SPEEDIE AND ASSOCIATES
Geotechnical • Environmental • Materials Engineers
A UES Company

www.speedie.net

REPORT ON GEOTECHNICAL INVESTIGATION

DESIGNATION: FIRA Luxury Boutique Hotel

LOCATION: 18211 North Pima Road
Scottsdale, Arizona

CLIENT: Alpha CO Land Holdings, LLC

PROJECT NO: 251041SA

DATE: June 19, 2025

Registered Professional Engineer (Civil)
CERTIFICATE NO.
37292
KEITH R.
GRAVEL
Date Signed 6/19/2025
ARIZONA, U.S.A.

3331 East Wood Street ■ Phoenix, AZ 85040 ■ Phone (602) 997-6391 ■ Fax (602) 943-5508
2026 North 3rd Street ■ Flagstaff, AZ 86004 ■ Phone (928) 526-6681 ■ Fax (928) 526-6685
3125 East 47th Street ■ Tucson, AZ 85713 ■ Phone (520) 514-9411 ■ Fax (520) 514-9474

SPEEDIE AND ASSOCIATES
Geotechnical ▪ Environmental ▪ Materials Engineers
A UES Company

TABLE OF CONTENTS

Registered Professional Engineer (Civil)
CERTIFICATE NO.
37292
KEITH R.
GRAVEL
Date Signed 6/19/2025
ARIZONA, U.S.A.

1.0 INTRODUCTION

This report updates the results of a subsoil investigation carried out (for a previous project) at the site of the proposed *FIRA Luxury Boutique Hotel* development to be located at the addresses of 18211 North Pima Road in Scottsdale, Arizona. The general project location is shown in Figure 1.0.1.

We understand that construction will consist of five story hotel building with two levels of underground parking on a 72,966 square foot vacant lot. The structure will be cast-in-place concrete construction. Structural loads are expected to be moderate to heavy and no special considerations regarding settlement tolerances are known at this time, although we assume that the industry standard of a maximum of 1-inch of total settlement will be allowed and used for design. Adjacent areas will be landscaped or paved to support light to moderate passenger and truck traffic. Storm water retention and disposal plans were not provided. **We are not aware of any proposed underground stormwater retention systems. If any are planned or required, we should be notified so that we may review the conditions and revise our recommendations accordingly.** Additional field and laboratory work may be required. CMP stormwater storage tanks are not recommended under or near structures, especially basement structures due to the risk of leaking and ongoing maintenance**.**

Figure 1.0.1 General Location Map

Speedie & Associates LLC (S&A) previously completed a subsoil investigation at the subject site for a project that did not proceed to construction (Project 151276SA). The data from the previous investigations has been used as the basis of this report and the recommendations provided herein. No additional field or laboratory work was completed for this updated report.

2.0 GENERAL SITE AND SOIL CONDITIONS

2.1 Site Conditions

The site is generally bounded on the north by a retail commercial plaza with associated parking lot, on the east by residential properties, on the south by Trailside View and on the west by access roads and additional parking lot. The proposed site is currently vacant land enclosed with a metal fence. Medium to dense covering of desert vegetation includes creosote bushes, trees, grasses, and weeds spread throughout the site. The ground surface elevation of the site appear to be approximately 2 feet below the neighboring roadways and land, due to a retention basin developed on 2008. Refer to the following photo taken from the online tool google maps.

Figure 2.1.1 – Google Maps View, Looking East

Based on a cursory review of historical aerial photographs, the site has been native desert land from before 1953, the earliest available photograph. Several natural drain washes oriented north-south across the site are visible. By 2006, the site had been graded, washes infilled, and the site was used as a temporary construction staging area. Development of the neighboring properties had been initiated. By 2007, several soil stockpiles are visible on the site. By 2008, the stockpiles had been removed and a retention basin covering most part of the site was visible. Apart from growth of vegetation, the site has remained relatively unchanged. Refer to the following aerial photographs:

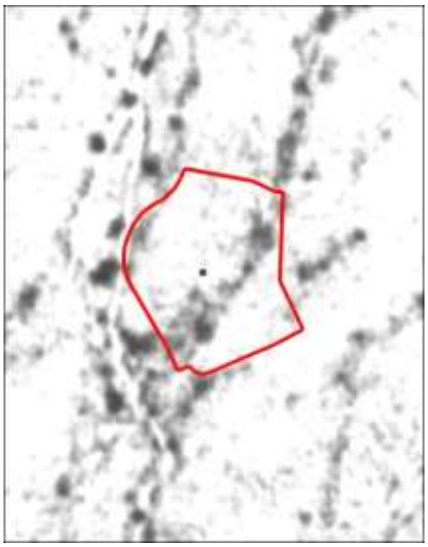


Figure 2.1.2 – Dated 1953

Figure 2.1.3 – Dated 2003

Figure 2.1.4 – Dated 2007

Figure 2.1.5 – Dated 2008

Figure 2.1.6– Dated 2013

Figure 2.1.7 – Dated 2022

"Historical Aerial Photography," *ArcGIS Web Application*. [Online]. Available: https://gis.maricopa.gov

2.2 Geologic Conditions

The site is **located outside known areas** that have undergone considerable subsidence due to groundwater removal. Areas of subsidence are known to produce earth fissuring, which has affected areas within several miles of the site. Subsidence is a basin wide phenomenon that would result in differential elevation changes over long distances, which would not affect the type of buildings proposed for this site. No evidence of earth fissures was observed on the site. Fissure gullies form over subsurface irregularities such as bedrock highs, which cause tensional stresses and differential subsidence. Where such anomalies are not present, subsidence tends to be uniform over a wide area, this having minimal effect on surficial structures. There are two documented cases of earth fissures near 40th Street and Lupine and near Cactus Road and the CAP canal. These were reported in the 1970's and have not shown any recent activity. Based on local experience, subsidence and earth fissures historically have **not** been a problem in the immediate area.

2.3 Seismic Design Parameters

The project area is located in a seismic zone that is considered to have low historical seismicity. The seismicity of the Phoenix area has had only two magnitude 3.0 events in over 100 years. Liquefaction is not considered a concern as groundwater exceeds 15 meters below ground surface.

Although borings were not advanced to 100 feet, based on the nature of the subsoils encountered in the borings and geology in the area, Site Class Definition, Class C may be used for design of the structures. In addition, the following seismic parameters may be used for design (based on ASCE 7-22 and ASCE 7-16, utilizing the ASCE Hazard Tool). The designer should choose the appropriate values for design:

Table 2.3.1 Seismic Parameters

Building Design Code:	ASCE7-22	ASCE7-16
MCE[1] spectral response acceleration for 0.2 second period, S_S:	0.23g	0.224g
MCE[1] spectral response acceleration for 1.0 second period, S_1:	0.066g	0.074g
Site coefficient, Fa:	--	1.3
Site coefficient, Fv:	--	1.5
MCE[1] spectral response acceleration adjusted for site class, S_{MS}:	0.26g	0.291g
MCE[1] spectral response acceleration adjusted for site class, S_{M1}:	0.096g	0.111g
5% Damped spectral response acceleration, S_{DS}:	0.17g	0.194g
5% Damped spectral response acceleration, S_{D1}:	0.064g	0.074g
NOTE1: MCE = maximum considered earthquake		

2.4 General Subsurface Conditions

S&A obtained information about the subsurface conditions at the site using six (6) hollow-stem auger soil borings to record the soil conditions and collect samples for laboratory testing. The borings were drilled to the depths of 20.1 to 31.0 feet below existing grade (bgs).

A veneer of cobbles (6 inches thick) was observed in three of the six borings. The subsoil conditions at the site generally consist clayey sand and silty/clayey sand to a depth of 4.0 feet below existing grades, underlain by interbedded layers of silty sand, well-graded sand (with and without clay and silt) clayey sand, extending to a depth 31.0 feet below existing grade, the maximum depth of the investigation. In addition, amounts of cobbles and weak degrees of calcareous cementation were noted in the soil profile. The standard penetration resistance test (SPT) values ranged from 16 to 50+ blows per foot (bpf), with the majority of the SPT values being greater than 50 blows per foot, starting at 5 feet below existing grades. Based on visual and tactile observation, the shallow upper soils were primarily in a ‘dry’ state at the time of investigation.

Laboratory testing indicates in-situ dry densities of the upper soils range from 101.6 to 116.0 pounds per cubic foot (pcf) with water contents of 2.4 to 4.8 percent at the time of investigation. Liquid limits range from non-plastic (NP) to 30 percent with plasticity indices of NP to 13 percent. The upper soils exhibit a volume increase (**swell**) due to wetting of < 1**.0 percent** when compacted to moisture and density levels normally expected during construction. ‘Undisturbed’ samples displayed minor to moderate (1.6 to 2.1 percent) compression under incremental loading a maximum confining load of 3,200 pounds per square foot (psf) and moderate to **significant** (2.0 to **7.9 percent**) additional compression due to inundation (**hydro-collapse**).

3.0 ANALYSIS AND RECOMMENDATIONS

3.1 Analysis

It is assumed that the majority of the proposed structure will be supported over the 2-level basement parking garage on the order of 25 feet below existing grade. Based on available drawings, small portions of the basement may be at 1 level below grade. The parking garage will cover the majority of the site. Depending on proximity of the basement to the property lines and infrastructure, shoring may be required. Due to anticipated moderate concentrated loads it is primarily recommended to support the proposed structure on basement level footings or structural mat bearing on undisturbed very dense native soils. **It is recommended to support all foundations in the same bearing media.**

Therefore, if any portion of the hotel structure (porte-cochere) will be **partially** supported beyond the perimeter of the basement walls, consideration will need to be given to deepening footings or using drilled shafts to transfer loads to the same bearing media as the basement level. **Placement of footings bearing in wall backfill material is not recommended.** Any footings located in the **backfill zone** next to the basement wall should be **deepened** below the 'line of influence' to avoid surcharge on the wall. **Footings should be situated such that they are not located within any wall backfill zone** and that a 45-degree plane below an upper foundation does not intersect the walls of an adjacent structure. Special backfill considerations are recommended to mitigate potential settlement of slabs located over the backfill zone (see section 'Fill and Backfill').

'Wet utilities' and/or settlement sensitive utilities should not be placed within the backfill zone of basement walls (except where service connections are required). If there are settlement concerns at connections, we recommend using flexible connections or designing deep support systems.

For below-grade slabs (at basement level), it is anticipated that the underlying soil moisture content will remain relatively constant. Accordingly, no remedial action (such as removal and replacement) is recommended to reduce swell potential. However, if a structural mat slab is not used, it is recommended to design the slab to "float" (i.e., not attached to the foundations) to allow for some minor movement in response to minor moisture changes.

It is assumed that the main structure will be supported on basement level spread footings. For standard foundations to perform as expected, attention must be paid to provide proper drainage to limit the potential for water infiltration of deeper soils. It is assumed that the landscape plan will use mostly low water use or "green" desert type plants (xeriscape). It is preferred to keep irrigated plants at least 5 feet away from structures with irrigation schedules set and maintained to run intermittingly. **We recommend against connecting deck drains and roof drains to the basement shoring wall drain/waterproofing system. Unpaved planter areas should be sloped at least 5 percent for a distance of at least 10 feet away from the building.** It is understood that this may not be possible due to ADA maximum slope requirements for the adjacent sidewalks and patios. The slope may be reduced to 2 percent provided extra care is taken to ensure sidewalks and other hardscape features do not create a "dam" that prevents positive drainage away from the buildings, creating a "pond" adjacent to the building. Roof drainage should also be directed away from the building in paved scuppers. Pre-cast loose splash blocks should not be used as they can be dislodged and/or eroded. Deck/Roof drains should not be allowed to discharge into planters adjacent

to the structure. It is preferred that they be directed to discharge to pavement scuppers (per photo example), retention basins or discharge points located at least 10 feet away from the building.

It is reiterated that shallow spread footings are recommended for isolated structures (if any) since this is the most economical system available. However, this shallow system relies on the dry strength of the unsaturated native soils. A limited depth of re-compaction is recommended for any at-grade structures to increase density of the near surface soils that are more likely to encounter seasonal moisture changes, or deeper foundations. **The deeper native soils are moisture sensitive and could experience differential settlement if subjected to significant surface water infiltration.** Recognizing the need to minimize significant water penetration adjacent to the building perimeter that could detrimentally impact the building foundation, the following additional recommendations are made to protect foundations:

1. Take extra precaution to backfill and compact native soil fill to 95 percent in all exterior wall locations.
2. Avoid utility trenches passing through retention basins leading to the building. If unavoidable, backfill the trench with MAG Section 728 ½-sack CLSM to cut off preferred drainage paths.
3. Avoid placing retention basins next to building foundations. **A distance of at least 10 feet should be maintained between structures and the location of any retention basin maximum fill level.**
4. Underground stormwater retention tanks (if planned) should be located away from foundations to avoid encroaching on the 1:1 line of influence of structure foundations (and far enough for excavation stability). Underground stormwater systems should be fully sealed and watertight to prevent seepage into the foundation zone. **CMP retention tanks are not recommended under any structures, rigid pavement, hardscape or settlement sensitive elements due to potential long-term backfill settlement.**
5. Create and maintain positive drainage away from the exterior wall for a minimum of 10 feet.
6. Avoid sidewalks, curbs or other elements that create a dam that could cause water to pond within 5 feet of the perimeter wall.
7. Include no irrigated landscape materials in the first 3 feet next to the building. **Tree wells are not recommended in basement wall backfill.**
8. Between 3 feet and 5 feet, include only landscape materials that can be irrigated with a maximum of 1 gallon per hour emitter heads. Set and maintain irrigation controllers to prevent 24/7 flows.
9. Any landscape materials requiring greater than 1 gallon per hour irrigation, including turf, shall be at least 5 feet from the outside face of the building.
10. All irrigation feeder lines, other than those that supply individual emitters, shall not be placed closer than 5 feet to the building.

Groundwater is not expected to be a factor in the design or construction of foundations and underground utilities. Shallow excavation operations (upper 5 feet) should be relatively straightforward with standard equipment. **However, deeper excavations will encounter hard/dense and/or cemented soils**

with/without cobbles that may impede excavation progress resulting in hard dig conditions. Secondly, there may be some stability issues with attempting to advance excavations into this zone and with “Running Sand”. Likewise for caisson construction (if needed) will be difficult in the cobble laden soils and “Running Sand” and will likely require full length casing and/or slurry to maintain open shafts or expect large over-runs on concrete volumes. It should be noted that the fact that a boring was advanced to a particular depth should not lead to the assumption that it is necessarily excavatable by conventional means. The excavating contractor should make his own determination of suitable equipment.

Drywell and interceptor chambers (if planned) should be designed with solid upper walls to mitigate potential wetting of subsoils supporting foundations. Drilled piers (if planned) must be located well away from drywell and interceptor chambers (at least 3 shaft diameters) to help mitigate potential collapse or runout of drain rock should sloughing occur in the shaft excavation. Drilled piers must also be designed to avoid imposing lateral loads on retention tanks. To avoid axial load interaction and interference with backfill/open cut they should be located at least 3 shaft diameters or 15 feet away (whichever is greater).

If underground storm water storage is needed, consideration could be given to using sealed underground retention systems (such as precast concrete vaults such as StormCapture™ or Duramax Pipes) with drywells and interceptor chambers. These can be designed to be closer or within the building footprint under garage floor slabs and for potential traffic loading. Foundations systems must still be designed to avoid imposing lateral loads in the vicinity of these retention systems. The foundations must also be sequenced in order to avoid compromising the retention systems and drywells during foundation excavation.

The following recommendations regarding below-grade, basement wall water-proofing and drainage assume that water infiltration from the surface will likely be relatively low-volume, short-term and should dissipate quickly and that the drainage from the podium deck will be directed to a piped drainage system and not be allowed to discharge into the basement wall water-proofing system. The lower level foundations will bear on the dense/hard native soils. To handle low-volume nuisance surface water, it is recommended to include vertical strip or sheet geo-composite drains (i.e. Cetco Aquadrain, AWD Amerdrain) to prevent any hydrostatic build-up that could compromise the wall water-proofing system. Where drainage swales and/or retention basins are planned within 15 feet of basement walls, sheet geo-composite drains and waterproofing is recommended. Although soils at the foundation elevations are permeable, it is recommended to include a detail to bring wall drainage into the basement level above the footings directed to a sump pump system. Drain pipes below basement slabs must be solid wall type. This will reduce the potential for wall drainage to wet the bearing soils causing a loss of support and differential settlement. This will also alleviate additional hydrostatic pressure against the basement walls. **Alternatively**, wall drainage may be connected to an external dry well system.

For exterior slabs on grade, frequent jointing is recommended to control cracking and reduce tripping hazards should differential movement occur. It is also recommended to pin the landing slab to the building floor/stem wall. This will reduce the potential for the exterior slab lifting to block the operation of out-swinging doors. Pinning should be designed by the structural engineer, but typically consists of 24-inch-long No. 4 reinforcing steel dowels placed at 12-inch centers.

3.2 Site Preparation

It is assumed that the basement level excavation will remove all surface features within the storage building limits including vegetation, debris, rubble, foundations, utilities, undocumented fills, and loose surface soils. The remainder of the site at-grade should be stripped of all vegetation, debris, rubble, and obviously loose surface soils. Special attention should be given to the retention area removing any soft wet soils if encountered.

For isolated structures (if any) to be built outside the perimeter of the basement level, the foundations should be deepened to bear on dense native soils at a minimum depth of 4 feet below existing grades or finished grade, whichever is deeper. This can be accomplished by over-excavating the planned footing width and backfilling with a ‘2-sack’ slurry mix (CLSM per M.A.G. Standard Specification section 728). A representative of the geotechnical engineer should examine the subgrade once sub-excavation is complete and prior to backfilling to ensure removal of deleterious materials and confirm bearing media. Fill placement and quality should be as defined in the "Fill and Backfill" section of this report.

Basement level footings excavations must be clean, dry, and free of loose and deleterious material. The footing must extend down into the dense gravelly soil layer. A representative of the geotechnical engineer should examine the footing subgrade to verify adequate cleaning and suitable bearing stratum. Inadvertent footing over-excavation in structures with below-grade levels supported on spread footing foundations should be backfilled with a ‘2-sack’ slurry mix (CLSM per M.A.G. Standard Specification section 728) and **not** engineered soil fill. If loose soils are encountered at the bottom of the excavation, they should be over-excavated down to hard soils and backfilled with a slurry mix in a similar manner.

If any utility is located within 5 feet of any proposed foundation, relocation and/or abandonment of the utility should be provided. They should either be removed and replaced with engineered fill or abandoned in-place. In the case of manholes and pipelines, it may be possible to abandon them in-place. The tops of manholes should be removed and filled with a weak (~500 psi) cementitious grout. Pipelines larger than 6 inches should be capped and filled with grout. If the contractor decides to abandon the pipes in-place, the onus should be put on him to demonstrate that the trench backfill is adequately compacted. Speedie and Associates should be notified of the circumstance for our review. If removal of a pipeline is not possible, the foundations should be deepened to bear in undisturbed soil so that the zone of influence under the foundation does not encroach on the pipeline and/or trench. This zone is any area below a 45° line drawn down and away from the bottom of the foundation edges.

Prior to placing structural fill below footing bottom elevation (at-grade structures, if any), the exposed grade should be scarified to a depth of 8 inches, moisture conditioned to optimum (±2 percent) and compacted to at least 95 percent of maximum dry density as determined by ASTM D-698. Pavement areas should be scarified, moisture-conditioned and compacted in a similar manner.

All cut areas and areas above footing bottom elevation that are to receive floor slab only fill should be scarified 8 inches, moisture conditioned to at least optimum to 3 percent above optimum and compacted to at least 90 percent but not more than 95 percent of maximum dry density as determined by ASTM D-698.

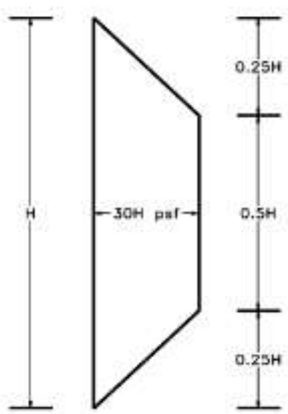
3.3 Excavation and Temporary Cut Slopes

Care should be taken during excavation not to endanger nearby existing structures, roadways, utilities, drainage channels, etc. Depending on proximity, existing structures (including utilities) may require shoring, bracing, or underpinning to provide structural stability and protect personnel working in the excavation.

All excavations must comply with current governmental regulations including the current OSHA Excavation and Trench Safety Standards. Preliminary indications are that the upper ~5 feet of silty gravelly soils would be classified as Type C. Side slopes for open-cut excavation should be cut back at 1.5:1 (horizontal to vertical). Below 5 feet cementation of the gravelly soils will allow for excavation at 1:1 and likely ¾:1 upon further evaluation once exposed. For excavation slope greater than 20 feet a slope stability analysis will be required, contact our office if needed. All slopes should be protected from erosion due to run-off or long-term surcharge at the slope crest. Construction equipment, building materials, excavated soil and vehicular traffic should not be allowed within 8 feet or one-third the slope height, whichever is greater, from the top of slope. **All cut slopes should be observed by the Soils Engineer or contractors qualified person during excavation.** Adjustments to the recommended slopes may be necessary due to wet zones, loose strata and other conditions not observed in the borings. Localized shoring may also be required. Shotcrete or soil stabilizer on the slope face may be useful in preventing erosion due to run-off and/or drying of the slope. Shotcrete protection is recommended for slopes that will remain open for extended periods of time (more than a week). Provision should be made for drainage (such as weep holes) to mitigate potential build-up of hydrostatic pressure below the shotcrete. If seepage from the slopes is encountered during construction, Speedie should be notified so that these recommendations can be reviewed.

3.4 Shoring

Portions of the excavation cuts may encroach on adjacent roadways, adjacent property, utilities and/or buildings. In areas where open-cut excavation is not feasible, consideration must be given to a shoring system. A standard system made up of steel soldier piles, lagging and tiebacks (or interior bracing), depending on depth and loading is one option. This system typically requires pre-drilling and installing heavy steel shoulder beams spaced on 8 to 10-foot centers and backfilled with lean grout. As the excavation progresses, wood lagging can be installed, and tieback anchors installed and tensioned. Cantilever systems may not be possible in the deeper cut areas. For the relatively short periods of time required to install lagging and tiebacks, excavations should stand at vertical. **Sloughing soils may be encountered and require special procedures**. For preliminary design of braced temporary shoring systems, we recommend the following conservative pressure diagram.



H=Depth of Excavation
γ=Unit Wet Soil Weight= 125 pcf

If shoring is required, it may be incorporated into the below-grade wall system whether the wall is cast-in-place or constructed of gunite in top-down construction.

Prior to any excavation work commencing, consideration should be given to pre-construction surveys of surrounding buildings, roadways, utilities, etc. It is recommended that each line of shoring be monitored for movement during the construction period, or at least until the at-grade level is in-place. Frequent monitoring of surrounding elements should also be provided during the construction period.

3.5 Foundation Design

If site preparation is carried out as set forth herein, the following bearing capacities can be utilized for design. No at grade spread footings for occupied structures should be allowed within the basement wall backfill zone due to potential for post construction total and differential settlement. At-grade portions (if any) of the proposed structure should be supported on deep foundations bearing in the same media as the basement foundations. Footings for low screen walls and other hardscape features are acceptable only if settlement risk can be tolerated. **If differential settlement cannot be tolerated, then window walls should be supported on basement walls or grade beams tied to drilled shafts.** See additional discussion in the Section "Fill and Backfill".

Table 3.5.1 Foundation Bearing Capacities

Structure	Foundation Type	Foundation Depth (1)	Bearing Medium	Bearing Capacity	Comments
Minor Structures	Spread	1.5 ft.	Compacted Subgrade	1,500 psf	2
At-Grade Portions	Spread	4.0 ft.	Dense Native or CLSM	2,500 psf	3
	Drilled Shaft	25 ft. min.	Dense/Hard Undisturbed Native Soils	See charts	4
Basement Level Structures	Spread	2.0 ft.	Dense/Hard Undisturbed Native Soils	8,000 psf	5

Comments:

1. Foundation Depth refers to minimum depth below finished existing grade within 5 feet, slab level, or basement finished floor elevation, whichever is greater.
2. Minor structures such as screen walls, planter walls, etc. not connected to any main structure. The bottom of footing excavation should be scarified to a depth of 8 inches, moisture-conditioned to optimum (±2 percent) and compacted to at least 95 percent of maximum dry density as determined by ASTM D-698.
3. Shallow spread footings bearing on dense native soils at a minimum depth of 4 feet below finished grade or deeper to be out of the zone of influence of the basement walls or through any basement wall backfill material. Footings can be deepened by over-excavating the planned footing width and backfilling with 2-sack slurry.
4. Drilled shafts bearing on dense native soils at least 25 feet below existing grade or finished grade, whichever is greater. Porte Cochere may be supported on shorter shafts (10 feet deep, minimum). Shafts must be deepened to dense bearing material if loose soils are encountered at design tip depth. It is possible that lateral and uplift loading will govern the required length of the shaft and not the end bearing capacity. Refer to the drilled shaft capacity charts in the appendix.
5. Basement level is assumed to be at approximately 25 feet below existing grade or greater. Shallow spread footings bearing on **dense/very dense undisturbed** native soils at a **minimum depth of 2 feet below** finished basement level floor elevation. For any isolated footings not exposing suitable bearing soils, the footing should be over-excavated to remove unsuitable soils and backfilled back to bottom of footing with MAG Spec 728 2-sack CLSM or structural concrete.

These bearing capacities refer to the total of all loads, dead and live, and are net pressures. They may be increased one-third for wind, seismic or other loads of short duration. These values may be increased by one-third as the allowable toe pressure for retaining walls. All footing excavations should be level and cleaned of all loose or disturbed materials. **Positive drainage away from the proposed buildings must always be maintained.**

Continuous wall footings and isolated rectangular footings should be designed with minimum widths of 16 and 24 inches respectively, regardless of the resultant bearing pressure. Lightly loaded interior partitions (less than 800 plf) may be supported on reinforced thickened slab sections (minimum 12 inches of bearing width).

Caissons should consist of drilled shaft foundations bearing in the dense to dense native soils. A minimum caisson length 25 feet is recommended for portions of the main structure supported at grade (if any). The Porte Cochere may be supported on shorter shafts, at least 10 feet below existing grade or finished grade, whichever is deeper. **Sloughing could occur** in silty sand and well graded sand layers (if encountered) resulting in concrete quantities higher than neat dimension calculations. Drilled shafts should not be left open overnight to reduce the amount of sloughing (or possible seasonal water intrusion) that may occur. A minimum shaft diameter of 30 inches is recommended to allow access for cleaning and inspection. All caissons should be examined by a representative of the Geotechnical Engineer to verify cleaning, depth, dimensions, and proper bearing strata. Straight shaft caissons may be "machine cleaned" provided the contractor can show the ability to adequately remove loose material. Adjacent caisson base (tip) elevations should not vary by more than 45 degrees.

A minimum allowable distance of 3 caisson diameters, center-to-center, is recommended between caissons for reasons of construction safety and to reduce **axial** group action. This limitation ensures that newly placed caissons are not damaged during the subsequent placement of adjacent caissons. This distance may be reduced to 2 diameters if one of the caissons has been in place for enough time to allow concrete to set and cure. A load bearing reduction factor of 0.7 should be applied to individual caissons within a proximity of two diameters, center-to-center, of each other. If adjacent caissons are of different diameters, an average diameter should be used for determining spacing. A separate set of group reduction factors should be applied for **lateral** load conditions. Alternatively, Speedie and Associates can provide lateral load analysis for selected cases using L-Pile+ (Version 2013, by Ensoft) on request (at additional cost).

Estimated settlements under design loads are on the order of ¾ to 1-inch, virtually all of which will occur during construction. Post-construction differential settlements will be on the order of one-half the total settlement, under existing and compacted moisture contents. Additional localized settlements (or heave) of the same magnitude could occur if native supporting soils were to experience a significant increase in moisture content. **Positive drainage away from structures and controlled routing of roof runoff must be provided and maintained to prevent ponding adjacent to perimeter walls.** Planters requiring heavy watering should **not** be placed adjacent to or within 5 feet of the building. Care should be taken in design and construction to ensure that domestic and interior storm drain water is contained to prevent seepage. Roof drainage should be directed to paved areas or storm drains. They should not discharge into planters adjacent to the structures.

Continuous footings and stem walls should be reinforced to distribute stresses arising from small differential movements, and long walls should be provided with control joints to accommodate these movements. Reinforcement and frequent control joints are suggested to allow slight movement and prevent minor floor slab cracking especially in floor areas to be covered with hard tile.

3.6 Lateral Pressures

The following lateral pressure values may be utilized for the proposed construction assuming unsaturated conditions. These lateral earth pressures are not applicable for submerged soils or hydrostatic loading. Additional recommendations may be required if such conditions exist and/or are to be included in the design.

Active Pressures	
Unrestrained Walls	35 psf/ft
At-Rest Pressures	
Restrained Walls	60 psf/ft
Passive Pressures	
Continuous Footings	300 psf/ft
Spread Footings	350 psf/ft
Coefficient of Friction (w/ passive pressure)	0.35
Coefficient of Friction (w/out passive pressure)	0.45

All backfill must be compacted to not less than 95 percent (ASTM D-698) to mobilize these passive values at low strain. Expansive soils should not be used as retaining wall backfill, except as a surface seal to limit infiltration of storm/irrigation water. The expansive pressures could greatly increase active pressures.

3.7 Fill and Backfill

Native fine grained soils are considered suitable for use in general grading and engineered fills, provided oversized material greater than 3 inches is screened out. Soils with a classification (USC) of CL should **not** be used in the top foot of pad fill or as wall backfill. The silty fine sand soils may be sensitive to excessive moisture content and will become unstable at elevated moisture content. Accordingly, it may be necessary to compact soils on the dry side of optimum, especially in asphalt pavement areas. The reduced moisture content under slabs-on-grade should only be used upon approval of the engineer in the field.

Successful backfill of below-grade basement walls can be difficult to achieve given the sometimes-tight access, especially a narrow slot between a shored excavation and formed wall. A well-graded granular import (i.e. MAG Spec 702 Select or AB) should be specified for backfill of below grade walls. Placement and compaction must be carefully controlled to minimize the potential for post construction settlement should the backfill zone be subjected to water infiltration. Even the most well controlled fills could experience additional settlement on the order of 1 inch if subjected to significant moisture increases. **Accordingly, it is recommended to design and construct a structural slab over the backfill zone in the most critical areas such as interior slabs or reinforce and pin the landing/entry slabs to the building stem wall to span over the backfill zone.** This will reduce the potential for the exterior slab dropping and creating a tripping hazard. Critical areas can be considered to include not only concrete walkways and slabs, but also concrete and asphaltic concrete paving. Paving over wall backfill zones should be detailed to minimize the effects of backfill settlement. **Utility lines (especially gravity sewer lines), except for building service connections, should be avoided in this zone or be designed to span over the zone.** Where critical piping sensitive to settlement is required, grade beams to transfer across the fill zone should be considered. Heavy compaction equipment should not be used next to basement walls, only **hand operated** equipment.

There may be cases where there is a narrow (5± foot) backfill zone between shoring and cast in place basement walls. If it is not possible to compact well graded granular fill, the use of washed crushed rock (pea gravel or ASTM C33 No. 56 or 57 rock or other approved uniform graded rock) is allowable. Contrary to popular belief that this material "falls" into place at maximum relative density when dumped, it is recommended to vibrate this material in place in 4-foot maximum lifts. This will allow the material to settle into a maximum relative density condition and reduce the potential for post construction settlement. This material should be brought up to grade under any hardscape surface. In any areas where fine grained soils are used to complete the surface, a filter fabric (Mirafi 140N or equal) should be placed between the coarse rock and surface fines to prevent fined from filtering down into the open graded rock.

In areas where it is not possible to fill and properly compact with stone due to interference from utilities, etc., it is recommended to use 1-sack slurry per MAG Section 728 Controlled Low Strength Material.

A pre-construction meeting should be held prior to starting the basement wall backfill to discuss the staging process and the procedures used for backfilling, to help minimize the potential for basement wall backfill settlement.

If imported common fill for use in site grading is required, it should be examined by a Soils Engineer to ensure that it is of low swell potential and free of organic or otherwise deleterious material. In general, the fill should have 100 percent passing the 3-inch sieve and no more than 40 percent passing the #200 sieve. For the fine fraction (passing the 40 sieve), the liquid limit and plasticity index should not exceed 30 percent and 10 percent, respectively. It should exhibit less than 1.5 percent swell potential when compacted to 95 percent of maximum dry density (ASTM D-698) at a moisture content of 2 percent below optimum, confined under a 100 psf surcharge, and inundated.

Fill should be placed on subgrade which has been properly prepared and approved by a Soils Engineer. Fill must be wetted and thoroughly mixed to achieve optimum moisture content, ±2 percent. Fill should be placed in horizontal lifts of 8-inch thickness (or as dictated by compaction equipment) and compacted to the percent of maximum dry density per ASTM D-698 set forth as follows:

A.	Building Areas	
	1. Below at-grade footing level	95
	2. Below basement level footings	2 sack CLSM
	3. Below slabs-on-grade (non-expansive soils)	95
B.	Pavement Subgrade or Fill	95
C.	Utility Trench Backfill	95
D.	Aggregate Base Course	
	1. Below floor slabs	95
	2. Below asphalt paving	100
E.	Landscape Areas	90

3.8 Utilities Installation

Trench excavations for **very shallow** utilities (less than 5 feet) can be accomplished by conventional trenching equipment. Excavation into the more highly cemented, or cobble laden soils may require heavy equipment. The fact that a boring or test pit was advanced to a certain depth does not mean that the soils may be excavated by normal means. The excavating contractor must make their own assessment as to excavatability. Trench walls should stand near vertical for the short periods of time required to install shallow utilities **although some sloughing may occur in the upper coarse-grained soils requiring laying back of side slopes** and/or temporary shoring. Adequate precautions must be taken to protect workmen in accordance with all current governmental regulations.

Backfill of narrow utility trenches **above** bedding and initial backfill zones may be carried out with native excavated material, provided material greater than 3-inches is either removed/screened out or reduced in size. This material should be moisture-conditioned, placed in 8-inch lifts and mechanically compacted. Water settling is not recommended. Compaction requirements are summarized in the "Fill and Backfill" section of this report. Native soils do not meet the typical granular bedding and initial backfill (see MAG Standard Specification Section 601.4.4) requirements of large diameter CMP tanks. These materials need to meet MAG Standard Specification Section 601 or the drainage engineers design and manufacture recommendations.

Underground CMP storm water storage tanks are not recommended below structures, rigid pavement, pavers, hardscape, or settlement sensitive elements due to potential long-term backfill settlement. Flexible pavement over the backfill zone will be subject to long-term maintenance for the same reason.

3.9 Slabs-On-Grade

To facilitate fine grading operations and aid in concrete curing, a 4-inch-thick layer of granular material conforming to the gradation for aggregate base (A.B.) as per M.A.G. Specification Section 702 should be utilized beneath the slab. Dried subgrade soils **must** be re-moistened prior to placing the aggregate base if allowed to dry out, especially if fine-grained soils are used in the top 12-inches of the pad.

The native soils can store a significant amount of moisture, which could increase the natural vapor drive through the slab. Accordingly, if moisture sensitive flooring and/or adhesive are planned, (or desire to add a layer of protection against possible vapor encroachment) the use of a vapor barrier or low permeability concrete should be considered. Vapor barriers should be a minimum 15-mil thick polyolefin (or equivalent), which meets ASTM E 1745 Class A specifications. Vapor barriers do increase the potential for slab curling and water entrapment under the slab. Accordingly, if a vapor barrier is used, additional precautions such as low slump concrete, frequent jointing and proper curing will be required to reduce curling potential and detailed to prevent the entrapment of outside water sources.

3.10 Asphalt/Concrete Pavement Design

If earthwork in paved areas is carried out to finish subgrade elevation as set forth herein, the subgrade will provide adequate support for pavements. The location designation is for reference only. **The designer/owner should choose the appropriate sections to meet the anticipated traffic volume and life expectancy.** The section capacity is reported as daily ESALs, Equivalent 18-kip Single Axle Loads. Typical heavy trucks impart 1.0 to 2.5 ESALs per truck depending on load. It takes approximately 1200 passenger cars to impart 1 ESAL.

Table 3.10.1 Pavement Sections

Area of Placement	Flexible (AC Pavement)			Rigid (PCC Pavement)	
	Thickness AC (0.39)	Thickness ABC (0.12)	Daily 18-kip ESALs	Thickness PCCP	Daily 18-kip ESALs
Auto Parking	2.0"	4.0"	6	4.5"	6
Truck Parking, Main Drives, & Fire Lanes	3.0"	4.0"	25	6.0"	25
	3.0"	6.0"	57	7.0"	57

Notes:

1. Designs are based on AASHTO design equations and ADOT correlated R-Values.
2. The PCCP thickness is increased to provide better load transfer and reduce potential for joint & edge failures. Design PCCP per ACI 330.
3. Full depth asphalt or increased asphalt thickness can be increased by adding 1.0-inch asphalt for each 3 inches of base course replaced.

Pavement Design Parameters:

Assume:	One 18-kip Equivalent Single Axle Load (ESAL)/Truck
Life:	20 years

Subgrade Soil Profile:

% Passing #200 sieve:	28 percent
Plasticity Index:	10 percent
k:	125 pci (assumed)
R value:	46 (per ADOT tables)
M_R:	26,000 (Maximum per AASHTO design)

These designs assume that all subgrades are prepared in accordance with the recommendations contained in the "Site Preparation" and "Fill and Backfill" sections of this report, and paving operations carried out in a proper manner. If pavement subgrade preparation is not carried out immediately prior to paving, the entire area should be proof-rolled at that time with a heavy pneumatic-tired roller to identify locally unstable areas for repair.

Pavement base course material should be aggregate base per M.A.G. Section 702 Specifications. Asphalt concrete materials and mix design should conform to M.A.G. 710. It is recommended that a ½ inch or ¾ inch mix designation be used for the pavements. While a ¾ inch mix may have a somewhat rougher texture, it offers more stability and resistance to scuffing, particularly in truck turning areas. Pavement installation should be carried out under applicable portions of M.A.G. Section 321 and municipality standards. The asphalt supplier should be informed of the pavement use and required to provide a mix that will provide stability and be aesthetically acceptable. Some of the newer M.A.G. mixes are very coarse and could cause placing and finish problems. A mix design should be submitted for review to determine if it will be acceptable for the intended use.

For sidewalks and other areas not subjective to vehicular traffic a 4-inch section of concrete will be sufficient. For trash and dumpster enclosures a thicker section of 6 inches of concrete is recommended.

Portland Cement Concrete Pavement must have a minimum 28-day flexural strength of 550 psi (compressive strength of approximately 3,700 psi). It may be cast directly on the prepared subgrade with proper compaction (reduced) and the elevated moisture content as recommended in the report. Lacking an aggregate base course, attention must be paid to using low slump concrete and proper curing, especially on the thinner sections. No reinforcement is necessary. Joint design and spacing should be in accordance with ACI recommendations. Construction joints should contain dowels or be tongue and grooved to provide load transfer. Tie bars are recommended on the joints adjacent to unsupported edges. Maximum joint spacing in feet should not exceed 2 to 3 times the thickness in inches. Joint sealing with a quality silicone sealer is recommended to prevent water from entering the subgrade allowing pumping and loss of support.

Proper subgrade preparation and joint sealing will reduce (but not eliminate) the potential for slab movements (thus cracking) on the expansive native soils. Frequent jointing will reduce uncontrolled cracking and increase the efficiency of aggregate interlock joint transfer.

4.0 GENERAL

The scope of this investigation and report includes only regional published considerations for seismic activity and ground fissures resulting from subsidence due to groundwater withdrawal, not any site-specific studies. The scope does not include any considerations of hazardous releases or toxic contamination of any type.

Our analysis of data and the recommendations presented herein assume that soil conditions do not vary significantly from those found at specific sample locations. Our work has been performed in accordance with generally accepted engineering principles and practice; this warranty is in lieu of all other warranties expressed or implied.

We recommend that a representative of the Geotechnical Engineer observe and test the earthwork and foundation portions of this project to ensure compliance to project specifications and the field applicability of subsurface conditions which are the basis of the recommendations presented in this report. If any significant changes are made in the scope of work or type of construction that was assumed in this report, we must review such revised conditions to confirm our findings if the conclusions and recommendations presented herein are to apply.

Respectfully submitted,
SPEEDIE & ASSOCIATES, LLC

Guadalupe Carrillo, E.I.T.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
38785
BRIAN E
LINGNAU
Date Signed 6/19/2025
ARIZONA U.S.A.

Brian Lingnau, Ph.D., P.E.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
37292
KEITH R.
GRAVEL
Date Signed 6/19/2025
ARIZONA, U.S.A.

Keith R. Gravel, P.E.

SPEEDIE AND ASSOCIATES
Geotechnical • Environmental • Materials Engineers
A UES Company

APPENDIX
(S&A Report 151276SA)

FIELD AND LABORATORY INVESTIGATION

SOIL BORING LOCATION PLAN

SOIL LEGEND

LOG OF TEST BORINGS

TABULATION OF TEST DATA

CONSOLIDATION TEST

MOISTURE-DENSITY RELATIONS

SWELL TEST DATA

DRILLED SHAFT CAPACITY CHARTS

SPEEDIE AND ASSOCIATES
Geotechnical • Environmental • Materials Engineers
A UES Company

FIELD AND LABORATORY INVESTIGATION

On August 19, 2015, soil test borings were drilled at the approximate locations shown on the attached Soil Boring Location Plan. All exploration work was carried out under the full-time supervision of our field engineer, who recorded subsurface conditions and obtained samples for laboratory testing. The soil borings were advanced with a truck-mounted CME-75 drill rig utilizing 7-inch diameter hollow stem flight augers. Detailed information regarding the borings and samples obtained can be found on an individual Log of Test Boring for each drilling location.

Laboratory testing consisted of moisture content, dry density, grain-size distribution, and plasticity (Atterberg Limits) tests for classification and pavement design parameters. Remolded swell tests were performed on samples compacted to densities and moisture contents expected during construction. Compression tests were performed on a selected ring sample to estimate settlements and determine effects of inundation. All field and laboratory data are presented in this appendix.

DR: NA	CHK: XXX	DATE: 06/17/25	PROJECT NO.: 251041SA	SHEET: 1 OF 1

SOIL BORING LOCATION PLAN	FIRA LUXURY BOUTIQUE HOTEL 18211 NORTH PIMA ROAD SCOTTSDALE, ARIZONA	SPEEDIE AND ASSOCIATES A UES Company

SOIL LEGEND

SAMPLE DESIGNATION	DESCRIPTION	
AS	**Auger Sample**	A grab sample taken directly from auger flights.
BS	**Large Bulk Sample**	A grab sample taken from auger spoils or from bucket of backhoe.
S	**Spoon Sample**	Standard Penetration Test (ASTM D-1586) Driving a 2.0 inch outside diameter split spoon sampler into undisturbed soil for three successive 6-inch increments by means of a 140 lb. weight free falling through a distance of 30 inches. The cumulative number of blows for the final 12 inches of penetration is the Standard Penetration Resistance.
RS	**Ring Sample**	Driving a 3.0 inch outside diameter spoon equipped with a series of 2.42-inch inside diameter, 1-inch long brass rings, into undisturbed soil for one 12-inch increment by the same means of the Spoon Sample. The blows required for the 12 inches of penetration are recorded.
LS	**Liner Sample**	Standard Penetration Test driving a 2.0-inch outside diameter split spoon equipped with two 3-inch long, 3/8-inch inside diameter brass liners, separated by a 1-inch long spacer, into undisturbed soil by the same means of the Spoon Sample.
ST	**Shelby Tube**	A 3.0-inch outside diameter thin-walled tube continuously pushed into the undisturbed soil by a rapid motion, without impact or twisting (ASTM D-1587).
--	**Continuous Penetration Resistance**	Driving a 2.0-inch outside diameter "Bullnose Penetrometer" continuously into undisturbed soil by the same means of the spoon sample. The blows for each successive 12-inch increment are recorded.

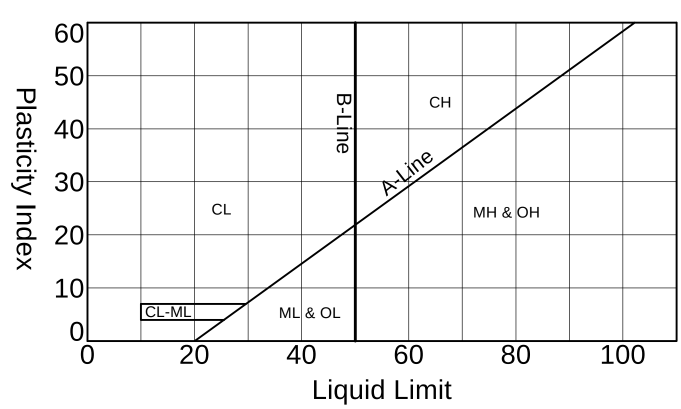
CONSISTENCY			RELATIVE DENSITY	
Clays & Silts	Blows/Foot	Strength (tons/sq ft)	Sands & Gravels	Blows/Foot
Very Soft	0 - 2	0 - 0.25	Very Loose	0 - 4
Soft	2 - 4	0.25 - 0.5	Loose	5 - 10
Firm	5 - 8	0.5 - 1.0	Medium Dense	11 - 30
Stiff	9 - 15	1 - 2	Dense	31 - 50
Very Stiff	16 - 30	2 - 4	Very Dense	> 50
Hard	> 30	> 4		

MAJOR DIVISIONS			SYMBOLS (LETTER)	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS; MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS; MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS; MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS; MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

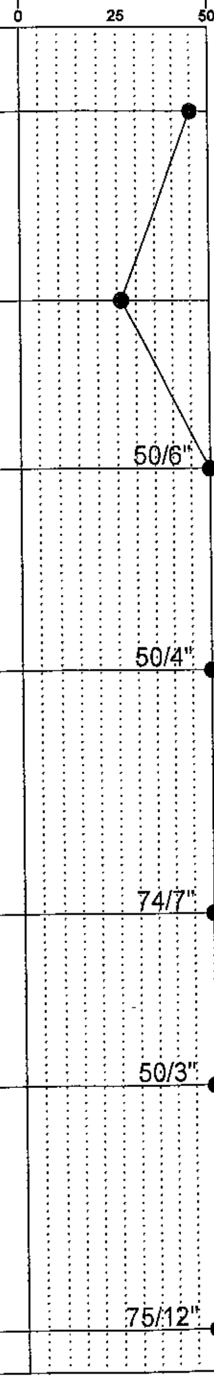
NOTE: DUAL OR MODIFIED SYMBOLS MAY BE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS OR TO PROVIDE A BETTER GRAPHICAL PRESENTATION OF THE SOIL

MATERIAL SIZE	PARTICLE SIZE			
	Lower Limit		Upper Limit	
	mm	Sieve Size ◆	mm	Sieve Size ◆
SANDS				
Fine	0.075	#200	0.42	#40
Medium	0.420	#40	2.00	#10
Coarse	2.000	#10	4.75	#4
GRAVELS				
Fine	4.75	#4	19	0.75" ×
Coarse	19	0.75" ×	75	3" ×
COBBLES	75	3" ×	300	12" ×
BOULDERS	300	12" ×	900	36" ×

◆U.S. Standard ×Clear Square Openings



Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot
0–3.0	Dense Light Brown CLAYEY SAND (SC-Dry) with Some Gravel	RS-1	2.0	2.4	116.0	
3.0–8.0	Medium Dense Light Brown WELL GRADED SAND with CLAY (SW/SC-Dry to Moist) with Trace Gravel	S-2	6.5	NT	NT	
8.0–17.0	Very Dense Light Brown CLAYEY SAND (SC-Dry) with Trace Gravel	S-3	10.5	NT	NT	50/6"
		S-4	15.3	NT	NT	50/4"
17.0–31.0	Very Dense Light Brown SILTY SAND (SM-Dry) with Weak Calcareous Cementation	S-5	21.1	NT	NT	74/7"
		S-6	25.2	NT	NT	50/3"
31.0	End of Boring	S-7	31.0	NT	NT	75/12"



Rig Type: **CME-75**
Boring Type: **Hollow Stem Auger**
Surface Elevation: **N/A**

Boring Date: **8-19-15**
Field Engineer/Technician: **R. Markley**
Driller: **B. Anderson**
Contractor: **Geomechanics SW**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 1**

Trailside Hotel

Trailside View east of Pima Road

Scottsdale, Arizona

Project No.: **151276SA**

_SPEEDIE 151276SA.GPJ GENGEO.GDT 9/8/15

Rig Type:	**CME-75**
Boring Type:	**Hollow Stem Auger**
Surface Elevation:	**N/A**

Visual Classification

Depth (feet)	Visual Classification
0 – 0.5	6.0" Cobbles
0.5 – 4.0	Medium Dense Light Brown CLAYEY SAND (SC-Dry to Moist) withSome Gravel
4.0 – 9.0	Medium Dense Light Brown WELL GRADED SAND with CLAY (SW/SC-Dry to Moist) with Trace Gravel
9.0 – 18.0	Very Dense Light Brown CLAYEY SAND (SC-Dry) with Trace Gravel
18.0 – 30.2	Very Dense Light Brown SILTY SAND (SM-Dry) with Trace Gravel, Weak Calcareous Cementation
30.2	End of Boring

Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
S-1	2.5	NT	NT	
BS-2	5.0	NT	NT	
S-3	6.5	NT	NT	
S-4	11.3	NT	NT	69/10"
S-5	15.7	NT	NT	91/8"
S-6	20.1	NT	NT	50/2"
S-7	25.3	NT	NT	50/4"
S-8	30.2	NT	NT	50/3"

Boring Date:	**8-19-15**
Field Engineer/Technician:	**R. Markley**
Driller:	**B. Anderson**
Contractor:	**Geomechanics SW**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 2**

Trailside Hotel

Trailside View east of Pima Road

Scottsdale, Arizona

Project No.: **151276SA**

Rig Type: **CME-75**
Boring Type: **Hollow Stem Auger**
Surface Elevation: **N/A**

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0–25–50)
0–4.0	Dense Light Brown CLAYEY SAND (SC-Dry) with Some Gravel; 1-10% Cobble	RS-1	2.0	3.3	108.8	
4.0–9.0	Very Dense Brown WELL GRADED SAND with CLAY (SW/SC-Dry) with Some Gravel, 1-10% Cobble	S-2	6.1	NT	NT	80/7"
9.0–13.0	Very Dense Light Brown CLAYEY SAND (SC-Dry) with Trace Gravel	S-3	10.4	NT	NT	50/5"
13.0–20.1	Very Dense Light Brown SILTY SAND (SM-Dry) with Trace Gravel, Weak Calcareous Cementation	S-4	15.3	NT	NT	50/4"
20.1	End of Boring	S-5	20.1	NT	NT	50/1"

Boring Date: **8-19-15**
Field Engineer/Technician: **R. Markley**
Driller: **B. Anderson**
Contractor: **Geomechanics SW**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 3**

Trailside Hotel

Trailside View east of Pima Road

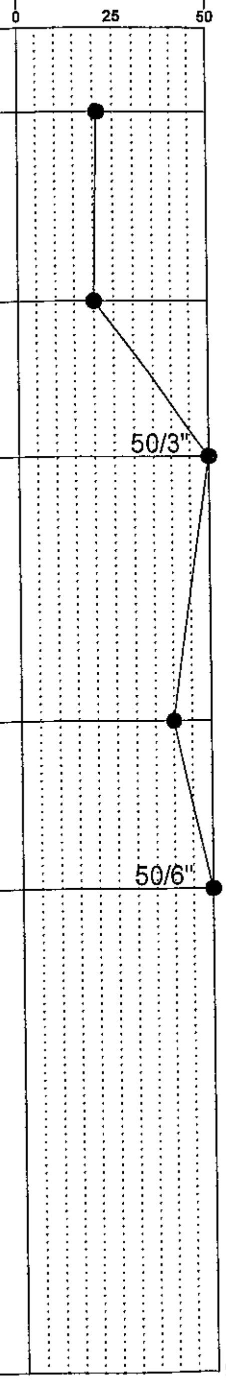
Scottsdale, Arizona

Project No.: **151276SA**

_SPEEDIE 151276SA.GPJ GENGEO.GDT 9/8/15

Rig Type:	**CME-75**
Boring Type:	**Hollow Stem Auger**
Surface Elevation:	**N/A**

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot
0 – 0.5	6.0" Cobbles					
0.5 – 4.0	Medium Dense Brown SILTY, CLAYEY SAND (SC/SM-Dry to Moist) with Some Gravel	RS-1	2.0	4.8	109.5	
4.0 – 12.0	Medium Dense Brown WELL GRADED SAND with CLAY (SW/SC-Dry to Moist) with Trace Gravel	S-2	6.5	NT	NT	
	Very Dense, Some Gravel	S-3	10.2	NT	NT	50/3"
12.0 – 17.0	Dense Light Brown WELL GRADED SAND with SILT (SW/SM-Dry) with Trace Gravel	S-4	16.5	NT	NT	
17.0 – 20.5	Very Dense Light Brown SILTY SAND (SM-Dry) with Trace Gravel, Weak Calcareous Cementation	S-5	20.5	NT	NT	50/6"
20.5	End of Boring					



Boring Date:	**8-19-15**
Field Engineer/Technician:	**R. Markley**
Driller:	**B. Anderson**
Contractor:	**Geomechanics SW**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 4**

Trailside Hotel

Trailside View east of Pima Road

Scottsdale, Arizona

Project No.: **151276SA**

Rig Type: **CME-75**
Boring Type: **Hollow Stem Auger**
Surface Elevation: **N/A**

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0–25–50)
0 – 0.5	6.0" Cobbles					
0.5 – 4.0	Dense Light Brown <u>SILTY, CLAYEY SAND</u> (SC/SM-Dry) with Trace Gravel and Weak Calcareous Cementation	S-1	2.5	NT	NT	
4.0 – 8.0	Very Dense Light Brown <u>SILTY SAND</u> (SM-Dry) with Little Gravel	RS-2	5.5	3.9	101.6	50/6"
8.0 – 17.0	Very Dense Light Brown <u>WELL GRADED SAND with SILT</u> (SW/SM-Dry) with Trace Gravel, Weak Calcareous Cementation	S-3	11.3	NT	NT	90/10"
		S-4	16.0	NT	NT	75/12"
17.0 – 21.5	Medium Dense Light Brown <u>WELL GRADED SAND with CLAY</u> (SW/SC-Dry) with Trace Gravel	S-5	21.5	NT	NT	
21.5	End of Boring					

Boring Date: **8-19-15**
Field Engineer/Technician: **R. Markley**
Driller: **B. Anderson**
Contractor: **Geomechanics SW**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 5**

Trailside Hotel

Trailside View east of Pima Road

Scottsdale, Arizona

Project No.: **151276SA**

_SPEEDIE 151276SA.GPJ GENGEO.GDT 9/8/15

Rig Type:	CME-75
Boring Type:	Hollow Stem Auger
Surface Elevation:	N/A

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot
0	Medium Dense Brown CLAYEY SAND (SC-Dry) with Trace Gravel	RS-1	2.0	4.4	102.2	
3.0	Very Dense Light Brown SILTY SAND (SM-Dry) with Trace Gravel and Weak Calcareous Cementation	S-2	6.5	NT	NT	55/12"
8.0	Dense Brown WELL GRADED SAND with CLAY (SW/SC-Dry) with Trace Gravel	S-3	11.5	NT	NT	
	Very Dense	S-4	15.8	NT	NT	78/9"
20.1	End of Boring	S-5	20.1	NT	NT	50/1"

Boring Date: 8-19-15
Field Engineer/Technician: R. Markley
Driller: B. Anderson
Contractor: Geomechanics SW

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 6**

Trailside Hotel

Trailside View east of Pima Road

Scottsdale, Arizona

Project No.: **151276SA**

SPEEDIE 151276SA.GPJ GENGEO.GDT 9/8/15

TABULATION OF TEST DATA

SOIL BORING or TEST PIT NUMBER	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE INTERVAL (ft)	NATURAL WATER CONTENT (Percent of Dry Weight)	IN-PLACE DRY DENSITY (Pounds Per Cubic Foot)	PARTICLE SIZE DISTRIBUTION (Percent Finer)					ATTERBERG LIMITS			UNIFIED SOIL CLASSIFICATION	SPECIMEN DESCRIPTION
						#200 SIEVE	#40 SIEVE	#10 SIEVE	#4 SIEVE	3" SIEVE	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX		
B- 1	RS-1	RING	1.0 - 2.0	2.4	116.0	NT	NT	NT	NT	NT	NT	NT	NT		
B- 2	BS-2	BULK	0.0 - 5.0	NT	NT	21	47	69	79	100	28	15	13	SC	CLAYEY SAND with GRAVEL
B- 3	RS-1	RING	1.0 - 2.0	3.3	108.8	NT	NT	NT	NT	NT	NT	NT	NT		
B- 4	RS-1	RING	1.0 - 2.0	4.8	109.5	23	39	57	73	100	23	18	5	SC-SM	SILTY, CLAYEY SAND with GRAVEL
B- 5	RS-2	RING	5.0 - 5.5	3.9	101.6	22	37	61	85	100	NP	NP	NP	SM	SILTY SAND
B- 6	RS-1	RING	1.0 - 2.0	4.4	102.2	42	62	79	91	100	30	17	13	SC	CLAYEY SAND

Sieve analysis results do not include material greater than 3". Refer to the actual boring logs for the possibility of cobble and boulder sized materials.

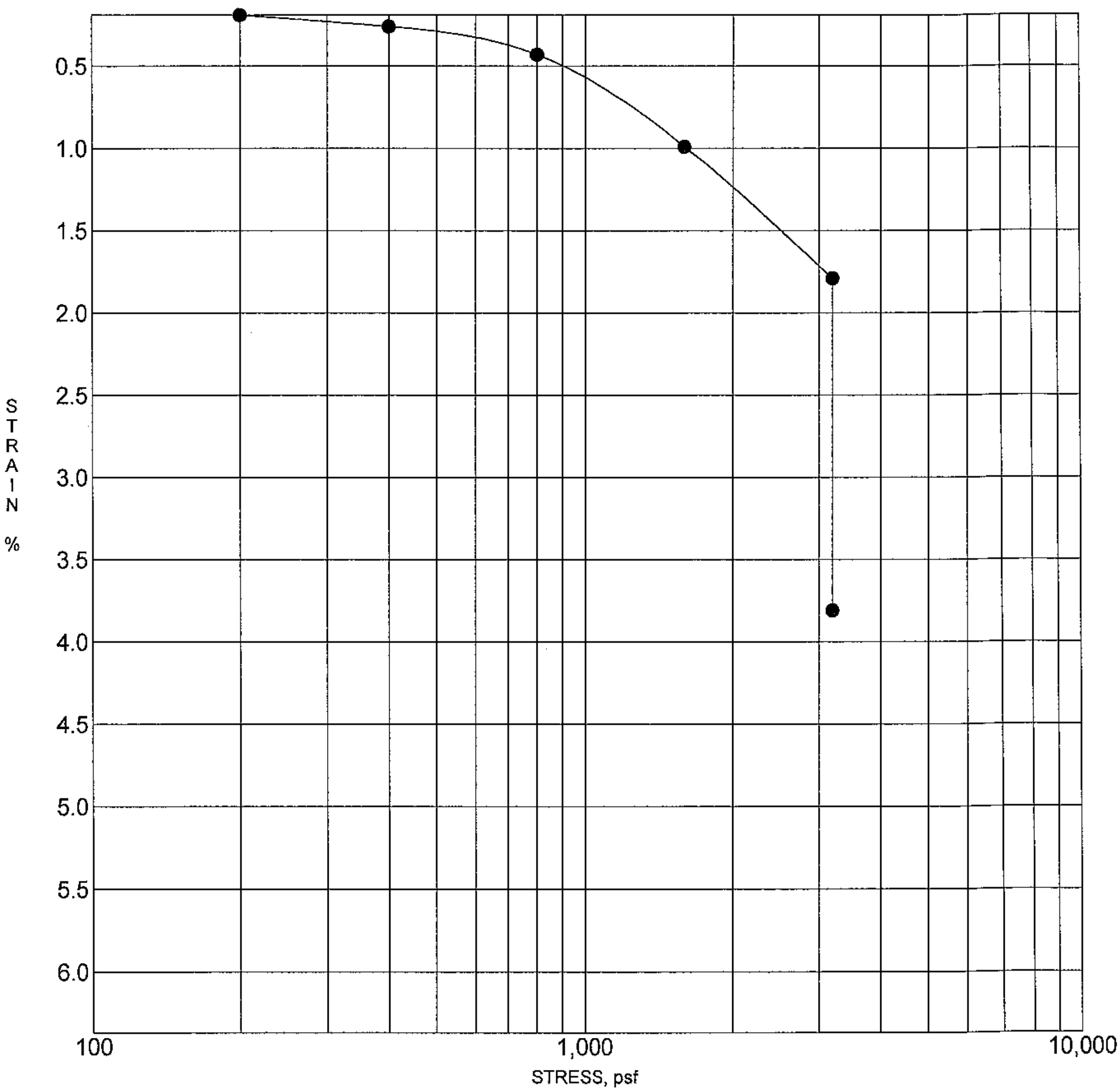
NT=Not Tested

Trailside Hotel
Trailside View east of Pima Road
Scottsdale, Arizona
Project No. 151276SA

SPEEDIE AND ASSOCIATES

CONSOLIDATION TEST

PROJECT: Trailside Hotel PROJECT NO.: 151276SA
LOCATION: Trailside View east of Pima Road DATE: 8/19/15
BORING NO.: B- 5 SAMPLE NO.: RS-2 SAMPLE DEPTH: 5 to 5.5 LABORATORY NO.:
LIQUID LIMIT: NP PLASTIC LIMIT: NP PLASTICITY INDEX: NP
CLASSIFICATION: SM ASTM SOIL DESCRIPTION: SILTY SAND



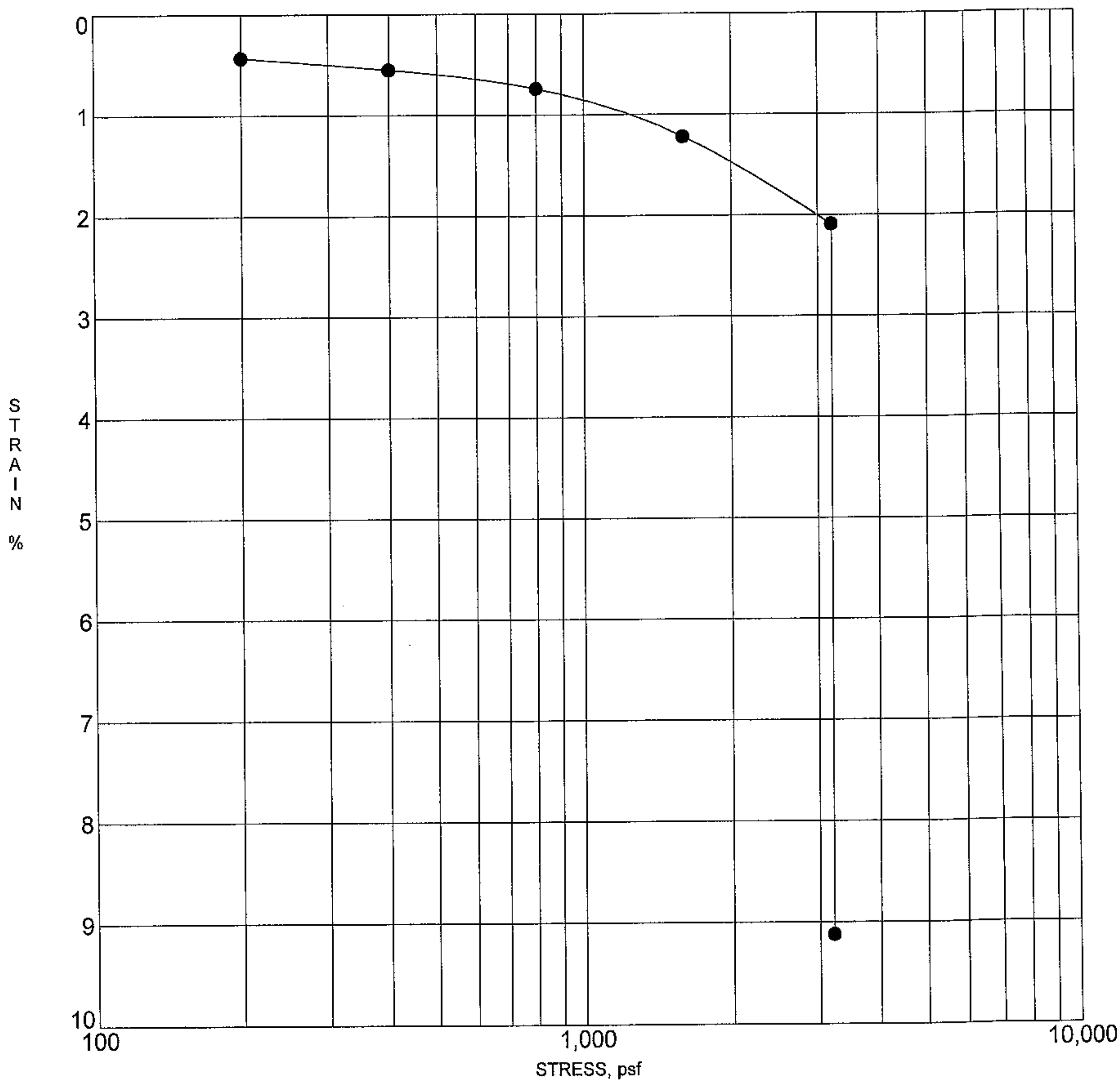
Sample inundated at end of test at 3200 psf

SPEEDIE AND ASSOCIATES

GEOTECH CONSOLIDATION 151276SA.GPJ GENGEO.GDT 9/8/15

CONSOLIDATION TEST

PROJECT: Trailside Hotel
PROJECT NO.: 151276SA
LOCATION: Trailside View east of Pima Road
DATE: 8/19/15
BORING NO.: B- 4 SAMPLE NO.: RS-1 SAMPLE DEPTH: 1 to 2
LABORATORY NO.:
LIQUID LIMIT: 23 PLASTIC LIMIT: 18 PLASTICITY INDEX: 5
CLASSIFICATION: SC-SM ASTM SOIL DESCRIPTION: SILTY, CLAYEY SAND with GRAVEL



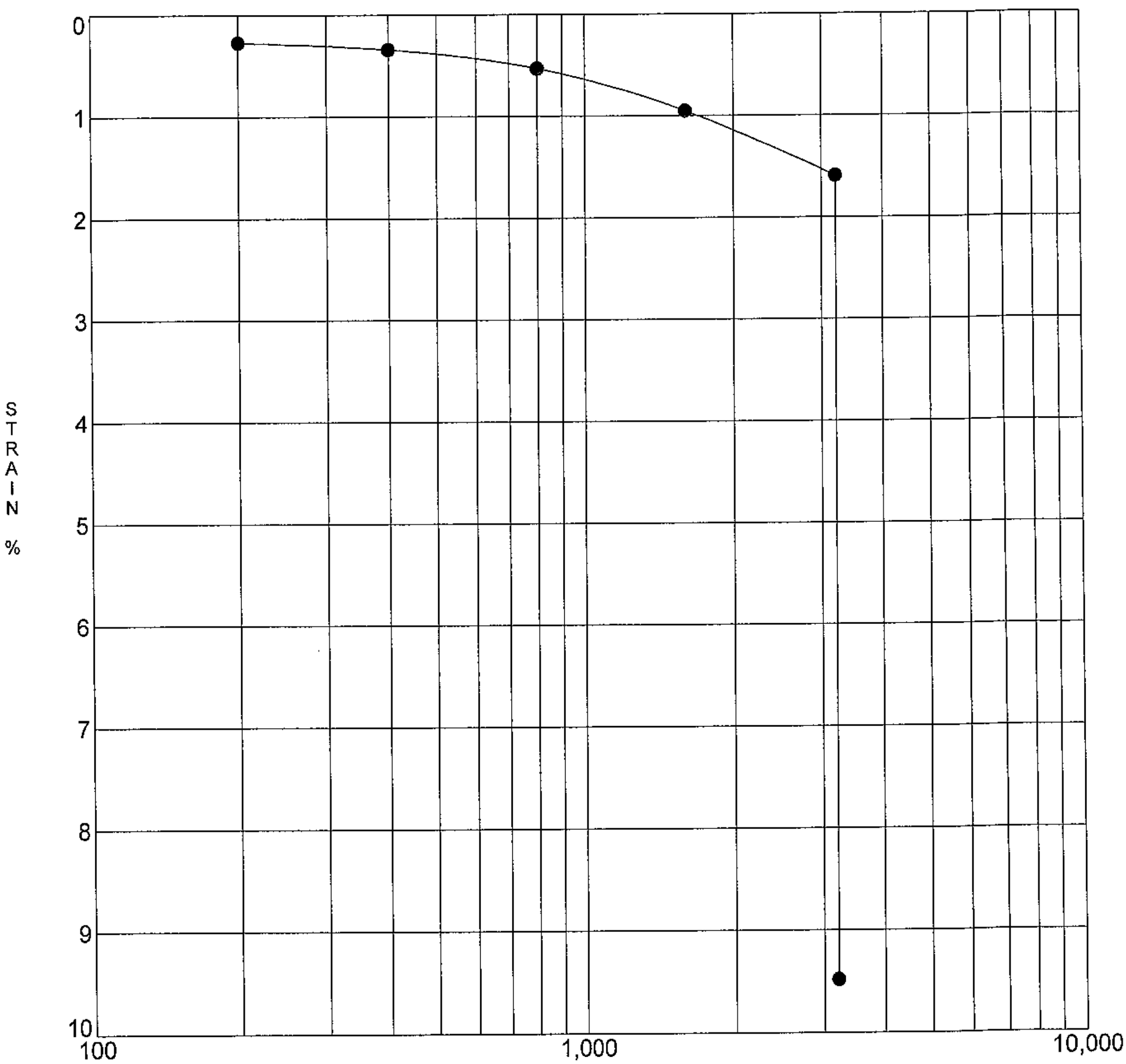
Sample inundated at end of test at 3200 psf

SPEEDIE AND ASSOCIATES

GEOTECH CONSOLIDATION 151276SA.GPJ GENGEO.GDT 9/8/15

CONSOLIDATION TEST

PROJECT: Trailside Hotel
PROJECT NO.: 151276SA
LOCATION: Trailside View east of Pima Road
DATE: 8/19/15
BORING NO.: B- 6 SAMPLE NO.: RS-1 SAMPLE DEPTH: 1 to 2
LABORATORY NO.:
LIQUID LIMIT: 30 PLASTIC LIMIT: 17 PLASTICITY INDEX: 13
CLASSIFICATION: SC ASTM SOIL DESCRIPTION: CLAYEY SAND



Sample inundated at end of test at 3200 psf

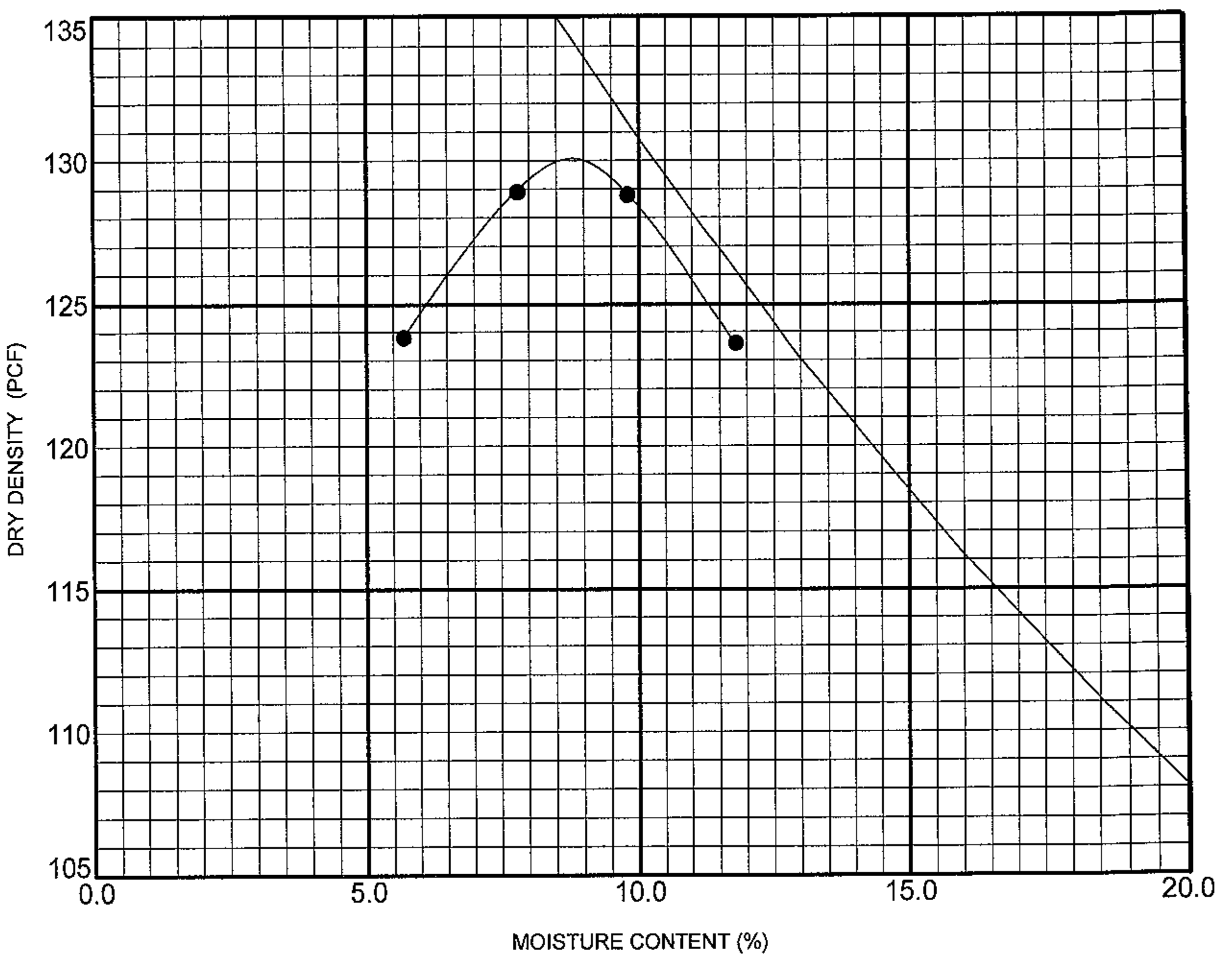
SPEEDIE AND ASSOCIATES

GEOTECH CONSOLIDATION 151276SA.GPJ GENGEO.GDT 9/8/15

MOISTURE-DENSITY RELATIONS

PROJECT: Trailside Hotel
PROJECT NO.: 151276SA
LOCATION: Trailside View east of Pima Road
DATE: 8/19/15
BORING NO.: B- 2 SAMPLE NO.: BS-2 SAMPLE DEPTH: 0 to 5
LABORATORY NO.:
METHOD OF COMPACTION: D698A
LIQUID LIMIT: 28 PLASTIC LIMIT: 15 PLASTICITY INDEX: 12
CLASSIFICATION: SC ASTM SOIL DESCRIPTION: CLAYEY SAND with GRAVEL

MAXIMUM DRY DENSITY: **130.0 PCF**
OPTIMUM MOISTURE CONTENT: **8.8%**



GEOTECH PROCTOR 151276SA.GPJ 9/8/15

SPEEDIE AND ASSOCIATES

SWELL TEST DATA

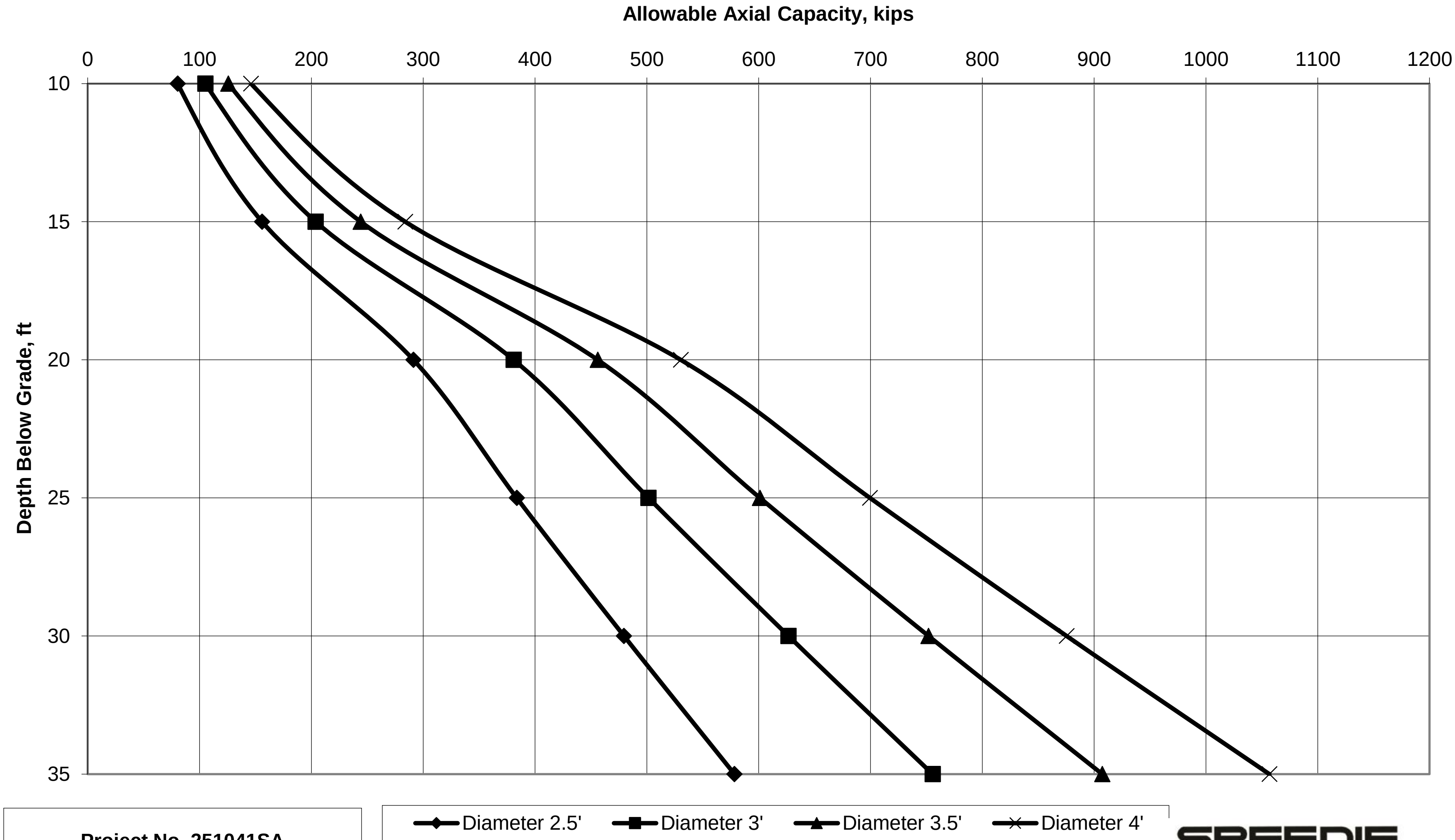
BORING or TEST PIT No.	SAMPLE DEPTH, ft	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	REMOLDED DRY DENSITY (pcf)	INITIAL MOISTURE CONTENT (%)	PERCENT COMPACTION	FINAL MOISTURE CONTENT (%)	CONFINING LOAD (psf)	TOTAL SWELL (%)
B- 2, BS-2	5.0	130.0	8.8	123.6	6.9	95.1	10.7	100	0.0

Trailside Hotel
Trailside View east of Pima Road
Scottsdale, Arizona
Project No. 151276SA

SPEEDIE AND ASSOCIATES

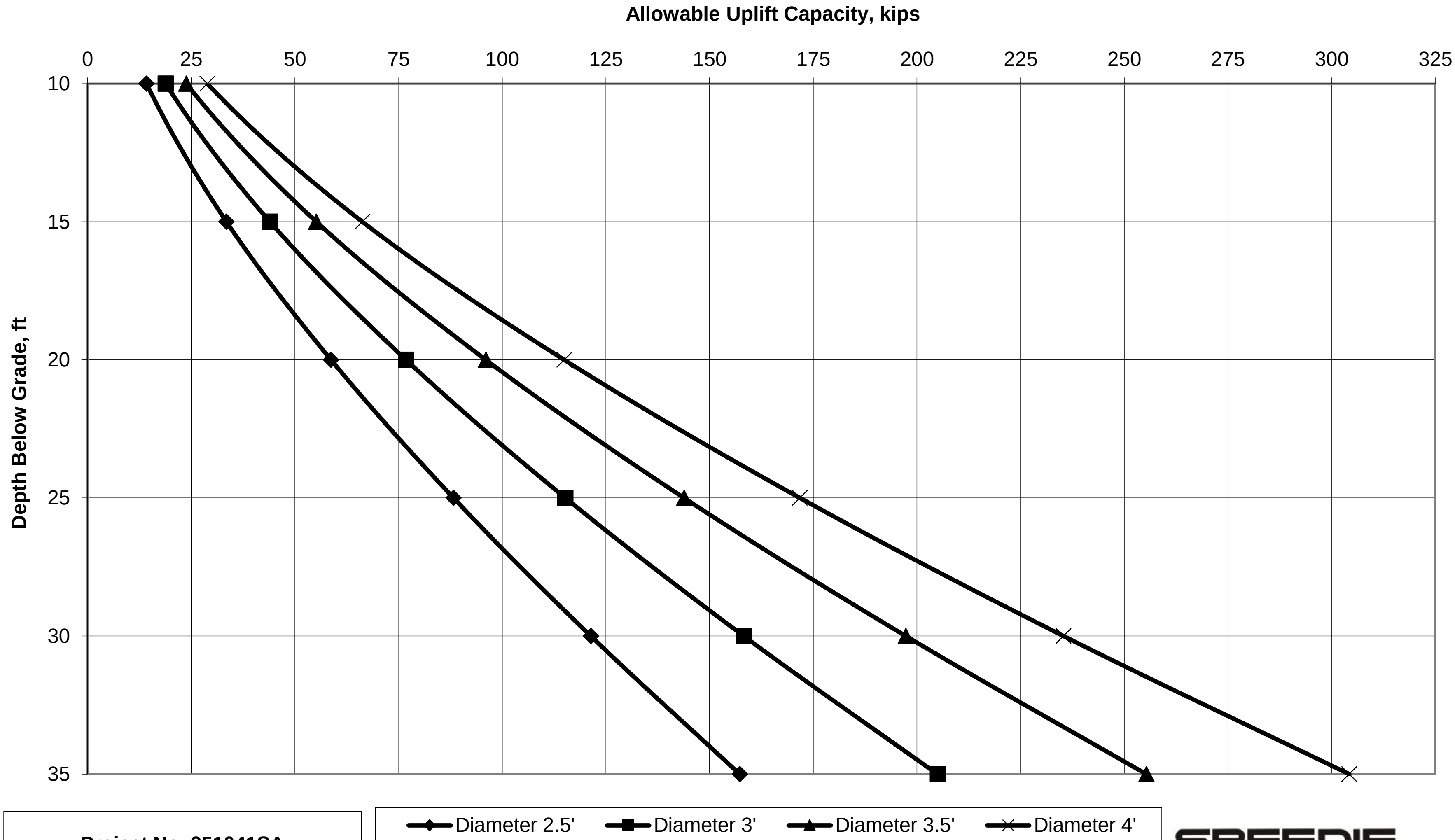
SWELL TEST 151276SA.GPJ GENGEO.GDT 9/8/15

Drilled Shaft Axial Capacity
At Grade



Project No. 251041SA
FIRA Luxury Boutique Hotel

Drilled Shaft Uplift Capacity
At Grade



Project No. 251041SA
FIRA Luxury Boutique Hotel

SPEEDIE AND ASSOCIATES