REPORT ON PRELIMINARY GEOTECHNICAL INVESTIGATION

DESIGNATION: Tempe Parking Structure

LOCATION: NWC University Drive & G Street
           Tempe, Arizona

CLIENT: ASU - CPMG

PROJECT NO: 180516SA

DRAFT DATE: May 2, 2018
# TABLE OF CONTENTS

1.0 **INTRODUCTION** ........................................................................................................................................... 1

2.0 **GENERAL SITE AND SOIL CONDITIONS** ........................................................................................................... 1

   2.1 Site Conditions .................................................................................................................................................... 1
   2.2 Geological Conditions ........................................................................................................................................... 2
   2.3 Seismic Design Parameters .................................................................................................................................. 3
   2.4 General Subsurface Conditions ............................................................................................................................ 3

3.0 **ANALYSIS AND RECOMMENDATIONS** ............................................................................................................ 4

   3.1 Analysis ............................................................................................................................................................... 4
   3.2 Site Preparation .................................................................................................................................................... 7
   3.3 Excavation and Temporary Cut Slopes .................................................................................................................. 8
   3.4 Shoring ................................................................................................................................................................. 9
   3.5 Foundation Design ................................................................................................................................................. 10
   3.6 Lateral Pressures .................................................................................................................................................. 13
   3.7 Fill and Backfill ................................................................................................................................................... 13
   3.8 Utilities Installation .............................................................................................................................................. 15
   3.9 Slabs-on-Grade ................................................................................................................................................... 15
   3.10 Asphalt & Concrete Pavement ............................................................................................................................. 15

4.0 **GENERAL** ........................................................................................................................................................... 17

**APPENDIX** – Current Field and Laboratory Data
1.0 INTRODUCTION

This report presents the results of a preliminary subsoil investigation carried out at the site of the proposed Tempe Parking Structure to be constructed at the northwest corner of University Drive and G Street in Tempe, Arizona.

We understand that future design and construction will consist of a parking structure. The structure will be about 6 stories above grade with one level below grade. The first phase, of three, will be for approximately 940± spaces. This report is intended to provide preliminary data for all three phases. Maximum column structural loads will be moderate to heavy and no special considerations regarding settlement tolerances known at this time. Adjacent areas will be landscaped or paved to support moderate volumes of auto traffic and service truck traffic. The need for and methods used to collect and dispose of storm water are not known at this time. If underground tanks are used, it may have an impact on the foundation design.

This project will be issued as a Design-Build project. This report is intended to be preliminary for inclusion in the bridging documents for the potential final design-build team. That team should determine the extent of any future final investigation needed for the selected design concept.

2.0 GENERAL SITE AND SOIL CONDITIONS

2.1 Site Conditions

The site is currently bounded on the north and east by Alpha Drive, on the south by University Drive, on the southwest by light rail tracks followed by Veterans Way and on the northwest by an adjacent vacant lot followed by 6th Street. The majority of the site is currently occupied with undocumented granular fill and partially occupied with a construction yard on the east side. The site was originally partially agricultural land with small structures until the early 1960’s when residential type buildings with pools were constructed as part of ASU student housing, aka “fraternity row”. The site remained residential until around the year 2010 when the majority of the buildings started being demolished and cleared. In 2016 there appears to be large stockpiles of material on site. Please refer to the following historical aerial photos:
2.2 Geological 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. The closest known earth fissures are located in the Chandler Heights area and in East Mesa, many miles from this site. Based on local experience, subsidence and earth fissures historically have not been a problem in this 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 three magnitude 3.0 events in over 100 years.

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 supported on drilled shafts. In addition, the following seismic parameters may be used for design (based on 2008 USGS maps adopted by 2012/15 IBC):

<table>
<thead>
<tr>
<th>Table 2.3.1 Seismic Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCE\textsuperscript{1} spectral response acceleration for 0.2 second period, ( S_2 ):</td>
</tr>
<tr>
<td>MCE\textsuperscript{1} spectral response acceleration for 1.0 second period, ( S_1 ):</td>
</tr>
<tr>
<td>Site coefficient, ( F_a ):</td>
</tr>
<tr>
<td>Site coefficient, ( F_v ):</td>
</tr>
<tr>
<td>MCE\textsuperscript{1} spectral response acceleration adjusted for site class, ( S_{MS} ):</td>
</tr>
<tr>
<td>MCE\textsuperscript{1} spectral response acceleration adjusted for site class, ( S_{M1} ):</td>
</tr>
<tr>
<td>5% Damped spectral response acceleration, ( S_{DS} ):</td>
</tr>
<tr>
<td>5% Damped spectral response acceleration, ( S_{D1} ):</td>
</tr>
</tbody>
</table>

\textsuperscript{1}MCE = maximum considered earthquake

2.4 General Subsurface Conditions

The subsoil conditions at the site consist primarily of silty gravel, clayey gravel, clayey sand and sandy silty clay to a depth of 5 to 9 feet underlain by silty sand, well graded sand, sandy silty clay, poorly graded gravel and silty gravel to the termination depths of the borings at 8.0 to 50.1 feet below existing grade. It should be noted that fill material was encountered in each of the borings ranging from 0.5 to 9 feet in depth. Auger refusal on cobbles was encountered in borings B-1 to B-3. Boring B-4 was advanced to 50 feet using a Tubex downhole hammer system and encountered weathered bedrock at a depth of 48 feet below existing grade. Standard Penetration Resistance Test (SPT) values range from 5 to 50+ blows per foot. Loose/soft soils were encountered in borings B-3 to a depth of 5 feet and B-4 to a depth of 10 feet below existing grades. It is likely that loose soils also exists to similar depths below boring B-1 & B-2, however to the auger refusal on the cobble laden fill this was not confirmed. Based on visual and tactile observation, the soils were in a ‘dry to moist’ state at the time of investigation.
Laboratory testing indicates in-situ dry densities of the soils ranged from 88 to 93 pcf and water contents from 8.3 to 10.0 percent at the time of investigation. Liquid limits range from non-plastic to 28 percent with plasticity indices at non-plastic to 5 percent. The upper 5 feet of clayey soils exhibit volume increase due to wetting of 2.1 percent when compacted to moisture and density levels normally expected during construction. ‘Undisturbed’ samples in the upper soils displayed significant (6.5%) compression under incremental loading and minor (2.5%) additional compression due to inundation under a maximum confining load of 6,400 psf. ‘Undisturbed’ samples in the soils at a depth of 10 feet displayed significant (8.0%) compression under incremental loading and minor (2.5%) additional compression due to inundation under a maximum confining load of 12,800 psf. Direct shear tests on undisturbed samples of the lower soils at a depth of 15 feet indicated cohesion of 706 psf and an angle of friction of 37 degrees.

3.0 ANALYSIS AND RECOMMENDATIONS

3.1 Analysis

It is assumed that the main parking structure with one level below grade would have a finished floor elevation of approximately 10 feet below existing grade. Based on the soils borings completed to date, this will leave approximately 8 feet of medium dense, compressible material which is not suitable for bearing due to the expected moderate to heavy concentrated loads. Therefore it is primarily recommended to support the proposed structure on drilled shafts (caissons). In lieu of using drilled shafts, consideration may be given deepening the lower level to 1½ - 2 levels below grade in order to support the structure on shallow spread footing or mat foundation bearing on the dense sand gravel cobble (SGC) layer of soils at approximately 20 feet below existing grade. Shallow spread footing can also be over-excavated the planned footing width to contact the SGC layer and backfilled with slurry. Another option is to use spread footings bearing on “stone columns” such as Rammed Aggregate Piers®/ GeoPiers™/Vibro Piers to transfer loads through the loose soil layer.

For any portion of the structure that will be partially supported beyond the perimeter of the basement walls, drilled shafts or other deep ground improvement options combined with structural slabs spanning the backfill are recommended to transfer loads to the same bearing media.

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 for any surface structures should be situated such that they are not located within any backfill zone and that a 45-degree plane below an upper foundation does not intersect the walls of an adjacent structure. This will prevent the imposition of foundation surcharge loads on the walls. If this is not practical, the design of the wall should consider the additional lateral surcharge and transfer of load...
over the wall backfill zone. In addition, only lightly loaded foundations (single story structures) should be considered for this (not moderate to heavily loaded foundations).

As indicated, the upper soils contain fill material to a depths as great as 9 feet underlain by the relatively inconsistent, soft/loose, fine grained soils to depths of 18 to 20 feet. In addition laboratory testing indicates the soils are susceptible to moisture induced collapse. This could result in excessive differential settlement resulting in cracking problems. Accordingly, for shallow spread footing options for any at grade lighter structures, recommendations are made to over-excavate and re-compact the bearing soils to increase density and reduce the potential for collapse. Attention must be paid to provide proper drainage to limit the potential for water infiltration of deeper soils.

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 intermittently. Unpaved planter areas should be sloped at least 5 percent for a distance of at least 10 feet away from the building. While this is the ideal condition, we recognize that this is not always possible in order to meet ADA slope requirements for the adjacent sidewalks. The slope may be reduced to 2 percent provide extra care is taken to ensure sidewalks and other hardscape features do not create a “dam” that prevents positive drainage away from the buildings that creates a "pond" adjacent to the building. Sidewalks should not be placed (or planters graded) that could create 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. 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 (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 light ‘at-grade’ structures, if any are planned, and outside the basement level zone of influence, 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 to increase density of the near surface soils that are more likely to encounter seasonal moisture changes. 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 the basin maximum fill level.

4. Create and maintain positive drainage away from the exterior wall for a minimum of 10 feet.

5. Avoid sidewalks, curbs or other elements that create a dam that could cause water to pond within 5 feet of the perimeter wall.

6. Include no irrigated landscape materials in the first 3 feet next to the building.

7. 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.

8. 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.

9. All irrigation feeder lines, other than those that supply individual emitters, shall not be placed closer than 5 feet to the building.

Excavation operations should be relatively straightforward in the upper soils using conventional equipment although the presence of cobbles at deeper depths may require the use of heavier equipment. 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. Very dense SGC may require more aggressive removal techniques. Secondly, there may be some stability issues with attempting to advance excavations into the SGC zone and with “Running Sand”. Likewise for the caisson construction will be difficult in the cobble laden soils and “Running Sand” and will likely require casing and/or slurry to maintain open shafts or expect large over-runs on concrete volumes.

Groundwater is not expected to be a factor in the design or construction of foundations and underground utilities. The following recommendations regarding below-grade, basement wall water-proofing and drainage are based on the assumption 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 wall water-proofing system. The lower level foundations will bear on the dense granular native soil. 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. While it is expected that the soils at foundation elevation to be relatively 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. This will reduce the potential for wall drainage to wet the bearing soils causing a loss of support and differential settlement.

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 and blocking the operation of out-swinging doors. Pinning typically consists of 24 inch long No. 4 reinforcing steel dowels placed at 12-inch centers.

3.2 Site Preparation

The site should be stripped of all vegetation, debris, rubble undocumented fills and obviously loose surface soils. Excavation for the lower level should remove all of the undocumented fills. Outside of the basement areas, it is recommended to over-excavate the entire site at least 18 inches below existing grade to aid in locating shallow buried hazards. Old foundation elements (if any) should be removed in their entirety along with soil disturbed by this activity. All resulting excavations should be widened as necessary to allow access for compaction equipment.

For the at grade shallow footing option (if any), subsoils should be further over-excavated at least 2 feet below proposed footing bottom elevation, or existing grade, whichever is deeper, extending at least 5 feet beyond the footing edges within all footing areas. It may be more feasible to just over-excavate the entire building pad if the building footprint is relatively small.

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 proper bearing media. Fill placement and quality should be as defined in the "Fill and Backfill" section of this report.

If any utility is located within 10 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. Sub-excavation of foundations, curb-and-gutter and any underground utilities should be provided to ensure complete removal of all structures, deleterious materials and disturbed soils.
Prior to placing structural fill below footing bottom elevation (at grade structures), 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 that are to receive only slab-on-grade (or sidewalk) fill should be scarified 8 inches, moisture-conditioned to at least optimum to 3 percent above optimum and lightly but uniformly compacted to at least 90 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, 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 stiff partially cemented upper fine-grained soils would be classified as Type C. Side slopes for open-cut excavation should be cut back at 1½:1 (horizontal to vertical). The 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 10 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 or stabilize loose layers. 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 will likely encroach on adjacent roadways, adjacent property, and/or buildings. In areas where open-cut excavation is not feasible, consideration must be given to a shoring system. It is suggested to consider using a soldier pile and lagging shoring system for basement walls as this will eliminate a lot of the wall backfill issues discussed below. 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.

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.

Locally, excavations have been braced using the Soil Nail technology. Several firms have experience in the immediate area. This system generally consists of excavating the cut face in increments on the order of 5 feet, installing passive tie back soil nails (anchors) and constructing a reinforced concrete (Shotcrete) face. Consideration may be given to using this system due to the local success, speed of installation and apparent economical cost. Due to the adjacent properties, soil nails may not be acceptable as the soil nails...
would need to be drilled into the adjacent property. Specialized contractors should make their own evaluations. Tiebacks installations are expected to encroach on other private/public property. The owner and/or contractor will have to obtain permission as required prior to tieback installation.

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. Mixing foundation elements within a single structure (footings and drilled shafts for example) should be avoided where possible.

#### Table 3.5.1 Foundation Design

<table>
<thead>
<tr>
<th>Structure</th>
<th>Foundation Type</th>
<th>Bearing Medium</th>
<th>Bearing Depth</th>
<th>Allowable Bearing Capacity</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor At-Grade Structures</td>
<td>Spread</td>
<td>2 ft. Engineered Fill</td>
<td>1.5 ft.</td>
<td>2,000 psf</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Drilled Shafts</td>
<td>Native SGC Soils</td>
<td>20 ft.</td>
<td>See Design Curves</td>
<td>2</td>
</tr>
<tr>
<td>Main Structure with 1 level Below Grade</td>
<td>Spread</td>
<td>500 psi slurry</td>
<td>3.0 ft.</td>
<td>10,000 psf</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Aggregate/Stone Piers or GeoPiers™</td>
<td>Native SGC Soils</td>
<td>TBD (~12 ft)</td>
<td>~6,000 psf</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Drilled Shafts</td>
<td>Native SGC Soils</td>
<td>10 ft.</td>
<td>See Attached Design Curves</td>
<td>5</td>
</tr>
<tr>
<td>Main Structure with 2 levels Below Grade</td>
<td>Mat</td>
<td>Native SGC Soils</td>
<td>4 ft.</td>
<td>k = 250 pci</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Spread</td>
<td>Native SGC Soils</td>
<td>4 ft.</td>
<td>10,000 psf</td>
<td>7</td>
</tr>
</tbody>
</table>
Notes:
1. Spread footings bearing 2 feet below finished grade on at least 2 feet of engineered fill. Assumed the structure is not located within the Main Structure basement wall backfill zone.

2. Drilled shaft capacity charts are found in the Appendix. The tip elevation must terminate in very dense SGC soils at a minimum depth of 20 feet below existing grade. If loose soils are encountered, then shaft depth must be increased to dense bearing soils. Shaft capacity includes skin friction and end bearing.

3. Depth refers to bottom of spread footing below lower level floor elevation. Footings should be over-excavated to remove unsuitable soils and backfilled back to bottom of footing with cementitious slurry with 500 psi compressive strength CLSM per MAG Standard Specification Section 721 or structural concrete.

4. Spread footings supported on highly compacted “stone columns” extended down to the very dense native soils. These proprietary foundation systems are typically designed by the specialty contractor. Contact Hayward Baker or Western Ground Improvement for details and performance criteria.

5. Drilled shaft capacity charts are found in the Appendix. The tip elevation must terminate in very dense SGC soils at a minimum depth of 10 feet below lower level finished grade. If loose soils are encountered, then shaft depth must be increased to dense bearing soils. Shaft capacity includes skin friction and end bearing.

6. Depth refers to bottom of concrete mat foundation below lower level floor elevation bearing on very dense native SGC soils. A modulus of subgrade reaction k of 250 psi may be used for design. A maximum contact pressure of 10 ksf is assumed at the bottom of the mat.

7. Depth refers to the bottom of spread footings below lowest level floor elevation bearing on very dense native SGC. In 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 500 psi compressive strength CLSM per MAG Standard Specification Section 721 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 for wind, seismic, or other loads of short duration.

Continuous masonry wall footings and isolated rectangular footings should be designed with minimum widths of 16 and 24 inches respectively, regardless of the resultant bearing pressure.

Estimated settlements of basement level spread footings under maximum design loads are on the order of ¾ to 1-inch, virtually all of which will occur during construction. Rammed aggregate or vibro-replacement stone improved soils settlement is expected to be similar depending on load, diameter and length (typically >30% reduction in comparison to untreated soils). Additional localized settlements 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 to prevent ponding adjacent to perimeter walls. Planters requiring heavy watering should be considered with caution. Care should be taken
in design and construction to insure that domestic and interior storm drain water is contained to prevent seepage.

The rammed aggregate (stone) or vibro-replacement stone column soil improvement techniques to reduce the amount of settlements consists of a properly designed short aggregate pier system, such as the Vibro-piers or Geopier\textsuperscript{TM} foundation system, that allows for founding the proposed structure on a shallow spread foundation. The system typically has a depth-to-diameter ratio of 2 to 2.5 and is made up of very stiff, densely compacted aggregate piers. The piers are constructed by forming a cavity in the soil matrix by drilling or similar excavation methods. The soil at the bottom of the cavity is pre-stressed and densified using a large tamper. Once the soil at the bottom of the cavity is pre-stressed, well-graded aggregate base stone is placed in the hole in 18-inch lifts and compacted, using a high energy tamping system until the hole is filled. The building’s slab and foundations then rest on the pier system. The short aggregate piers can typically be installed to depths in the range of 10 to 15 feet. A proposed depth of 12 feet should be used to ensure the aggregate piers are bearing in the dense native SGC soils.

Caissons should consist of drilled shaft foundations bearing in the dense to very-dense clayey sand zone. A minimum caisson tip depth of 20 feet (at grade structures) and 10 feet (1 Level below grade) below the finished floor elevations are recommended. Actual shaft lengths may be reduced to accommodate pier caps and/or grade beams. Design and construction should assume straight shaft caissons. Sloughing could occur in the sand layer resulting in concrete quantities higher than neat dimension calculations. A minimum shaft diameter of 30 inches is recommended to allow 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 of the diameters should be used for determining spacing. A separate set of reduction factors apply to lateral group action. These can be provided on request if needed. In addition, lateral load analysis using L-Pile can be provided on request at additional cost. All caissons should be examined by a representative of the Geotechnical Engineer to verify cleaning, depth, dimensions and proper bearing strata.
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 control joints are suggested to allow slight movement and prevent minor floor slab cracking.

3.6 Lateral Pressures

The following equivalent fluid lateral pressure values may be utilized for the proposed construction.

Active Pressures
- Unrestrained Walls: 35 pcf
- At Rest
  - Restrained Walls: 60 pcf

Passive Pressures
- Continuous Footings: 300 pcf
- Spread Footings or Drilled Piers: 350 pcf
- 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 clayey soils are considered suitable for use in general grading fills but should not be used in the top 12-inches of pad fill or as retaining wall backfill. The top 12-inches of pad fill should be completed with an approved low or non-expansive soil, either approved imported common borrow or select granular soil. The native soils below 8 feet are approved as low or non-expansive soil. If select granular soils are used, the 4 inches of under-slab aggregate base may be included as part of the top 18-inches. Otherwise, 18-inches of approved common borrow should be used in addition to the normal 4 inches of aggregate base.

It is preferred to use well graded granular soil for wall backfill. Successful backfill of basement level walls can be difficult to achieve given generally tight access. Placement and compaction must be carefully controlled in order to minimize the potential for post construction settlement should the backfill zone be subjected to water infiltration. Even the most well controlled fine grained fills such as the native soils could experience additional settlement on the order of one to two percent of the wall height, or more, if subjected to significant moisture increases. Using well-graded granular fill will reduce that settlement potential to ½
Accordingly, it is recommended to design and construct a structural slab to span over the backfill zone in the most critical areas 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/storm drain lines, should be avoided in this backfill zone except for perpendicular building service connections. Where critical piping sensitive to settlement is required, grade beams to transfer across the fill zone should be considered. **Tree wells are not recommended in basement wall backfill.**

**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 60 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:

<table>
<thead>
<tr>
<th>A. Building Areas</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Below footing level</td>
<td>95 or 500 psi slurry</td>
</tr>
<tr>
<td>2. Below slabs-on-grade (non-expansive soils)</td>
<td>95</td>
</tr>
<tr>
<td>3. Below slabs-on-grade (expansive soils)</td>
<td>90-95 (max)</td>
</tr>
<tr>
<td>(Not recommended for the top 12-inches of at-grade pads)</td>
<td></td>
</tr>
<tr>
<td>B. Pavement Subgrade or Fill</td>
<td>95</td>
</tr>
<tr>
<td>C. Utility Trench Backfill</td>
<td>95</td>
</tr>
<tr>
<td>D. Aggregate Base Course</td>
<td></td>
</tr>
<tr>
<td>1. Below floor slabs</td>
<td>95</td>
</tr>
<tr>
<td>2. Below asphalt paving</td>
<td>100</td>
</tr>
<tr>
<td>E. Landscape Areas</td>
<td>90</td>
</tr>
</tbody>
</table>
3.8 Utilities Installation

Trench excavations for shallow utilities can be accomplished by conventional trenching equipment, although cobble laden soils may impede progress and require the use of heavier equipment. Trench walls may not stand near-vertical for the periods of time required to install utilities. Trenches penetrating looser sandy deposits may experience sloughing of side walls and necessitating cutting back of side slopes and/or shoring. Adequate precautions must be taken to protect workmen in accordance with all current governmental regulations.

Backfill of trenches above bedding zones may be carried out with native excavated material provided over-sized material (>3 inches) is first removed. 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.

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 Course (A.B.C.) 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 A.B.C. if allowed to dry out, especially if native soils are used in the top 18 inches of the pad.

The native upper soils are capable of storing 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, 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. Vapor barriers are not required below the lower basement level slabs.

3.10 Asphalt & Concrete Pavement

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

<table>
<thead>
<tr>
<th>Area of Placement</th>
<th>Flexible (AC Pavement)</th>
<th>Rigid (PCC Pavement)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thickness</td>
<td>Daily 18-kip ESALs</td>
</tr>
<tr>
<td></td>
<td>AC (0.39)</td>
<td>ABC (0.12)</td>
</tr>
<tr>
<td>Auto Parking</td>
<td>2.0&quot;</td>
<td>4.0&quot;</td>
</tr>
<tr>
<td>Truck Parking, Main Drives, &amp; Fire Lanes</td>
<td>3.0&quot;</td>
<td>4.0&quot;</td>
</tr>
<tr>
<td></td>
<td>3.0&quot;</td>
<td>8.0&quot;</td>
</tr>
</tbody>
</table>

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 330R-87.
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: 62%
- Plasticity Index: 4%
- k: 150 pci (assumed)
- R value: 36 (per ADOT tables)
- Mₐ: 21,900 (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 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 reinforcing 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 are based on the assumption 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, INC.

Ray C. Markley, E.I.T.

Keith R. Gravel, P.E.

Gregg A. Creaser, P.E.
APPENDIX

FIELD AND LABORATORY INVESTIGATION

SOIL BORING LOCATION PLAN

SOIL LEGEND

LOG OF TEST BORINGS

TABULATION OF TEST DATA

CONSOLIDATION TEST RESULT

MOISTURE DENSITY RELATION

SWELL TEST DATA

DIRECT SHEAR TEST DATA

DRILLED SHAFT CAPACITY CHARTS
FIELD AND LABORATORY INVESTIGATION

On March 20, 2018 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 staff engineer, who recorded subsurface conditions and obtained samples for laboratory testing. The soil auger borings were advanced with a truck-mounted CME-75 drill rig utilizing 7-inch diameter hollow stem flight augers. The hammer borings were advanced with a truck-mounted CME-75 utilizing the Tubex hammer drilling equipment. Detailed information regarding the borings and samples obtained can be found on an individual Log of Test Boring prepared 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 in order to estimate settlements and determine effects of inundation. All field and laboratory data is presented in this appendix.
### SOIL LEGEND

#### Sample Designation

<table>
<thead>
<tr>
<th>DESIGNATION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS</td>
<td>Auger Sample</td>
</tr>
<tr>
<td>BS</td>
<td>Large Bulk Sample</td>
</tr>
<tr>
<td>S</td>
<td>Spoon Sample</td>
</tr>
<tr>
<td>RS</td>
<td>Ring Sample</td>
</tr>
<tr>
<td>LS</td>
<td>Liner Sample</td>
</tr>
<tr>
<td>ST</td>
<td>Shelby Tube</td>
</tr>
</tbody>
</table>

#### Continuous Penetration Resistance

Driving a 2.0-inch outside diameter "Bullnose Penetrometer" continuously into undisturbed soil by a rapid motion, without impact or twisting (ASTM D-1587).

#### Consistency

<table>
<thead>
<tr>
<th>Clays &amp; Silts</th>
<th>Blows/Foot</th>
<th>Strength (tons/sq ft)</th>
<th>Sands &amp; Gravels</th>
<th>Blows/Foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>0 - 2</td>
<td>0 - 0.25</td>
<td>Very Loose</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>0.25 - 0.5</td>
<td>Loose</td>
<td>5 - 10</td>
</tr>
<tr>
<td>Firm</td>
<td>5 - 8</td>
<td>0.5 - 1.0</td>
<td>Medium Dense</td>
<td>11 - 30</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
<td>1 - 2</td>
<td>Dense</td>
<td>31 - 50</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
<td>2 - 4</td>
<td>Very Dense</td>
<td>&gt; 50</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Relatable Density

#### Major Divisions

- **Coarse Grained Soils**
  - More than 50% of material 3/8" #4 and sievesize
  - Clean Gravels (little or no fines)
  - Gravels with Finest (appreciable amount of fines)
  - Gravelly Sands (little or no fines)
  - Clean Sands (little or no fines)
  - Sands with Finest (appreciable amount of fines)

- **Fine Grained Soils**
  - More than 50% of material is smaller than No. 40 #4 sieve
  - Silts and Clays (less than 50)
  - Liquid Limit Greater than 50
  - Liquid Limit Less Than 50

- **Highly Organic Soils**
  - Liquid Limit Greater than 50
  - Liquid Limit Less Than 50

#### Symbol and Letter Graphs

#### Typical Descriptions

- **GW** - Well-Graded Gravels
- **GP** - Poorly-Graded Gravels
- **GM** - Silty Gravels
- **GC** - Clayey Gravels
- **SW** - Well-Graded Sands
- **SP** - Poorly-Graded Sands
- **SM** - Silty Sands
- **SC** - Clayey Sands
- **ML** - Organic Silts and Very Fine Sands
- **CL** - Organic Silt and Fine Sand
- **OL** - Organic Silt
- **MH** - Organic Silts and Organic Silts
- **CH** - Organic Clays
- **OH** - Organic Silts

#### Material Size

<table>
<thead>
<tr>
<th>MATERIAL SIZE</th>
<th>PARTICLE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lower Limit</td>
</tr>
<tr>
<td></td>
<td>mm Sieve Size</td>
</tr>
<tr>
<td>Sands</td>
<td></td>
</tr>
<tr>
<td>Fine</td>
<td>0.075</td>
</tr>
<tr>
<td>Medium</td>
<td>0.420</td>
</tr>
<tr>
<td>Coarse</td>
<td>2.000</td>
</tr>
<tr>
<td>Gravels</td>
<td></td>
</tr>
<tr>
<td>Fine</td>
<td>4.75</td>
</tr>
<tr>
<td>Coarse</td>
<td>19</td>
</tr>
<tr>
<td>Cobbles</td>
<td>75</td>
</tr>
<tr>
<td>Boulders</td>
<td>300</td>
</tr>
</tbody>
</table>

**U.S. Standard**

**Clear Square Openings**

---

**NOTE:** Dual or Modified Symbols May Be Used to Indicate Borderline Soil Classifications or To Provide A Better Graphical Presentation of The Soil.
Auger Refusal on Cobbles

FILL: Medium Dense Brown SILTY GRAVEL with SAND (GM-Dry to Moist) with 1-10% Cobble

Very Dense, 1 to 5% Cobble at 5'

FILL: Medium Dense Brown SILTY GRAVEL with SAND (GM-Dry to Moist) with 1-10% Cobble

Auger Refusal on Cobbles

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

Visual Classification

FILL: Medium Dense Brown SILTY GRAVEL with SAND (GM-Dry to Moist) with 1-10% Cobble

Very Dense, 1 to 5% Cobble at 5'

Water Level

Depth | Hour | Date
--- | --- | ---
Free Water was Not Encountered

NT = Not Tested

Log of Test Boring Number: B-1

ASU Novus Parking Structure
NWC University Drive & G Street
Tempe, Arizona

Project No.: 180516SA

Sample Number | Depth of Sample | Natural Water Content (%) | In-Place Dry Density (P.C.) | Penetration Resistance Blows per Foot
--- | --- | --- | --- | ---
S-1 | 2.5 | NT | NT | 0
S-2 | 6.5 | NT | NT | 61/12"
**Visual Classification**

- **FILL:** Loose Brown CLAYEY SAND (SC-Dry to Moist) with Gravel, 1-10% Cobble
- **FILL:** Dense Brown CLAYEY GRAVEL with SAND (GC-Dry) 1 to 5% Cobble

**Auger Refusal on Cobbles**

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

**Details:**

- **Boring Date:** 3-20-18
- **Driller:** R. Quezada
- **Contractor:** Resilient Drilling
- **Field Engineer/Technician:** J. Miller

**Water Level**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Hour</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<tr>
<td>5</td>
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<tr>
<td>10</td>
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<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Free Water was Not Encountered*
Auger Refusal on Cobbles

Very Dense Brown/Gray SILTY GRAVEL with SAND (GM-Dry) with 5-15% Cobble

Dense Brown WELL GRADED SAND (SW-Dry) with Little Gravel

Stiff Brown SANDY SILTY CLAY (CL/ML-Dry) with Trace Gravel

Very Stiff Brown SANDY SILTY CLAY (CL/ML-Dry)

Loose Brown SILTY SAND (SM-Dry) with Little Gravel

Medium Dense

Free Water was Not Encountered

NT = Not Tested

Rig Type: Hollow Stem Auger

Surface Elevation: N/A

Visual Classification

FILL: Loose Brown POORLY GRADED GRAVEL with SAND (GP-Dry to Moist) with 1-10% Cobbles

Loose Brown SILTY SAND (SM-Dry) with Little Gravel

Sample Number | Depth of Sample | Natural Water Content (%) | In-Place Density (P.C.F.) |
--- | --- | --- | --- |
RS-1 | 2.0 | 10.0 | 93.3 |
BS-2 | 5.0 | NT | NT |
S-3 | 6.5 | NT | NT |
S-4 | 11.5 | NT | NT |
S-5 | 16.5 | NT | NT |
S-6 | 21.5 | NT | NT |

Boring Date: 3-20-18
Field Engineer/Technician: J. Miller
Driller: R. Quezada
Contractor: Resilient Drilling

Log of Test Boring Number: B-3

ASU Novus Parking Structure
NWC University Drive & G Street
Tempe, Arizona
Project No.: 180516SA
Visual Classification

FILL: Medium Dense Brown SILTY GRAVEL with SAND (GM-Dry to Moist) with 1-10% Cobble

Trace Gravel

Very Dense Brown/Gray POORLY GRADED GRAVEL with SAND (GP-Dry to Moist) with 1-10% Cobble

5-15% Cobble at 25'

Very Dense Brown/Gray SILTY GRAVEL with SAND (GM-Dry) with 5-15% Cobble

Very Dense Brown/Gray POORLY GRADED GRAVEL with SAND (GM-Dry) with 5-15% Cobble

Very Dense Gray POORLY GRADED GRAVEL with SAND (GP-Dry) with 5-15% Cobble

Very Dense Gray SILTY GRAVEL (WEATHERED BEDROCK) (GM-Dry)

End of Boring

Boring Date: 3-20-18
Field Engineer/Technician: J. Miller
Driller: R. Quezada
Contractor: Resilient Drilling

<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Hour</th>
<th>Date</th>
<th>Water Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td></td>
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</tr>
<tr>
<td>5</td>
<td>5.0</td>
<td>NT</td>
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<tr>
<td>10</td>
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<td>45</td>
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</tr>
<tr>
<td>50</td>
<td>50.1</td>
<td>NT</td>
<td>NT</td>
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</table>

NT = Not Tested
### TABULATION OF TEST DATA

<table>
<thead>
<tr>
<th>Soil Boring or Test Pit Number</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>Sample Interval (ft)</th>
<th>Natural Water Content (Percent of Dry Weight)</th>
<th>In-Place Dry Density (Pounds Per Cubic Foot)</th>
<th>Particle Size Distribution (Percent Finer)</th>
<th>Atterberg Limits</th>
<th>Plasticity Index</th>
<th>Unified Soil Classification</th>
<th>Specimen Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3</td>
<td>BS-2</td>
<td>BULK</td>
<td>0.5 - 5.0</td>
<td>NT</td>
<td>NT</td>
<td>#200 SIEVE: 62  #40 SIEVE: 99  #10 SIEVE: 99  #4 SIEVE: 100  #2 SIEVE: 100</td>
<td>LIQUID LIMIT: 24  PLASTIC LIMIT: 20  PLASTICITY INDEX: 4</td>
<td>CL-ML</td>
<td>SANDY SILTY CLAY</td>
<td></td>
</tr>
<tr>
<td>B-3</td>
<td>RS-1</td>
<td>RING</td>
<td>1.0 - 2.0</td>
<td>10.0</td>
<td>93.3</td>
<td>NT: NT  #200 SIEVE: 99  #40 SIEVE: 99  #10 SIEVE: 100  #4 SIEVE: 100</td>
<td>NT: NT  NT: NT  NT: NT: NT</td>
<td>NT: NT  NT: NT  NT: NT: NT</td>
<td>ML</td>
<td>SILT with SAND</td>
</tr>
<tr>
<td>B-4</td>
<td>RS-3</td>
<td>RING</td>
<td>10.0 - 11.0</td>
<td>8.3</td>
<td>88.4</td>
<td>NT: NT  #200 SIEVE: 99  #40 SIEVE: 99  #10 SIEVE: 100  #4 SIEVE: 100</td>
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<td>NT: NT  NT: NT  NT: NT: NT</td>
<td>ML</td>
<td>SANDY SILT</td>
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<tr>
<td>B-4</td>
<td>RS-4</td>
<td>RING</td>
<td>15.0 - 16.0</td>
<td>NT</td>
<td>NT</td>
<td>#200 SIEVE: 63  #40 SIEVE: 87  #10 SIEVE: 96  #4 SIEVE: 100  #2 SIEVE: 100</td>
<td>NT: NP  NP: NP  NP: NP: NP</td>
<td>NP: NP  NP: NP  NP: NP: NP</td>
<td>ML</td>
<td>SANDY SILT</td>
</tr>
</tbody>
</table>

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

ASU Novus Parking Structure
NWC University Drive & G Street
Tempe, Arizona
Project No. 180516SA
Sample inundated at end of test at 6400 psf
CONSOLIDATION TEST

PROJECT: ASU Novus Parking Structure
LOCATION: NWC University Drive & G Street
BORING NO.: B-4
SAMPLE NO.: RS-3
SAMPLE DEPTH: 10 to 11
PROJECT NO.: 180516SA
DATE: 3/20/18
LABORATORY NO.: 

LIQUID LIMIT: 28
PLASTIC LIMIT: 23
PLASTICITY INDEX: 4
CLASSIFICATION: ML
ASTM SOIL DESCRIPTION: SILT with SAND

Sample inundated at end of test at 12800 psf
SHEAR TEST DIAGRAM

NORMAL PRESSURE, psf

Shear Strength, psf

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>Cohesion, psf</th>
<th>Friction Angle</th>
<th>DD</th>
<th>MC%</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4</td>
<td>15.0</td>
<td>706.0</td>
<td>37.0</td>
<td>92.5</td>
</tr>
</tbody>
</table>

PROJECT ASU Novus Parking Structure - NWC University
JOB NO. 180516SA
DATE 3/20/18

Drive & G Street
MOISTURE-DENSITY RELATIONS

PROJECT: ASU Novus Parking Structure
LOCATION: NWC University Drive & G Street
BORING NO.: B-3 SAMPLE NO.: BS-2 SAMPLE DEPTH: 0.5 to 5
METHOD OF COMPACTION: D698A
LIQUID LIMIT: 24 PLASTIC LIMIT: 20 PLASTICITY INDEX: 4
CLASSIFICATION: CL-ML ASTM SOIL DESCRIPTION: SANDY SILTY CLAY

MAXIMUM DRY DENSITY: 116.5 PCF
OPTIMUM MOISTURE CONTENT: 11.0%

<table>
<thead>
<tr>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>125</td>
</tr>
<tr>
<td>5.0</td>
<td>120</td>
</tr>
<tr>
<td>10.0</td>
<td>115</td>
</tr>
<tr>
<td>15.0</td>
<td>110</td>
</tr>
<tr>
<td>20.0</td>
<td>105</td>
</tr>
<tr>
<td>25.0</td>
<td>100</td>
</tr>
<tr>
<td>30.0</td>
<td>95</td>
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</tbody>
</table>

DATE: 3/20/18
LABORATORY NO.: 180516SA
PROJECT NO.: 180516SA
## SWELL TEST DATA

<table>
<thead>
<tr>
<th>BORING or TEST PIT No.</th>
<th>SAMPLE DEPTH, ft</th>
<th>MAXIMUM DRY DENSITY (pcf)</th>
<th>OPTIMUM MOISTURE CONTENT (%)</th>
<th>REMOLDED DRY DENSITY (pcf)</th>
<th>INITIAL MOISTURE CONTENT (%)</th>
<th>PERCENT COMPACTION</th>
<th>FINAL MOISTURE CONTENT (%)</th>
<th>CONFINING LOAD (psf)</th>
<th>TOTAL SWELL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3, BS-2</td>
<td>5.0</td>
<td>116.5</td>
<td>11.0</td>
<td>110.4</td>
<td>9.5</td>
<td>94.8</td>
<td>18.1</td>
<td>100</td>
<td>2.1</td>
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</table>

ASU Novus Parking Structure  
NWC University Drive & G Street  
Tempe, Arizona  
Project No. 180516SA
Drilled Shaft Axial Capacity
At-Grade Structures

Allowable Axial Capacity, kips

- Diameter 2'
- Diameter 2.5'
- Diameter 3'
- Diameter 4'

Project No. 180516SA
Tempe Parking Garage
Drilled Shaft Uplift Capacity
At-Grade Structures

Allowable Uplift Capacity, kips

Depth Below Grade, ft

0 25 50 75 100 125 150 175 200 225 250 275 300

0 15 20 22.5 25 30

Diameter 2'
Diameter 2.5'
Diameter 3'
Diameter 4'

Project No. 180516SA
Tempe Parking Garage

DRAFT
Drilled Shaft Uplift Capacity
with 1 Level Below Grade

Depth Below Lower Level Grade, ft

Allowable Uplift Capacity, kips

Diameter 3'
Diameter 4'
Diameter 5'
Diameter 6'

Project No. 180516SA
Tempe Parking Garage

DRAFT