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Report on Geotechnical Investigation

Designation:	**Sedona Parking Structure**
Location:	**430/460 Forest Road** **Sedona, Arizona**
Client:	**Gabor Lorant Architects, Inc.**
Project Number:	**210124SF**
Draft Date:	**March 31, 2021**

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1.0 INTRODUCTION

1.1 PROJECT INFORMATION

This report presents the results of a subsoil investigation carried out at the site of a proposed parking structure, located at 430/460 Forest Road in Sedona, Arizona.

At this time, it is our understanding that construction will consist of a parking structure with one level below grade at the street side and possibly two levels below grade at the north side of the site due to the site slope. The construction is assumed to be cast in place or pre-cast concrete construction. Structural loads are expected to be moderate to heavy, and no special considerations regarding settlement tolerances are known at this time. Preliminary grading concepts indicate that cuts may be 20 feet in depth on the north side of the structure. Adjacent areas will be landscaped or paved to support moderate passenger and light truck traffic. Landscaped areas will be utilized for storm water retention and disposal.

1.2 FIELD AND LABORATORY INVESTIGATION

On February 11, 2021 through February 13, 2021, five structural soil 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 borings were excavated with a CME-75 truck mounted drill rig utilizing 7-inch diameter hollow stem augers. The sandstone can be augered with moderate difficulty and in some locations the borings were terminated as "auger refusal" to avoid excessive wear on equipment. It is probable that the borings could have been advanced, with difficulty, beyond the depth indicated. Three borings were advanced with HQ diamond bit coring equipment to depths of 35 feet below existing grade after encountering sandstone to provide information for foundation design, slope stability analysis and excavation. Detailed information regarding the borings and samples obtained can be found on an individual Log of Test Boring prepared for each location. Photographs of the rock cores are contained in the Appendix of this report.

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 samples in order to estimate settlements and determine effects of inundation. Unconfined compressive strength tests were performed on selected rock cores to provide information regarding excavation, slope stability and foundation parameters. All field and laboratory data are presented in this Appendix.

2.0 SITE CONDITIONS

2.1 PROPERTY DESCRIPTION

The rectangular shaped site is approximately 1.24 acres and is bounded on the north and west by residential property, on the south by Forest Road, and on the east by commercial property with Smith Road beyond. At the time of the investigation the site was covered with a moderate growth of native shrubs, juniper trees and low lying grasses. The site slopes up to the north at approximately 12%. The east side of the site is currently developed with a residential structure. Two dirt driveways exist providing access to the existing residential structure on site and an adjacent residential structure to the west. Site drainage is in the form of sheet flow towards the south.

2.2 LOCAL GEOLOGY

The site is located in the Transition Zone Province near its northern boundary with the Colorado Plateau Province. The geology is dominated by flat lying Paleozoic age sedimentary rocks of the Supai Formation mantled with a thin veneer of residual soil. The Supai Formation is predominantly sandstone with interbedded silty sandstone and shaley siltstone. The Supai Formation tends to be red, reddish brown and reddish orange in color. The bedding is generally thick and massive. The massive units are silica cemented and are typically hard to very hard. Localized shaley interbeds often interrupt the massive units and are generally decomposed to predominantly decomposed and very soft. Structurally the Supai Formation generally dips gently to the northeast and north west at very low angles. Regional tectonic stresses have produced very high angle joints typically between 75° and 90°.

2.3 SUBSURFACE CONDITIONS

2.3.1 Field Results

Subsurface conditions are somewhat consistent. The upper soils consist of thin veneer, between 1 and 2 feet, of silty fine sand and clayey fine sand with subordinate amounts of sandstone gravel. Underlying the upper soil is moderately weathered to fresh sandstone of the Supai Formation. The sandstone is generally medium hard to very hard with thick bedding typically 3 to 5 feet in thickness and interrupted with sequences of one to two feet of thin bedded sandstone and shaley siltstone.

Due to the thin soil veneer overlying the Supai Formation, Standard Penetration Resistance Tests (SPT) were not obtained in the upper soil. SPT values of 50+ bpf were encountered in the strata identified as sandstone. Based on visual and tactile observation, the upper soils were typically in a dry state, below the plastic limit, at the time of investigation. Groundwater was not encountered during this investigation; however, it is not uncommon to have seasonally perched water that may be trapped and encountered at or near the soil/bedrock interface.

2.3.2 Laboratory Results

Laboratory testing of the soils indicates liquid limits in the range of 20 to 21 percent with plasticity indices ranging from non-plastic to 4 percent. In-place densities of the upper soils is in the range of 90 to 108 pcf with moisture contents on the range of 9 to 16 percent. Volume increase due to wetting of the upper soils is generally on the order of 1.4 percent when re-compacted to moistures and densities normally expected during construction and confined to 100 psf. Five rock core samples were selected for compressive strength testing. Unconfined compressive strength of the selected rock cores ranged from 2,550 psi to 10,000 psi with an average compressive strength of 7,275 psi. Unit weight of the rock varies from 158 pcf to 168 pcf.

2.4 SEISMIC PARAMETERS

The project area is located in a seismic zone that is considered to have low to moderate historical seismicity. Sedona, AZ has a moderate earthquake risk, with a total of 8 earthquakes since 1931. The USGS database shows that there is a 23.33% chance of a major earthquake within 50km of Sedona, AZ within the next 50 years. The largest earthquake within 30 miles of Sedona, AZ was a 4.7 Magnitude in 2014. Liquefaction is not a concern with the shallow depth to bedrock.

Although borings were not advanced to 100 feet, based on the nature of the subsoils encountered in the borings and geology in the area, a Site Class Definition, Class B may be used for design of the structures bearing on rock. In addition, the following seismic parameters may be used for design (based on ASCE 7-16 (IBC 2018), utilizing the ATC Hazards by Location Online Tool):

2.4.1 Seismic Parameters

Site Class:	B
MCE[1] spectral response acceleration for 0.2 second period, S_S:	0.294
MCE[1] spectral response acceleration for 1.0 second period, S_1:	0.093
Site coefficient, **Fa:**	0.9
Site coefficient, **Fv:**	0.8
MCE[1] spectral response acceleration adjusted for site class, S_{MS}:	0.265
MCE[1] spectral response acceleration adjusted for site class, S_{M1}:	0.074
5% Damped spectral response acceleration, S_{DS}:	0.177
5% Damped spectral response acceleration, S_{D1}:	0.049

Notes:

1. MCE = maximum considered earthquake

3.0 ANALYSIS

This report herein assumes that the information contained in Section 1.1 Project Information is accurate and that cuts will be 20 feet or less. If cuts greater than 20 feet are required to reach finished grades, this office should be contacted for additional recommendations. Analysis of the field and laboratory data indicates that subsoils at the site are suitable, for support of the proposed structures bearing on sandstone. Foundation options provided include standard shallow foundations bearing on moderately weathered to fresh sandstone.

As indicated, the site is developed on the east side of the site and structures are still present. Given that proposed construction consists of below grade parking levels, the existing foundations and utilities should not be a major concern as they should be removed during either demolition or excavation for the parking structure. The on-site native soils are suitable for reuse as engineered fill.

The upper soils do not present any concerns with respect to expansion potential due to moisture fluctuations. The upper soils are expected to consolidate due to increased moisture under light loads. Proper site preparation for minor structures should reduce the potential for settlement due to moisture infiltration.

Preparation of the site will be challenging as excavation of the bedrock will require heavy duty rock excavation equipment. It is anticipated that Caterpillar D10/D12 track mounted tractors or equivalent with single shank rippers will be moderately effective. It is anticipated that ripping will result in large blocks that will require secondary fragmentation. Excavation with track mounted bucket excavators equipped with hydraulic hammers is expected to result in very slow production. A combination of track mounted tractors and bucket excavators for secondary fragmentation may result in acceptable production rates.

Although blasting may be the most production effective method, given the number of structures between ¼ and ½ miles, the misconception of structural damage perceived by building owners and potential disruption to downtown activities, blasting is not recommended. If blasting is considered and approved by the City of Sedona, an extensive pre-blast survey of all structures within ½ mile should be performed. The purpose of a pre-blast survey is to document the condition of a structure prior to its exposure to blasting vibrations. The pre-blast survey documentation can be used to verify or refute claims of damage as a result of felt vibrations. If blasting is pursued, additional recommendations for project specifications can be provided.

4.0 RECOMMENDATIONS

4.1 EARTHWORK

4.1.1 Site Preparation

The entire area to be occupied by the proposed construction should be stripped of all vegetation, debris, rubble and obviously loose surface soils. As indicated, excavation of the basement level will require rock excavation equipment/techniques. Underground utilities should be relocated or abandoned in place to reduce the potential of the abandoned utility from becoming a conduit for water infiltration.

After removal of the upper soils, prior to placing structural fill, the excavated 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. Scarification of rock is not necessary. Pavement areas should be treated in a similar manner.

4.1.2 Site Drainage

Attention must be paid to provide proper drainage to limit the potential for water infiltration to below grade elements for minor structures such as screen walls using conventional standard spread footings bearing on soil and slabs on grade to perform as expected. It is assumed that the adjacent exterior areas will most likely consist of hardscape or pavement. If installed, the landscape plan should 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 and slabs on grade with irrigation schedules set and maintained to run intermittingly. **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. A “French Drain” or sump-pump system should be installed to remove water from below concrete slabs-on-grade and prevent hydrostatic pressure along below grade walls. Roof drainage should also be directed away from the building in paved scuppers and **NOT** connected to the underdrain system. 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; especially vault/pit wall backfill (if any). 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 conventional shallow spread footings bearing on soil are recommended for minor structures not connected to the main structure since this is the most economical system available and if loading conditions allow. 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, or deeper foundations. Recognizing the need to minimize significant water penetration adjacent to the foundations of minor structures that could detrimentally impact the foundation, the following additional recommendations are made to protect foundations:

1) Take extra precaution to backfill and compact native soil fill to 95 percent.
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 or underground storage tanks (USTs) 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 and 15 feet from any USTs.
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.

4.1.3 Fill and Backfill

Native soils, are suitable for use in grading fills and can be placed directly beneath structures. Excavated rock may be reused provided it is reduced in size and fill contains sufficient fines to avoid nesting of rocks. A minimum of 15% fines passing the #200 sieve is recommended. Oversized material (> 3 inches) should be removed or reduced in size.

4.1.3.1 Fill Specification		
Specification	**Below Concrete Slabs**	**Below Foundations**
Passing 3”/75mm	100%	100%
Passing #200/0.075mm	≤60%	15-60%
Liquid Limit	<30%	<30%
Plasticity Index	<10%	<10%
Swell[1]	<1.5	<2.0

Notes:
1. 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.

Clean Cinders are not acceptable beneath foundations. **For fill placed beneath foundations, it should meet the above specifications in addition to containing at least 15 percent passing the 200 sieve.** Although “clean” cinders often times meet our fill specifications for placement beneath building slabs, they may pose difficulties during construction. Due to their granular nature and lack of sufficient fines, “clean” cinders are a free draining material. As a result, they may be difficult to properly moisture condition and water may infiltrate the cinders and saturate the underlying soils. This could result in an unstable support for foundations and building slabs. Excess water, as a result of moisture conditioning, is often observed at the interface between the fill and underlying less permeable material. This often results in free water accumulating in foundation excavations prior to the placement of concrete. Free water and loose saturated soils would need to be removed prior to placement of concrete. “Clean” cinders also pose difficulties in trenching operations due to the inability to excavate neat trenches. With the lack of fines and cohesive soils, the clean cinders generally slough and vertical walls are hard to maintain. If a cinder based product is used for import fill above foundation bottom elevation, consideration should be given to a “dirty” cinder product that meets the fill criteria for placement beneath foundations.

Imported common fill for use in site grading should be examined by a Soils Engineer to ensure that it is of low swell potential and free of organic or otherwise deleterious material. 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 (optimum to +3 percent for under slab fill). Granular fill (ASTM Classification GW, GP, SW, SP) can be placed on the dry side of optimum at the discretion of the geotechnical engineer on record.

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 as set forth below. Frozen material shall not be placed, nor shall fill be placed upon frozen grade.

4.1.3.2 Compaction Specifications	
Building Areas	
Total Amount of Fill Below Finished Grade > 5’	98%
Total Amount of Fill Below Finished Grade < 5’	95%
Pavement Subgrade/Fill	
Native/Import Fill	95%
Utility Trench Backfill	
> 2.0' Below Finish Subgrade	95%
Within 2.0' of Finish Subgrade (non-granular)	95%
Within 2.0' of Finish Subgrade (granular)	100%
Aggregate Base Course	
Below Concrete Slabs	95%
Below Asphalt Paving	100%
Landscaped Areas	
Miscellaneous fill	90%
Utility Trench - > 1.0’ Below Finished Grade	85%
Utility Trench - < 1.0’ Below Finished Grade	90%

4.2 FOUNDATION DESIGN

The following bearing capacities can be utilized for design.

4.2.1 Foundation Bearing Capacities

Structure	Foundation Type	Foundation Depth (1)	Bearing Medium	Bearing Capacity	Notes
Minor Structures	Spread	2.0 ft	Native Soil/ Engineered Fill	1.5 ksf	2
Major Structures	Spread	2.0 ft	Sandstone	16 ksf	3

Notes:

1. Foundation depth refers to the bottom of footing elevation from the lowest exterior grade within 5.0 feet of the foundation element. Interior footing bottoms may be reduced to 1.5 feet below bottom of slab elevation. All capacities are based on bearing media.
2. For minor structures such as screen walls, planter walls, etc. not connected to any main structure.
3. Bearing capacity is contingent upon bearing medium. Footings shall bear on moderately weathered to fresh sandstone. All foundation excavations in rock should be properly cleaned of all loose material in order to take advantage of the higher coefficient of friction for clean rock provided in Section 4.3 – Lateral Pressures

These bearing capacities refer to the total of all loads, dead and live, and are a net pressure. They may be increased one-third for wind, seismic or other loads of short duration. All footing excavations should be level and cleaned of all loose or disturbed materials. Positive drainage away from the proposed buildings must be maintained at all times.

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).

Estimated settlements under design loads for soil are on the order of ¾ to 1.0 inch, virtually all of which will occur during construction. Post-construction differential settlements will be negligible, under existing and compacted moisture contents. Additional localized settlements of the same magnitude could occur if native supporting soils were to experience a significant increase in moisture content. Estimated settlements under design loads for rock is less than ¼ inch. As indicated in Section 4.1.2 Site Drainage, positive drainage away from structures, and controlled routing of roof runoff should be provided to prevent ponding adjacent to perimeter walls. Caution must be used when considering planters requiring heavy watering. Care should be taken in design and construction to insure that domestic and interior storm drain water is contained to prevent seepage.

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.

4.3 Lateral Pressures

The following lateral pressure values may be utilized for the proposed construction:

4.3.1 Lateral Pressures		
Active Pressure		
	Unrestrained Walls	**35 pcf**
At-Rest Pressure		
	Restrained Walls	**60 pcf**
Passive Pressures		
	Continuous Footings	**300 pcf**
	Spread Footings or Drilled Piers	**350 pcf**
Coefficient of Friction		
	With Passive Pressure	**0.35**
	Without Passive Pressure	**0.45**
	Clean Bedrock	**0.60**

All backfill must be compacted to not less than 95 percent (ASTM D-698) to mobilize these passive values at low strain. Expansive native 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. The exposed rock cut must be cleaned of all loose debris by high pressure air or water to take advantage of the higher coefficient of friction.

4.4 Utility Installation

Trench excavations for utilities cannot be accomplished by conventional trenching equipment. Rock excavation equipment/techniques will be required for excavation of the sandstone. Shallow trench walls, less than 3 feet, should stand near-vertically for the short periods of time required to install utilities. If trenches are greater than shoulder-height, precautions must be taken to protect workmen. Trenches greater than 3 feet in depth should be laid back following the recommendations contained in Section 4.7 - Temporary and Permanent Slopes. All trenches should be in accordance with OSHA Excavation Standard 1926 Subpart P.

Pipe bedding and shading should be per M.A.G. Specification Section 601.4 (and any City of Sedona modifications). Backfill of trenches above bedding and initial backfill zones may be carried out with native excavated material provided material greater than 3 inches is broken down or removed. Material used for backfill of trenches should be moisture-conditioned, placed in 8 inch lifts and mechanically compacted. Water settling is not recommended. Compaction requirements are summarized in Section 4.1.3 Fill and Backfill of this report.

4.5 Slab-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.

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 and/or cause a tripping hazard. Pinning typically consists of 24-inch long reinforcing steel dowels placed at 12-inch center.

4.6 Pavement

It must be noted that all new asphalt pavements will eventually crack. Cracking in asphalt pavement is typical and should be expected over the life of the pavement. In fact, it has been our experience of late that the new asphalt binders that are available, we are seeing the onset or earlier aging and block shrinkage cracking. These require routine maintenance to prevent accelerated deterioration.

Accordingly, it is highly recommended to establish a maintenance program where the cracks are routinely filled as they appear beginning at about the second year of life. It is also recommended that surface fog seal coats be considered beginning at about year 5 and every 5 years after. This will help preserve the pavements, extending the service life.

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 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. In residential subdivisions, the worst offender, construction traffic, is often over looked. The designer/owner should choose the appropriate sections to meet the anticipated traffic volume and life expectancy.

4.6.1 Pavement Sections

Area of Placement	AC Pavement (Flexible)			PCCP Pavement (Rigid)	
	Thickness (in.)		Daily 18-kip ESALs	Thickness (in.)	Daily 18-kip ESALs
	AC	AB		PCCP	
Parking/ Driveways	2.5	4.0	8	5.0	11
	3.0	4.0	16	6.0	27
	3.0	6.0	34	7.0	58
	4.0	6.0	148	8.5	173

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” asphalt for each 3.0” of aggregate base replaced.

<u>Pavement Design Parameters</u>:

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

<u>Subgrade Soil Profile:</u>

Average % Passing #200 sieve: 27%
Plasticity Index: 4%
k: 200 pci (assumed)

R value:	59 (per ADOT tables)
M_R:	18,457 (per AASHTO design)

These pavement sections 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. Site drainage should be designed to ensure positive drainage of the base and sub base materials. Improper grading of sub base materials will drastically reduce the overall life of the pavement.

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 (and any City of Sedona modifications) using the Marshall mix design criteria and PG 70-10 for the asphalt grade. Reducing the air void content to 3 percent will aid in reducing thermal cracking typical in the area. It is recommended that a 12.5mm or 19.0mm mix designation be used for the pavements. While a 19.0mm 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.

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.

4.7 TEMPORARY AND PERMANENT 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.

In accordance with Building Code requirements, all occupied structures should be set back from the crest (top edge) of the slope such that the outer edge of the nearest foundation is no closer than a distance equal to at least one third (⅓) of the total height of the slope. See specific building code requirements for additional detail and/or placement of structures at the bottom (toe) of slopes.

4.7.1 Cut/Fill Slopes - Soil

Generally, permanent cut or fill slopes in soil or decomposed sandstone should be no steeper than 2 horizontal to 1 vertical (2(h):1(v)). Where particular conditions make it appropriate to vary from these slopes, these must be addressed on a case by case basis, either in this report or at special request directed to a representative of this office. Where fills are made on hillsides or slopes, the slope of the original ground upon which the fill is to be placed shall be plowed or scarified deeply or where the slope ratio of the original ground is steeper than 5(h):1(v), the bank shall be stepped or benched to remove all loose soils and to provide a level surface for placement of fill. Ground slopes which are flatter than 5 to 1 may require benching when considered necessary by a representative of this office. The benches should be cut wide enough to remove loose surface soils and allow proper compaction of fills. A minimum bench width of 8 feet is typically recommended for the first lift (toe) of any fill placed on a slope. This width may be reduced at the direction of the field engineer depending on the presence of loose soils, slope steepness, exposed rock and lift thickness. A keyway shall also be constructed at the toe of the slope. The key width shall be ½ times the height of the slope or at least 1½ times the width of the compaction equipment. The key bottom shall be sloped 2% toward the slope. The key shall be excavated into dense soil or rock formation to a minimum depth of 18 inches unless approved otherwise by the engineer.

Placement and obtaining compaction of fill adjacent to fill slopes may be very difficult. Depending on soil type and final slope configuration, it may be necessary to over-build the slope and cut back to the final configuration to obtain the required degree of compaction.

4.7.2 Cut Slopes - Rock

To determine cut slope angles in rock, a kinematic slope stability analysis of rock discontinuities was conducted with ROCKPACK III to identify critical potential failure planes that may be associated with the sandstone bedrock formation. Lacking outcrops exposed on the slope surfaces, the discontinuity data were obtained from fractures and bedding plans present in the rock cores. Orientation of the discontinuities is assumed to be present in all directions. Therefore, the discontinuity data used for this kinematic analysis is assumed to be representative for the bedrock unit that will eventually be exposed regardless of the slope direction. Once construction has begun and discontinuities are visible, a representative of this office should visit the site to confirm the assumptions used are valid.

Observations of rock discontinuities observed in the cores, combined with available rock strength data, are used to:

- Evaluate rock structure relationships that could result in the slope failure and that may potentially affect stability of the proposed finish cut slopes.
- Qualitatively analyze slope stability to assess failure potential and identify preferential cut slope geometry compatible with the geological conditions.

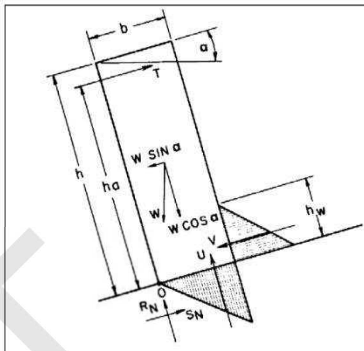
Kinematic analysis of slope was accomplished by performing a computer-aided stereographic projection analysis of discontinuity measurements with orientations assumed in all directions. This method analyzes 3-dimensional rock discontinuity geometry data relative to the proposed cut slope geometry (slope face direction and cut slope angles) using stereonet analytical projection of the 3-dimensional discontinuity data points to a 2-dimensional planar surface (paper space).

For the kinematic analysis, all discontinuities are assumed to be continuous and through going, when in reality many of them are not. Even a small percentage of intact rock along a discontinuity can be enough to make it safe from sliding. Also, the stereonet procedure is a “cohesion-equals-zero” analysis, in which the effects of cohesion are ignored. When this assumption is made, the fundamental limiting equilibrium equation for calculating safety factor (FS) is - $FS = \tan \Phi / \tan \theta$ (C.F. Watts, 2003).

The results of the kinematic analysis is used to identify critical bedrock discontinuity planes that daylight in the slope face at an inclination angle that is flatter than the natural or cut slope angle and at an angle steeper than the friction angle of the bedrock. Concentrations are defined based on the contoured plot of discontinuity dip vectors. This contoured data analysis is used to define the relative importance and the potential for various types of failure modes within bedrock units with respect to the geometry of the slope. Where this analysis suggests a particular slope angle may be potentially unstable, examination of discontinuity concentrations relative to the slope geometry can be used to identify alternative slope angles or excavation orientations, that when subjected to a kinematic analysis, may prove to be favorable relative to the stability of the slopes. However, all natural or cut slopes, over time can be subject to localized rock “pop-outs” and small detachments of rock blocks in response to natural weathering and erosion cycles.

To define the critical discontinuities and the stereonet critical zone limits for potential plane failures, analysis was performed on finish cut slopes of 76° (1/4(h):1(v)) and 63° (1/2(h):1(v)). The results of the stereonet analysis indicate that slopes configured at 63° (½(h):1(v)) show no signs of plane failure where slopes as there are no critical bedrock discontinuity planes that would daylight in the slope face. Slopes configured at 76° exhibit critical bedrock discontinuity planes daylighting on the slope face indicting a risk to global failure. To define the critical discontinuities and the stereonet critical zone limits for potential toppling failures, analysis was performed on finish cut slope of 63°, ½ (h):1(v). Analysis for toppling shows a moderate potential for toppling if the high angle joints dip into the cut face. Stereonet plots of the kinematic analyses are attached.

The program TOPPLE was then used to provide a more quantitative way to examine potential topples since toppling was identified in the kinematic analysis. TOPPLE calculates the sum of the moments of the potential toppling blocks. When the resulting forces is zero, the block is in equilibrium, with the overturning moment equal to the resisting moment. A negative sum of moments indicates that toppling is not likely to occur. The model used is very simplistic analysis used for a very complex mode of failure with numerous factors that can easily deviate from the conditions that must be assumed. The following assumptions are made in the TOPPLE analysis:



FORCES — SINGLE TOPPLING BLOCK

- The block is fixed at the lower downhill corner such that it cannot slide down the plane. Rotation around point “O” is the only movement permitted.
- The reaction force R_N is a point force and is acting at the point “O” at the beginning of rotation.
- Hydrostatic pressure has been removed from the analysis assuming a dry slope. A water column height (h_w) is 0.

The results of the TOPPLE analysis indicates that provided the width of the block (b) is at least 1/3 of the height (h), the slope should be in equilibrium and toppling should not be an issue.

Based on the analysis performed, it is our professional opinion that a cut slope in the moderately weathered to fresh sandstone configured at 63°, ½(h):1(v), will perform satisfactorily against global failure. As indicated, toppling may occur if the block width is less than 1/3 the block height. Furthermore, wedge failure analysis was not performed due to the unknown orientation of joints and infinite possibilities of intersecting planes. Once construction has begun and discontinuities are visible, a representative of this office should visit the site to further evaluate the risk of topping and wedge failures. The preference for remediation of potential toppling blocks and wedge failures is to simply remove the threatening blocks.

5.0 CONCLUSION

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.

Clay W. Spencer, R.G.

Gregg A. Creaser, P.E.

APPENDIX

SOIL BORING LOCATION PLAN

SOIL LEGEND

ROCK TERMINOLOGY

LOG OF TEST BORINGS

TABULATION OF TEST DATA

MOISTURE-DENSITY RELATIONS

SWELL TEST DATA

DIRECT SHEAR

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORES

ROCK CORE PHOTOGRAPHS

KINEMATIC ANALYSIS STEREONETS

– APPROXIMATE SOIL BORING LOCATIONS

Drawing Courtesy of Google Earth

SOIL BORING LOCATION PLAN					Sedona Parking Structure 460 and 430 Forest Road Sedona, Arizona	SPEEDIE AND ASSOCIATES GEOTECHNICAL/ENVIRONMENTAL/MATERIALS ENGINEERS 2026 N. 3rd Street FLAGSTAFF, ARIZONA 86004
DR:GJC	CHK:CWS	REV:	DATE: 2-3-21	PROJECT NO. 210124SF		

TMPLT_2000.CAD 10/23/00

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.

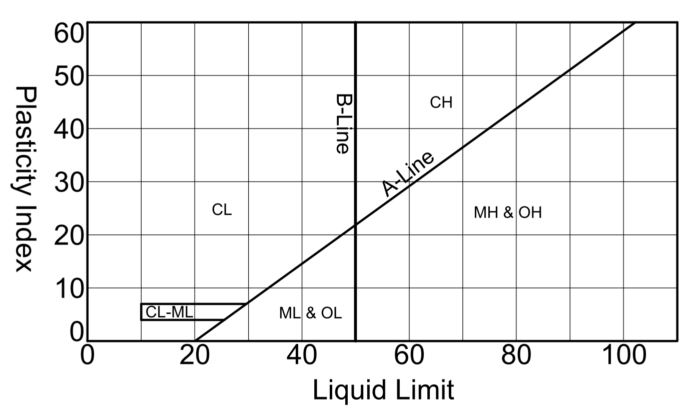
CONSISTENCY			RELATIVE DENSITY	
Clays & Silts	Blows/Foot	Strength (tons/sq ft)	Sands & Gravels	Blows/Foot
Very Soft	0 - 2	0 - 0.25	Very Loose	0 - 4
Soft	2 - 4	0.25 - 0.5	Loose	5 - 10
Firm	5 - 8	0.5 - 1.0	Medium Dense	11 - 30
Stiff	9 - 15	1 - 2	Dense	31 - 50
Very Stiff	16 - 30	2 - 4	Very Dense	> 50
Hard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



ROCK TERMINOLOGY

SCALE OF RELATIVE HARDNESS	
Term	Field Identification
Extremely Soft	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.
Very Soft	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocketknife. Scratched with fingernail.
Soft	Can be peeled by a pocketknife with difficulty. Cannot be scratched with fingernail. Shallow indention made by firm blow of a geology pick.
Medium Hard	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.
Hard	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.
Very Hard	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.

STRATIFICATION TERMS	
Term	Characteristics
Laminations	Thin beds (<½ inch)
Fissile	Tendency to break along laminations.
Parting	Tendency to break parallel to bedding, any scale.
Foliation	Non-depositional, e.g., segregation and layering of minerals in metamorphic rocks.

ROCK TERMINOLOGY

SCALE OF RELATIVE ROCK WEATHERING	
Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with a geologists pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock fabric may be evident. May be reduced to soil with hand pressure.

JOINT AND BEDDING/FOLIATION SPACING TERMS		
Spacing	Joint Spacing Terms	Bedding/Foliation Spacing Terms
<2 in.	Very Close	Very Thin (Laminated)
2 in. to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately Close	Medium
3 ft. to 10 ft.	Wide	Thick
>10 ft.	Very Wide	Very Thick (Massive)

Depth (feet)	Graphic Log	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		Rig Type: **CME-75** Boring Type: **Hollow Stem Auger** Surface Elevation: **N/A**					
		Medium Dense Reddish Brown CLAYEY SAND (SC-Dry)					
1.8 – 2.0		Hard Reddish Brown SANDSTONE	AS-1	2.0	NT	NT	
		Auger Refusal on Bedrock	RS-2	2.2	NT	NT	50/2"
5							
10							

Boring Date: **2-11-21**
Field Engineer/Technician: **G. Chott**
Driller: **B. Anderson**
Contractor: **Resilient Drilling**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

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

Sedona Parking Structure

430/460 Forest Road

Sedona, Arizona

Project No.: **210124SF**

_SPEEDIE 210124SF.GPJ GENGEO.GDT 3/31/21

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

Depth (feet)	Visual Classification	Layer Depth	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 / 25 / 50)
0	Medium Dense Reddish Brown CLAYEY SAND (SC-Dry) with Some Gravel	1.0					
	Soft to Medium Hard; Decomposed to Predominantly Decomposed Reddish Brown SANDSTONE	2.0	BS-1	2.0	NT	NT	
5	Soft to Medium Hard; Predominantly Decomposed to Moderaely Weathered Reddish Brown SANDSTONE	5.0					
	Auger Refusal on Bedrock		S-2	5.2	NT	NT	50/3"
10							

DRAFT

Boring Date:	2-11-21
Field Engineer/Technician:	G. Chott
Driller:	B. Anderson
Contractor:	Resilient Drilling

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES

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

Sedona Parking Structure

430/460 Forest Road

Sedona, Arizona

Project No.: **210124SF**

_SPEEDIE 210124SF.GPJ GENGEO.GDT 3/31/21

GEOTECHNICAL SERVICES
EXPLORATION LOG

PROJECT NO.: 210124SF | DATE: 2-11-21 | GEOL/TECH: G. Chott
PROJECT NAME: Sedona Parking Structure | DRILLER: B. Anderson
LOCATION: 430/460 Forest Road | CONTRACTOR: Resilient Drilling
BORING NO.: B- 3 | STATION: N/A
RIG TYPE: CME-75 | ELEVATION: N/A

DEPTH IN FEET	NX CORING DATA %RECOVERY	NX CORING DATA %RQD	PENETRATION RESISTANCE BLOWS/12"	DRILLING DATA NOTES	DRILLING DATA AVERAGE RATE (Min/Ft)	VISUAL CLASSIFICATION & REMARKS
0–1.5						Medium Dense Reddish Brown CLAYEY SAND (SC-Dry) with Some Gravel
1.5–2.3			50/3"			SANDSTONE Reddish Brown Decomposed to Predominantly Decomposed Soft Fine Grained;
2.3–5.0						SANDSTONE Reddish Brown Moderately Weathered Soft to Medium Hard Fine Grained;
5.0–10.8						Began Coring At 5' SANDSTONE Reddish Brown Moderately Weathered to Slightly Weathered Hard Fine Grained; Close Joints/Fractures;
10	100	35				
10.8–17.2						SANDSTONE Reddish Brown Slightly Weathered Hard Fine Grained; Moderately Close to Wide Joints/Fractures;
15	100	68				
17.2–22.0						SANDSTONE Mottled Light Brown/Brown Moderately Weathered to Slightly Weathered Hard Fine Grained; Close Joints/Fractures;
20	100	94				
22.0–26.0						SANDSTONE Reddish Brown Moderately Weathered to Slightly Weathered Hard Fine Grained; Moderately Close to Wide Joints;/Fractures
25	100	66				
26.0–35.0						SANDSTONE Reddish Brown Moderately Weathered to Slightly Weathered Hard Fine Grained; Moderately Close to Wide Joints;/Fractures
30	100	56				
35	100	96				
35.0						End of Boring

	Depth	Hour	Date
∇	*Free Water was Not Encountered*		
▼			

DRAFT

SPEEDIE ROCK 210124SF.GPJ GENGEO.GDT 3/31/21

SPEEDIE AND ASSOCIATES

GEOTECHNICAL SERVICES
EXPLORATION LOG

PROJECT NO.: 210124SF
DATE: 2-12-21
GEOL/TECH: G. Chott
PROJECT NAME: Sedona Parking Structure
DRILLER: B. Anderson
LOCATION: 430/460 Forest Road
CONTRACTOR: Resilient Drilling
BORING NO.: B- 4
STATION: N/A
RIG TYPE: CME-75
ELEVATION: N/A

DEPTH IN FEET	NX CORING DATA %RECOVERY	NX CORING DATA %RQD	PENETRATION RESISTANCE BLOWS/12"	DRILLING DATA NOTES	DRILLING DATA AVERAGE RATE (Min/Ft)	GRAPH	VISUAL CLASSIFICATION & REMARKS
0 – 1.4							Medium Dense Reddish Brown CLAYEY SAND (SC-Dry) with Some Gravel
1.4 – 2.0							SANDSTONE Reddish Brown Decomposed to Predominantly Decomposed Soft Fine Grained;
2.0 – 10.0							SANDSTONE Reddish Brown Moderately Weathered Soft to Medium Hard Fine Grained; Very Slow Hard Drilling from 5 to 10 feet
5							
10							Began Coring At 10' SANDSTONE Reddish Brown with Some Mottling Moderately Weathered to Slightly Weathered Hard to Very Hard Fine Grained; Close to Wide Joints/Fractures;
15	96	93					
20	100	98					
25	95	93					
30	100	96					
35	100	86					35.0 End of Boring

	Depth	Hour	Date
∇	*Free Water was Not Encountered*		
▼			

SPEEDIE AND ASSOCIATES

SPEEDIE ROCK 210124SF.GPJ GENGEO.GDT 3/31/21

GEOTECHNICAL SERVICES
EXPLORATION LOG

PROJECT NO.: 210124SF
DATE: 2-12-21
GEOL/TECH: G. Chott
PROJECT NAME: Sedona Parking Structure
DRILLER: B. Anderson
LOCATION: 430/460 Forest Road
CONTRACTOR: Resilient Drilling
BORING NO.: B- 5
STATION: N/A
RIG TYPE: CME-75
ELEVATION: N/A

DEPTH IN FEET	NX CORING DATA %RECOVERY	NX CORING DATA %RQD	PENETRATION RESISTANCE BLOWS/12"	DRILLING DATA NOTES	DRILLING DATA AVERAGE RATE (Min/Ft)	GRAPH	VISUAL CLASSIFICATION & REMARKS
							Medium Dense Reddish Brown CLAYEY SAND (SC-Dry) with Some Gravel — 1.5
							SANDSTONE Reddish Brown Decomposed to Predominantly Decomposed Soft Fine Grained; — 3.0
							SANDSTONE Reddish Brown Moderately Weathered Medium Hard to Hard Fine Grained;
5							Very Slow Hard Drilling from 5 to 10 feet
10							
15							15.0
							Began Coring At 15'
							SANDSTONE Reddish Brown with Some Mottling Moderately Weathered to Slightly Weathered Hard to Very Hard Fine Grained; Close to Wide Joints/Fractures;
20	100	82					
25	93	84					
							Dissolution Vug at 26'
30	98	95					
35	100	90					35.0
							End of Boring

	Depth	Hour	Date
▽	*Free Water was Not Encountered*		
▼			

SPEEDIE AND ASSOCIATES

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-2	RING	2.0 - 2.2	NT	NT	22.1	32	48	64	100	NP	NP	NP	SM	SILTY SAND with GRAVEL
B- 2	BS-1	BULK	0.0 - 2.0	NT	NT	35.3	49	57	69	100	20	16	4	SC-SM	SILTY, CLAYEY SAND with GRAVEL
B- 3	RS-1	RING	2.0 - 2.2	NT	NT	24.5	34	44	53	100	21	17	4	GC-GM	SILTY, CLAYEY GRAVEL with 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

Sheet 1 of 1

Sedona Parking Structure
430/460 Forest Road
Sedona, Arizona
Project No. 210124SF

SPEEDIE AND ASSOCIATES

MOISTURE-DENSITY RELATIONS

PROJECT: Sedona Parking Structure — **PROJECT NO.:** 210124SF

LOCATION: 430/460 Forest Road — **DATE:** 2/11/21

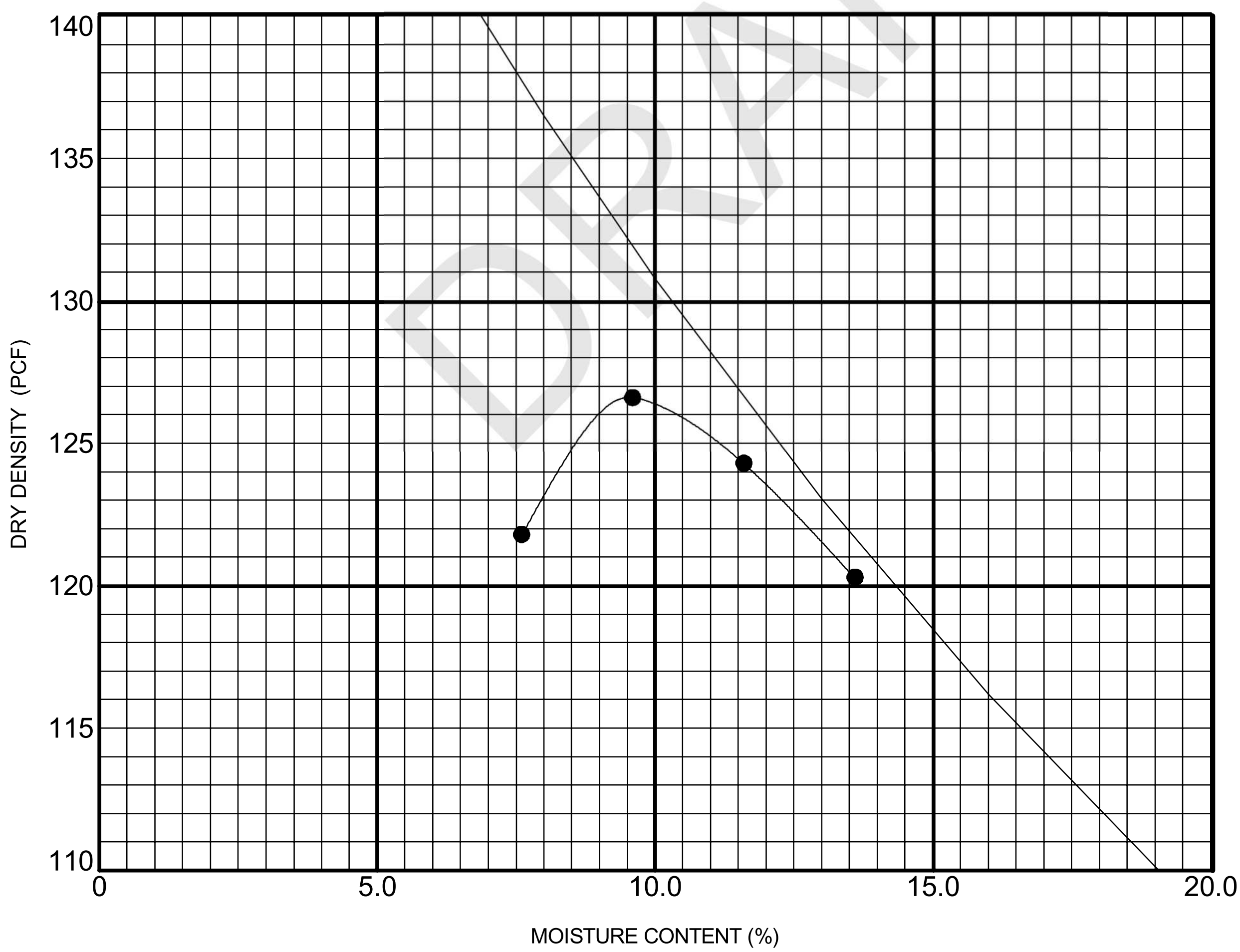
BORING NO.: B- 2 — **SAMPLE NO.:** BS-1 — **SAMPLE DEPTH:** 0 to 2 — **LABORATORY NO.:** CMA37

METHOD OF COMPACTION: D698A

LIQUID LIMIT: 20 — **PLASTIC LIMIT:** 16 — **PLASTICITY INDEX:** 4

CLASSIFICATION: SC-SM — **ASTM SOIL DESCRIPTION:** SILTY, CLAYEY SAND with GRAVEL

MAXIMUM DRY DENSITY: **126.6 PCF** — OPTIMUM MOISTURE CONTENT: **9.6%**



SPEEDIE AND ASSOCIATES

GEOTECH PROCTOR 210124SF.GPJ 3/31/21

SWELL TEST DATA

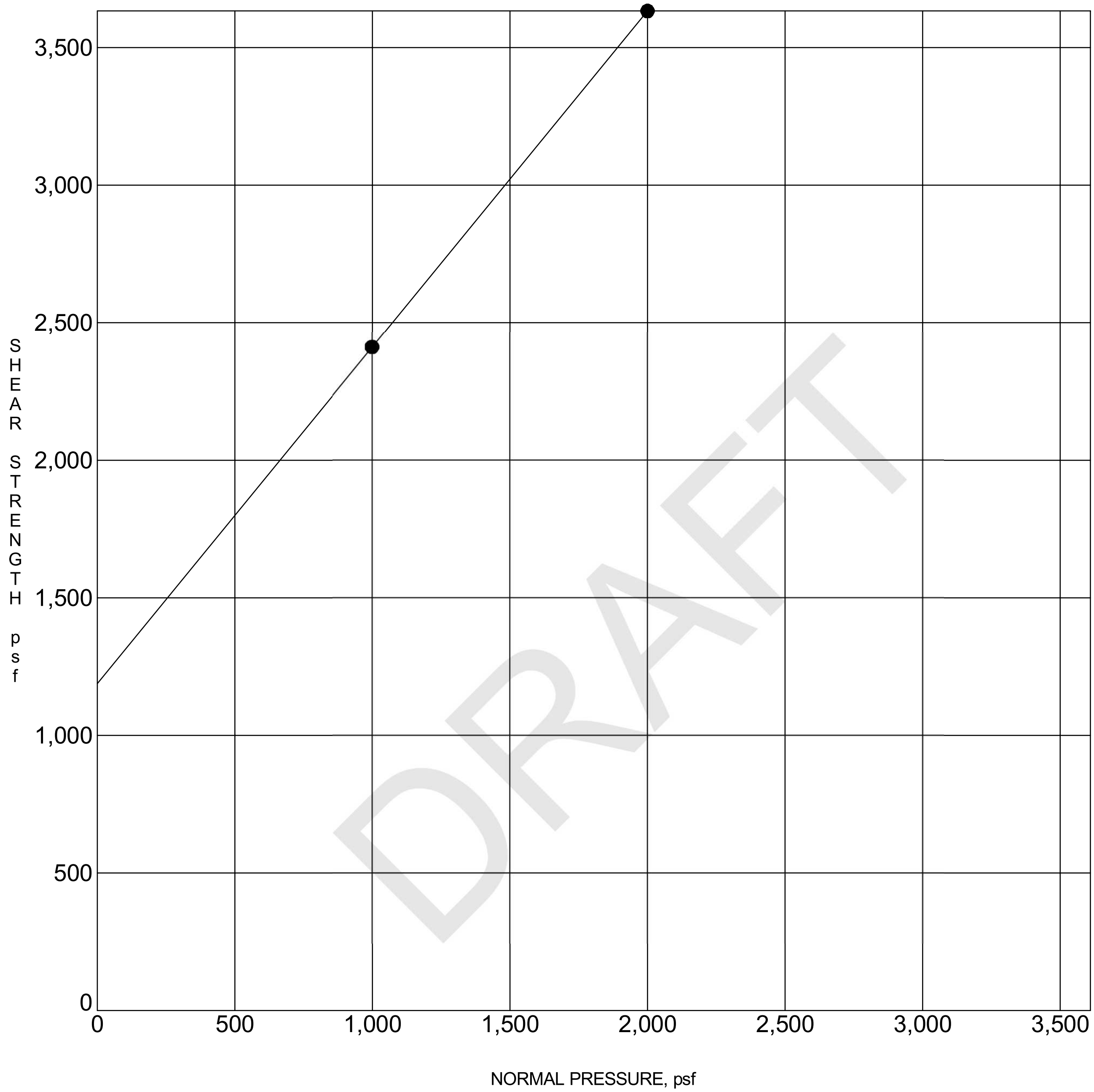
BORING or TEST PIT No.	SAMPLE DEPTH, ft	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	REMOLDED DRY DENSITY (pcf)	INITIAL MOISTURE CONTENT (%)	PERCENT COMPACTION	FINAL MOISTURE CONTENT (%)	CONFINING LOAD (psf)	TOTAL SWELL (%)
B- 2, BS-1	2.0	126.6	9.6	120.6	7.8	95.3	14.8	100	1.4

Sedona Parking Structure
430/460 Forest Road
Sedona, Arizona
Project No. 210124SF

SPEEDIE AND ASSOCIATES

SWELL TEST 210124SF.GPJ GENGEO.GDT 3/29/21

SHEAR TEST DIAGRAM



	Specimen Identification		Cohesion, psf	Friction Angle	DD	MC%
●	**B- 3**	**2.0**	**1000.0**	**45.0**	**98.7**	**13.0**

PROJECT **Sedona Parking Structure - 430/460 Forest Road**

JOB NO. **210124SF**
DATE **2/11/21**

SPEEDIE AND ASSOCIATES

GEOTECH DIRECT SHEAR 210124SF.GPJ 3/29/21

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS (ASTM D2938)

PROJECT: Sedona Parking Structure	Project NO. 210124SF
LOCATION: Forest Road w/o Smith Road, Sedona, AZ	
CLIENT: Gabor Lorant Architects, Incoporated	

MATERAL:	DATE SAMPLED:
SOURCE:	DATE SUBMITTED: 2/16/21
SAMPLE LOCATION: B-3, C-1	DATE TESTED: 2/18/21

REMARKS:

SAMPLE NUMBER:	CMA40	CMA46	CMA47	CMA51
CORE LOCATION:	B-3 @ 10'	B-4 @ 15	B-4 @ 20'	B-4 @ 32'
DATE TESTED:	02/18/21	02/18/21	02/18/21	02/18/21
SAMPLE DIAMETER (in.)	2.38	2.38	2.38	2.38
SAMPLE LENGTH (in.)	4.77	4.69	4.67	4.76
CAPPED LENGTH (in.)	5.18	5.16	5.30	5.19
SPECIMEN AREA (sq.in.)	4.46	4.463	4.456	4.445
TIME OF TEST	12:00	12:05	12:10	12:15
LAB TECH ID	SWH	SWH	SWH	SWH
MOISTURE CONDITION	Dry	Dry	Dry	Dry
ORIENTATION OF CORE TO SOURCE	Perp	Perp	Perp	Perp
TOTAL LOAD (lb.)	33,860	36,530	35,830	11,350
TYPE OF FRACTURE	Columnar	Columnar	Columnar	Shear
COMPRESSIVE STRENGTH (psi)	7598	8185	8040	2553
LENGTH TO DIAMETER CORRECTION	1.00	1.00	1.00	1.00
CORRECTED COMPRESSIVE STRENGTH (psi)	7600	8190	8040	2250
WEIGHT OF SAMPLE (grams)	904.8	887.2	887	875.6
UNIT WEIGHT (pcf)	162.3	161.4	162.3	157.7

DATE REQUESTED:		REQUESTED BY:	CLIENT
DATE SAMPLED:		SAMPLED BY:	G. Chott
DATE RECEIVED:		SUBMITTED BY:	G. Chott
DATE PLACED:		REVIEWED BY:	C. Spencer

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS (ASTM D2938)

PROJECT:	Sedona Parking Structure	Project NO. 210124SF
LOCATION:	Forest Road w/o Smith Road, Sedona, AZ	
CLIENT:	Gabor Lorant Architects	

MATERAL:		DATE SAMPLED:	
SOURCE:		DATE SUBMITTED:	2/16/21
SAMPLE LOCATION:	B-5, C-1	DATE TESTED:	2/18/21

REMARKS:

SAMPLE NUMBER:	CMA53			
CORE LOCATION:	B-5 @ 20'			
DATE TESTED:	02/18/21			
SAMPLE DIAMETER (in.)	2.38			
SAMPLE LENGTH (in.)	4.78			
CAPPED LENGTH (in.)	5.48			
SPECIMEN AREA (sq.in.)	4.45			
TIME OF TEST	12:20			
LAB TECH ID	SWH			
MOISTURE CONDITION	Dry			
ORIENTATION OF CORE TO SOURCE	Perp			
TOTAL LOAD (lb.)	44,500			
TYPE OF FRACTURE	Columnar			
COMPRESSIVE STRENGTH (psi)	10000			
LENGTH TO DIAMETER CORRECTION	1.00			
CORRECTED COMPRESSIVE STRENGTH (psi)	44500			
WEIGHT OF SAMPLE (grams)	939.2			
UNIT WEIGHT (pcf)	168.2			

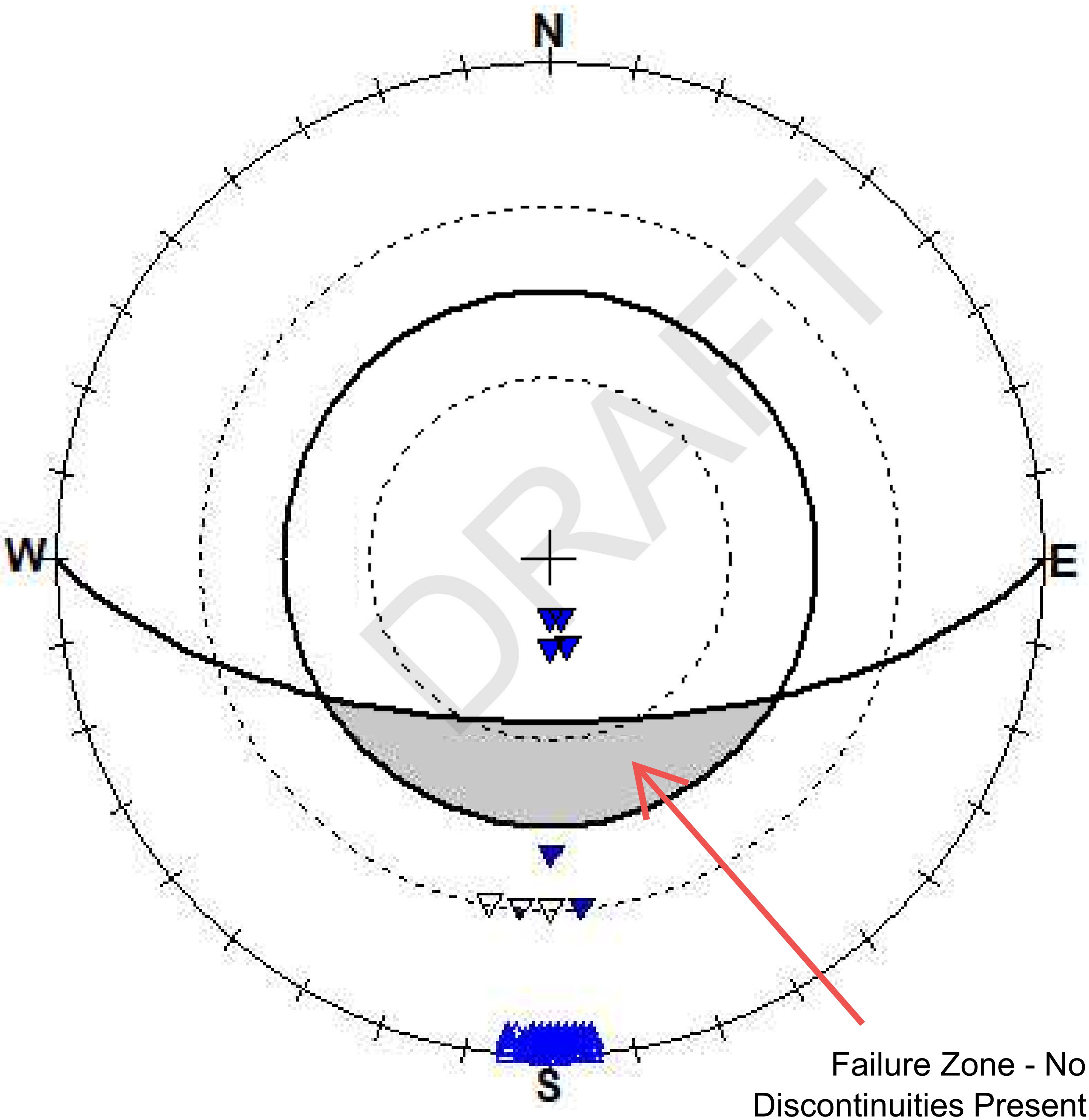
DATE REQUESTED:		REQUESTED BY:	CLIENT
DATE SAMPLED:		SAMPLED BY:	G. Chott
DATE RECEIVED:		SUBMITTED BY:	G. Chott
DATE PLACED:		REVIEWED BY:	C. Spencer

Boring B-3

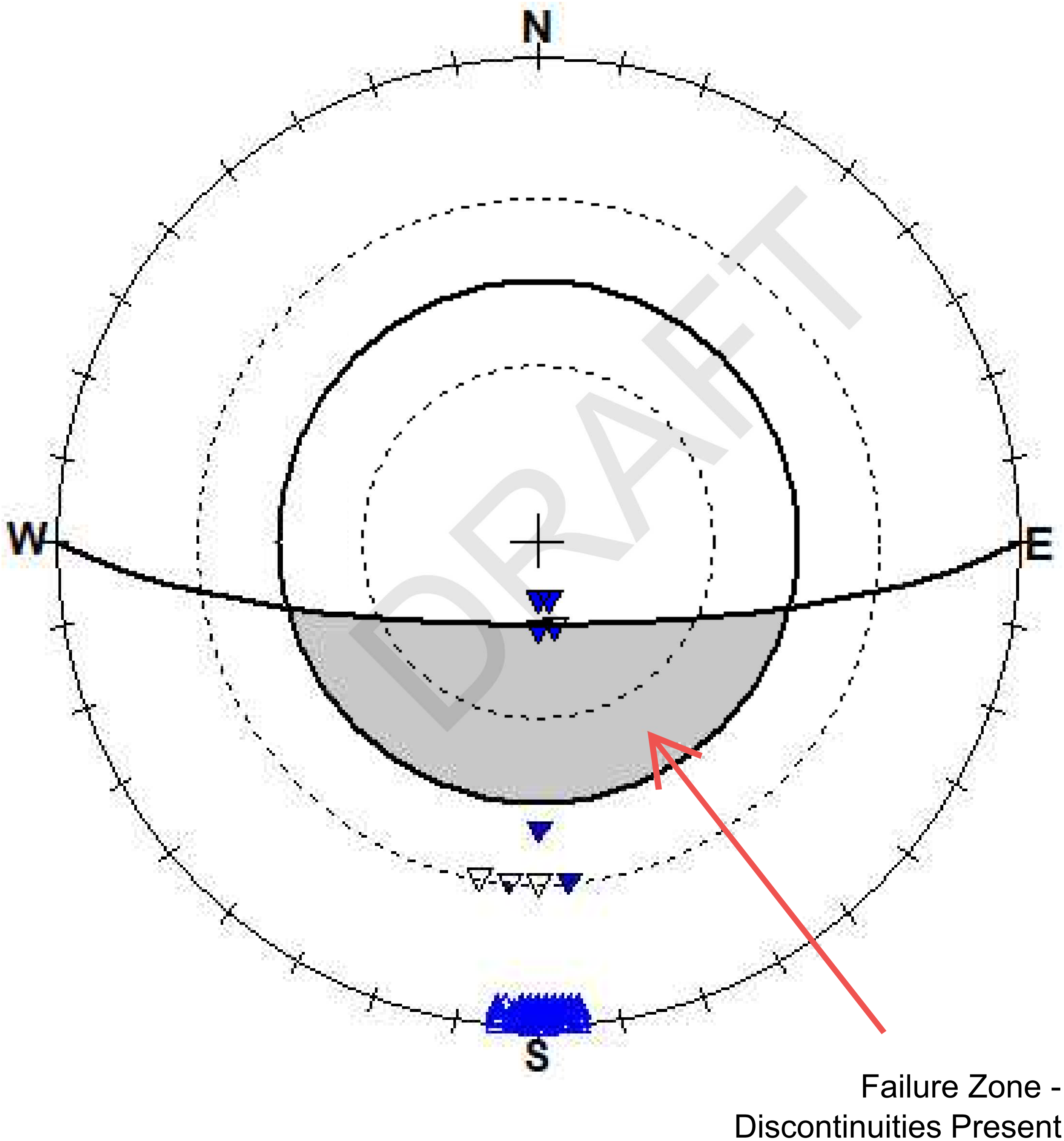
Boring B-4

Boring B-5

1/2:1 Kinematic Plane Failure Analysis



1/4:1 Kinematic Plane Failure Analysis



1/2:1 Kinematic Wedge Failure Analysis