasibility of constructing the proposed improvements. This report should not be used for evaluating the feasibility of developing the site for other uses or for the detailed design of the proposed project, because this report does not contain sufficient or appropriate information for such use.

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 is reasonably representative of field conditions and are conducive to interpolation and extrapolation.

As noted previously, additional geotechnical investigations will be needed for design and construction. Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by a qualified geotechnical consulting firm during grading, excavation, and foundation construction. If design- and construction-phase geotechnical services are performed by others they must accept full responsibility for all geotechnical aspects of the project.

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.

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Enclosures:	References	
	Figure 1	- Site Location Map
	Figure 2	- Site Plan – Proposed
	Figure 3	- Site Plan - Existing
	Appendix	- Logs of CPTs

Distribution: Addressee (via email)

REFERENCES

1. California Department of Conservation, Division of Mines and Geology (1998), “Seismic Hazard Zone Report for the Mount Wilson 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 030”.

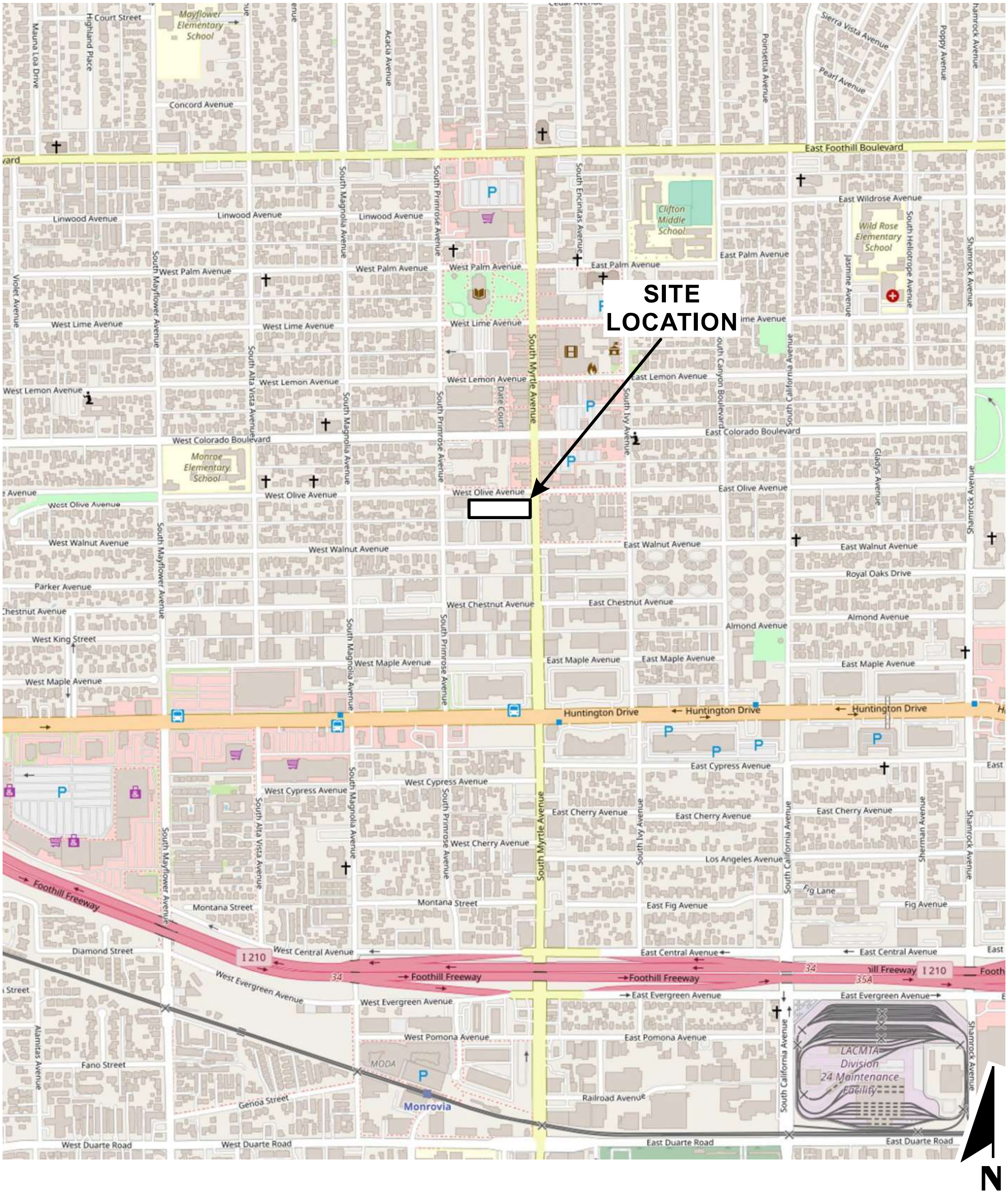
2. http://www.historicaerials.com, Aerial Photography from the Past and Present”, National Environmental Title Research, LLC.

3. California Department of Conservation, Division of Mines and Geology (1998), “Seismic Hazard Zone Map, Mount Wilson Quadrangle”.

4. Geotechnical Professionals, Inc., “Revised Geotechnical Feasibility Investigation, Avalon Bay Monrovia, 825 South Street, Monrovia, California,” Project No. 2775.I, dated December 6, 2016 (Revised April 2, 2018).

5. California Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Website, https://seismicmaps.org/

6. United States Geological Survey (2014), 2008 National Seismic Hazard Maps, Source Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008.cfm



0 1000 2000 FEET

BASE MAP REPRODUCED FROM CALTOPO © 2022

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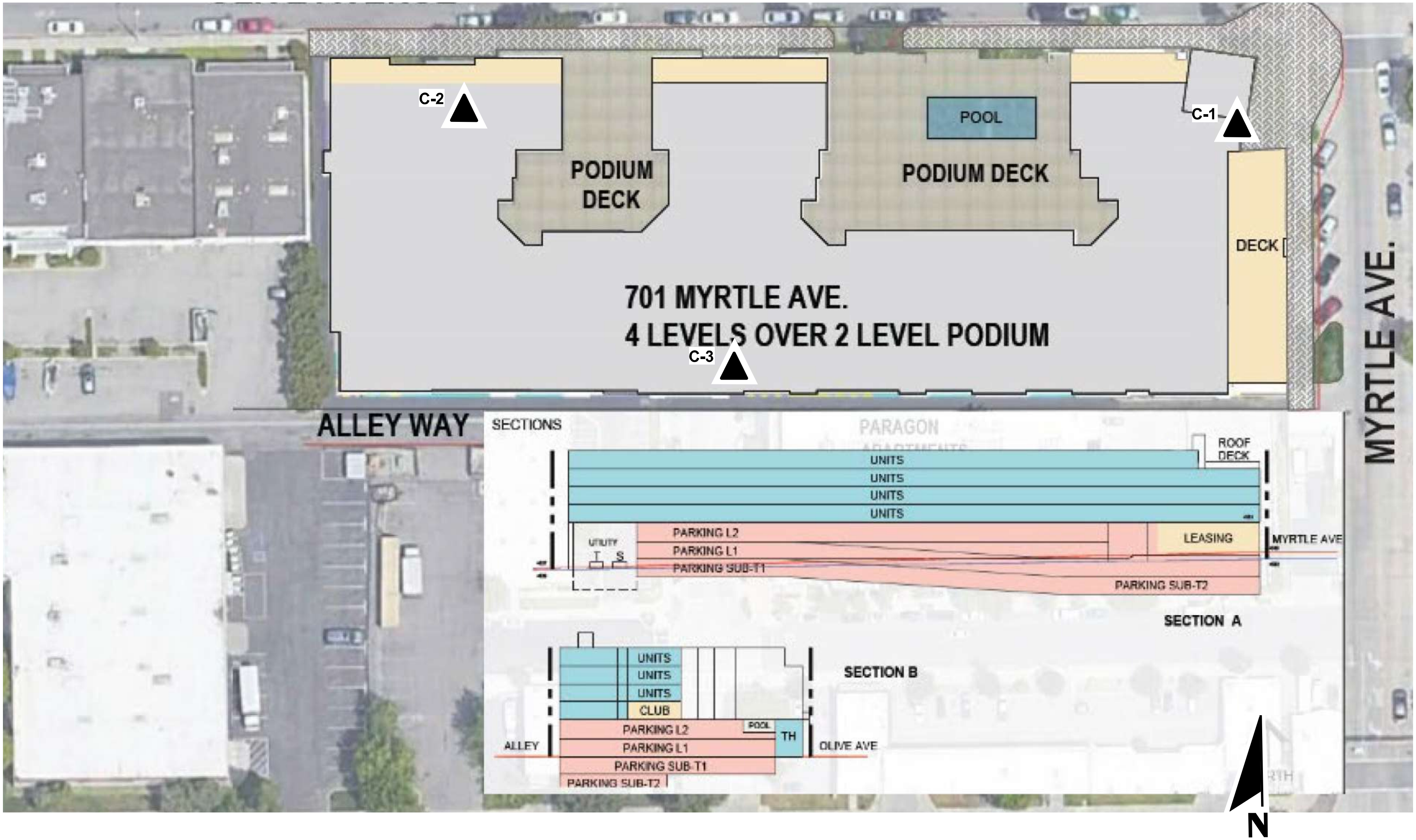
SCALE: 1" = 1000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

C-3 ▲ APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

0 60 120 FEET

BASE MAP REPRODUCED FROM PROPERTY OVERVIEW PLAN PROVIDED BY BLAKE GRIGGS PROPERTIES: NOT DATED

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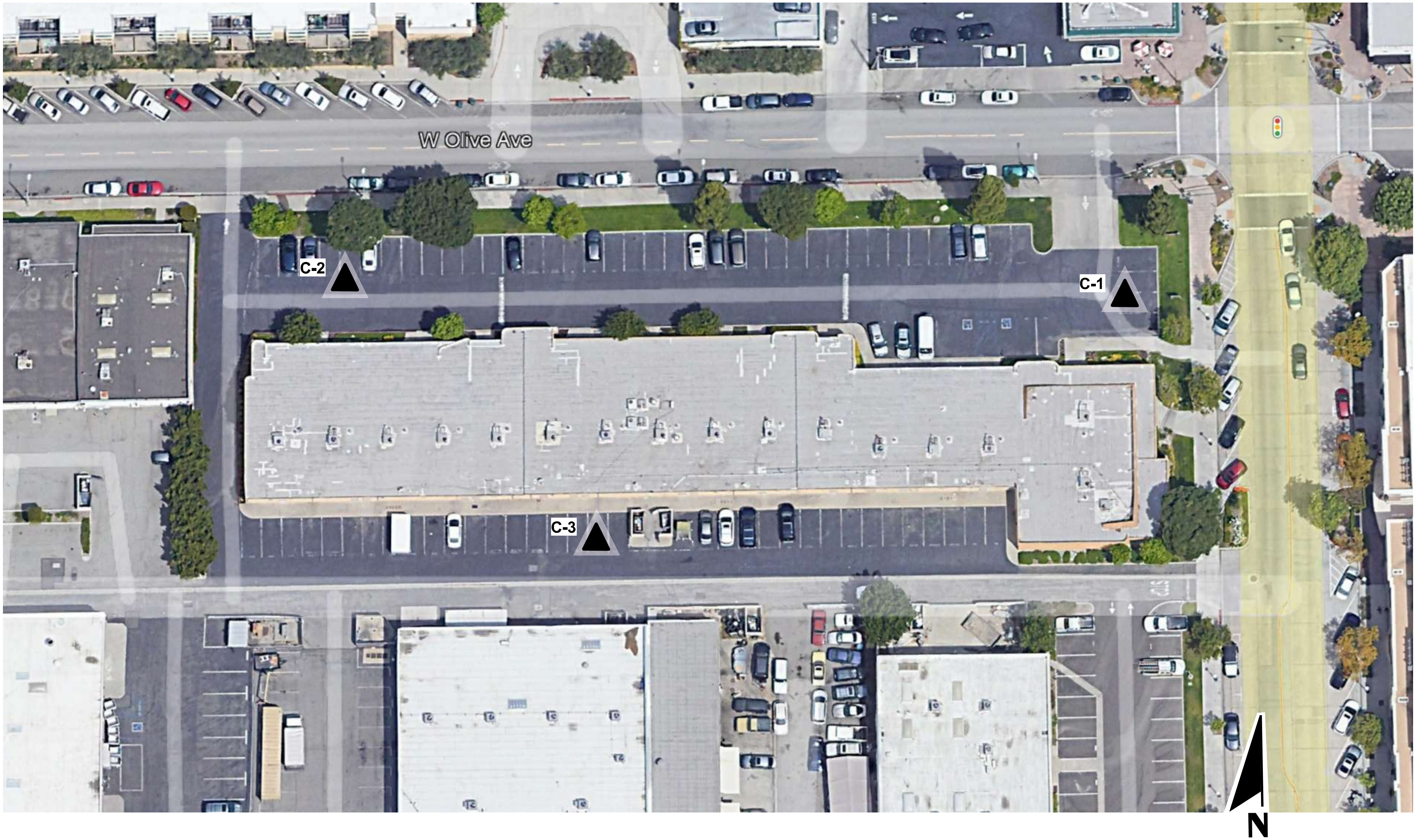
SITE PLAN

(PROPOSED CONDITIONS)

FIGURE 2



EXPLANATION

C-3 ▲ APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

0 60 120 FEET

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2022

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SCALE: 1" = 60'

SITE PLAN

(EXISTING CONDITIONS)

FIGURE 3

APPENDIX

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing three Cone Penetration Tests (CPT's) at the site. The soundings were advanced to a ranging from approximately 40 feet to 71 feet below existing grade. The locations of the CPTs are shown on the Site Plans, Figures 2 and 3.

The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT described in this report was conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface.

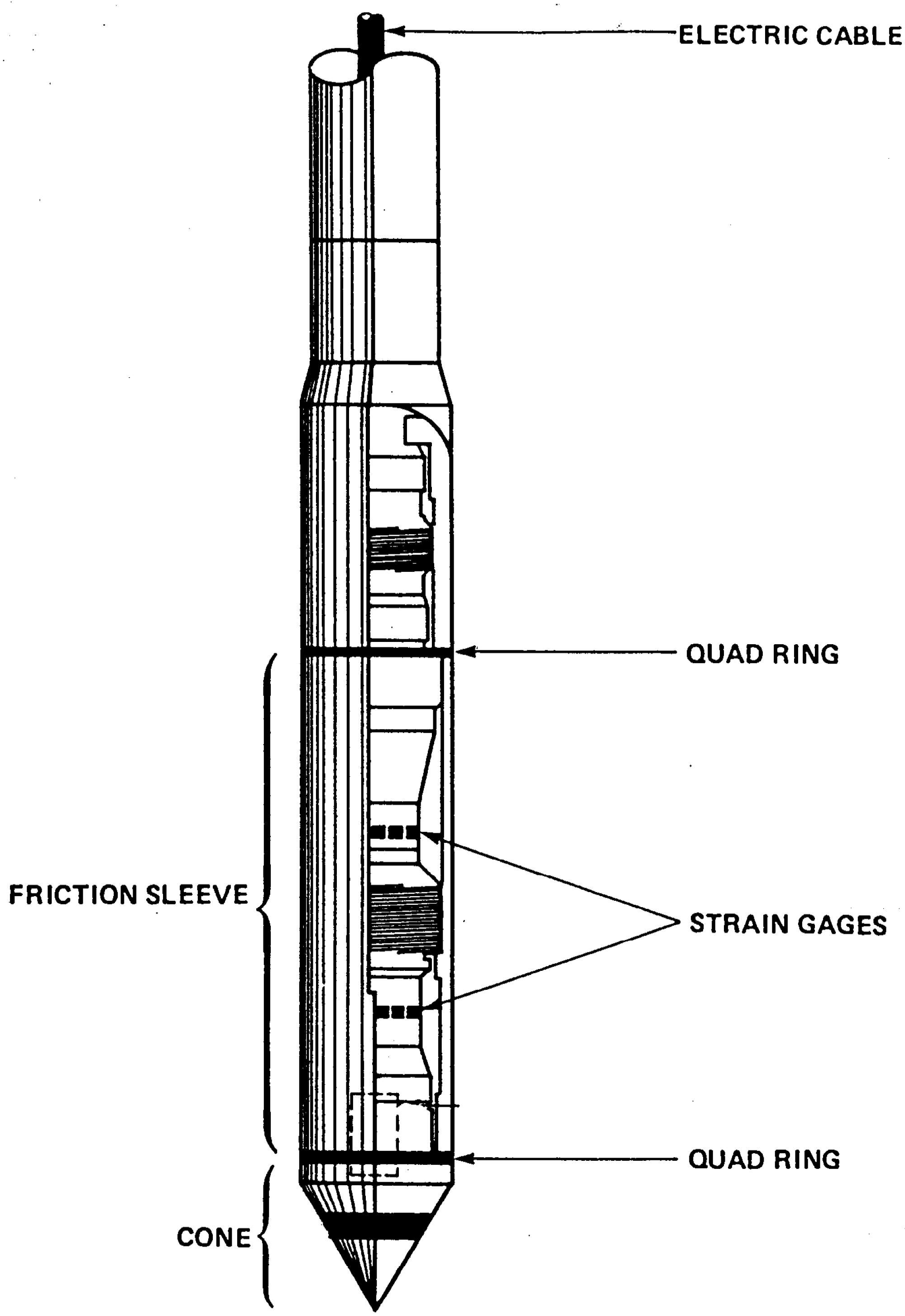
Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations, which utilize the CPT data.

A computer plot of the reduced CPT data acquired for this investigation is presented in Figures A-2 through A-4 of this appendix. The field testing and computer processing for the current investigation was performed by Kehoe Testing under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

A seismic cone penetration test provided shear wave velocity measurements of the soil profile. A standard cone penetrometer is equipped with two sets of geophones located approximately 3 feet apart on the cone penetrometer. At 10-foot intervals, a shear wave source is activated at the ground surface using an air-actuated hammer. A seismograph measures the travel time of the shear wave detected at each set of geophones. The time difference provides the velocity of the shear wave in the layer between the two geophone sets.

Seismic cone penetration tests were performed at CPT C-3. The cone penetration test at location C-3 was performed to refusal at 71 feet below existing grade and the subsequent data was used in order to estimate the average shear wave velocity for the upper 100 feet of soil profile. Table A-1 provides the shear wave velocity from the surface and the interval of soil between the geophones.

The CPT locations were laid out in the field by measuring from existing features at the site. The ground surface elevation at the CPT locations was estimated from Google Earth and should be considered very approximate.



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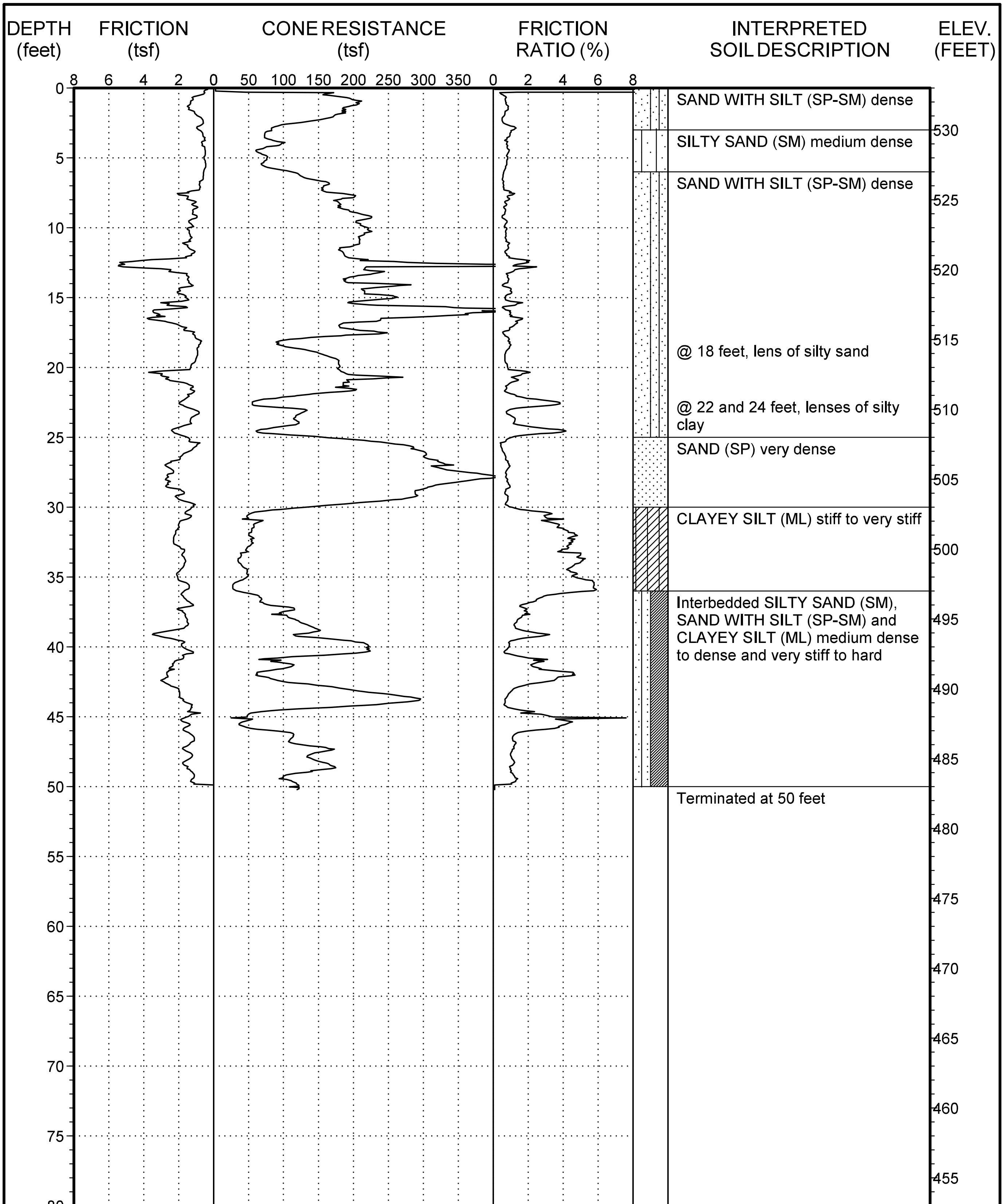
CONE PENETROMETER

FIGURE A-1



Date performed: 10-20-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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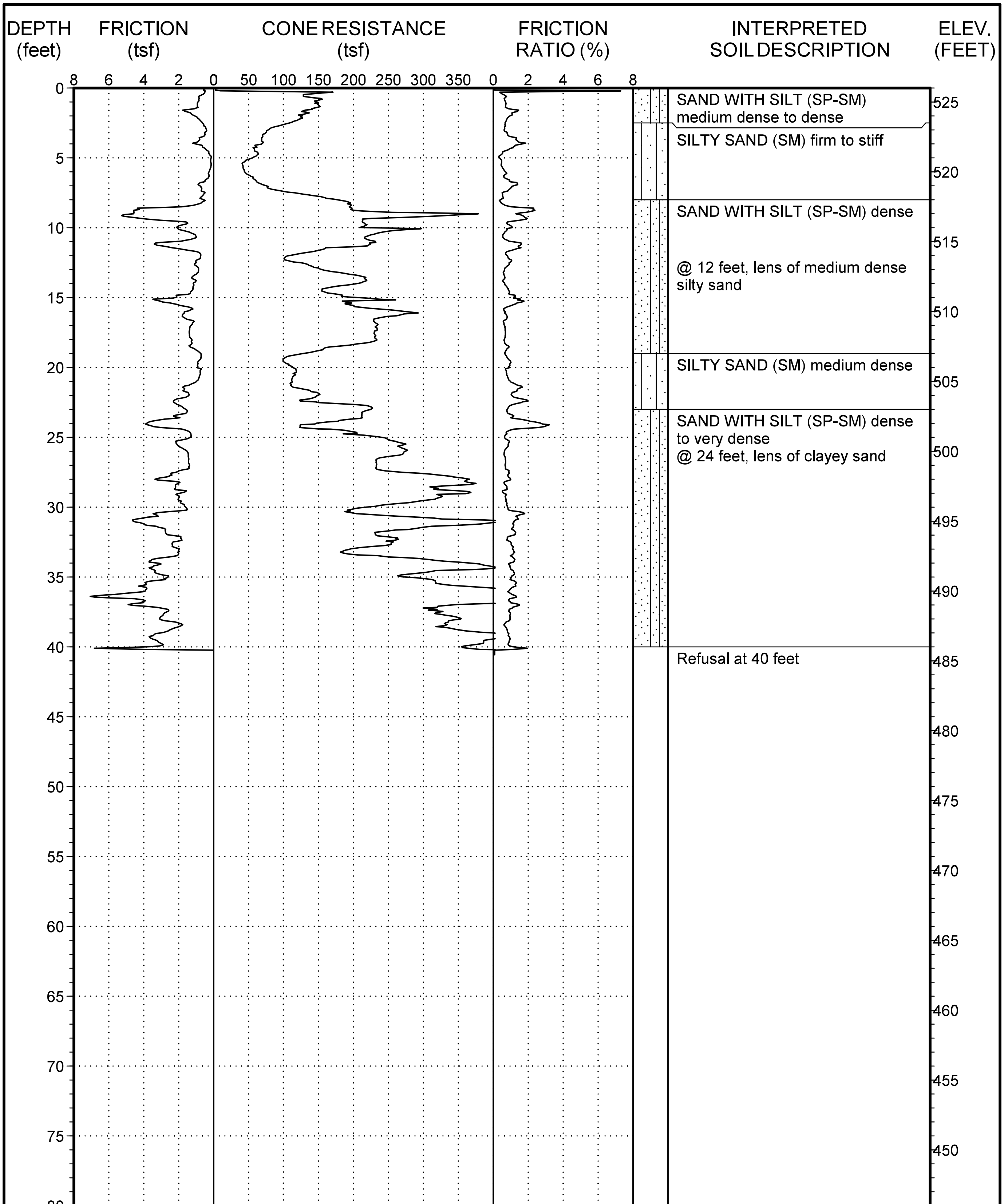
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:10-20-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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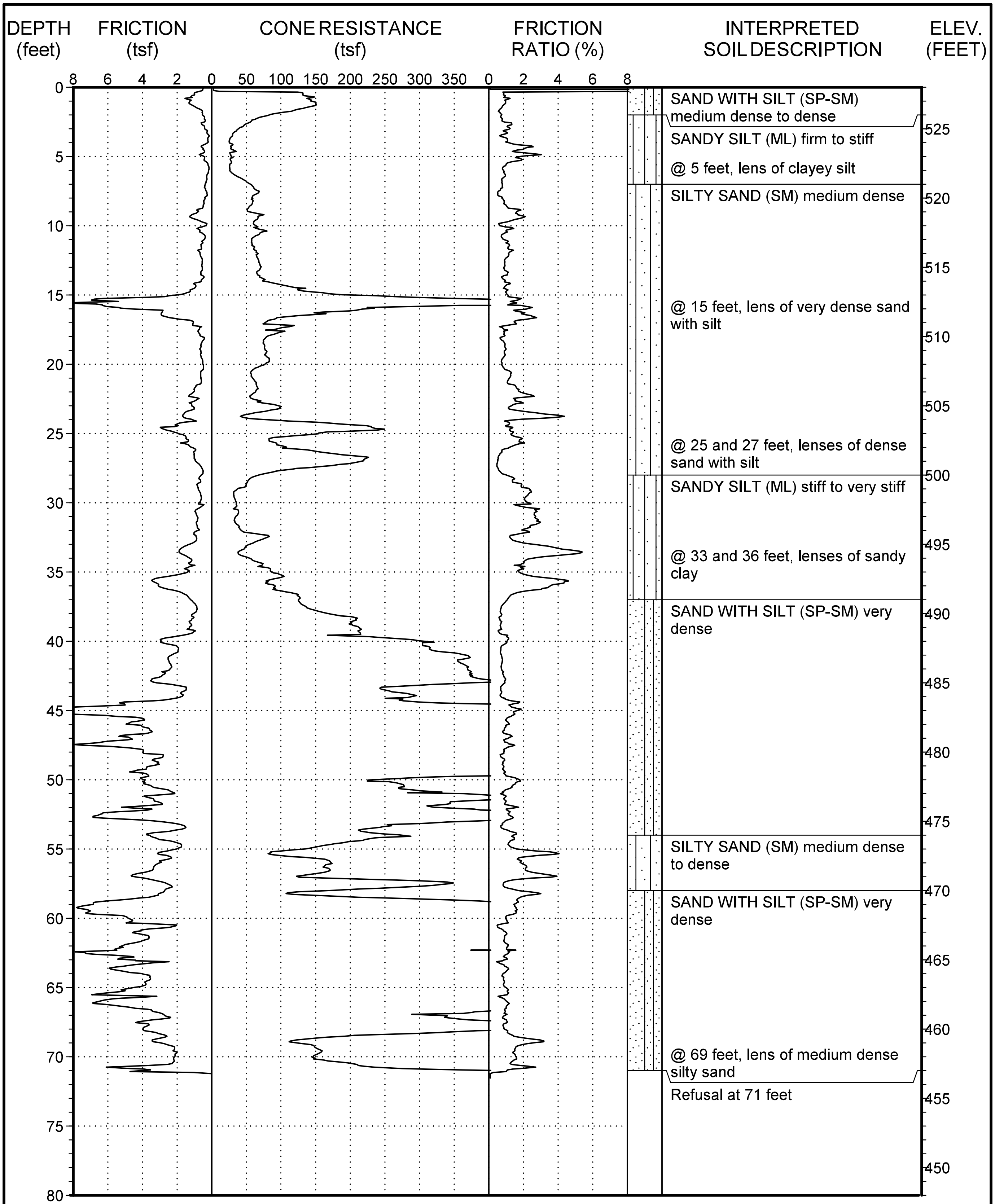
LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 10-20-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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LOG OF CPT NO. C-3

FIGURE A-4