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Report on Geotechnical Investigation

Registered Professional Engineer (Civil)
CERTIFICATE NO.
43813
TODD B.
HANKE
Date Signed 4/11/2023
ARIZONA, U.S.A.

Designation:	**Mount Elden Lookout Road Box Culvert**
Location:	**Mount Elden Lookout Road, Flagstaff, AZ**
Client:	**Coconino County Public Works Department**
Project Number:	**230072SF**
Date:	**April 11, 2023**

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TABLE OF CONTENTS

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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 box culvert at Schultz Creek Crossing beneath Mount Elden Lookout Road, located approximately at 1000 feet east of the intersection of W. Mount Elden Lookout Road and Schultz Pass Road, in Flagstaff, Arizona.

At this time, it is our understanding that construction will consist of a four-sided 4 foot by 8 foot precast concrete box culvert and precast wing walls on the inlet and outlet. The new concrete box culvert will be installed to replace the existing metal pipe culvert. The box culvert will be designed per the ADOT Typical Single Box Detail SD 6.01-5, and the wing walls will be designed per ADOT Typical Outlet Wings 25° to 45° Detail SD 6.08-5. The roadway that traverses over the box culvert will be reprofiled, and the asphalt will be removed and replaced. A traffic count study was not available at the time of this report; however, traffic loads are expected to consist of moderate passenger and light truck traffic. Preliminary grading concepts indicate that cuts and fills will be 10 feet or less. Adjacent areas will remain as they currently exist or be landscaped for storm water retention and disposal.

1.2 FIELD AND LABORATORY INVESTIGATION

On February 20, 2023, six structural borings and one pavement boring were drilled at the approximate locations shown on the 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 tracked skid steer utilizing 12-inch diameter solid stem augers. Detailed information regarding the soil borings and samples obtained can be found on an individual Log of Test Boring prepared for each location.

Laboratory testing consisted of moisture content, dry density, grain-size distribution, plasticity (Atterberg Limits) tests, and R-values for classification and pavement design parameters. Remolded swell tests were performed on samples compacted to densities and moisture contents expected during construction. All field and laboratory data are presented in the appendix of this report.

2.0 SITE CONDITIONS

2.1 PROPERTY DESCRIPTION

The existing metal culvert is approximately 30 feet in length and crosses Mount Elden Lookout Road perpendicularly from north to south. The site is bounded on the north, east, and west by the Coconino National Forest and on the south by multiple single family residences. At the time of the investigation, the site was partially covered in snow, which created significantly wetter conditions than might typically be observed. The surrounding National Forest land was moderately vegetated with mature Ponderosa Pines and native grasses. The areas surrounding both the north and south ends of the culvert were covered with wood debris, and significant sediment appeared to have been washed away from the outlet of the culvert (south end) from previous flooding events. Roughly 4 horizontal feet of the outlet was exposed at the time of the field investigation. As previously mentioned, the culvert flows from north to south, with about 60 feet of evident channelized flow on the north side leading to a meandering trapezoidal channel on the south side. Refer to the following Figures.

Figure 2.1.1 – North Side

Figure 2.1.2 – South Side

2.2 LOCAL GEOLOGY

The subject site is located within the Coconino Plateau in the San Francisco Volcanic Field. The geologic profile of the region varies significantly due to historical volcanic activity. Shallow basalt boulders and/or bedrock is fairly typical of the region, as well as localized deposits of volcanic tuff, generally overlying Moenkopi Sandstone or Kaibab Limestone.

2.3 SUBSURFACE CONDITIONS

2.3.1 Field Results

Subsurface conditions are somewhat consistent, though slight variations are observed in the upper 6 inches to 1 foot of soil between the northern and southern ends of the culvert due to flooding and buildup of sediment. The upper soils on the south side of the culvert, labeled as Borings B-1, B-2, and B-3, consist of 1 to 3 feet of wet, dark brown silty gravel, overlying shallow basalt cobbles and boulders. Boring B-3 differs slightly, with approximately 1 foot of roadway fill, classified as moist red sand with silt, overlying 3 feet of moist brown silty gravel with sand. Basalt cobbles and boulders are present at a depth of 4 feet. Auger refusal occurred at depths ranging from 1 to 4 feet in the southern borings, coinciding with the depths at which basalt cobbles and boulders are encountered. On the southern end of the culvert, the embankment for the road is washed away and the soil profile for the road is exposed. Due to this erosion, the roadway fill is deposited more on the east side of the outlet. Basalt cobbles and boulders are also visible at ground surface elevation on the southern side of Mount Elden Lookout Road.

The upper soils on the north side of the culvert, labeled as Borings B-5, B-6, and B-7, consist of approximately 6 inches of moist to wet black peat and other organic materials, overlying 1.5 to 6.5 feet of moist to wet, brown to dark brown silty sand, with increasing amounts of gravel at depths of 3 to 4 feet. Auger refusal on basalt cobbles and boulders occurs at depths ranging from 2 to 7 feet.

A pavement core, removed in the area of Boring B-4 before drilling, exposed a 9.25-inch thick pavement section consisting of two Hot Mix Asphalt (HMA) overlays, indicating that the most recent overlay is approximately 1.25 inches thick and the older overlay is 0.5 inches thick. The rest of the 7.5-inch thick pavement section appears to consist of millings with cinders that may have been oil conditioned during installation. No aggregate base (AB) is evident below the pavement. Below the pavement is approximately 2.25 feet of fill, classified as moist red silty sand with gravel. Below the fill layer is 3 feet of moist brown silty sand. Auger refusal on basalt cobbles and boulders occurred at a depth of 6 feet.

Based on visual and tactile observation at the time of the investigation, the upper soils were typically in a <u>moist to wet state</u>, and above optimum moisture for the soils. Groundwater was not encountered during this investigation; however, it is not uncommon to have seasonally perched water that may be encountered at or near the soil/bedrock interface. Additionally, the topography to the north of the culvert may encourage pools of standing water to form in and around the culvert during periods of excessive precipitation.

2.3.2 Laboratory Results

Laboratory testing indicates that site soils are generally non-plastic (NP). In-place densities of the upper soils are on the order of 75 to 93 pcf with moisture contents on the order of 14 to 42 percent. Volume increase due to wetting of the upper silty soils are generally less than 1.0 percent when re-compacted to moistures and densities normally expected during construction and confined to 100 psf.

Based on the sieve and PI data and ADOT correlations, the correlated R-values (Rc) range from 59 to 81, with an average of 70 and a standard deviation of 11. A sample tested from Boring B-4 indicated a laboratory R-value of 69, at 300 psi. Based on the laboratory test results and following ADOT design procedures, an **R-value of 70 was selected** and used for design of the new pavements. This assumes that the existing native soils will be used to support any new roadway pavement.

2.4 SEISMIC PARAMETERS

The project area is located in a seismic zone that is considered to have moderate historical seismicity. Flagstaff, Arizona has a moderate earthquake risk, with a total of 8 earthquakes since 1931. The USGS database shows that there is a 27.7% chance of a magnitude 5.0 or greater earthquake within 30 miles of Flagstaff, Arizona within the next 50 years. The largest earthquake within 30 miles of Flagstaff, AZ was a magnitude 4.7 earthquake in 2014. Liquefaction should not be a concern as groundwater exceeds 15 meters below the surface.

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

2.4.1 Seismic Parameters	
Site Class:	**C**
MCE[1] spectral response acceleration for 0.2 second period, $\mathbf{S_S}$:	**0.320 g**
MCE[1] spectral response acceleration for 1.0 second period, $\mathbf{S_1}$:	**0.098 g**
Site coefficient, $\mathbf{F_a}$:	**1.3**
Site coefficient, $\mathbf{F_v}$:	**1.5**
MCE[1] spectral response acceleration adjusted for site class, $\mathbf{S_{MS}}$:	**0.417 g**
MCE[1] spectral response acceleration adjusted for site class, $\mathbf{S_{M1}}$:	**0.147 g**
5% Damped spectral response acceleration, $\mathbf{S_{DS}}$:	**0.278 g**
5% Damped spectral response acceleration, $\mathbf{S_{D1}}$:	**0.098 g**
NOTE 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 10 feet or less, and fills 5 feet or less. If cuts and/or fills greater than the amounts assumed are required to reach finished grades, this office should be contacted for additional recommendations.

Analysis of the field and laboratory data indicates that the silty sandy subsoils at the site are generally suitable for support of the proposed structures, subject to some remedial earthwork. Laboratory testing indicates that the native silty sandy soils exhibit a negligible potential for volume increase (swell) due to inundation. The shallow upper fine grained soils were of variable density and are susceptible to additional compression due to inundation. Compressible soils can settle under anticipated structural loads and cause excessive settlement, resulting in damage to the overlying structures. However the very dense basalt boulders that were encountered at depths of 1 to 7 feet below existing grades, will provide sufficient support for the structure, subject to some minor remedial earthworks.

The borings located north of the roadway contained a thin layer of organic peat. This material will not be suitable for reuse as fill. The underlying native silty soils may be reused if they can be selectively stockpiled. Any reuse of existing material will need to be examined to ensure it meets the criteria outlined in Section 4.1.2 Fill and Backfill.

It is understood that the box culvert and wing walls will be placed at an elevation of 7,135 feet, which is approximately 7 feet below the existing grade elevation of 7,142 feet. The road in the area of the culvert will be adjusted and lowered to an elevation that is approximately 2 feet lower than it currently is, to a finish grade elevation of 7,140 feet. Basalt cobbles and boulders are encountered at depths ranging from 1 to 7 feet below the existing grades. While typically considered an ideal bearing media for foundations, placing the box culvert directly onto basalt boulders may negatively affect the box culvert by creating areas of point loading under the culvert. This could result in additional stress and causing cracks in the concrete at the invert and crown. Accordingly, recommendations are made to conduct a limited amount of over-excavation to allow for a uniform thickness of low to non-expansive fill between the basalt boulders and the bottom of the box culvert. The over-excavated material may be selectively stockpiled on-site for reuse as engineered fill between the basalt cobbles and boulders, and the material used as bedding directly below the culvert and wing walls. This will reduce the risk for damage to the culvert by point loading, as well as create a uniform bearing media for the structures, thereby reducing (not eliminating) the risk for differential settlement of the structures. Any organic material (peat) should be removed and can be reused in landscaped areas.

Preparation of the site should be relatively straight forward while removing the upper soils overlying the basalt boulders, but deeper excavations will likely be difficult. The presence of cobbles and boulders at varying depths may interfere with ‘neat’ foundations excavations and result in soil disturbance. This may result in variable amounts of engineered fill needed to bring structures to grade, and/or the need to re-compact soils in the areas disturbed. Excavation difficulties may be encountered in areas where the underlying rock is highly competent or frequent, and will likely require the use of rock excavation/trenching equipment and rock removal techniques.

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, **organic material**, rubble, any undocumented fill, obviously loose surface soils, and any other deleterious materials. Depending on time of year of construction and previous precipitation, the surface soils may be wet and require time to dry and/or stabilization prior to their re-use. If the soils are still wet when construction begins, the exposed grade may become unstable under excavating equipment. Additional time may be required to allow the soils to dry when exposed, or alternative methods such as lime stabilization or rock fill may be required to stabilize the exposed grade.

Special attention will need to be given to the area around the existing culvert, especially in the inlet and outlet areas. Depending on most recent rain events, it would not be uncommon to find several feet of soft wet soil buildup in these areas. Any loose wet soils will need to be 'mucked' out and removed and replaced with engineered fill. If stabilization is required, there are several options available including allowing time to dry out, removal and replacement with additional subbase (usually 2 feet of stockpiled drier soils is sufficient), cement treating (MAG Section 311), or the use of geogrid such as Tensar BX1200 or equivalent with an additional 12 inches of aggregate base. Due to the number of means and methods available, potential for utility conflicts, and other restrictions, there is no one method that may work best.

Complete removal of the existing culvert, including adjacent asphalt surfaces and any aggregate base, will be required. Millings and aggregate base from the existing pavement may be selectively stockpiled for reuse as a subbase layer and as general fill provided they meet MAG/Coconino County Specifications. Cinder roadway fill should not be reused under new asphalt pavements or under precast concrete structures. Depending on the season of construction, it may be possible to encounter perched water at the soil/rock interface, or in the drainage channel. This may require groundwater removal techniques during construction. Care should be taken when selecting groundwater removal techniques to mitigate the risk of subsidence in the surrounding areas, which may affect nearby structures and pavement.

At the time of the investigation, utilities were present around the northern end (inlet) of the existing culvert and under the roadway. If any utility is located within 5 feet of any proposed foundation, relocation and/or abandonment of the utility should be provided. They should either be removed and replaced with engineered fill, or abandoned in place. If left in place, it is critical that utilities are appropriately plugged to prevent water from infiltrating into vault locations. At a minimum, all utilizes must be removed at all footing intersections and backfilled with a lean one sack cementitious slurry. Pipelines larger than 6 inches should be capped and filled with grout to prevent water from infiltrating underneath foundations. If the contractor decides to abandon pipes in-place, the onus should be put on them to demonstrate that the trench backfill is adequately compacted. Speedie & Associates should be notified of the circumstances for our review. If removal of a pipeline is not possible, the foundations should be deepened to bear in undisturbed soil so that the zone of influence under the foundation does not encroach on the pipeline and/or trench. This zone is any area below a 45° line drawn down and away from the bottom of the foundation edges.

Based on conditions observed while drilling, it is likely that the depth at which basalt rock is encountered will be highly variable. It is recommended to over-excavate a minimum of 1.0 foot below the planned bottom-of-culvert and wing wall elevation or deeper if loose conditions are encountered to remove any deeper soft/wet soils, extending at least 3 feet beyond culvert/footing edges and re-compacted as set forth herein. The intent is to over-excavate down to undisturbed dense native soils to allow for sufficient placement of engineered fill between the variable basalt rock and the bedding used directly beneath the culvert and wing walls. Placing the culvert directly onto the basalt rock may cause point loading beneath the culvert, resulting in cracking of the concrete. The amount of engineered fill needed to create a level bearing media for the culvert may vary due to the mix of cobbles and boulders. Additionally, the culvert should be installed so that an appropriate amount of cover is provided to protect the structure from excessive strain, as it will be situated under the roadway. Sufficient width should be planned for any trench excavations for the culvert prior to installation of any structures to allow for proper placement and compaction of subgrade or bedding.

The silty fine sand soils may be sensitive to excessive moisture content and will become unstable at elevated moisture content. Accordingly, it may be necessary to compact soils on the dry side of optimum. The reduced moisture content should only be used upon approval of the engineer in the field. 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.

After removal of the upper soils and 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 100 percent of maximum dry density as determined by ASTM D-698. Pavement areas should be treated in a similar manner.

4.1.2 Fill and Backfill

Native soils classified as silty sand, silty sand with gravel, and silty gravel with sand, are suitable for use in roadway, general grading fills, and as engineered fill under structures. The highly organic "peat" layer present in the upper 6+ inches of soil should not be reused under any structures or roadways. It is recommended that approved native soils be reused as engineered fill between the basalt rocks and the bedding material used to prevent point loading of the culvert. Appropriate bedding materials are typically considered to be Class 2 aggregate base (AB), or cement treated slurry.

Placement of bedding should conform to the requirements of ADOT Special Provisions Subsection 501-3.02, and any Coconino County modifications. It is recommended that a minimum of 1.0 foot of low-expansive engineered fill be placed under the bedding of all structures in addition to AB or cement treated slurry material used as bedding for structural support. A minimum thickness of 6-inches of AB should be used beneath all structures if utilizing aggregate base. Oversized material (> 3inches) in the prepared subgrade should be removed or reduced in size. Any imported material should meet the fill specification as outlined in Table 4.1.2.1.

4.1.2.1 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 or as fill under Coconino County roadways. **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 unstable support for foundations. 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.

4.1.2.2 Compaction Specifications	
Box Culvert/ Wing walls	
Native/Import Fill	100%
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 Wing walls or Box Culvert	100%
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

4.2.1 Shallow Foundations

The following bearing capacities can be utilized for design.

4.2.1.1 Foundation Bearing Capacities

Structure	Foundation Type	Foundation Depth	Bearing Medium	Bearing Capacity	Notes
Box Culvert, Wing Walls	Bedding	1.5 ft	1.0 ft Engineered Fill	3,000 psf	1

Notes:

General –Foundation depth refers to the bottom of footing elevation from the lowest exterior grade within 3.0 feet of the foundation element. All capacities are based on outlined bearing media. The amount of engineered fill indicated are considered minimums.

1. Native silty soils may be removed and recompacted for use as engineered fill below structures.
2. Foundation depths listed are considered a minimum, and may vary during excavation of basalt boulders and bedrock.
3. Box culvert mat design bearing on the bedding material above the dense to very dense basalt cobbles and boulders. A subgrade modulus of k_s = 150 pci (30” plate), with a net bearing of 3,000 psf can be used for design.
4. Design of the box culvert should include evaluation of potential scour and an appropriate scour protection or cutoff walls should be provided.

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.

Estimated settlements under design loads are on the order of ¼ to ½ inch for the wing walls and ½ to ¾ inch for the precast box culvert, 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.

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.

4.4 Utility Installation

Basalt boulders and cobbles were encountered as shallow as 1 foot during the investigation; however, the depth to rock is known to be variable. Trench excavations for shallow utilities installed in the upper silty sandy soils can be accomplished by conventional trenching equipment. Deeper excavations that extend into the underlying boulders or bedrock will require specialized rock trenching equipment. It should be noted that the fact that a boring was advanced to a certain depth should not lead to the assumption that it is necessarily excavatable by conventional means. The excavating contractor must make their own assessment as to excavatability. Sloughing is likely to occur in looser and/or sandier soils, requiring laying back of side slopes and/or temporary shoring. If trenches are greater than shoulderheight, precautions must be taken to protect workmen. 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 Coconino County modifications). Backfill of trenches above bedding 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 4.1.2 Fill and Backfill of this report.

4.5 PAVEMENT

It is understood that the existing roadway will be regraded, lowered in elevation, and then repaved following the installation of the proposed box culvert.

A detailed traffic study was not provided, however it is assumed that this road will provide service for primarily light cars and trucks, and less frequent heavy truck traffic (including possible garbage collection and delivery vehicles). For the purposes of this analysis and lacking detailed traffic information, it is assumed that the proposed road will be classified as Local Street as outlined by the Coconino County Engineering Design and Construction Manual. A minimum section of 4-inches of asphalt concrete on 8 inches of aggregate base is recommended.

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.

Pavement Design Parameters:

Assume:	One 18 kip Equivalent Single Axle Load (ESAL)/Truck
Life:	40 years
Initial Serviceability:	4.5
Terminal Serviceability:	3.0
Level of Reliability (minor collector):	0.75
Seasonal Variation Factor:	3.5

Subgrade Soil Profile:

Average % Passing #200 sieve:	26%
Plasticity Index:	0%
k:	150 pci (assumed)
R value:	70 (per ADOT tables)
M_R:	23,300 psi (per AASHTO design)
R-value	69 at 300 psi (AASHTO T190)

4.5.1 Pavement Sections

Area of Placement	Asphalt Concrete Pavement (Flexible)			Portland Cement Concrete Pavement	
	Thickness, in.		Daily 18 kip	Thickness, in.	Daily 18 kip
	AC	AB	ESALs	PCCP	ESALs
Minor Collector	4.0	8.0	341	6.0	21
	5.0	8.0	801	7.0	43
	5.0	10.0	1237	8.0	88

Notes:

1. Designs are based on AASHTO design equations and ADOT correlated R-Values.
2. The PCCP thickness is increased to provide better load transfer, and reduce potential for joint & edge failures. Design PCCP per ACI 330.
3. Full depth asphalt or increased asphalt thickness can be increased by adding 1.0-inch asphalt for each 3 inches of base course replaced
4. Pavement Design Parameters per guidelines outlined in Coconino County Engineering Design & Construction Manual.
5. Daily 18-kip ESALs based on a 40 year analysis period (based on Coconino County design guidelines).

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 Coconino County modifications) using the Marshall mix design criteria. Reducing the air void content to 3 percent will aid in reducing thermal cracking typical in the area. 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.

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.

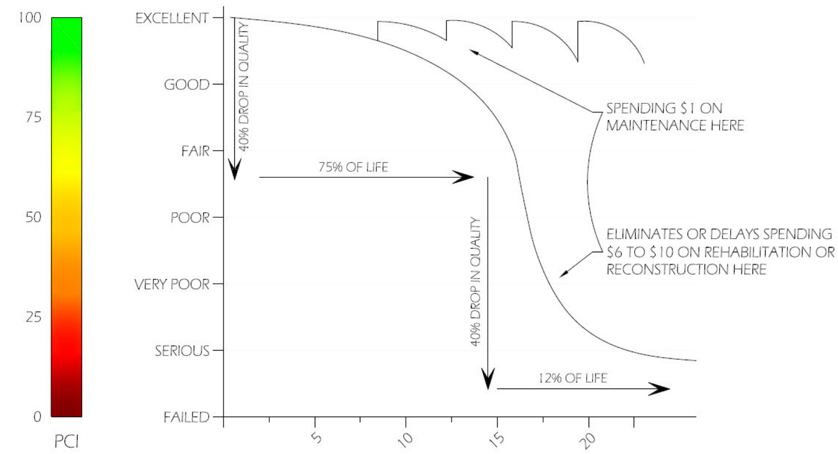
4.5.2 On-Going Pavement Maintenance

In order to achieve an extended life in the pavement it is highly recommended that a maintenance plan be used to address the aging process of the pavement. This will allow for a lower operational cost, while extending the life of the pavement.

It has been well documented that proper pavement maintenance will prolong the life of the pavement at a lower cost than letting the pavement age with no maintenance. Figure 4.5.2.1 shows how spending money on pavement preservation at the correct time will be a significantly more cost effective means for extending the life of the pavement. As long as the pavement remains in a fair to excellent state, the cost of pavement preservation is relatively small, however as the pavement deteriorates into the fair to serious range, the pavement life becomes significantly shorter and there is a change from preservation to pavement rehabilitation and reconstruction (at a significantly higher cost).

Once the pavement enters into this middle zone, light pavement preservation techniques, such as seal coats, crack sealing, etc. become a poor use of funds and the process results in just chasing the distress. This will result in the owner spending high amounts of money just to maintain the current pavement condition. The following figure provides a Pavement Condition Index (PCI) rating, which is a visual condition scale rating from 0 to 100, with 100 being a brand new pavement and 0 being a completely failed pavement. The PCI is a standardized method for rating a pavement based on a visual condition survey of the pavement. The rating procedure uses the pavement distress type and severity to calculate a PCI value for the pavement.

Figure 4.5.2.1 – Life Cycle Cost Analysis



All new asphalt pavements will eventually crack. Cracking in asphalt pavement is typical and should be expected over the life of the pavement. All pavements will age at different rates dues to numerous variables, including factors such as loading condition, environmental conditions, and moisture infiltration or drainage issues. Accordingly, it is highly recommended to establish a maintenance program that addresses this aging process. In general terms it is recommended that the cracks are routinely filled as they appear. Cracking for an overlay will typically result from reflective cracking of the surface below. These cracks will typically reflect through at a rate of approximately 1 year per inch of overlay thickness. This rate will depend on multiple factors, including things such as the mix design, severity of previous cracks, and construction techniques.

In general it should be anticipated that cracks will typically begin to appear around the 2nd to 3rd year of life. So the maintenance program should include a budget item to conduct crack sealing on a routine basis. The amount of crack sealing that is required will depend on how the pavement is aging. Once cracks are sealed, they will typically not need to be sealed for another 4 to 5 years. Therefore the budget estimates should assume that approximately 25 percent of the pavement areas will need isolated crack sealing every year. 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 surface as well as minimize the effects from moisture infiltration. Depending on the progression of the aging, more costly surface treatments such as thin overlays, chip seals, or slurry seals should be anticipated at the 10 to 15 year point of the pavement's life.

4.6 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.

Generally, permanent cut or fill slopes should be no steeper than 2 horizontal to 1 vertical (2:1). 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. Steeper cut slopes in stable rock may be possible (depending of geology), not very likely in soils. Determination of acceptable steeper slope ratios is predicated on a stability analysis of the specific geometry, determinations of soil and groundwater characteristics, structure setbacks, surcharge loads and slope stabilization.

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 horizontal to 1 vertical (5:1), 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.

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, LLC

Conrad Senior E.I.T.

Erin V Piechota

Registered Professional Engineer (Civil)
CERTIFICATE NO.
43813
TODD B.
HANKE
Date Signed 4/11/2023
ARIZONA, U.S.A.

Todd B. Hanke, P.E.

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

APPENDIX

SOIL BORING LOCATION PLAN

SOIL LEGEND

LOG OF TEST BORINGS

TABULATION OF TEST DATA

MOISTURE-DENSITY RELATIONS

SWELL TEST DATA

R-VALUE

— APPROXIMATE EXISTING METAL CULVERT

— APPROXIMATE SOIL BORING LOCATIONS

Drawing Courtesy of Google Earth

N

TMPLT 2000.CAD 10/23/00

SOIL BORING LOCATION PLAN

DR: CS	CHK: CWS	REV:	DATE: 02-20-23	PROJECT NO. 230072SF

Mt. Elden Lookout Road Box Culvert
Mount Elden Lookout Road
Flagstaff, Arizona

SPEEDIE AND ASSOCIATES
Geotechnical • Environmental • Materials Engineers
A UES Company
2026 N. THIRD ST. FLAGSTAFF, ARIZONA 86004

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.

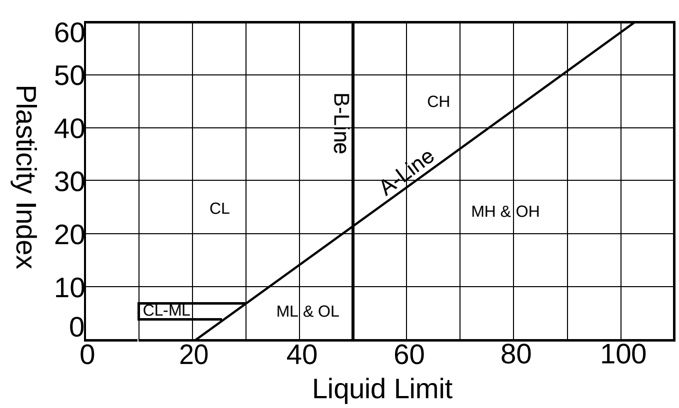
CONSISTENCY			RELATIVE DENSITY	
Clays & Silts	Blows/Foot	Strength (tons/sq ft)	Sands & Gravels	Blows/Foot
Very Soft	0 - 2	0 - 0.25	Very Loose	0 - 4
Soft	2 - 4	0.25 - 0.5	Loose	5 - 10
Firm	5 - 8	0.5 - 1.0	Medium Dense	11 - 30
Stiff	9 - 15	1 - 2	Dense	31 - 50
Very Stiff	16 - 30	2 - 4	Very Dense	> 50
Hard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

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

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

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

◆U.S. Standard ×Clear Square Openings



Depth (feet)	Graphic Log	Rig Type: Kubota SLV75 Boring Type: 12" Auger Surface Elevation: N/A 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		Dark Brown SILTY GRAVEL with SAND (GM-Wet)	3.0	BS-1	3.0	NT	NT	
		Auger Refusal on Basalt Cobbles and Boulders						
5								

Boring Date: **2-20-23**
Field Engineer/Technician: **C. Senior**
Driller: **C. Senior**
Contractor: **Speedie & Assoc.**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES — A UES Company

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

_SPEEDIE 230072SF.GPJ GENGEO.GDT 3/6/23

Depth (feet)	Graphic Log	Rig Type: Kubota SLV75 Boring Type: 12" Auger Surface Elevation: N/A 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		Dark Brown SILTY GRAVEL with SAND (GP/GM-Wet)					
1.0		Auger Refusal on Basalt Cobbles and Boulders					
5							

Boring Date: 2-20-23
Field Engineer/Technician: C. Senior
Driller: C. Senior
Contractor: Speedie & Assoc.

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES A UES Company

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

_SPEEDIE 230072SF.GPJ GENGEO.GDT 3/6/23

Depth (feet)	Graphic Log	Rig Type: Kubota SLV75 Boring Type: 12" Auger Surface Elevation: N/A 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		Red SAND with SILT (SM-Moist) **Cinder Roadway Fill**						
			1.0					
		Brown SILTY GRAVEL with SAND (GM-Moist)		RS-2	1.5	13.9	92.6	
		Wet, Increasing Gravel at 3'						
			4.0	BS-1	4.0	NT	NT	
		Auger Refusal on Basalt Cobbles and Boulders						
5								

Boring Date: **2-20-23**
Field Engineer/Technician: **C. Senior**
Driller: **C. Senior**
Contractor: **Speedie & Assoc.**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

SPEEDIE AND ASSOCIATES A UES Company

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

_SPEEDIE 230072SF.GPJ GENGEO.GDT 3/6/23

Depth (feet)	Graphic Log	Rig Type: Kubota SLV75 Boring Type: 12" Auger Surface Elevation: N/A 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		9.25" ASPHALT OVERLAY (0" AB)	0.8					
		Red <u>SILTY SAND with GRAVEL</u> (SM-Moist) **Fill**	3.0	BS-1	3.0	NT	NT	
5		Brown <u>SILTY SAND</u> (SM-Moist)	6.0	BS-2	6.0	NT	NT	
		Auger Refusal on Basalt Cobbles and Boulders						

Boring Date: **2-20-23**
Field Engineer/Technician: **C. Senior**
Driller: **C. Senior**
Contractor: **Speedie & Assoc.**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

Rig Type:	**Kubota SLV75**
Boring Type:	**12" Auger**
Surface Elevation:	**N/A**

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0–25–50)
0–0.5	Black 6" Peat & Organics (PT-Wet)					
0.5–7.0	Dark Brown SILTY SAND (SM-Moist)	RS-2	2.0	41.5	74.6	
	Wet, Increasing Gravel at 3'	BS-1	5.0	NT	NT	
7.0	Auger Refusal on Basalt Cobbles and Boulders					

Boring Date:	**2-20-23**
Field Engineer/Technician:	**C. Senior**
Driller:	**C. Senior**
Contractor:	**Speedie & Assoc.**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

_SPEEDIE 230072SF.GPJ GENGEO.GDT 3/6/23

Depth (feet)	Graphic Log	Rig Type: Kubota SLV75 Boring Type: 12" Auger Surface Elevation: N/A 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		Black 6" Peat & Organics (PT-Moist)	0.5					
		Dark Brown SILTY SAND with GRAVEL (SM-Moist)						
		Wet, Increasing Gravel at 4'						
5				BS-1	5.0	NT	NT	
		Auger Refusal on Basalt Cobbles and Boulders	6.0					

Boring Date: 2-20-23
Field Engineer/Technician: C. Senior
Driller: C. Senior
Contractor: Speedie & Assoc.

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

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

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

Depth (feet)	Graphic Log	Rig Type: Hand Tools Boring Type: Shovel & Spade Surface Elevation: N/A 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		Black 6" Peat & Organics (PT-Wet)					
0.5		Brown SILTY SAND (SM-Moist)					
2.0			BS-1	2.0	NT	NT	
		Auger Refusal on Basalt Cobbles and Boulders					
5							

Boring Date: **2-20-23**
Field Engineer/Technician: **C. Senior**
Driller: **C. Senior**
Contractor: **Speedie & Assoc.**

Water Level

Depth	Hour	Date
Free Water was Not Encountered		

NT = Not Tested

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Log of Test Boring Number: **B-7**

Mount Elden Lookout Road Box Culvert

3620 W. Schultz Pass Road

Flagstaff, Arizona

Project No.: **230072SF**

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	BS-1	BULK	0.0 - 3.0	NT	NT	15.1	32	43	52	100	NP	NP	NP	GM	SILTY GRAVEL with SAND
B-3	BS-1	BULK	1.0 - 4.0	NT	NT	13.7	33	45	55	100	NP	NP	NP	GM	SILTY GRAVEL with SAND
B-3	RS-2	RING	1.1 - 1.5	13.9	92.6	NT	NT	NT	NT	NT	NT	NT	NT		
B-4	BS-1	BULK	1.0 - 3.0	NT	NT	15.4	37	55	69	100	NP	NP	NP	SM	SILTY SAND with GRAVEL
B-4	BS-2	BULK	3.0 - 6.0	NT	NT	28.6	58	76	86	100	NP	NP	NP	SM	SILTY SAND
B-5	BS-1	BULK	0.0 - 5.0	NT	NT	33.3	68	83	91	100	NP	NP	NP	SM	SILTY SAND
B-5	RS-2	RING	1.0 - 2.0	41.5	74.6	NT	NT	NT	NT	NT	NT	NT	NT		
B-6	BS-1	BULK	0.0 - 5.0	NT	NT	37.9	63	75	84	100	NP	NP	NP	SM	SILTY SAND with GRAVEL
B-7	BS-1	BULK	0.0 - 2.0	NT	NT	38.4	71	84	91	100	NP	NP	NP	SM	SILTY 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

Mount Elden Lookout Road Box Culvert
3620 W. Schultz Pass Road
Flagstaff, Arizona
Project No. 230072SF

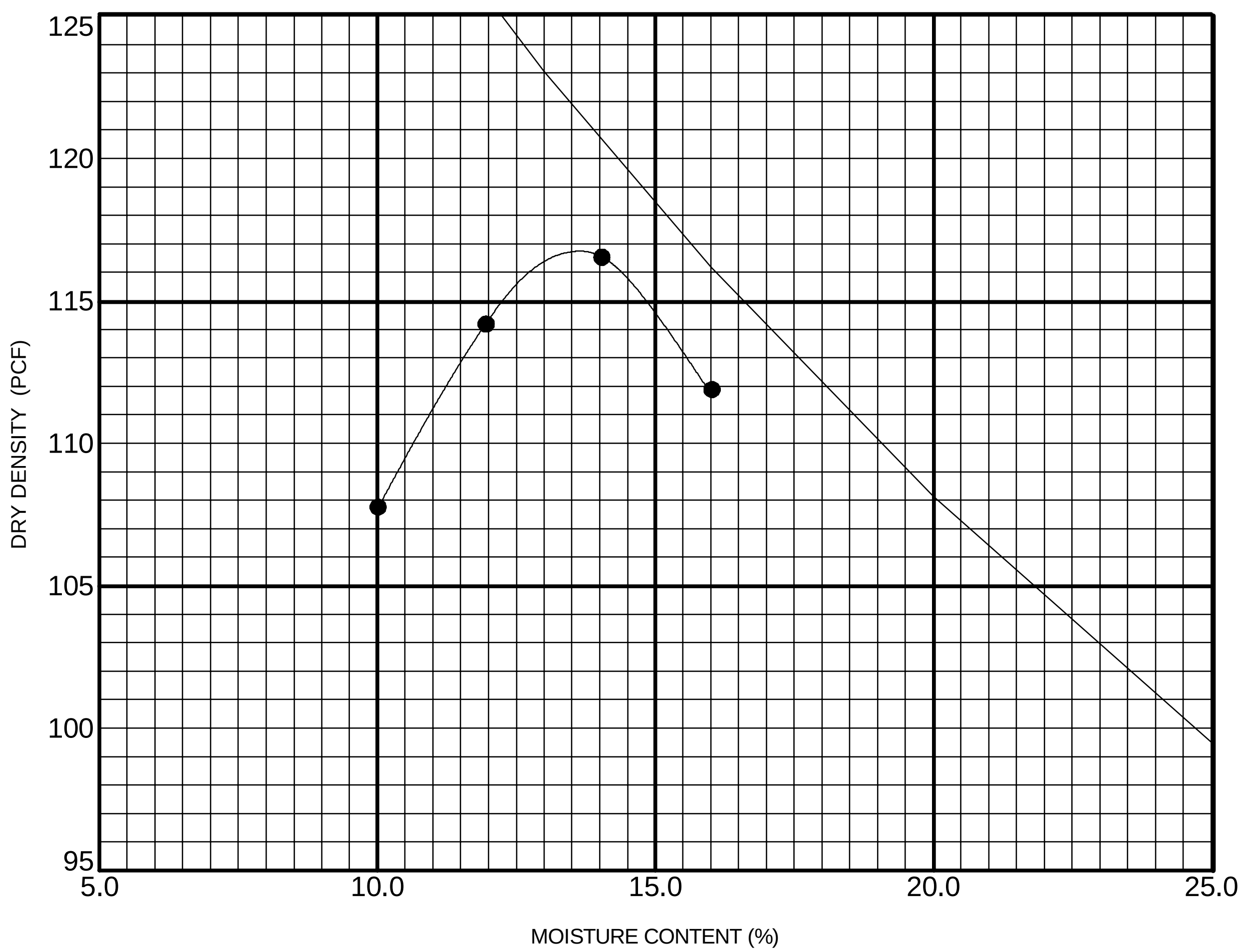
SPEEDIE AND ASSOCIATES

MOISTURE-DENSITY RELATIONS

PROJECT:	Mount Elden Lookout Road Box Culvert				PROJECT NO.: 230072SF
LOCATION:	3620 W. Schultz Pass Road				DATE: 2/20/23
BORING NO.: B-4	SAMPLE NO.: BS-2	SAMPLE DEPTH: 3 to 6			LABORATORY NO.: FVM65
METHOD OF COMPACTION:	D698A				
LIQUID LIMIT:	NP	PLASTIC LIMIT:	NP	PLASTICITY INDEX:	NP
CLASSIFICATION:	SM	ASTM SOIL DESCRIPTION:	SILTY SAND		

MAXIMUM DRY DENSITY: **116.7 PCF**

OPTIMUM MOISTURE CONTENT: **13.6%**



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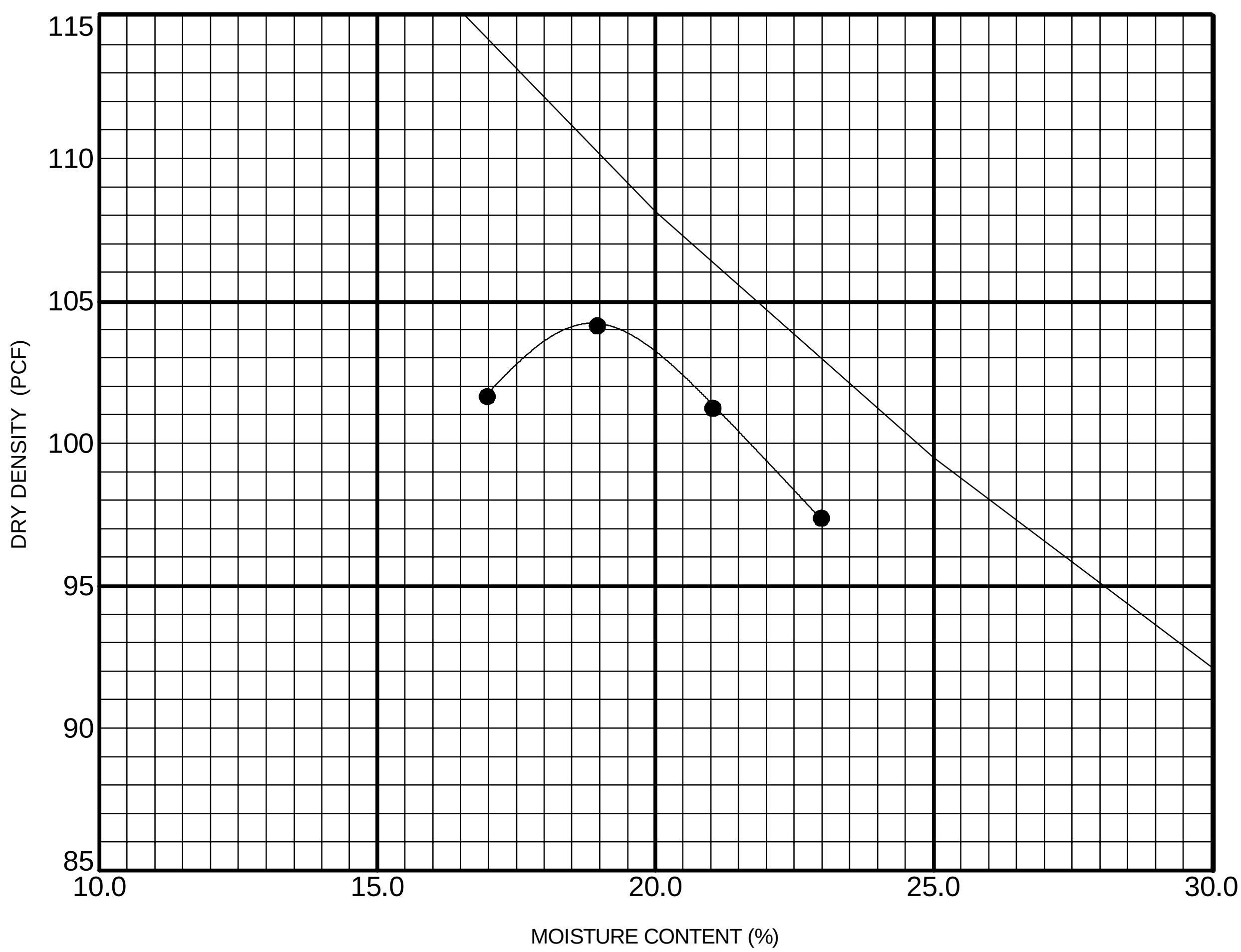
GEOTECH PROCTOR 230072SF.GPJ 3/3/23

MOISTURE-DENSITY RELATIONS

PROJECT: Mount Elden Lookout Road Box Culvert | PROJECT NO.: 230072SF
LOCATION: 3620 W. Schultz Pass Road | DATE: 2/20/23
BORING NO.: B-5 | SAMPLE NO.: BS-1 | SAMPLE DEPTH: 0 to 5 | LABORATORY NO.: FVM67
METHOD OF COMPACTION: D698A
LIQUID LIMIT: NP | PLASTIC LIMIT: NP | PLASTICITY INDEX: NP
CLASSIFICATION: SM | ASTM SOIL DESCRIPTION: SILTY SAND

MAXIMUM DRY DENSITY: **104.3 PCF** OPTIMUM MOISTURE CONTENT: **18.9%**



GEOTECH PROCTOR 230072SF.GPJ 3/3/23

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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-4, BS-2	6.0	116.7	13.6	111.2	11.7	95.3	17	100	0.3
B-5, BS-1	5.0	104.3	18.9	99.7	16.9	95.6	23.7	100	0.4

Mount Elden Lookout Road Box Culvert
3620 W. Schultz Pass Road
Flagstaff, Arizona
Project No. 230072SF

SPEEDIE AND ASSOCIATES

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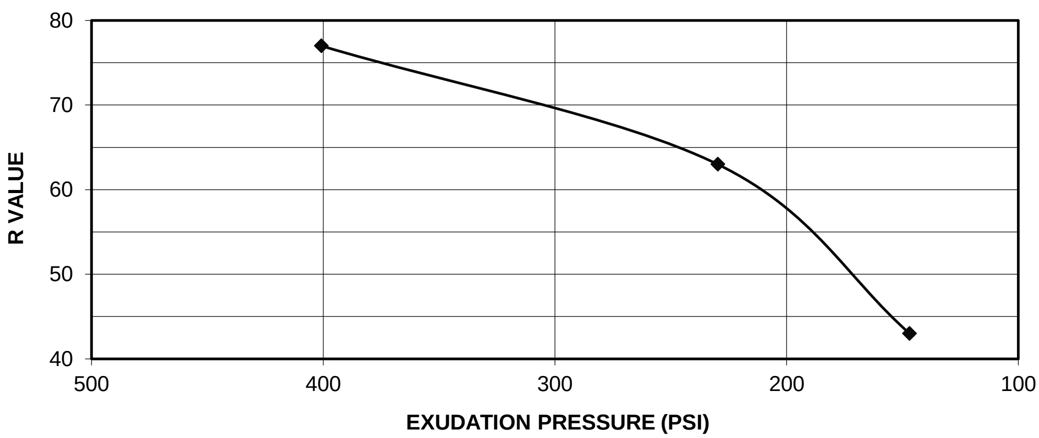
Geotechnical ▪ Environmental ▪ Materials Engineers
3331 E. WOOD ST., PHOENIX, AZ 85040 ▪ O: 602-997-6391 F: 602-943-5508

Resistance R-Value and Expansion Pressure of Compacted Soils

PROJECT:	**Mount Elden Lookout Road Box Culvert**	**JOB NO:**	**230072SF**
LOCATION:	**Mount Elden Lookout Road**	**LAB NO:**	**FVM65**
SAMPLE SOURCE:	**B-4 BS-2 @ 3-6'**	**REPORT DATE:**	**3/15/2023**

R VALUE CALCULATION - AASHTO T190

SPECIMEN I.D.	**A**	**B**	**C**
Moisture content	11.6%	12.7%	13.5%
Compaction Pressure (psi)	350	350	250
Specimen Height (inches)	2.44	2.41	2.44
Dry Density (pcf)	127.3	127.9	125.4
Ph @ 1000 (lb)	14.0	22.0	32.0
Ph @ 2000 (lb)	23.0	41.0	60.0
Displacement	4.13	3.96	5.09
Expansion Pressure (psi)	0.0	0.0	0.0
Exudation Pressure (psi)	401	230	147
R Value	78	65	43
Corrected R Value	77	63	43



R VALUE AT 300 PSI = 69