

September 09, 2024

Mr. Christopher Phillips
Keith
 301 East Atlantic Boulevard
 Pompano Beach, FL 33060

Re: Revision No. 1 - Report of Subsurface Exploration and Geotechnical Engineering Study
Gary Matzner Park
 6610 N. Kendall Drive
 Pinecrest, Florida
 NV5 Project No.: 18595

Dear Mr. Phillips:

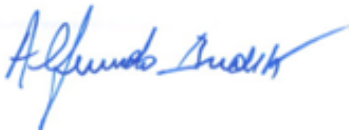
NV5, Inc. submits this report in fulfillment of the scope of services described in our Proposal No. 24-0493 dated July 08, 2024. The work was authorized by a client-provided professional services subconsultant agreement (Keith Project No. 13861.00) dated August 16, 2024. This report describes our understanding of the project, presents our evaluations, and provides our professional opinions and recommendations for foundation design and construction for the project.

NV5 previously performed a geotechnical study for the project and presented the results in a September 04, 2024 report titled *Report of Subsurface Exploration and Geotechnical Engineering Study, Gary Matzner Park, 6610 N. Kendall Drive, Pinecrest, Florida, NV5 Project No. 18595*.

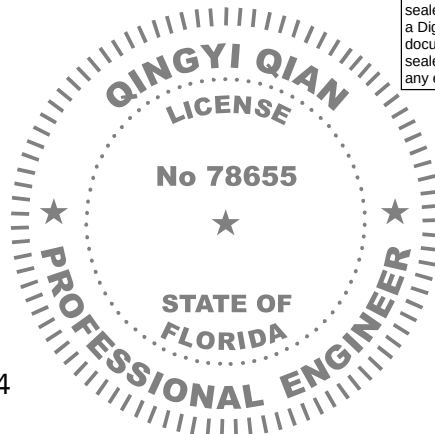
This current report includes updated boring locations.

This report should be read in its entirety. The user of this report is encouraged to contact NV5 to discuss our findings and to ensure clarity regarding the evaluations and recommendations contained herein.

Sincerely,
 NV5, Inc.



Alfredo Budik, P.E.
 Senior Engineer
 Florida License No. 43884



This item has been digitally signed and sealed by Qingyi Qian on 09/10/2024 using a Digital Signature. Printed Copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Qingyi Qian

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FIGURES

Drawing 1	Site Vicinity Map & Test Location Plan
Drawing 2	Boring Summary Sheet

APPENDICES

Appendix A	Boring Log Data (A-1 through A-6)
Appendix B	Field Permeability Test Data (B-1)



1.0 SITE AND PROJECT INFORMATION

The project site is located at 6610 N. Kendall Drive in Pinecrest, Florida. The site is bounded to the north by N. Kendall Drive, residential structures to the south, SW 65th Court to the east and SW 67th Avenue to the west. A site vicinity map is presented on Drawing 1. Based on the Miami-Dade County Property Appraiser's webpage, the site corresponds to Folio Numbers 20-5001-000-0180 and 20-5001-000-0780. We were provided with a boundary and topographic survey drawing prepared by Keith dated December 29, 2023. The site is trapezoidal in shape (curbed corners to the north) with longest dimensions in the north-south direction of about 295 feet and in the east-west direction of about 430 feet. The site is vacant. Site grades are estimated at about Elevation +9 to +11 feet with respect to the 1929 National Geodetic Vertical Datum (NGVD).

We were provided with a site plan drawing titled Gary Matzner Park prepared by Keith dated May 31, 2024. We understand the project consists of the construction of a park with a 1-level facility and restrooms, two play structures and a paved parking lot. Basement construction is not planned for this project. We estimate column and wall loads on the order of 75 kips and 4 kips per linear foot, respectively. We assume ground floor slabs will be loaded to around 150 pounds per square foot. When project drawings and structural loading are available, that information should be provided to us.

2.0 PURPOSE AND SCOPE OF WORK

The purpose of our services on this project is to perform a subsurface exploration and engineering analyses, and to provide recommendations for foundation design and construction. Specifically, this report provides:

- ◆ Drawings showing boring locations, a graphic summary of the generalized subsurface conditions, and boring logs with detailed descriptions of the materials encountered.
- ◆ Discussion of generalized subsurface conditions at the site including groundwater levels and hydraulic conductivity.
- ◆ Discussion of feasible foundation type(s) for the proposed construction.
- ◆ Design parameters for the recommended foundation type, including vertical and lateral load resistance.
- ◆ Estimates of foundation settlements.
- ◆ Modulus of subgrade reaction for any slabs on grade
- ◆ Recommendations for site preparation and grading, including the re-use of site-excavated materials for fill, fill placement and compaction, and slab subgrade preparation.
- ◆ Construction considerations including excavation support and dewatering, existing structure foundations, and impacts for adjacent structures.

3.0 FIELD EXPLORATION

NV5 performed a field exploration program comprising test borings and field permeability test at the approximate locations depicted on Drawing 1, and described below. The test locations were marked and identified in the field by NV5. It should be noted that the test locations shown are approximate. If accurate as-built test locations are required, they should be surveyed.



3.1 BORINGS

The subsurface conditions at the site were explored with a total of four (4) test borings at the approximate locations shown on Drawing 1. Three (3) borings were advanced to around 15 feet and one (1) to 10 feet below ground surface.

The borings were drilled with truck-mounted drill rig utilizing the rotary wash method. Samples of the subsurface materials were recovered at roughly 2-foot intervals within the upper 10 feet, and at approximately 5-foot intervals thereafter, where applicable, using a Standard Penetration Test split-spoon sampler (SPT) in substantial accordance with ASTM D-1586, "Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils." This test procedure drives a 1.4-inner-diameter split-tube sampler into the subsurface using a 140-pound hammer falling 30 inches. The total number of blows required to drive the sampler the second and third 6-inch increments is the SPT N-value, in blows per foot, and is an indication of material strength. Upon completion of the borings, the boreholes were backfilled with soil cuttings and the upper few feet closed with cement grout.

The soil/rock samples recovered from the borings were initially classified in the field. The collected samples were later re-examined in the laboratory to confirm field classifications. Visual soil classifications were made in accordance with ASTM D2487 and ASTM D2488. The results of the classification and consequent generalized stratification of the borings are shown in Drawing 2, the boring summary sheet, and in the records of test borings in Appendix A (sheets A-1 through A-6). Strata contacts shown on these drawings are approximate. The boring data reflect conditions at the specific test locations only, and at the time the borings were drilled.

3.2 FIELD PERMEABILITY TEST

NV5 performed also one (1) field permeability (drainage) test to 15 feet deep at the location shown on Drawing 1. The test was performed in general accordance with the South Florida Water Management District's *Usual Open Hole Procedure*. Test result is presented in Section 5 and Appendix B (Sheet B-1)

4.0 LOCAL GEOLOGY AND GEOLOGIC HAZARDS

4.1 LOCAL GEOLOGY

Miami-Dade County is located on the southern flank of a stable carbonate platform on which thick deposits of limestones, dolomites and evaporites have accumulated. The upper two hundred feet of the subsurface profile is composed predominantly of limestone and quartz sand. These sediments were deposited during several glacial and interglacial stages when the ocean was at elevations higher than present.

In many portions of Miami-Dade County, surface sand deposits of the Pamlico Formation are encountered. The Pamlico sands overlie the Miami Limestone. In western Miami-Dade County, portions of the Everglades Region interfinger with the Pamlico sand. The Everglades soil consists of peat and calcareous silt (marl).

The Miami Limestone is a soft to moderately hard, white, porous to very porous, sometimes sandy, oolitic calcareous cemented grainstone. The formation outcrops in portions of Miami-Dade County. The Miami Limestone has a maximum thickness of about 35 feet along the Atlantic Coastal Ridge



and thins sharply near the coastline and more gradually in a westerly direction. The Miami Limestone was formed about 130,000 years ago at a time when the sea level was twenty-five feet higher than it is today. This environment facilitated formation of concentrically layered sand sized carbonate grains called oolites. These grains formed by repeated precipitation of calcium carbonate around the nucleus of a sand or shell grain.

The Miami Limestone can be separated into two facies: the barrier bar oolitic facies and the tidal shoal limestone facies. The barrier bar facies are characterized by lenses of oolitic limestone separated by intermittent, 1-inch thick or less, uncemented sand layers (cross-bedded limestone). Zones of higher porosity are characteristic and parallel the bedding planes of the cross-bedded limestone. The tidal shoal limestone facies are characterized by a distinct lack of bedding planes. In addition, burrowing organisms have churned previously deposited sediments, which have resulted in high porosity channels in the rock. These ancient channels give the rock an appearance of a hardened sponge in some areas.

The Fort Thompson Formation underlies the Miami Limestone, and includes sand, sandstone, and limestone. The upper zones of the Fort Thompson Formation consist of sand having a thickness ranging from 5 to 35 feet. The remainder of the formation consists of coralline limestone, quartz sandstone, sandy limestone and freshwater limestone. The type of soils within the formation and the degree of cementation vary with lateral extent and depth.

The Fort Thompson Formation is underlain by the Tamiami Formation. The Tamiami Formation consists of sands, silts, clays, and sometime fossiliferous limestone. The upper portions of the Tamiami Formation are permeable and make up the lower reaches of the Biscayne Aquifer. This formation ranges in thickness from zero to 300 feet in South Florida.

4.2 GEOLOGIC HAZARDS

The South Florida area is relatively free of geologic hazards. The region is not considered seismically active. Consequently, hazards such as ground shaking, liquefaction, lateral spreading, and ground rupture that are normally associated with earthquakes and other seismic activity are generally not a factor for the design of structure foundations in South Florida.

Karst topography that is associated with the formation of sinkholes and other underground discontinuities in carbonate rock formations in the central and northern portions of Florida is generally not found in South Florida. Any discontinuities in the limestone due to solutioning of the rock are typically very limited in vertical and lateral extent and are usually not considered a factor in the design of foundations in the local practice.

5.0 SUBSURFACE CONDITIONS

5.1 BORINGS

In general, the subsurface conditions encountered in the borings are consistent with the geology described above. The detailed subsurface conditions are presented graphically in the attached boring summary sheet, and in more detail on the records of test boring sheets. We note that the tops of boring elevations have been estimated. For accurate elevations, the boring locations should be surveyed.

Layer 1 – Sand/Limestone Fragments:

This layer consists of one (1) to 4 feet of light brown and light brownish yellow limestone fragments with sand. Light gray to gray sand was recovered above the limestone fragments in borings B-2 and B-3, and gray silty sand in boring B-4. Topsoil about 2-inch thick was encountered at the top of the borings. SPT N-values of Layer 1 ranges from 11 to 74 blows per foot (bpf). The average of the recorded SPT N-values is around 44 bpf.

Layer 2 – Limestone:

This layer consists of light brown limestone found at one (1) to 4 feet below grade and extends to 10 and 15 feet below the existing grade. The thickness of the limestone varies from six (6) to 14 feet. The recorded SPT N-values in this layer range from six (6) to 47 bpf and the average of the recorded SPT N-values in the stratum is about 30 bpf.

Layer 3 – Sand:

This layer consists of light brownish gray sand found at 10 feet below grade in borings B-1 and B-2. The sand extends to the termination depth of the borings at 15 feet below the existing grade. The thickness of the sand is at least five (5) feet where encountered. The recorded SPT N-values in this layer are four (4) and 9 bpf and the average of the recorded SPT N-values is about 7 bpf.

For the layers described above, Table 1 below summarizes our estimates of engineering parameters considered pertinent to the design of foundations for low and mid-rise structures.

TABLE 1 - SUMMARY OF ESTIMATED PERTINENT ENGINEERING PARAMETERS

Layer ID	Description	Thickness (ft.)	SPT N-values		Modulus of Elasticity (ksf)	Unconfined Compressive Strength (ksf)
			Range	Avg.		
1	Sand/Limestone Fragments	1 - 4	11 - 74	44	450	-
2	Limestone	6 - 14	6 - 47	30	10,000	100 - 300
3	Sand	0 - 5+	4 - 9	6	200	-

We note that the values of allowable side shear estimated in Table 1 above are based on our experience with similar rock.

Groundwater

Groundwater was encountered in the borings at depths of about 6.7 to 7.9 feet below the existing ground surface. It should be noted that groundwater readings during drilling might not represent stabilized groundwater levels. Stabilized water levels would be best obtained by installing groundwater monitoring devices and taking readings over an extended period. NV5 can provide these services if they are of interest to the project development team.

The water depths reported above for the borings correspond to approximate elevations of +1.1 to +1.3 feet NGVD. On average, stabilized groundwater levels in the general vicinity of the project are expected to vary between elevations 0 and +4 feet NGVD, the variations being primarily as a result of seasonal rainfall and tidal fluctuations in nearby water bodies. Nonetheless, it should be noted that groundwater levels outside of this range could be encountered during construction. Storm and hurricane events and construction activities also result in variations in the groundwater levels. Notwithstanding the variations

acknowledged, we anticipate that groundwater at the site will generally be encountered within the upper 10 or so feet of the existing ground surface.

5.2 FIELD PERMEABILITY

The result of the open-hole field permeability (drainage) test performed at the site is presented in the table below:

TABLE 2 – SUMMARY OF FIELD PERMEABILITY TEST RESULT

Test ID	Test Depth (ft.)	Hydraulic Conductivity (cfs/ft ² -ft. head)
P-1	15	7.71×10^{-04}

It should be noted that the above result is un-factored and represent the conditions at the test location at the time of the test. To account for potential variations in hydraulic conductivity across the site the designer should apply an appropriate safety factor to the reported value. The permeability test data is presented in Appendix B.

6.0 EVALUATION AND DISCUSSION

6.1 FOUNDATION SUPPORT

Based on the results of the field exploration and our engineering analyses, we consider the site suitable for the proposed construction from a geotechnical engineering perspective. The primary concerns for foundation design and construction include support of the proposed new structure loads without unacceptable settlement, and minimizing the impacts of new construction to existing structures. Foundation options are discussed below. Detailed recommendations for foundation design and construction are presented in Section 7 of this report.

The proposed up to 1-level improvements can be supported on shallow foundations bearing on either the surficial sand/limestone fragments of Layer 1, or on the exposed limestone of Layer 2.

6.2 ESTIMATED SETTLEMENT

We estimate that shallow foundations that are properly designed and constructed as recommended herein will experience total settlements of one (1) inch or less and maximum differential settlements of ½ inch. Given the granular nature of the subsurface materials at the site, we predict settlement will occur coincidental with the application of the structure dead and live loads and for a short time after. The above settlement estimates are for foundations that are designed and constructed as recommended herein.

6.3 GROUND LEVEL SLAB

Ground floor slabs can be supported on grade assuming the site has been prepared as recommended herein.

6.4 IMPACTS TO ADJACENT STRUCTURES

Potential adjacent structure of concern for the proposed development includes the single-family residence to the south of the project site. Impacts to adjacent structures during construction

generally come from one of three (3) sources, namely settlement, ground movement due to nearby excavations, or vibrations.

6.4.1 Settlement Impacts

We do not expect the loads of the proposed 1-story improvements to cause significant areal settlements outside the foundation footprints.

Settlements can also derive from drawdown of groundwater levels due to dewatering. This is usually an issue for long-term dewatering by well-points. Dewatering is not expected to be required for this project. Nevertheless, detailed dewatering plan will be required to be developed by the contractor, if dewatering is needed, and it would be prudent to monitor adjacent foundations during any long-term dewatering activities.

6.4.2 Excavation & Ground Movement Impacts

Depending on the foundation construction details for adjacent structures, excavations within close proximity could result in settlement of existing foundations and floor slabs. Loss of ground around column pads could result in loss of lateral support around those foundation elements, which could be of consequence during say a storm or high-wind event. Additionally, slabs on grade could be impacted by loss of ground or nearby ground movement.

Considering the distance to the neighboring structures, settlement of existing foundations is unlikely.

6.4.3 Vibration Impacts

Construction-related vibrations could impact the existing structures as well. Such vibrations could derive from activities such as compaction, or even excavation/breaking up of the near surface limestone. In general, while such vibrations can be a nuisance to humans nearby, the damage caused to adjacent structures by vibrations from these activities are typically cosmetic in nature. Notwithstanding, methods that could potentially address mitigation of offsite vibration impacts and reduce complaints and damage to adjacent properties include modifying compaction procedures and techniques, and performing vibration monitoring at the structures during construction.

6.5 MISCELLANEOUS ENVIRONMENTAL IMPACTS

Environmental forces consist of sinkholes, freeze thaw damage, shrinking and swelling soils, and hurricane scour can affect the performance of a foundation system. Sinkholes, freeze-thaw, and shrinking/swelling soils are generally not of concern in the South Florida area.

While a detailed study of hurricane scour was outside the scope of this study, it is nonetheless our opinion that the foundation systems recommended herein when properly designed and constructed, will resist hurricane scour forces.

7.0 RECOMMENDATIONS

Our recommendations for geotechnical design and construction of the proposed project are provided below in the following sections.



7.1 SITE PREPARATION AND GRADING

1. Geotechnical site preparation for construction should consist of removal of all existing structures, foundations, pavements, underground utilities, and other deleterious materials within the proposed structure and pavement footprints plus a five-foot perimeter. Any voids created by the removal of these deleterious materials should be properly backfilled as described in the paragraphs below.

No information has been provided about previous or existing foundations at the site and we are not aware of the site development history beyond the current condition. Where old spread or other foundations are encountered, they should be removed and replaced with compacted fill if they interfere with new foundations or utilities. If the old foundations do not interfere with new construction they could be left in place. Backfilling of old foundation excavations should be performed in accordance with the recommendations provided in this report.

After site preparation as described above, areas for structures that will have slabs on grade or pavements should be proof rolled and densified. Proof-rolling should include planned development footprints plus a five-foot perimeter. The proof rolling should be observed by NV5 to identify and mitigate any weak subgrade conditions evidenced by yielding or rutting at the wheels of the roller. Proofrolling is not required where the limestone layer is exposed.

2. In general fill soils should consist of either inorganic, non-plastic sand having less than 10 percent material passing the No. 200 sieve, or crushed limestone with a maximum rock size of six (6) inches. In particular, fill soils placed within the upper 12 inches of the subgrade of building slabs on grade should consist of either sand with less than 10 percent passing the number 200 sieve, or crushed limestone with a maximum particle size of three (3) inches.

Based on our boring data the majority of the near-surface fill materials should satisfy the fill criteria. However, some materials might require localized sorting and moisture-conditioning prior to re-use. Topsoil and silty sand should not be used as structural fill. In any event, representative samples of the fill soils should be collected for classification and compaction testing. The maximum dry density, optimum moisture content, gradation, and plasticity should be determined. These tests are needed for quality control of the compacted fill.

3. Fill soils should be placed with loose lift thicknesses of not more than 12-inches, moisture-conditioned to within two (2) percent of the optimum moisture content based on ASTM D-1557, and compacted to a minimum 95 percent relative compaction¹. One test should be performed for each 2,500 square feet of fill area per lift of fill soils. If during the compaction process fill shows evidence of yielding under the weight of the roller, it should be removed and replaced with properly compacted granular fill as described herein. Fill particles exceeding one (1) inch in size should not be allowed to nest within the fill. Fill density tests are not required where the limestone layer is exposed.
4. The vibrations produced by the operation of the compactor should be monitored for potential adverse effect on adjacent existing structures, pavements, and utilities. If nearby structures

¹ Relative compaction refers to the in-place dry unit weight of a material expressed as a percentage of the maximum dry unit weight of the same material as determined in the laboratory using the Modified Proctor procedure (ASTM D1557).

will be affected by the vibration of the compactor, the compaction procedure may require modification as approved by the geotechnical engineer.

7.2 FOUNDATION SUPPORT

7.2.1 Proposed 1-Level Improvements

1. After preparing the site as described above the proposed structures may be supported on shallow spread foundations designed for the maximum allowable bearing pressure of **2,500 psf** bearing on either the limestone fragments or on the limestone layer.
2. Footings should be bottomed at least 18 inches below lowest adjacent grade.
3. To assure an adequate factor-of-safety against a general shearing failure, strip and continuous footings should be at least 18 inches wide, and isolated footings should be no less than 30 inches wide.
4. Lateral forces may be resisted by passive earth pressure acting on the vertical foundation faces and by friction acting between the bottoms of foundations and the supporting subgrade. We recommend using an equivalent fluid weight of 180 pounds per cubic foot (pcf) to compute passive resistance for moist soil above the water table, and 80 pcf to compute passive resistance in submerged soil. Passive resistance in the upper 12 inches of soil should be neglected unless it is confined by a slab or pavement. Frictional resistance may be computed using a factor of 0.30 times the sustained dead loads.

The above values include a factor of safety of at least 1.5. These values of resistance assume that the foundations are: 1) surrounded by in-situ materials densified by proof-rolling, clean sand fill which is compacted to 95 percent relative compaction, and 2) able to withstand horizontal movement on the order of $\frac{1}{4}$ to $\frac{3}{8}$ inch.

5. NV5 should inspect the footing excavations prior to placement to check for any unsuitable or weak materials in the footing subgrade. Any such materials will need to be removed and replaced with No. 57 stone or lean concrete.

7.2.2 Miscellaneous Structures

1. Lightly loaded miscellaneous structures such as guard huts, planters and small retaining walls be designed using an allowable bearing pressure of **2,000 psf**. The parameters presented above for lateral load resistance may be used in the design of these shallow footings. Footings must bear at a minimum depth of 12 inches below lowest adjacent grade. Continuous footings should be at least 16 inches wide and isolated footings should be at least 24 inches wide. Exposed bearing soils should be compacted to a minimum of 95 percent relative compaction.
2. With the shallow footing bearing pressure recommended above, we expect settlement of such footings for lightly loaded structures will be on the order of $\frac{1}{2}$ to $\frac{3}{4}$ inch, with differential settlement on the order of $\frac{1}{2}$ inch.

7.3 GROUND FLOOR SLABS

1. Ground floor slabs can be supported on grade provided the site is prepared as recommended under Section 7.1 *Site Preparation and Grading* above.
2. For design of slabs supported on grade we recommend a modulus of subgrade reaction of 150 pounds per cubic inch (pci).
3. Slabs should be reinforced for the loads that they will sustain and construction joints should be provided at frequent intervals.
4. A design groundwater level of +4 feet NGVD can be used for design of ground level and below grade slabs. Information for flood zone elevations (FEMA Flood Maps) is publicly available. We recommend that such information be relied upon for design flood water elevations for below-grade slabs. The design water levels should be the minimum flood elevations stated on the maps for the site or for nearby locations.
5. Slabs should be reinforced for the loads that they will sustain and construction joints should be provided at frequent intervals.
6. Slabs in contact with soil are subject to movement of moisture from the soil upward through the slab. To prevent such moisture vapor transmission, a moisture barrier should be placed on the slab subgrade, and should be protected from damage during construction. Construction joints should be provided with water stops in any permanently submerged areas.

7.4 EXCAVATION AND DEWATERING

1. Excavations into the near-surface materials will likely stand vertical for short periods of time only. The excavation sides will unravel over time as they are exposed to weather and construction traffic. In general, the Layer II limestone is expected to stand vertically unsupported if excavated. Localized weaker zones within the limestone layer could become loose if unsupported. Deeper excavations, especially those that extend below the groundwater table, as well as excavations that will remain open for longer periods of time will require support in the form of temporary shoring or sliding trench boxes to prevent instability of excavation walls and to protect workers from injury. All excavations should comply with Occupational Safety and Health Administration (OSHA) design and safety requirements. Shoring designs should be signed and sealed by a Florida-licensed professional engineer, and should be provided for the Owner's review.

Particular attention should be paid to any deep excavations and the potential impacts these could have on adjacent structures, especially where such excavations are close to project property lines.

2. Average groundwater elevation is expected to be approximately between Elevation 0 and +4 feet NGVD for this site. As stated above, groundwater levels outside this range could be encountered during construction. Dewatering is not expected for foundation construction. However, dewatering could be required for installation of deeper utilities and appurtenances. We judge that localized dewatering of foundation excavations can be accomplished using pumps and sumps. Dewatering of larger excavations and larger volumes could require the installation of well points or other dewatering systems.



It should be noted there are two components to the dewatering process. The first is extracting the water from the subsurface and the requirement of the project to maintain a dry excavation to allow construction to proceed. The other component is the ability to discharge the volume of water extracted. The contractor must ensure this capability exists for the site such that all dewatering and consequent effluent discharge will meet the requirements of the local jurisdictional agencies including Miami-Dade County, Florida Department of Environmental Protection (FDEP), Florida Department of Transportation, and South Florida Water Management District (SFWMD) as appropriate. This study did not include specific testing or analysis to determine if dewatering is feasible or if adequate discharge is available. Ultimately, dewatering of the site to facilitate construction is the contractor's responsibility.

During dewatering the adjacent properties must be monitored for adverse impacts from dewatering drawdown.

The dewatering subcontractor should submit a proposed design for dewatering operations to the owner for review and approval prior to commencing work.

7.5 OTHER RECOMMENDATIONS

1. NV5 should participate in the design development phases of this project in order to modify the recommendations provided above as changes occur during the design development process.
2. Construction activities could have adverse impact on structures outside the proposed structure footprints. We recommend that pre- and post-construction surveys of adjacent structures of concern be conducted to document conditions. We recommend also vibration monitoring be performed at adjacent structures during construction activities that could generate offsite vibrations.
3. NV5 should participate in the evaluation of field problems as they arise and recommend solutions. We should also be involved with site work activities so we can address needed changes to the foundation recommendations if site conditions different from those described herein are encountered. NV5 should observe and test the foundation installation to satisfy the requirements of the Florida Building Code and municipal agencies.

8.0 REPORT LIMITATIONS

This report has been prepared pursuant to approved proposal Consultant Agreement No. 24-0493 between Keith ("client") and NV5 dated July 08, 2024 and in general accordance with the standard of care ordinarily practiced by members of Consultant's profession performing similar services on similar projects in similar localities; no other warranty is expressed or implied. The report should be read in its entirety. NV5 is not responsible for misinterpretations arising from reading sections of the report only.

This report has been prepared for the exclusive use of the Owner and other members of the design/construction team for the specific site(s) and project(s) discussed in this report. The report should not be used for any other site(s) or project(s) without express written permission from NV5.

The evaluation and recommendations submitted in this report are based in part upon the data collected from the field exploration. These data were collected at specific locations and describe



subsurface conditions encountered at those specific locations at the time(s) the field explorations were made. Further, the plan area of the field test locations is relatively small as compared to the total site area. Consequently, subsurface conditions could be different at site locations other than those tested. The nature or extent of variations throughout the subsurface may not become evident until the time of construction. If variations later become evident, it may be necessary for NV5 to revisit the recommendations provided in this report.

In the event changes are made in the nature, design, or location(s) of the proposed project construction, the conclusions and recommendations contained in this report cannot not be relied upon unless the changes are reviewed by NV5, and the conclusions and recommendations herein are either verified or modified as needed in writing by NV5. Therefore, NV5 must be informed of any such changes if those changes are not addressed in this report. The scope of services performed by NV5 did not include any environmental assessment or investigation for the presence or absence of wetlands, sinkholes, chemically hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around the site.

NV5 should be retained to provide consultation to the ownership and design team during the design development phase of the project, to review final foundation specifications and review foundation design drawings in order to ascertain that its recommendations have been properly interpreted and implemented. Furthermore, NV5 should be retained to provide inspections during geotechnical construction. If NV5 is not selected to monitor/inspect foundation installation as recommended in this report, Keith agrees that NV5 shall not be responsible for another engineer's interpretation of the recommendations made in this report or for foundation performance resulting from another engineer's recommendations and/or foundation installation inspections. However, NV5 will continue to be responsible and liable for its data, analysis, recommendations, and designs provided in this report or in any subsequent reports or drawings issued by NV5.

9.0 CLOSURE

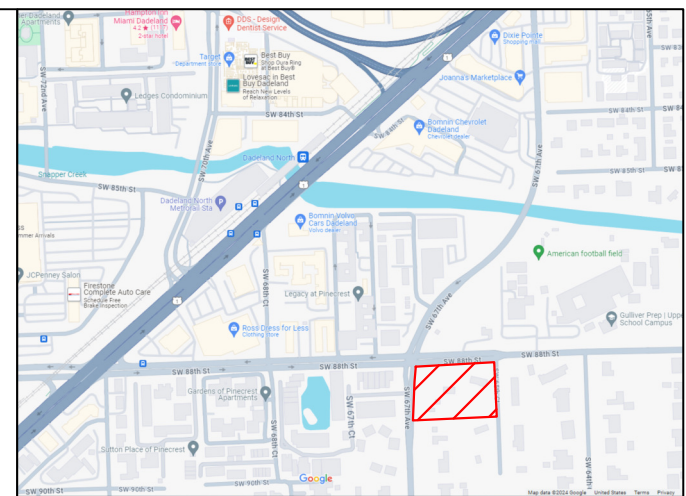
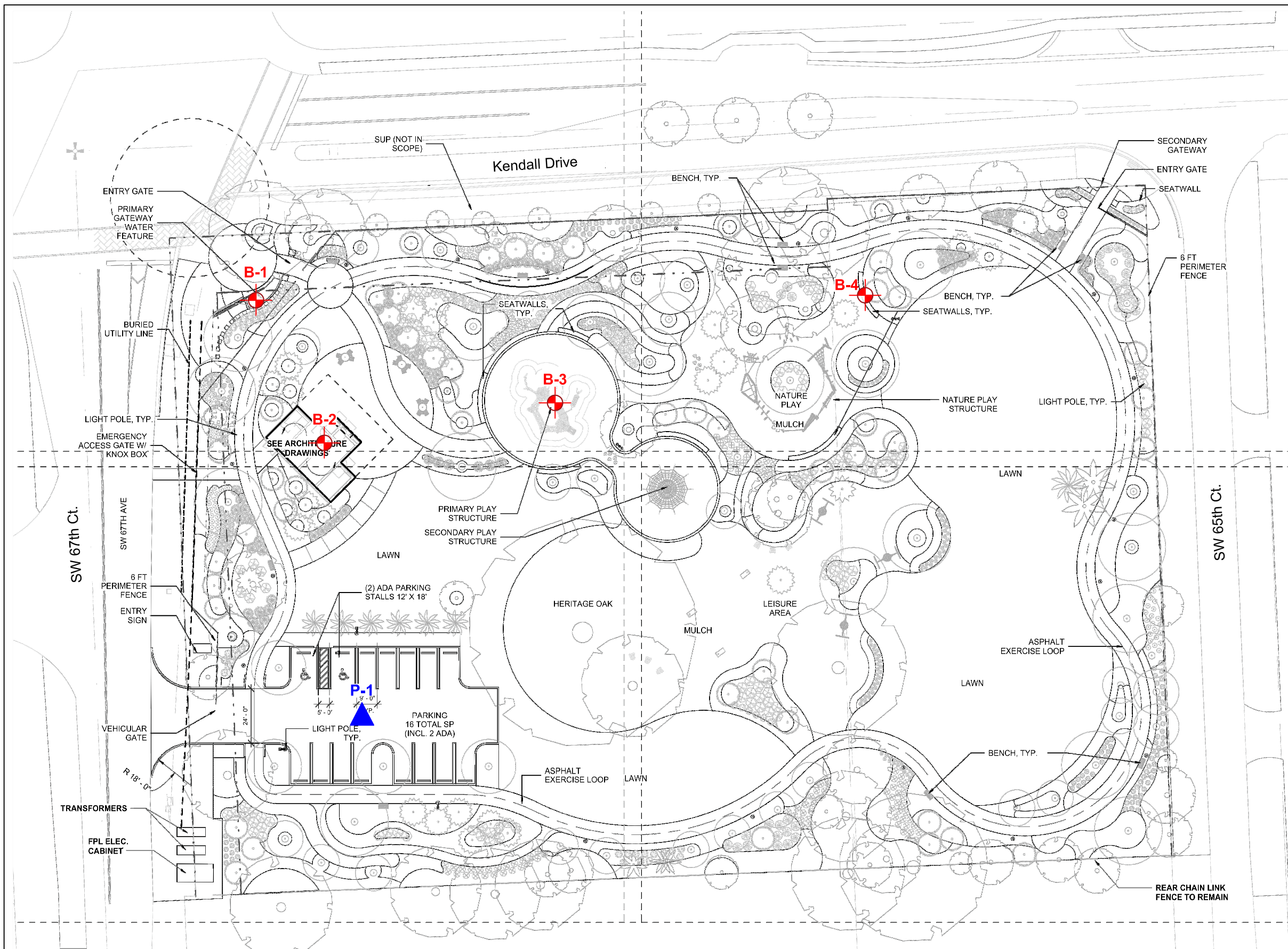
We appreciate the opportunity to provide specialized engineering services on this project and look forward to an opportunity to participate in construction related aspects of the development. If you have questions about information contained in this report contact the writer at 305.901-2151.



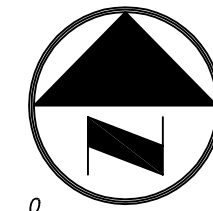
DRAWINGS

NIV5

CONSTRUCTION QUALITY ASSURANCE - INFRASTRUCTURE - ENERGY - PROGRAM MANAGEMENT - ENVIRONMENTAL



Site Vicinity Map



Approximate Scale in Feet

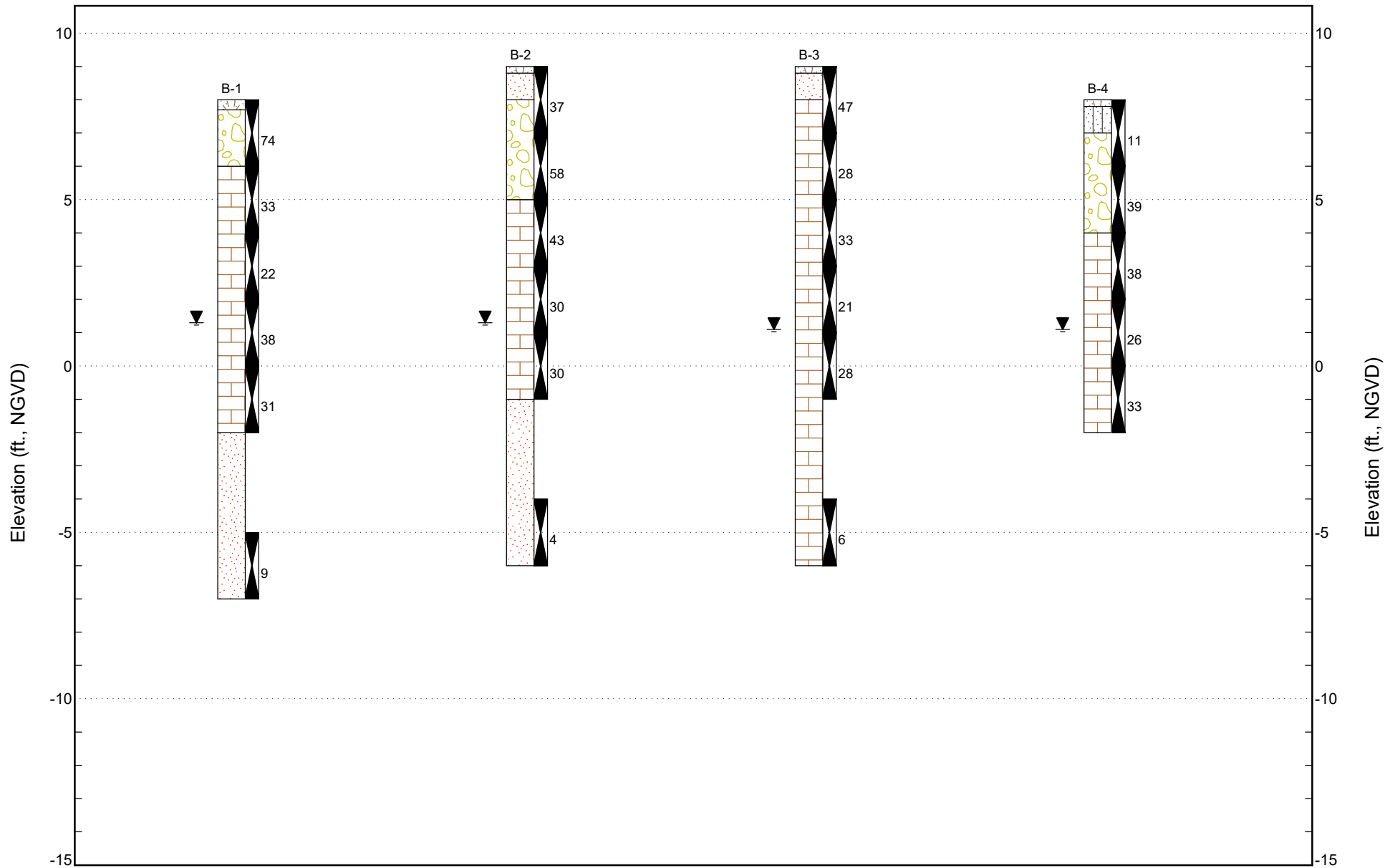
LEGEND:

- B-1** (red crosshair symbol) - Number & Approximate Location of Test Boring.
- P-1** (blue triangle symbol) - Number & Approximate Location of Percolation Test.

NOTES:

1. Test locations shown are approximate.
2. Test location symbols are not to scale.
3. Base drawing was taken from Sheet SP-101, Gary Matzner Park - Overall Site Plan, prepared by Keith, dated 08/09/2024.

NV5	DRAWING TITLE: Site Vicinity Map & Test Location Plan				DWN BY: ER	
	PROJECT NAME: Gary Matzner Park				CKD BY: AB	
	PROJECT LOCATION: 6610 N. Kendall Drive, Pinecrest, Florida			PROJECT NO: 18595	DATE: 09/09/2024	DWG NO: 1
APD BY: _____						



Note: Boring top elevations have been estimated



BORING SUMMARY SHEET

PROJECT NAME: Gary Matzner Park

PROJECT LOCATION: 6610 N. Kendall Drive, Pinecrest, Florida

PROJECT NUMBER: 18595

DATE: 09/03/2024

DRAWN BY: CR

CHECKED BY: AB

DRAWING NO: 2

LEGEND



Topsoil



Sand



Silty Sand



Limestone



Limestone Fragments



Standard Penetration Test
& SPT N-value



Water Level

APPENDIX A

BORING LOG DATA



PROJECT NAME Gary Matzner Park
PROJECT NUMBER 18595 **PROJECT LOCATION** 6610 N. Kendall Drive, Pinecrest, Florida
DATE STARTED 8/21/24 **COMPLETED** 8/21/24 **GROUND ELEVATION** 8 ft NGVD est. **HOLE SIZE** 3 inches
DRILLING CONTRACTOR NV5 **GROUND WATER LEVELS:** 6.7 ft / Elev 1.3 ft
DRILLING METHOD Rotary drill with mud, wash & casing
LOGGED BY J. Rivera/Y. Leyva **CHECKED BY** C. Rausseo
NOTES _____

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NGVD)
0							
	SPT	24	3-30-44-21 (74)	SM GP		LIMESTONE FRAGMENTS, very dense, light brownish yellow to light brown, with sand	7.7 6.0
	SPT	20	18-20-13- 13 (33)	LS		LIMESTONE, medium hard, light brown	5
5	SPT	24	12-10-12- 15 (22)			LIMESTONE, soft, light brown	
	SPT	24	15-25-13- 10 (38)			▼ LIMESTONE, medium hard, light brown	0
10	SPT	24	7-14-17-10 (31)			LIMESTONE, medium hard, light brown	-2.0
				SP			
15	SPT	20	3-3-6-5 (9)			SAND, loose, fine, light brownish gray, with a trace of limestone fragments	-5 -7.0

Boring terminated at 15.0 feet.

PROJECT NAME Gary Matzner Park
PROJECT NUMBER 18595 **PROJECT LOCATION** 6610 N. Kendall Drive, Pinecrest, Florida
DATE STARTED 8/22/24 **COMPLETED** 8/22/24 **GROUND ELEVATION** 9 ft NGVD est. **HOLE SIZE** 3 inches
DRILLING CONTRACTOR NV5 **GROUND WATER LEVELS:** 7.7 ft / Elev 1.3 ft
DRILLING METHOD Rotary drill with mud, wash & casing
LOGGED BY J. Rivera/Y. Leyva **CHECKED BY** C. Rausseo
NOTES _____

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NGVD)
0							
	SPT	24	13-9-28-22 (37)	SM SP		0.2 1.0 SAND, fine, gray	8.8 8.0
	SPT	24	30-30-28-30 (58)	GP		LIMESTONE FRAGMENTS, dense, light brown LIMESTONE FRAGMENTS, very dense, light brownish yellow to light brown, with sand	
5	SPT	24	25-18-25-25 (43)	LS		LIMESTONE, medium hard, light brown, with a trace of sand	
	SPT	24	12-16-14-16 (30)			▼ LIMESTONE, soft, light brown, with sand	
	SPT	24	8-18-12-8 (30)			LIMESTONE, soft, light brown, with sand	0
10							-1.0
				SP			
	SPT	20	3-2-2-4 (4)			SAND, very loose, fine, light brownish gray, with a trace of limestone fragments	-5
15							-6.0

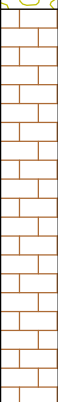
Boring terminated at 15.0 feet.



NOTES

Boring terminated at 15.0 feet.

PROJECT NAME Gary Matzner Park
PROJECT NUMBER 18595 **PROJECT LOCATION** 6610 N. Kendall Drive, Pinecrest, Florida
DATE STARTED 8/22/24 **COMPLETED** 8/22/24 **GROUND ELEVATION** 8 ft NGVD est. **HOLE SIZE** 3 inches
DRILLING CONTRACTOR NV5 **GROUND WATER LEVELS:** 6.9 ft / Elev 1.1 ft
DRILLING METHOD Rotary drill with mud, wash & casing
LOGGED BY J. Rivera/Y. Leyva **CHECKED BY** C. Rausseo
NOTES _____

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NGVD)
0							
	SPT	18	2-4-7-16 (11)	SM SM		0.2 1.0 SILTY SAND, medium dense, brown	7.8 7.0
	SPT	24	15-17-22- 15 (39)	GP		LIMESTONE FRAGMENTS, dense, light brownish yellow, with sand	5 4.0
5	SPT	24	9-15-23-15 (38)	LS		LIMESTONE, medium hard, light brown, with a trace of sand	
	SPT	7	12-14-12- 10 (26)			▼ LIMESTONE, soft, light brown	0
	SPT	22	10-15-18- 15 (33)			LIMESTONE, medium hard, light brown	
10							-2.0

Boring terminated at 10.0 feet.

KEY TO SYMBOLS

Symbol Description

Strata symbols

	Topsoil		Concrete		Limestone Fragments		Silty Sand with Peat
	Clay		Asphalt		Limestone and Sand		Shelly Sand
	Silt		Sandstone		Sand with Organics		Debris
	Sand		Silty sand		Sandstone and Sand		Cemented Sand
	Peat		Limestone		Sand with Silt		Silty Limestone Fragments

Misc. Symbols

	Groundwater level measured at boring completion. The date checked is indicated.
	Boring continues
	End of Boring

Soil Samplers

	SPT	Standard Penetration Test. 140 lb. hammer dropped 30"		AU	Auger Cuttings
	RC	Rock Core			

Notes:

1. Exploratory borings were drilled on 08/21/2024 and 08/22/2024 using a 3-inch-diameter rotary drill with mud, wash and casing.
2. Groundwater was encountered at depths between 6.7 and 7.9 feet below grade upon boring completion.
3. These logs are subject to the limitations, conclusions, and recommendations in this report.
4. Results of tests conducted on samples recovered are reported on the logs.

NOTES RELATED TO RECORDS OF TEST BORING AND GENERALIZED SUBSURFACE PROFILE

1. Groundwater level was encountered and recorded (if shown) following the completion of the soil test boring on the date indicated. Fluctuations in groundwater levels are common; consult report text for a discussion.
2. The boring location was identified in the field by offsetting from existing reference marks and using a cloth tape and survey wheel.
3. The borehole was backfilled to site grade following boring completion, and patched with asphalt cold patch mix when pavement was encountered.
4. The Record of Test Boring represents our interpretation of field conditions based on engineering examination of the soil samples.
5. The Record of Test Boring is subject to the limitations, conclusions and recommendations presented in the report text.
6. "Field Test Data" shown on the Record of Test Boring indicated as 11/6 refers to the Standard Penetration Test (SPT) and means 11 hammer blows drove the sampler 6 inches. SPT uses a 140-pound hammer falling 30 inches.
7. The N-value from the SPT is the sum of the hammer blows required to drive the sampler the second and third 6-inch increments.
8. The soil/rock strata interfaces shown on the Record of Test Boring are approximate and may vary from those shown. The soil/rock conditions shown on the Record of Test Boring refer to conditions at the specific location tested; soil/rock conditions may vary between test locations.
9. Relative density for sands/gravels and consistency for silts/clays and limestone are described as follows:

SPT Blows/ Foot	Sands/Gravels Relative Density	SPT Blows/Foot	Silt/Clay Relative Consistency	SPT Blows/ Foot	Limestone Relative Consistency
0-4	Very loose	0-2	Very Soft	0-20	Very Soft
5-10	Loose	3-4	Soft	21-30	Soft
11-30	Medium Dense	5-8	Medium Stiff	31-45	Medium Hard
31-50	Dense	9-15	Stiff	46-60	Moderately Hard
Over 50	Very Dense	16-30	Very Stiff	61-50/2"	Hard
		Over 30	Hard	Over 50/2"	Very Hard

10. Grain size descriptions are as follows:

<u>NAME</u>	<u>SIZE LIMITS</u>
Boulder	12 inches or more
Cobbles	3 to 12 inches
Coarse Gravel	3/4 to 3 inches
Fine Gravel	No. 4 sieve to 3/4 inch
Coarse Sand	No. 10 to No. 4 sieve
Medium Sand	No. 40 to No. 10 sieve
Fine Sand	No. 200 to No. 40 sieve
Fines	Smaller than No. 200 sieve

11. Definitions related to adjectives used in soil/rock descriptions:

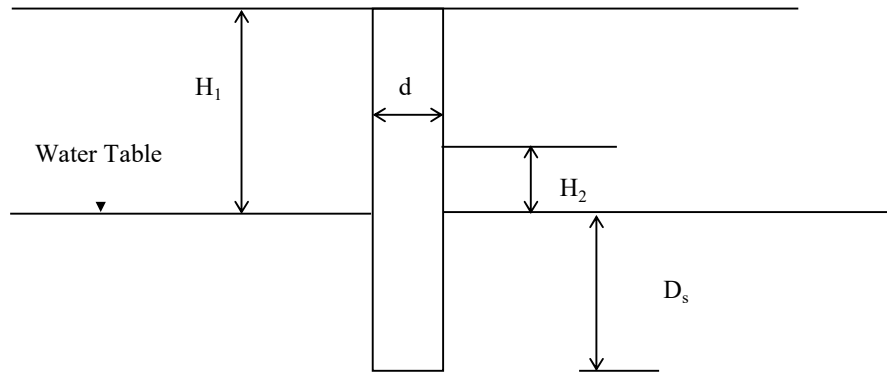
<u>PROPORTION</u>	<u>ADJECTIVE</u>	<u>APPROXIMATE ROOT DIAMETER</u>	<u>ADJECTIVE</u>
About 5%	with a trace	Less than 1/32"	Fine roots
About 5% to 12%	with	1/32" to 1/4"	Small roots
About ≥ 12%	silty, sandy, etc.	1/4" top 1"	Medium roots
		Greater than 1"	Large roots

APPENDIX B

FIELD PERCOLATION TEST DATA



**SOUTH FLORIDA WATER MANAGEMENT DISTRICT
" USUAL OPEN - HOLE TEST "**



HYDRAULIC CONDUCTIVITY

$$K = \text{Hydraulic Conductivity} = 4Q / [\pi d (2H_2^2 + 4H_2 D_s + H_2 d)]$$

7.71E-04 CFS/FT²-FT HEAD

Time (Min.)	Flow (GPM)		
1	15.00	Q = Average Flow Rate =	0.033420 CFS
2	15.00		
3	15.00	d = Diameter of Test Hole =	2.0 inches
4	15.00		
5	15.00	H ₂ = Head on Water Table =	7.3 feet
6	15.00		
7	15.00	D _s = Depth below Ground Water Table =	7.7 feet
8	15.00		
9	15.00		
10	15.00		

TEST LOCATION :		See Drawing No. 1
TEST ELEVATION :	+9.0'	NGVD (Estimated)
DEPTH TO WATER TABLE H ₁ :	7.3'	Below Existing Grade
DEPTH OF TEST HOLE :	15.0'	Below Existing Grade
AVERAGE FLOW RATE:	15.00	GPM

SOIL PROFILE :

0.0' - 0.5'	Dark brown Sand
0.5' - 3.0'	Light brownish yellow to light brown Limestone with sand
3.0' - 5.0'	Light brown to brownish yellow Limestone with sand
5.0' - 15.0'	Light brown Limestone with sand

NOTES: 1) The subsurface profile is determined by cuttings & should not be relied upon as an accurate record of material type or for transition zones.
2) K value calculated using PVC diameter of 2 inches

PERCOLATION TEST

	PROJECT NAME: Gary Matzner Park		
	PROJECT LOCATION: 6610 N. Kendall Drive, Pinecrest, Florida		
	PROJECT NO: 18595	TEST DATE: 8/23/2024	TEST NO: P-1
	TESTED BY: Y. Garcia		CHECKED BY: AB