Appendix F

Preliminary Geotechnical Investigation

GEOTECHNICAL PROFESSIONALS INC.

November 10, 2022

Sares Regis Group
3501 Jamboree Road, Suite 3000
Newport Beach, California 92660

Attention: Dave Powers
Senior Vice President

Subject: Updated Planning-Level Preliminary Geotechnical Investigation
Proposed Sunset Vine - SV2 Mixed-Use Development
6260-6290 Sunset Boulevard, 1460-1480 North Vine Street, and
6251-6265 Leland Way
Hollywood Area, Los Angeles, California
GPI Project No. 2910.2I

Dear Dave:

In accordance with your request, this report presents the results of our updated planning-level preliminary geotechnical investigation for the proposed mixed-use development at the subject site. This report was prepared to support the entitlements phase of the currently proposed project. We understand the project team is preparing an environmental document focusing on obtaining a Sustainable Communities Project Exemption (SCPE). This report supersedes our December 21, 2020 Planning Level Report and is based on updated project information provided by Eyestone Environmental and conceptual plans prepared by TCA Architects and dated August 30, 2022.

The purpose of our investigation was to determine, based on existing published data and prior limited subsurface exploration, if significant geotechnical constraints currently affect the site. We conducted three cone penetration tests (CPTs) in 2018 as part of our prior study. The results of the CPTs are included in Appendix A.

PROJECT DESCRIPTION

The project site is located at the southeast corner of Sunset Boulevard and Vine Street in the Hollywood Area of the City of Los Angeles, California. Sares Regis Group is planning to construct a mixed-use development around the existing Sunset Vine Tower building at 1480 Vine Avenue. The location of the site is shown on the Site Location Map Figure 1.

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The Sunset Vine Tower is a 19-story building with 3 levels of below grade parking. The lowest existing parking level (Existing Basement Level B3) is at about Elevation +319 feet to +321 feet.

The project includes the development of a new 201,134-square-foot, eight-story mixed-use building consisting of 170 new residential units and 16,680 square feet of ground-floor commercial space. The planned improvements will include a podium-style project with two levels of subterranean parking south and west of the existing tower. A small portion of the building near the northeast corner of the site is planned to be supported at grade with no subterranean levels. The development will include retail space and parking on the ground floor and the 2nd floor and residential space and amenities on Levels 3 through 8. The lowest proposed parking level (Basement Level 2) will be at about Elevation +324.5 feet, approximately 20 to 24 feet below existing grades.

Excavations for the subterranean levels and foundations are anticipated to extend on the order of 24 to 28 feet below existing grades and be made close to existing property lines and existing adjacent structures. Shoring of the excavations will be required. Column loads are not available at this preliminary stage; we anticipate the maximum loads to be on the order of 800 to 1200 kips.

The proposed site configuration is shown on the Exploration Location Plan, Figure 2. Two cross sections through the proposed building are shown on Figure 3, Building Sections.

We understand that a new stormwater infiltration system is being considered. We anticipate the infiltration system will include dry wells extending below the proposed structure, if determined to be feasible. Additional details of the proposed stormwater infiltration system have not yet been established.

Our recommendations are based upon the above structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to either confirm or modify our recommendations.

SCOPE OF WORK

Our scope of work included review of published information, including prior limited subsurface explorations, engineering evaluations, and preparation of this updated planning-level preliminary geotechnical letter report. We reviewed the Special Studies Fault Zone maps and Seismic Hazard Zone maps as part of our study.

In November 2018, we performed three CPTs to evaluate subsurface conditions at the site. The approximate exploration locations are shown on Figure 2. The CPT's were performed to depths of approximately 51½ to 75 feet below existing grade. A description of field procedures and logs of the CPTs are presented in the attached Appendix.

Engineering evaluations were performed to provide planning-level recommendations and an assessment of seismic hazards. The results of our evaluations are presented in the remainder of the report.

SITE CONDITIONS

The site is bounded on the north by West Sunset Boulevard and on the south by Leland Way. The proposed site is located south and east of the existing Sunset Vine Tower building at 1480 Vine Avenue. The Sunset Vine Tower building is a 19-story building with 3 levels of below grade parking. The property due east of the project site was recently developed and consist of a 7-story residential building with 2 subterranean levels for parking.

The site is approximately 75,938 square feet (1.74 acres) in plan and currently occupied by 1 and 2 story retail/restaurant buildings, asphalt paved parking and drives, a pool area, and a subterranean parking level access ramp for the Sunset Vine Tower building.

Ground surface elevations gently slope downward from approximate Elevation +350 feet along West Sunset Boulevard to approximate Elevation +343 feet along Leland Way in the southeast corner of the site.

SUBSURFACE CONDITIONS

Our preliminary CPT field explorations were performed in 2018; as such the presence of fill soils, or lack thereof, could not be evaluated. With the planned construction including subterranean parking levels, the upper 20 to 24 feet of materials would be excavated and exported off-site. We expect this excavation to remove existing fill in the footprint of the proposed building.

Based on the CPTs conducted at the site, the site soils generally consist of 11 to 15 feet of loose to medium dense silty sand and firm sandy silt, underlain predominantly by interbedded layers of very stiff to hard clays and silts to the depth explored. Discontinuous layers of medium dense to very dense sands and silty sands, approximately 3 to 15 feet in thickness, were encountered at variable depths of 37 to 59 feet below grade. The soils encountered are comparable to conditions reported on nearby sites.

The natural soils in the site vicinity are geologically mapped as Quaternary sediments that include older and younger alluvial-fan deposits.

The holes resulting from our CPTs caved back to depths of approximately 37 to 46 feet below grade. Groundwater was not encountered above those depths. Historical high groundwater levels are reported to be on the order of 50 to 60 feet below the existing ground surface and dip downward to the north (CDMG, 2001), although the site topography ascends from south to north. It is possible that actual ground water is deeper than historical high ground levels, but this still needs to be determined.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation and experience with nearby, similar projects, it is our opinion that from a geotechnical engineering viewpoint it is feasible to develop the site as proposed. Our current report update does not include additional field explorations nor field infiltration testing. Additional field explorations, laboratory testing and analyses will be required to develop design-level geotechnical recommendations. The following sections provide the results of our updated planning-level preliminary geotechnical investigation for the proposed mixed-use development.

UPDATED GEOLOGIC-SEISMIC HAZARDS

Seismic Design

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

We understand the seismic design of the proposed development will be in accordance with the 2022 California Building Code (CBC) and future 2023 Los Angeles Building Code (LABC). It is our understanding that the applicable sections for seismic design between the 2022 CBC and the most recent 2019 CBC will remain unchanged as they are both based on ASCE 7-16. Accordingly, for the 2019 CBC (and subsequent 2022 CBC and 2023 LABC), a Site Class D may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate internet website (https://seismicmaps.org/), the corresponding seismic design parameters from the CBC are as follows:

$S_S = 2.109g$	$S_{MS} = F_a * S_S = 2.109g$	$S_{DS} = 2/3 * S_{MS} = 1.406g$
$S_1 = 0.747g$	$S_{M1} = F_V * S_1 = 1.270g$	$S_{D1} = 2/3 * S_{M1} = 0.847g$

In accordance with the 2019 CBC and 2022 CBC, site-specific response spectra are required for structures located in a Site Class D (with S_1 greater than or equal to 0.2) unless, per the exceptions detailed in Section 11.4 8 of ASCE 7-16, the structure is designed using seismic response coefficient (Cs) determined by either:

- Equation 12.8-2 for values of $T \leq 1.5\ T_S$,
- 1.5 times the value computed by Equation 12.8-3 for values of $T_L \geq T > 1.5\ T_S$, or
- 1.5 times the value computed by Equation 12.8-4 for values of $T > T_L$.

If this exception is not taken and the structure will still be designed in accordance with the 2019 and 2022 CBC, GPI should be notified that site-specific response spectra is requested. Based on the mapped seismic parameters, the T_S period is approximately 0.6 seconds (therefore $1.5*T_S$ is approximately 0.9 seconds).

The above seismic code values should be confirmed by the Project Structural Engineer using the value above and the pertinent internet website and tables from the building code. The Project Structural Engineer should also evaluate the period of the proposed structure with respect to the T_S value above when reviewing whether a site-specific response analysis will be requested.

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 published information (earthquake.usgs.gov), the most significant fault in the proximity of the site is the Hollywood Fault, which is located about 0.34 miles north of the subject site.

Based on the OSHPD website (https://seismicmaps.org/), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 1.00g for a magnitude 7.0 earthquake (Hollywood Fault). This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. 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 development does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geologic Survey (CGS) or within a Preliminary Fault Rupture Study Area (PFRSA) as designated by the City of Los Angeles, which would require further studies of the fault. Therefore, ground rupture due to faulting is considered unlikely at this site. The closest fault to the site is the Hollywood fault, which is mapped about 0.34 miles to the north. The site is located outside of and approximately 0.25 miles south of the Alquist-Priolo Earthquake Fault Zone for the Hollywood fault.

Liquefaction, Seismic Settlement, and Seismic Induced Landslides

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 not located within a zone identified as having a potential for liquefaction nor earthquake induced landslide zones by the State.

Seismic ground subsidence, not related to liquefaction, occurs when loose, granular soils above the groundwater are densified during strong earthquake shaking. The 2020 LABC and ASCE 7-16 (ASCE, 2017) require that the ground motion used to evaluate liquefaction and seismic settlement be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCE_G) peak ground acceleration for Site Class B and a site coefficient, F_{PGA}. Accordingly, we considered a ground acceleration of 1.00g for a magnitude 7.0 earthquake as detailed previously.

Based on our preliminary analyses, we computed a potential dry seismic settlement of less than ¼ inch below the planned subterranean levels. The differential seismic settlement is also estimated to be less than ¼ inch across a span of 40 feet. The potential seismic induced settlement of the upper 11 to 15 feet of loose silty sand materials is expected to exceed tolerable levels if left in place.

Methane

The site is not located in either a Methane Zone or Methane Buffer Zone as mapped by the City of Los Angeles (NavigateLA; LADPW, 2004).

Expansive Soils

The clay soils that will be encountered at the lowest subterranean level are expected to have a relatively low (possibly medium) expansion potential and are not anticipated to impact the structure. Additional testing should be conducted during the design level investigation. Because of the poor drainage characteristics of the fine-grained clays and silts, these soils are not considered suitable for use as backfill behind retaining walls.

Sedimentation and Erosion

The majority of the ground surface at the site is relatively level and is, or will be, covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site. During construction, provisions should be in place to mitigate potential temporary erosion and sedimentation conditions.

EARTHWORK CONSIDERATIONS

Because the field exploration program was limited to CPTs, the presence of existing fills soils, or lack thereof, was not evaluated. It is likely, due to past development activities, some shallow fill exists at the site. Existing fills will need to be removed within building areas and replaced with properly compacted fill, where not removed by subterranean construction.

The planned excavation for subterranean parking levels will likely remove existing undocumented fills and low-density upper soils across the majority of the site. Removals are anticipated for remedial grading to support minor at-grade structures on spread footings. For planning purposes, we anticipate that it will be required to remove and recompact the materials within the upper 2 to 5 feet prior to constructing minor at-grade structures (screen walls, etc.).

Based on our preliminary findings the earthwork can be performed using conventional rubber-tired equipment. This should be further evaluated during a design level geotechnical investigation.

Additional laboratory testing should be conducted during the design level geotechnical investigation to evaluate the expansion potential of the clay soils that will be encountered at the lowest subterranean level. These potentially expansive soils are not anticipated to impact the structure if a subterranean level is planned but may need to be considered in the support of at grade floor slabs and exterior hardscape. Because of the poor drainage characteristics of the fine-grained clays and silts, these soils are not considered suitable for use as backfill behind retaining walls.

Groundwater is not anticipated to significantly impact the construction or long-term maintenance of the development.

The existing buildings on site are anticipated to be supported on shallow spread foundations. Shored excavations adjacent to spread footing supported buildings to remain will need to include surcharge loading due to the adjacent building floor slabs and foundations.

If the existing buildings to be demolished are supported on piles, care should be taken during site demolition to reduce the disturbance of the in-place soils. Removal of the piles should only include the upper portion of the piles (to within 5 feet of the bottom of proposed foundations), after excavating the adjacent soils and cutting the concrete and steel. Removal should not include bending or breaking the piles or pulling in an attempt to remove the entire element.

FOUNDATIONS

Based on our findings, we anticipate the proposed building may be supported on either spread footings (isolated and continuous) or a mat foundation underlain by the undisturbed, very stiff to hard clays and silts encountered at the anticipated subterranean levels. The at-grade portion of the building may need to be supported on deepened spread footings or deep foundations such as drilled piers or auger cast piles in order to address potential differential settlement between the at-grade and subterranean portions of the building.

An allowable net bearing capacity for spread footings on the order of 4 kips per square foot (ksf) for the undisturbed natural materials at the subterranean level or 2 ksf for properly compacted fill (minor at-grade foundations) is anticipated. We anticipate a mat foundation would have bearing pressures ranging from approximately 1,200 to 2,500 pounds per square foot (psf). Settlement analyses based on the actual design loads and the configurations of the proposed building foundations relative to the adjacent existing structures should be conducted during the design level investigation.

Resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of foundations and underlying soils and by passive soil pressures acting against the embedded sides of the foundations. For preliminary design, a coefficient of friction of 0.35 may be used and an allowable lateral bearing pressure equal to an equivalent fluid weight of 275 pounds per cubic foot may be used (to a maximum of 2,750 psf). This assumes the foundations are poured tight against compacted fill and/or undisturbed natural soils. These values may be used in combination without reduction and may be increased by 1/3 for short term loading conditions.

Details regarding adjacent structures and foundations should be considered in site planning and design so as not to remove support of adjacent structures during construction or impose additional surcharge loading and/or settlement on the adjacent structures. Spread and mat foundations should extend below a 1:1 plane drawn upward from the bottom of the adjacent footing to avoid surcharging the adjacent structure. Alternatively, deep foundations such as auger cast piles or drilled piers would be required to support the proposed structure if the proposed structure foundations would surcharge the adjacent structures or induce excessive settlement on the adjacent properties.

WALLS BELOW GRADE

Preliminary lateral earth pressures for use in conceptual design of walls below grade are provided below:

Wall Condition	Equivalent Fluid Pressure
Cantilever walls with level, drained backfill comprised of non-expansive granular (free-draining) soils (Static Active Pressure Condition)	40 pcf (1)
Restrained basement walls with drained conditions (Static At-Rest Pressure Condition)	65 pcf (1)
Additional lateral pressure increases for seismic loading (PGA = 1.0g) Active Pressure Condition: At-Rest Pressure Condition:	 33 pcf 8 pcf
Surcharge Pressures	See below

Note (1) does not include hydrostatic pressures or seismic earth pressures.

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. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the walls due to normal street traffic.

SHORING

Where there is not sufficient space for sloped embankments, which appears to be the majority of the site, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied-back with earth anchors. Driven or vibrated soldier piles may also be a considered if potential vibration impacts on adjacent structures is mitigated. The tie-back anchors may require permission from adjacent property owners and be subject to requirements of the City of Los Angeles. Rakers providing support to the soldier piles from inside the excavation would be an option if tie-backs are not allowed or suitable.

PAVEMENTS

The upper silty sands and deeper clays are anticipated to have R-values on the order of 30 and 5, respectively. Where pavements are supported by the upper silty sands recompacted as engineered fill, we anticipate an asphalt pavement section of 3 inches of asphalt concrete over 7 inches of aggregate base for access driveways and portland cement concrete drives on the order of 7 inches thick. Where pavements are supported by the deeper clay soils, we anticipate an asphalt pavement section of 3½ inches of asphalt concrete over 10 inches of aggregate base for access driveways and portland cement concrete drives on the order of 8 inches thick.

STORMWATER INFILTRATION

We anticipate the infiltration system will include dry wells extending below the proposed structure and into the more permeable sand layers encountered at depth, if determined to be feasible. The depth and thickness of the sand layers that would be optimal for infiltration varied in our explorations and in nearby explorations by others.

Additionally, the infiltration of storm water into subsurface soils should also not adversely impact foundation supporting soils. The City of Los Angeles guidelines require the system invert to be set back at least 10 feet from foundations if the infiltration of stormwater will not impact foundation bearing soils. This should be further evaluated during the design-level geotechnical investigation.

The depth to actual groundwater will also impact the maximum depth of the infiltration system. The City of Los Angles requires the invert of stormwater infiltration system to be at least 10 feet above the groundwater elevation and that the groundwater elevation may be based on current ground water elevations. According to state maps, historical high groundwater levels are reported to be on the order of 50 to 60 feet below the existing ground surface and dip downward to the north. Current groundwater levels have not been established at the site. It is possible that actual ground water levels are deeper than historical high ground levels, but this still needs to be determined.

Accordingly, our recommendation to further assess storm water infiltration is to drill a boring at or close to the perspective dry well location to:

- Identify the depth and thickness of target sand layer(s) for potential infiltration at that location,
- Measure the depth to current ground water so that a possible deeper ground water surface could be used for infiltration design, and
- Conduct infiltration testing at this location in the target sand layer.

Although the ground surface elevation is highest along Sunset Boulevard (the northern property boundary) and lowest along Leland Way (the southern property boundary), it may be advantages to plan the dry well locations near Sunset Boulevard because the groundwater is anticipated to be deeper along Sunset Boulevard (the northern property boundary) than Leland Way (the southern property boundary).

LIMITATIONS

The planning-level preliminary geotechnical investigation reported herein was performed for the exclusive use by Sares-Regis Group 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 are 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 - Exploration Location Plan
Figure 3 - Proposed Building Sections
Appendix - Cone Penetration Tests

REFERENCES

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

2. City of Los Angeles, 2004, Department of Building and Safety (LADBS), “Methane Code: Ordinance 175790,” effective date March 29, 2004

3. City of Los Angeles, 2016, Preliminary Fault Rupture Study Areas, dated March 16, 2016, http://geohub.lacity.org/datasets/9a1a1c350c9043a2b2fce10c0530f769_2

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6. Geocon West, Inc, Proposed Multi-Family Residential Development, 6250 Sunset Project, 6234-6258 West Sunset & 6235-6249 Leland Way, Los Angeles, California, Tract: TR 5840, Lots 1-8, Project A9202-06-01, dated October 6, 2016.

7. Geotechnical Professionals Inc., Planning-Level Preliminary Geotechnical Investigation, Proposed Apartment Development, 6266 West Sunset Boulevard, Los Angeles, California, GPI Project No. 2910.I, dated December 19, 2018

8. Martin, G.R. and Lew, M., (1999), “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California,” Southern California Earthquake Center Publication.

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10. Seed, H.B. and DeAlba, P., (1986), "The Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands," Proceedings of Use of In-Situ Tests in Geotechnical Engineering, ASCE Special Publication No. 6, pp 281-302.

11. Youd, T.L. and Idriss, I.M. (1997), “Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,” Technical Report NCEER-97-0022.

0 2000 4000 FEET

BASE MAP REPRODUCED FROM GOOGLE MAPS © 2018

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SUNSET VINE - SV2

GPI PROJECT NO. 2910.2I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1

EXPLANATION

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

BASE MAP REPRODUCED FROM SITE PLAN SHEET A-1.3 BY TCA ARCHITECTS, DATED 8/30/22

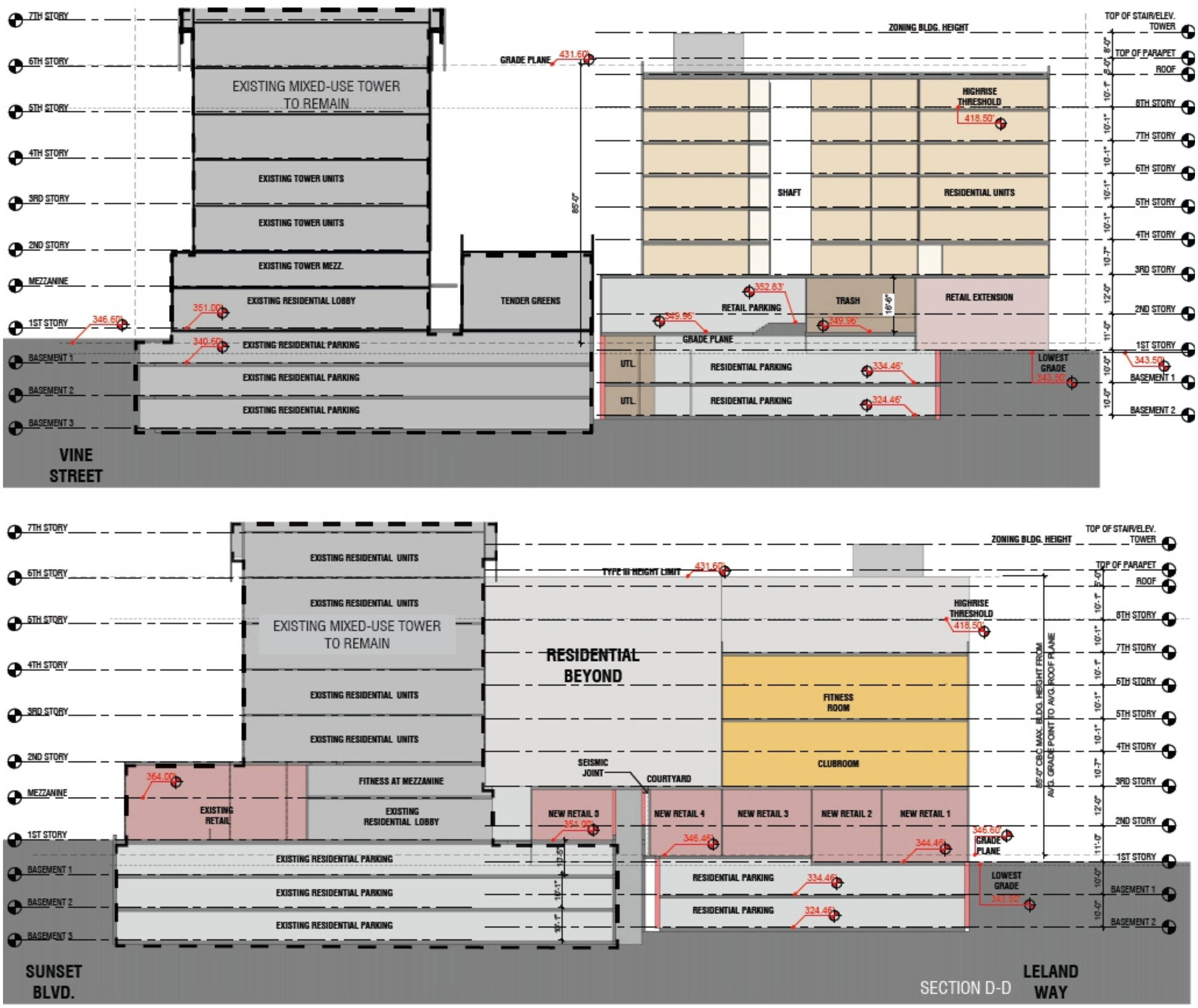
GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET VINE - SV2

GPI PROJECT NO. 2910.2I | SCALE: 1" = 50'

EXPLORATION LOCATION PLAN

FIGURE 2



0 50 100 FEET

BASE PLAN REPRODUCED FROM BUILDING SECTIONS C & D
SHEET A-2.2 BY TCA ARCHITECTS, DATED 8/30/22

GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET VINE - SV2

GPI PROJECT NO.: 2910.2I | SCALE: 1" = 50'

PROPOSED BUILDING SECTIONS

FIGURE 3

APPENDIX

APPENDIX

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing three cone penetration tests (CPT’s) at the site on November 29, 2018. The soundings were advanced to depths of 52 to 75 feet below existing grades. The approximate locations of the CPT’s are shown on the Exploration Location Plan, Figure 2.

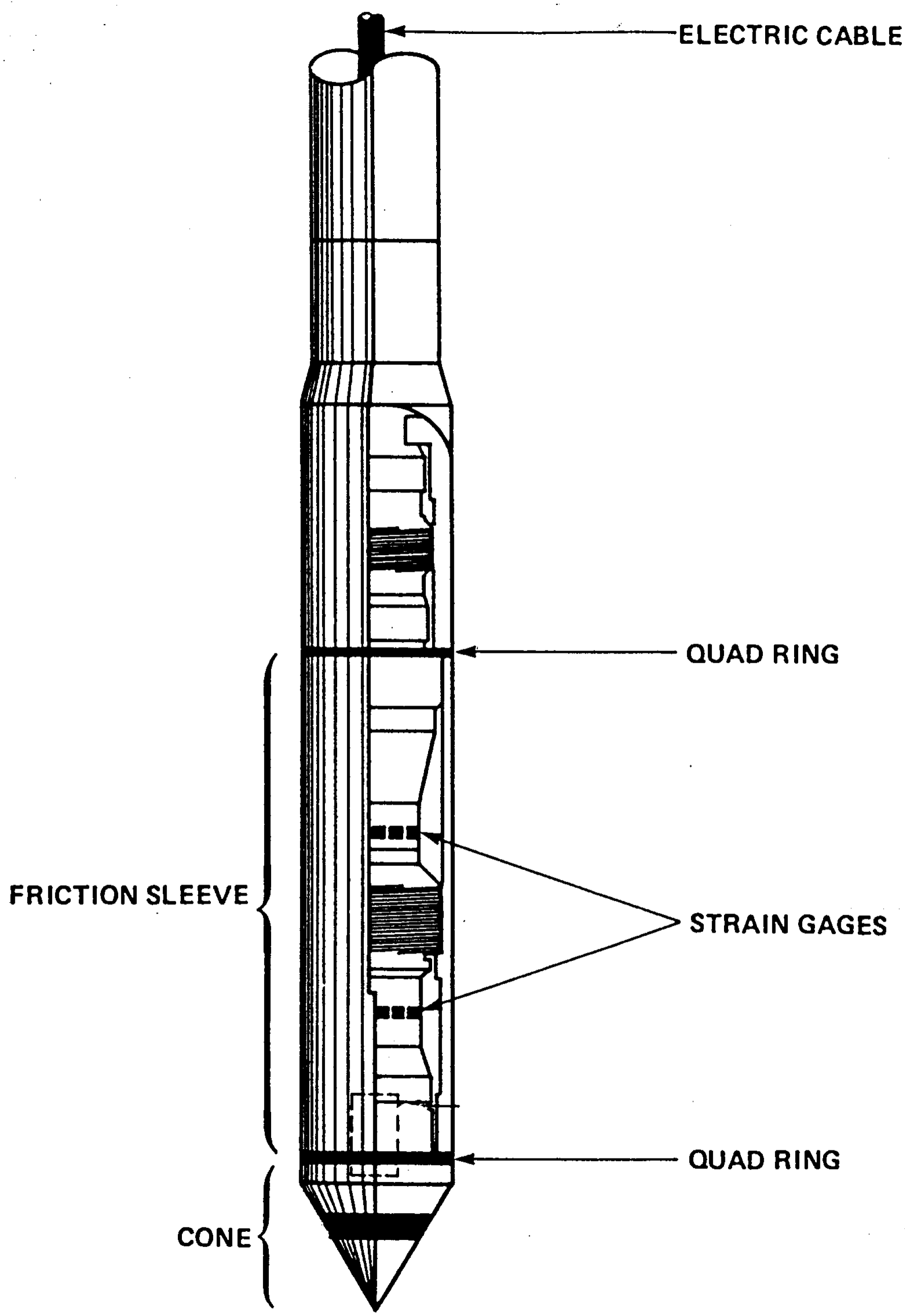
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D5778) 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.

Computer plots of the reduced CPT data acquired for this investigation are 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.

The CPT locations were laid out in the field by measuring from existing features at the site. Upon completion, the CPT hole was backfilled above caving with a bentonite plug and capped with quick set grout. The ground surface elevations at the CPT locations were estimated from Google Earth and should be considered very approximate.



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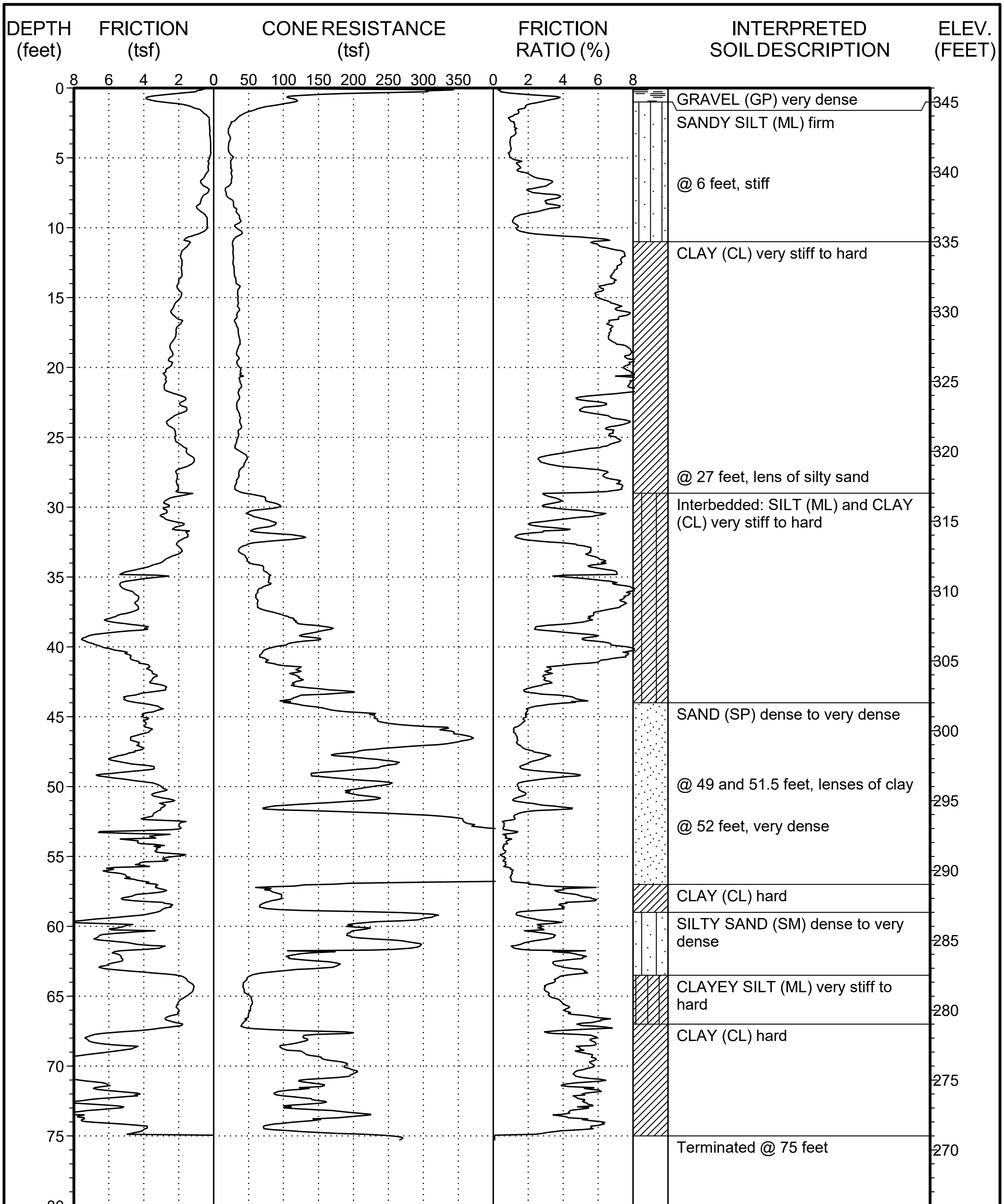
CONE PENETROMETER

FIGURE A-1



Date performed: 11-29-18

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.

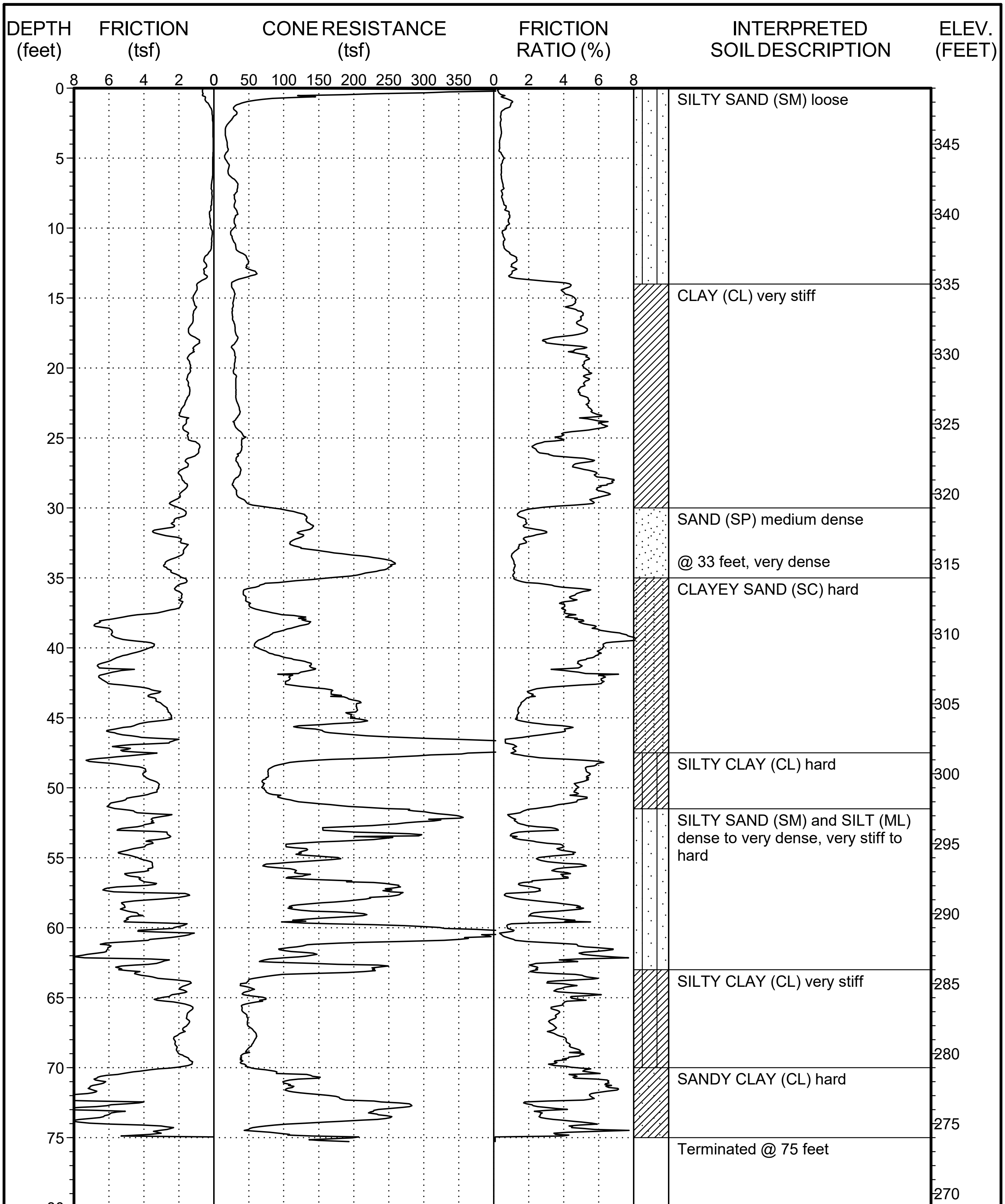
GPI

PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-1

FIGURE A-2



Date performed:11-29-18

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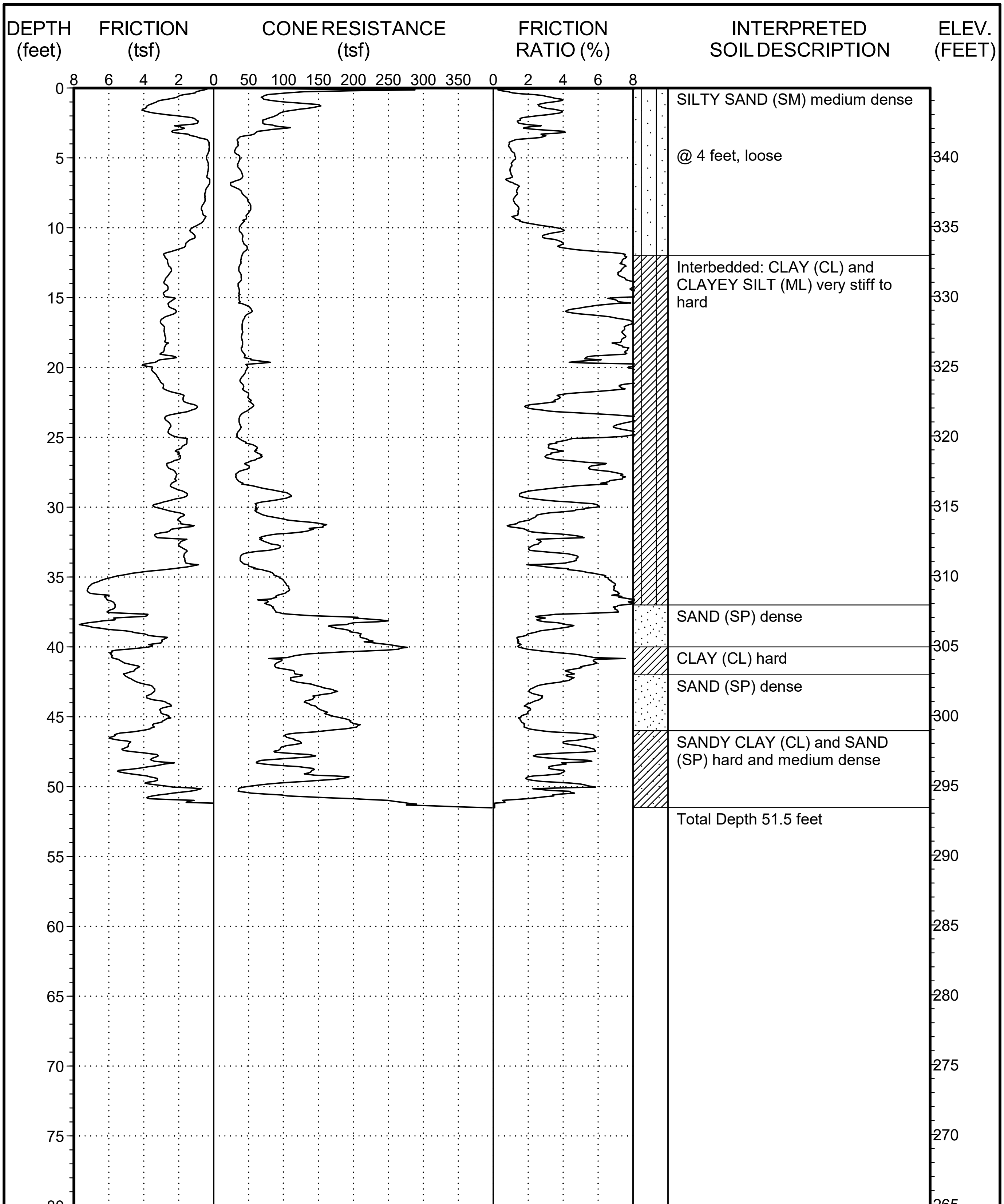
GPI

PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-2

FIGURE A-3



Date performed:11-29-18

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GPI

PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-3

FIGURE A-4