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 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 our borings are consistent with the geology described above. The detailed subsurface conditions are presented graphically in the attached boring summary sheet in drawing 2, and in more detail on the records of test boring logs in appendix A. It should be noted that the ground surface elevations shown for the borings have been estimated. If accurate elevations are required, the boring locations should be surveyed. The subsurface conditions disclosed by the borings can be generalized as described below.

<u>Layer 1 – Sand:</u>
This surficial layer consists of brown, gray and light gray sand with occasional limestone fragments and shells that extends three (3) to 10 feet below the existing grade in the borings. In borings B-1 and B-5 silty sand was recovered at the top of the layer with up to two (2) feet in thickness. SPT N-values recorded in the sand layer range from two (2) to 20 blows per foot (bpf), with an average value of about 9 bpf, indicating the layer is typically loose.

<u>Layer 2 –Peat/Silt:</u>
Beneath the surficial sand the borings encountered dark brown peat and/or brown and dark gray silt encountered at three (3) and 20 feet below grade, extending to depths of 13 and 23 feet below the existing grade. The thickness of this compressible layer in the borings are seven (7) to 20 feet. The stratum is mostly very soft and soft with recorded SPT N-values that range from less than one (1) to 15 bpf. The average of the recorded SPT N-values is about one (1) bpf.

<u>Layer 3 –Limestone:</u>
This layer was encountered at 13 and 23 feet below grade and extends to 39 and 50 feet below grade with a thickness that range 17 to 37 feet in the borings. SPT N-values in the limestone range from less than one (1) to greater than 50 bpf. The average of the recorded SPT N-values is around 16 bpf.

<u>Layer 4 – Sand:</u>
Beneath the limestone in the deep borings is an 18- to 34-foot-thick layer of light gray and light greenish gray sand with some limestone fragments encountered at 39 and 50 feet below grade and extend to 68 and 73 feet below grade. In borings B-1 and B-2 thin lenses of limestone up to two (2) feet in thickness were recovered within this layer. The recorded SPT N-values in layer 4 sand averages about 10 bpf, and the values range from three (3) to 23 bpf.

<u>Layer 5 – Limestone/Sandstone with Interbedded Sand Layers and Sand Zones:</u>
Layer 5 consists of gray and light gray limestone and/or sandstone encountered at about 68 and 73 feet below grade that extends to the maximum boring termination depths at 120 feet below grade. The thickness of Layer 5 is at least 47 feet. Sand zones and layers four (4) to

13 feet in thickness were observed within Layer 5 at depths of about 80 and 100 feet below grade. The recorded SPT N-values in Layer 5 range from three (3) to greater than 50 bpf. The average of the recorded SPT N-values is about 18 bpf.

For the layers described above, Table 1 below summarizes our estimates of engineering parameters considered pertinent to the design of foundations for high-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)	Allowable Side Shear (ksf)
			Range	Avg.			
1	Sand	2 – 10	2 – 20	9	250	-	-
2	Peat/Silt	7 - 20	<1 – 15	1	100	-	-
3	Limestone	17 - 37	<1 – 50+	16	5,000 - 10,000	50 - 300	3
4	Sand	5 – 34	3 – 23	10	250	-	-
5	Limestone/Sandstone with Interbedded Sand	47+	3 – 50+	18	5,000 – 15,000	50 – 500	3 – 5

We note that the values of allowable side shear estimated in Table 1 above are based on our experience and laboratory data from similar rock that we have tested.

Groundwater

Groundwater was encountered in the borings at depths between about 1.6 and 4.5 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 depth reported above corresponds to approximate elevations of -1.3 to +0.6 feet NAVD based on our assumed elevations at the boring locations. On average, stabilized groundwater levels in the general vicinity of the project are expected to vary between elevations -1.5 and +2.5 feet NAVD, the variations being primarily the result of seasonal rainfall. Nonetheless, it should be noted that groundwater levels outside of this range could be encountered during construction. Storm and hurricane events and construction activities could 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 five or so feet of the existing ground surface.

5.2 FIELD PERMEABILITY

The results of the open-hole field permeability tests performed to 10 feet below existing ground surface at the site are presented in the table below:

TABLE 2 – SUMMARY OF FIELD PERMEABILITY TEST RESULTS

Test ID	Hydraulic Conductivity (cfs/ft^2-ft. head)
P-1	5.42×10^{-5}
P-2	3.28×10^{-5}
P-3	1.39×10^{-4}
P-4	2.63×10^{-4}

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

6.0 EVALUATION AND DISCUSSION

6.1 GENERAL

We consider the site suitable for the proposed project from a geotechnical perspective. The primary concern for foundation design and construction includes support of the proposed new structure loads without unacceptable settlement. Foundation support options are discussed below, and detailed foundation design and construction recommendations including sizes, lengths, and axial and lateral load capacities are presented in Section 7 of this report.

6.2 FOUNDATION SUPPORT

Given the subsurface conditions encountered in the borings, and the anticipated structure loads, we conclude that deep foundation support is appropriate for the proposed development. Consistent with current practice in the South Florida area we consider augered, cast-in-place (ACIP) piles to be the most feasible foundation type for this project. Other deep foundation systems such as driven piles and drilled shafts are not considered feasible. In addition to the noise nuisance, vibrations from driven pile foundations could adversely impact existing buildings on the site as well as those on adjacent properties. Additionally, it would be difficult to penetrate the hard zones in the limestone and sandstone rock at the site to sufficient depths to provide adequate uplift capacity on the driven piles. Drilled shafts are typically economically feasible and attractive only where they are used to carry very large loads that sufficiently justify the slower installation rates and other installation difficulties attendant with such foundations.

We conclude that the 21-level tower can be supported on 18-inch-diameter piles on the order 78 to 95 feet long below existing grade. The 3-level podium and villas can be supported on 14-inch diameter piles on the order of 42 feet below grade. Low capacity 14-inch diameter piles on the order of 35 feet below grade can be used for miscellaneous structures or as intermediate piles supporting the first-floor slab.

6.3 GROUND LEVEL SLABS

The ground level slabs should be structurally supported due to the proximity of the compressible peat and silt layers.

6.4 ESTIMATED SETTLEMENT

Assuming overall average base pressures of around 3.5 and 0.6 kips per square foot (ksf) for the 21-level tower and 3-level podium and villas, respectively, and based on the subsurface conditions, and the pile foundation system recommended herein, we conclude settlements of one (1) inch may be expected. The granular nature of the subsurface materials at the site will result in the majority of the tower settlement occurring during construction and for a short time period (typically less than three to six months) following substantial completion of the top level. Additional small settlements of the tower could occur after structural completion as interior walls, cladding, finishes etc. are added to the building.

As the structure height increases the tower should become stiffer thereby reducing the potential for differential edge-to-center settlements. Differential movements of the pile cap system will result in redistribution of loads in the tower and among the piles.

At the ground level, the settlements will manifest as an areal drop in grade rather than abrupt differential movement between the pile caps and the immediately adjacent soil grade. As a result, lightly loaded structures that are close to the tower foundation could be impacted by this areal drop in grade. The zone of influence and the rate of settlement attenuation away from the tower footprint is determined by the magnitude of the settlement, and the geometry and layout of the tower foundations, in particular the location of heavy cores with respect to the edges of the footprint.

The project design and the construction schedule should be planned to accommodate the anticipated structure settlements. Connections to the tower such as lateral piping and duct banks should be deferred until tower construction is near completion.

Depending on the relative timing of the podium and tower construction, the potential exists for those podium columns closest to the tower to experience additional settlement due to settlement from the tower construction. It will therefore be prudent to delay the construction of the podium until the tower is almost complete. If this will not be possible, then the adjacent podium columns should be designed to accommodate this additional settlement.

6.5 IMPACTS OF PEAT AND SILTY MATERIALS

It is noteworthy that the borings encountered seven (7) to 20 feet of very soft to stiff peat and silty soils at about four (4) to 10 feet below grade These materials are highly compressible and will undergo consolidation when subjected to new stresses. It is therefore desirable to keep new fills to an absolute minimum to prevent consolidation settlement of the layer and the consequent potential impacts to any pavements or miscellaneous structures supported on shallow footings, particularly where these materials are closer to the ground surface. Structures that could be susceptible to such impacts include pavements, water features, entrance ramps, and other landscaping that requires filling. These compressible materials could also result in down-drag on perimeter pile foundations as the adjacent soil settles against the pile shafts. Structures supported on shallow foundations over the compressible peaty and silty materials could be subject to the effects of long-term secondary compression of the material. NV5 should evaluate the project grading plans to assess any potential adverse impacts with respect to the peat layer, including downdrag forces on the piles.

The weak materials have implications for ground floor slab support as well. To avoid slab settlement associated with compression of these materials it will be prudent to structurally support ground floor slabs.

6.6 IMPACTS TO EXISTING STRUCTURES

The primary potential adjacent structures of concern for the proposed development are the 2-level residential buildings to the north and east of the site. The new 3-level garage podium will be 5-feet west, and the 3-level villas about 15 feet south of the existing 2-level residential structures.

Impacts to adjacent structures during construction generally come from one of three sources, namely settlement, ground movement due to nearby excavations, or vibrations. The discussion below is general in nature and NV5 can perform additional and more specific evaluation of potential impacts to adjacent structures as the project foundation design progresses and more information on the adjacent structure becomes available. <u>It will be important to obtain as-built foundation information for the adjacent structures as early as possible in the project development schedule</u>.

<u>It could become necessary to include a contingency to address repairs that might be needed at nearby properties due to impacts from construction of the podium and villas. It will also be prudent to perform pre-construction condition observations of the adjacent properties and to monitor them for the impacts discussed below during construction.</u>

6.6.1 Settlement Impacts

The tower, podium and villas are not expected to cause area settlement outside their footprint. Settlements can also derive from drawdown of groundwater levels due to dewatering. This is usually an issue for long-term dewatering by well-points. For this project, we anticipate there will be a need for some dewatering during construction and drawdown effects could be observed outside of planned excavation footprints. <u>A detailed dewatering plan will be required to be developed by the contractor</u>.

6.6.2 Excavation & Ground Movement Impacts

<u>Excavations for the proposed development could negatively impact the neighboring structures considering their proximity to the new development. Excavations could result in movement of existing ground level slabs</u>. Support of excavation will have to be properly designed to limit ground movement at the top of the excavations.

It would be prudent to plan underpinning at adjacent foundations and ground level slabs that are close to proposed excavations. Such underpinning would likely comprise chemical grouting or permeation grouting of the Layer 1 sand. <u>One of the important considerations in any plan for underpinning of adjacent foundations is that often access to the neighboring property is required for this work to be done</u>.

6.6.3 Vibration Impacts

<u>Construction-related vibrations could impact the existing structures around the site as well. Such vibrations could derive from activities such as sheet pile installation or compaction</u>. 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 the use of non-vibratory techniques such as secant ACIP piles or a deep mix (DM) for excavation support, modifying compaction procedures and techniques, and performing vibration monitoring at the structures during construction.

6.7 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 South Florida. 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. We conclude therefore that these specific environmental forces have a low risk (on a scale of low, moderate, high) of adversely affecting deep foundation performance at this site provided the foundation system is designed and constructed as recommended herein.

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 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.

   We are not aware of the development history of the site beyond its current condition. If old subsurface structures 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 preparation as described above, areas for structures that will have slabs on grade or pavements should be densified with at least five overlapping passes of a 20-ton roller as it operates at its maximum vibrational frequency, and a travel speed of not more than 2 feet per second. The densification should be observed by NV5 to identify and mitigate any weak subgrade conditions evidenced by yielding or rutting at the wheels of the roller. Proof-rolling should include planned development footprints plus a five-foot perimeter.

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 inches.

   Based on our boring data the majority of the near-surface granular materials should satisfy the fill criteria. However, some materials might require localized sorting and moisture-conditioning prior to re-use. Silty materials nor peat should 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 compaction[1]. 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.

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 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 Augered Cast-In-Place (ACIP) Piles

1. Our recommended pile tip elevations, allowable pile axial capacities, and grout strengths for foundation support are presented in the table below.

TABLE 3 - SUMMARY OF ALLOWABLE PILE CAPACITIES

Pile Diameter (in)	Min. Pile Tip Elevation (ft., NGVD)	Allowable Compression (kips)	Vertical Spring Constant (kpi)	Allowable Tension (kips)	Allowable Lateral Load (kips)	Minimum Grout Strength (ksi)
Tower						
18	-77	420	420	210	20	7.0
18	-94	600	600	300	8	8.0
3-Level Podium/Villas						
14	-40	230	230	115	8	5.0
Miscellaneous Structures/Intermediate Piles						
14	-33	80	80	40	8	5.0

Notes:
a) Minimum tip elevation based on an average site grade of +2 feet NAVD at the time of the borings.
b) Required grout strength is 56-day test for the 18-inch-diameter piles, and 28-day test for the 14-inch piles.

2. The vertical spring constant is the working pile load divided by the estimated pile settlement and is based on our experience and a review of available pile load test data in similar subsurface conditions. The initial spring constant value should be refined as the structural model is developed. The design value used should match the settlement estimates. For analysis of transient loads, a value of 1,200 and 800 kpi may be used for the tower and podium piles, respectively.

3. We performed the lateral load analyses using the LPILE computer program to estimate the performance of the piles under lateral loading. In the analyses, we considered the simultaneous application of about 25 percent of the compression loads in Table 4 along with

[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).

the lateral loads. A fixed head condition was assumed for the pile. A p-modification factor of 0.4 was applied to the soil resistance values to consider the effect of pile grouping since the LPILE program analyzes a single-pile condition only. No y-modification was applied. The maximum bending moments associated with the recommended lateral loads for a fixed head are presented in Table 4 below.

TABLE 4 - SUMMARY OF PILE MAXIMUM MOMENTS UNDER LATERAL LOAD

Pile Diameter (in)	Allowable Lateral Load (kips)	Maximum Bending Moment (in-kips)	Depth to Zero Moment (ft.)
18	20	1100	14
14	8	440	12

Notes:

a) Lateral load capacities based on maximum pile head movement of ¼ to 3/8 inch.
b) Bending moments listed above are un-factored.
c) The depths in table above reference to the bottom of pile cap/top of pile.

It should be noted that the lateral load capacities provided above assume pile reinforcement of approximately one (1) to two (2) percent. If the actual pile reinforcement differs significantly from this assumption, it might become necessary to revisit the lateral recommendations provided in Table 4 above.

4. Pile reinforcing should be designed by the structural engineer to resist the tension and lateral forces applied to the pile systems. We recommend that piles resisting tension loads be reinforced over their entire length. Piles resisting lateral loads should be reinforced for the maximum bending moments listed in the table below. It should be noted that the depths in the table below are referenced to the top of the pile. If the pile is not reinforced over the entire length, we recommend as a minimum, a single No. 7 bar be installed the full length of the pile to verify cross-section continuity.

5. Resistance to lateral loads can also be provided by passive pressure acting on the pile caps or grade beams. However, this resistance should not be considered in combination with the lateral capacity of the piles as the deflections required to mobilize the passive resistance might be larger than those associated with the pile lateral capacity. Equivalent fluid densities of 180 and 80 pounds per cubic foot may be used to compute the passive pressures acting against the sides of the pile caps and grade beams above and below the groundwater table respectively. Passive resistance of the upper one foot of soil should be neglected, unless it is confined by a slab or pavement. Frictional resistance between the soil and bottom foundation elements should be ignored.

   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 limestone, in-situ soil densified by compaction, and 2) able to withstand horizontal movement on the order of ¼ to 3/8 inch.

6. Pile reinforcing should be designed by the structural engineer to resist the forces applied to the pile systems. We recommend that piles resisting tension loads be reinforced over their entire length. The information provided in Table 4 above should be used to design the reinforcing for piles resisting lateral loads. If the pile is not reinforced over the entire length, we recommend as a minimum, a single No. 7 bar be installed the full length of the pile to verify pile cross-section continuity.

It should be noted that the lateral load capacities provided above assume pile reinforcement of approximately one (1) to 2 percent. If the actual pile reinforcement differs significantly from this assumption, it might become necessary to revisit the lateral recommendations provided in Table 4 above.

7. Foundations should be designed so that a minimum center-to-center pile spacing of three pile diameters is maintained.

8. We recommended that a load test program be performed for the project prior to the start of production foundation installation. This will allow for the test results to be analyzed, and for recommendations to be revised if necessary. Based on load test results pile capacities and/or lengths may be adjusted. The pile load test program should consist of one (1) compression load test (ASTM D 1143), one (1) tension load test (ASTM D 3689) and one (1) lateral load test (ASTM D 3966) for each pile diameter configuration and tip elevation chosen. Load tests should be performed and results interpreted in accordance with the most current edition of the Florida Building Code. We recommend the use of strain gauge pairs in all test piles to evaluate load transfer. Upon final selection of the load test location, NV5 will provide recommendations for the locations (w.r.t. to elevation) of the pile instrumentation. The minimum test loads should be twice the pile working capacity. We recommend the compression load test(s) be designed to allow overloading of the test pile (s) to 2.5 times the design working load after completion of the standard compression test loading and unloading procedure.

   Test piles should not be used as production piles. Upon approval by NV5, reaction piles may be installed in production locations provided such piles are properly installed to meet the project specifications and are monitored for movement during load testing.

   NV5 should review and approve the contractor's load testing submittal with respect to test locations, test pile installation, and load testing equipment and procedures. NV5 should also monitor and report the results of test pile installation and load testing.

   We note that the borings encountered zones of very hard rock at the site. These are indicated on the boring summary sheet shown on Drawing 2 as material with refusal type SPT N-values typically exceeding 50 bpf. Some of these materials are encountered at elevations above the recommended pile tip elevations. The contractor must mobilize the appropriate equipment in order to drill through this hard rock and achieve the tip elevations recommended herein.

9. Piles should be installed within three (3) inches of specified plan location, and within two (2) percent of vertical or batter line.

10. During grouting of the pile excavation, the auger should be raised at a rate consistent with the capacity of the pump to ensure the entire pile shaft is uniformly grouted and to prevent caving of soils into the pile excavation. The actual grout volume for each ACIP pile should be at least 15 percent greater than the theoretical volume. A grout head of at least 10 feet should be maintained throughout the grouting of the pile shaft. Production piles should be installed in a manner similar to the successfully tested pile.

11. If during pile grouting any abnormalities such as sudden pressure drop or low grout take for a given interval of pile length are observed, the auger should be re-advanced to about five feet

below the elevation where the anomaly was observed and the pile shaft properly re-grouted. Pumping should continue while the auger is rotated back down to the required remedial depth.

12. New piles should not be installed close to previously installed piles before the existing pile grout has started to set. Per the Florida Building Code, piles should not be installed closer than six (6) diameters within 12 hours.

13. Grout should be sampled during piling installation at a minimum frequency corresponding to the greater of one set of at least six cubes each morning and afternoon during production or one set of at least six cubes for each 50 cubic yards of grout placed. Cubes should be tested for compressive strength at intervals of seven, 14, and 28 days for grout design of less than 7 ksi. At least three cubes should be tested at 28 days. For grout design of 7 ksi or greater, grout cubes should be tested at intervals of seven, 28, and 56 days. At least three cubes should be tested at 56 days. Any remaining cubes should be retained for subsequent intermediate breaks if required.

14. The steel reinforcement should be installed into the pile shaft immediately upon withdrawal of the grouting auger. Spacers should be fitted to the reinforcing cages to assure that they remain centered within the grouted shaft and maintain the required side cover. If obstructions are encountered during insertion of the steel cage, the cage should be extracted, the pile shaft re-drilled to the originally drilled pile tip elevation and re-grouted to the ground surface, and the reinforcement re-installed.

15. An NV5 inspector should provide full-time quality control inspection to document the excavation and grouting of each pile and to provide, in conjunction with a licensed office engineer, any necessary field adjustments of tip elevations.

7.2.2 Miscellaneous Structures

1. Lightly-loaded miscellaneous structures such as planters <u>that have tolerance for settlement</u> may be designed using an allowable bearing pressure of 1,000 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. It these structures do not tolerate settlements they should be supported on piles.

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 1.5 inches, with differential settlement one the order of ¾ inch.

7.3 GROUND FLOOR SLABS

1. Ground floor slabs should be structurally supported due presence of the compressible material encountered in the borings close to the surface.

2. A design groundwater level of +2.5 feet NAVD 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.

3. Slabs should be reinforced for the loads that they will sustain and construction joints should be provided at frequent intervals.

4. 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. 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.

   <u>Particular attention should be paid to any deep excavations such as for thick pile caps and elevator shafts, and the potential impacts these could have on adjacent structures, especially where such excavations are close to project property lines</u>.

2. Average groundwater elevation is expected to be approximately between Elevation -1.5 and +2.5 feet NAVD for this site. <u>As stated above, groundwater levels outside this range could be encountered during construction</u>. Some dewatering is anticipated for foundation excavations particularly for the deep shear walls and elevator shafts. Additionally, 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 such 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. <u>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 Broward County, Florida Department of Environmental Protection (FDEP), Florida Department of Transportation, and South Florida Water Management District (SFWMD) as appropriate</u>. This study did not include specific testing or analysis to determine if dewatering is feasible or if adequate discharge is available. <u>Ultimately, dewatering of the site to facilitate construction is the contractor's responsibility</u>.

   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. Construction activities could have adverse impacts 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. NV5 can prepare a protocol for monitoring of adjacent structures.

2. 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.

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 our approved Consultant Agreement between Hollywood Moon Development ("client") and NV5 March 8, 2023 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.

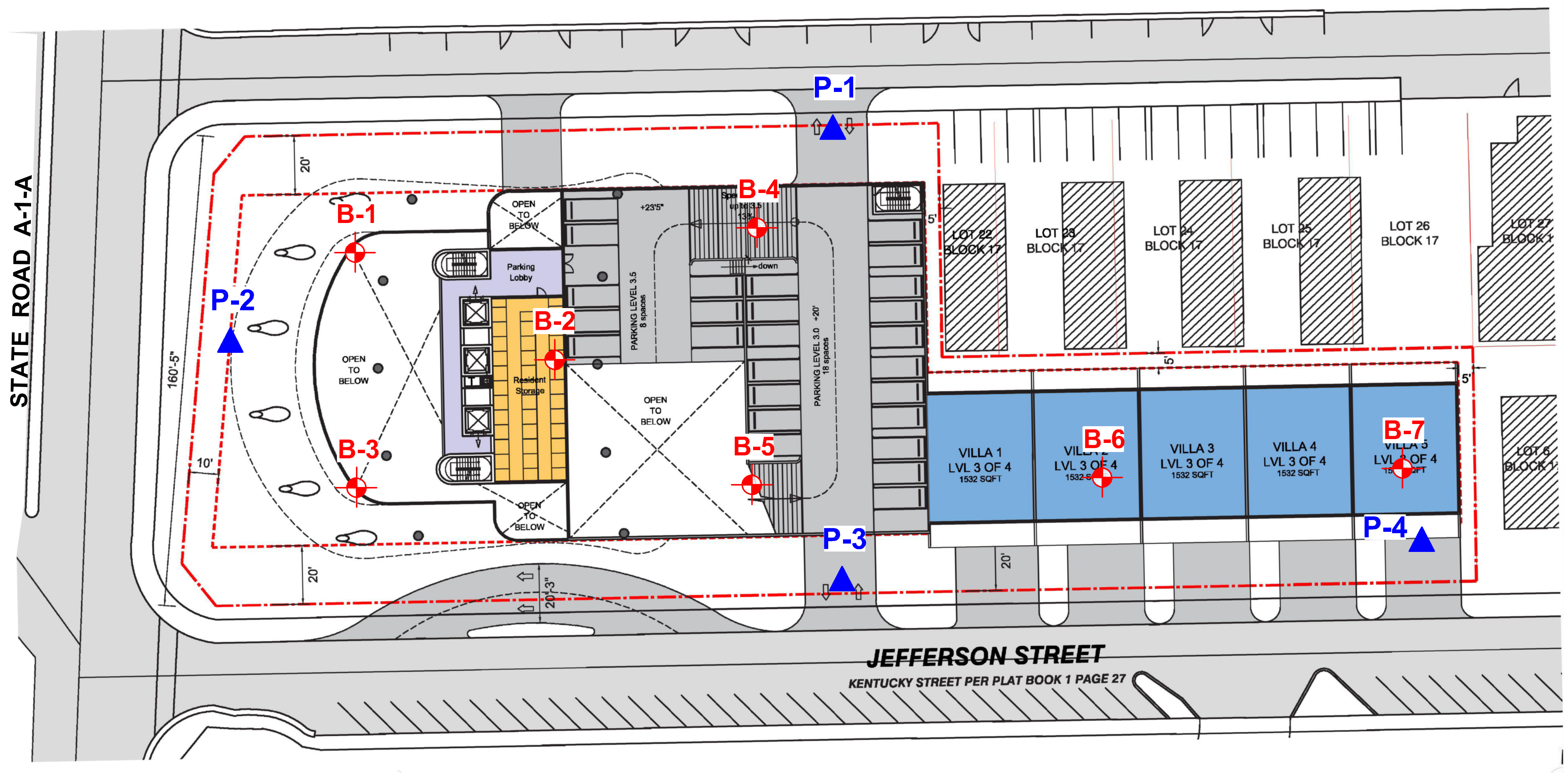
NV5 should be retained to provide consultation to the ownership and design team during the design development phase of the project, to review final foundation specifications and review foundation design drawings in order to ascertain that its recommendations have been properly interpreted and implemented. Furthermore, NV5 should be retained to provide inspections during geotechnical construction. If NV5 is not afforded the opportunity to participate in foundation installation as recommended in this report, client agrees that NV5 has no responsibility for the interpretation of the recommendations made in this report or for foundation performance.

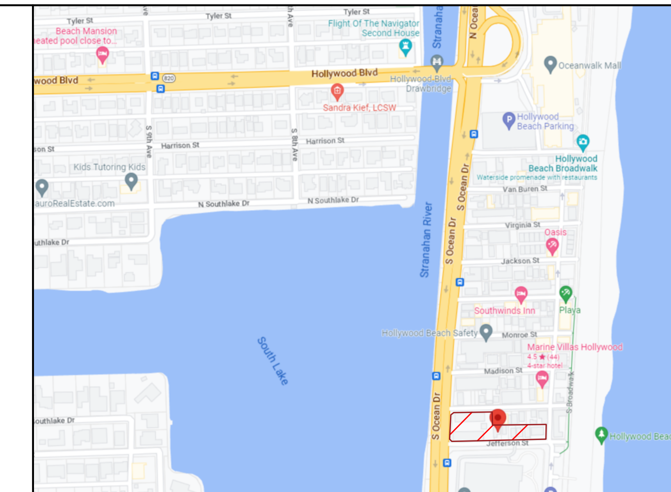
9.0 CLOSURE

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.

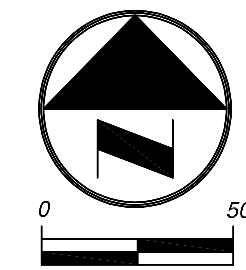
****

DRAWINGS





Site Vicinity Map



Approximate Scale in Feet

LEGEND:

B-1 - Number & Approximate Location of Test Boring.

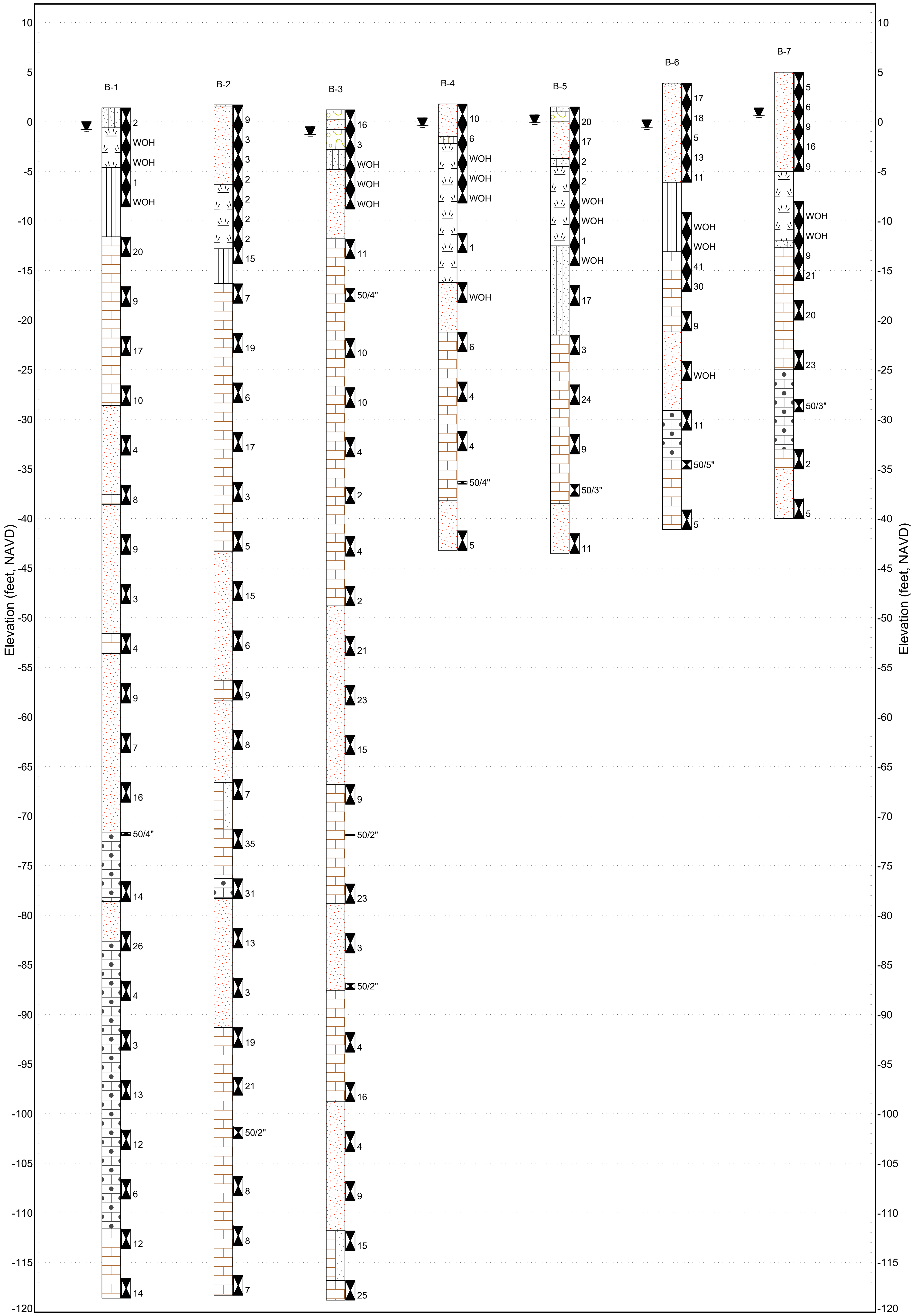
P-1 - Number & Approximate Location of Percolation Test.

NOTES:

1. Test locations shown are approximate.
2. Test location symbols are not to scale.
3. Base drawing was taken from Sheet A007, Level 3P, prepared by Arquitectonica, dated 02/21/2023.

NV5

DRAWING TITLE:	*Site Vicinity Map & Test Location Plan*					**DWN BY:**	CR
PROJECT NAME:	*901 South Ocean Drive – 21-Level Condominium*					**CKD BY:**	AB
PROJECT LOCATION:	*901 South Ocean Drive, Hollywood, Florida*	**PROJECT NO:** 18119	**DATE:** 04/06/2023	**DWG NO:** 1		**APD BY**	—



Note: Boring top elevations have been estimated

BORING SUMMARY SHEET

PROJECT NAME: 901 South Ocean Drive – 21-Level Condominium

PROJECT LOCATION: 901 South Ocean Drive, Hollywood, Florida

PROJECT NUMBER:18119

DATE: 04/10/2023

DRAWN BY: CR

CHECKED BY: AB

DRAWING NO: 2

LEGEND

- Topsoil
- Concrete
- Sand
- Silt
- Sandstone
- Silty Sand
- Limestone
- Limestone Fragments
- Peat
- Limestone and Sand
- Standard Penetration Test & SPT N-value
- Water Level

APPENDIX A

BORING LOG DATA

BORING NUMBER B-1

PROJECT NAME 901 South Ocean Drive – 21-Level Condominium
PROJECT NUMBER 18119 **PROJECT LOCATION** 901 South Ocean Drive, Hollywood, Florida
DATE STARTED 3/29/23 **COMPLETED** 3/30/23 **GROUND ELEVATION** 1.4 ft NAVD est. **HOLE SIZE** 3 inches
DRILLING CONTRACTOR NV5 **GROUND WATER LEVELS:** 2.2 ft / Elev -0.8 ft
DRILLING METHOD Rotary drill with mud, wash & casing
LOGGED BY J. Johnson / Y. Garcia **CHECKED BY**
NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NAVD)
0	SPT	75	1-1-1-WOH (2)	SM	2.0	SILTY SAND, very loose, fine, dark brown to brown, with a trace of roots and limestone fragments	-0.6
	SPT	67	1-WOH-WOH-WOH (WOH)	PT		SILTY PEAT, very soft, dark brown, with a trace of sand	
5	SPT	100	1-WOH-WOH-1 (WOH)		6.0	SILTY PEAT, very soft, dark brown	-4.6
	SPT	67	1-WOH-1-1 (1)	ML		SILT, very soft, brown, with sand, trace of roots and limestone fragments	
10	SPT	50	WOH-WOH-WOH-WOH (WOH)			SILT, very soft, brown, with sand, trace of roots and limestone fragments	
					13.0		-11.6
15	SPT	50	2-11-9-10 (20)	LS		LIMESTONE, very soft, light brown to brown, with sand	
20	SPT	67	6-4-5-9 (9)			LIMESTONE, very soft, light brown to gray, with sand	
25	SPT	67	8-7-10-9 (17)			LIMESTONE, very soft, light brown to gray, with sand	
30	SPT	75	11-5-5-4 (10)		30.0	LIMESTONE, very soft, light brown, with a trace of sand	-28.6
35	SPT	58	5-3-1-1 (4)	SP		SAND, very loose, fine, light brown to brown, with a trace of limestone fragments	

Elevation scale: 0, -5, -10, -15, -20, -25, -30

(Continued Next Page)

BORING NUMBER B-1

PROJECT NAME 901 South Ocean Drive – 21-Level Condominium

PROJECT NUMBER 18119 **PROJECT LOCATION** 901 South Ocean Drive, Hollywood, Florida

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NAVD)
35				SP			-35
39.0	SPT	92	2-2-6-37 (8)			SAND, loose, fine to medium, gray, with a trace of limestone fragments	-37.6
40.0				LS		LIMESTONE, very soft, light brown, with a trace of sand	-38.6
40							-40
45	SPT	50	5-5-4-3 (9)	SP		SAND, loose, fine, light gray, with a trace of limestone fragments	-45
50	SPT	0	2-2-1-WOH (3)			NO RECOVERY (Possible: SAND)	-50
53.0							-51.6
55.0	SPT	25	1-3-1-8 (4)	LS		LIMESTONE, very soft, gray, with sand	-53.6
55							-55
60	SPT	75	4-5-4-6 (9)	SP		SAND, loose, fine, gray, with a trace of limestone fragments	-60
65	SPT	67	5-3-4-7 (7)			SAND, loose, fine, light greenish gray	-65
70	SPT	75	4-6-10-16 (16)			SAND, medium dense, fine to medium, light gray, with sandstone fragments	-70
73.0							-71.6
75	SPT	100	50/4" (100)	SS		SANDSTONE, hard, gray	

(Continued Next Page)

BORING NUMBER B-1

PROJECT NAME 901 South Ocean Drive – 21-Level Condominium

PROJECT NUMBER 18119 **PROJECT LOCATION** 901 South Ocean Drive, Hollywood, Florida

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft., NAVD)
75							-75
80	SPT	75	3-4-10-8 (14)	SS	80.0	SANDSTONE, very soft, gray to light greenish gray, with sand, trace of silt	-78.6
							-80
85	SPT	92	22-18-8-17 (26)	SP	84.0	SAND, medium dense, medium, light gray, with a trace of shells	-82.6
				SS		SANDSTONE, soft, gray to light brown, with sand	-85
90	SPT	50	14-3-1-5 (4)			SANDSTONE, very soft, light gray, with sand	-90
95	SPT	42	2-1-2-1 (3)			SANDSTONE, very soft, light gray, with sand	-95
100	SPT	67	28-4-9-20 (13)			SANDSTONE, very soft, light gray to gray, with a trace of sand	-100
105	SPT	42	8-7-5-4 (12)			SANDSTONE, very soft, light gray, with a trace of sand	-105
110	SPT	67	2-3-3-3 (6)			SANDSTONE, very soft, gray, with sand	-110
					113.0		-111.6
115	SPT	58	11-8-4-5 (12)	LS		LIMESTONE, very soft, light gray to light brown, with a trace of sand	

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