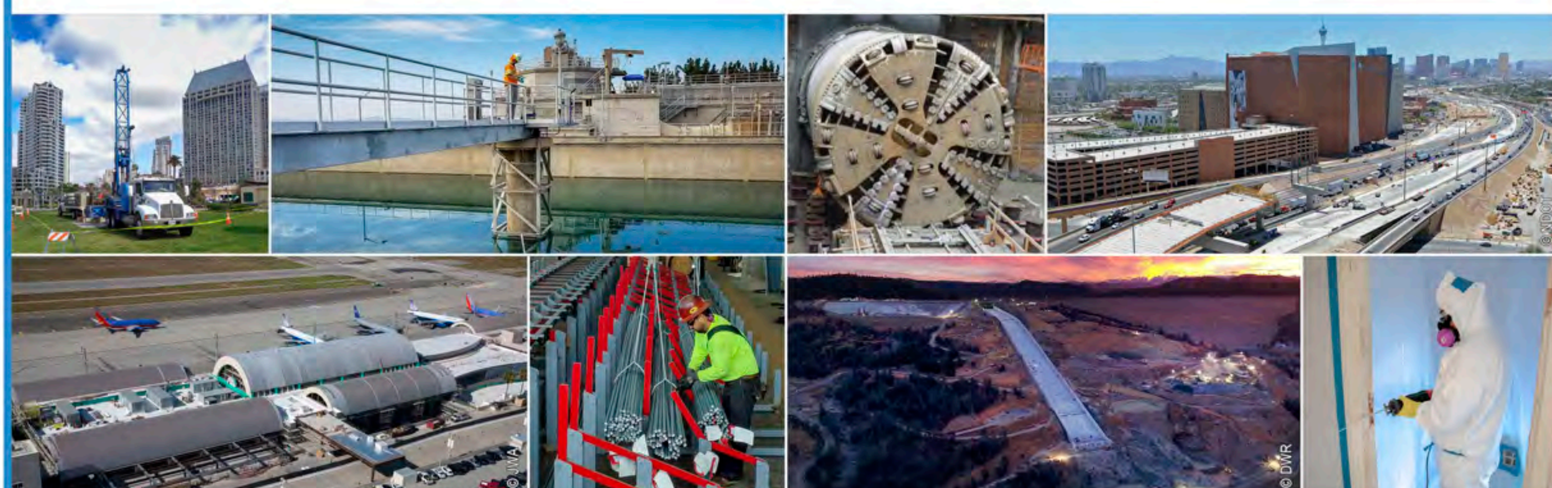
Geotechnical Evaluation

Carefree Highway Improvements
Cave Creek Road to Scottsdale Road

Scottsdale, Arizona

TY Lin
1475 North Scottsdale Road | Scottsdale, Arizona 85257

April 5, 2023 | Project No. 607272001



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Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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April 5, 2023
Project No. 607272001

Mr. Carlos Sanchez-Soria, PE
TY Lin
1475 North Scottsdale Road
Scottsdale, Arizona 85257

Subject: Geotechnical Evaluation
Carefree Highway Improvements
Cave Creek Road to Scottsdale Road
Scottsdale, Arizona

Dear Mr. Sanchez-Soria:

In accordance with our proposal dated February 10, 2022, and your authorization, Ninyo & Moore has performed a preliminary geotechnical evaluation for the above-referenced site. The attached report presents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

Stephen V. Hargus, PE
Senior Engineer

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Managing Principal Engineer

SVH/SDN/tlp

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1. INTRODUCTION

In accordance with your request, Ninyo & Moore has conducted a geotechnical evaluation in support of the planned Carefree Highway improvements between Cave Creek Road and Scottsdale Road in Scottsdale, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site and provide geotechnical recommendations for design and construction. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

Our scope of services for the project included the following:

- Reviewing, as needed, available published and in-house geotechnical reports, topographic information, soil surveys, geologic literature, and aerial photographs of the project area.
- Obtaining a permit from City of Scottsdale to access their right-of-way prior to our field work.
- Conducting a site visit to conduct a geologic reconnaissance and selecting proposed exploration locations.
- Contacting Arizona 811 to evaluate utility locations prior to our field work.
- Coring the existing pavement at four locations using an electronic core machine. The pavement core samples were collected from each location and returned to our laboratory to be measured and photo-documented.
- Drilling, logging, and sampling 10 exploratory soil boring extending to depths between approximately 8.8 and 25 feet below ground surface (bgs).
- Collecting soil samples in the borings at 2.5- to 5.0-foot intervals using ASTM International (ASTM) Methods D1586 (Standard Penetration Test) with split-barrel sampling of soils) and D3550 (ring-lined barrel sampling of soils) for laboratory testing and analysis.
- Collecting four near-surface soil samples within the existing wash along the south side of the Carefree Highway alignment using hand operated equipment in order to evaluate the soil's erosive and sediment transport characteristics.
- Performing laboratory testing to evaluate the on-site soil's in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, and chemical testing to evaluate corrosivity (including soil pH, resistivity, sulfate, and chloride).
- Preparing this report presenting our findings, conclusions, and recommendations regarding the design and construction of the project.

Our scope of services did not include environmental consulting services such as hazardous waste sampling or analytical testing at the site. A detailed scope of services and estimated fee for such services can be provided upon request.

3. SITE DESCRIPTION

The project site is located along Carefree Highway between Cave Creek Road and Scottsdale Road in Scottsdale, Arizona, a distance of about 1.8 miles (Figure 1). The site is currently a two-lane, asphalt concrete (AC) paved roadway with a center turn lane over much of the alignment. At both the Cave Creek Road and Scottsdale Road intersections are commercial developments. The Carefree Highway alignment is bounded to the north and south by low-density residential developments, interspersed with native desert. Black Mountain is located approximately one-quarter mile to the north of the alignment. Several natural drainage features cross, or run parallel to, the Carefree Highway alignment. Many of these features are uncontrolled and allow flows to pass over the Carefree Highway AC pavement.

According to the Cave Creek, Arizona 7.5-Minute United States Geological Survey Topographic Quadrangle Maps (2021), the Carefree Highway alignment is at an average elevation 2,151 feet relative to mean sea level (MSL) between Cave Creek Road and Scottdale Road. The roadway generally slopes downward from east to west with the intersection at Scottsdale Road at approximately 2,261 feet relative to MSL and the intersection at Cave Creek at approximately 2,041 feet relative to MSL

4. PROJECT DESCRIPTION

This project will widen Carefree Highway to a four-lane arterial street. Project features will include a raised landscaped median, increased travel lane capacity, safety improvements at key intersections, and a new shared use path/trail on the south side of Carefree Highway. In addition, the project will consider a roundabout at 60th Street. This project will also include the design of drainage improvements including box culverts and/or bridges at key natural wash crossing and the installation of fiber optic/ITS conduits that are assumed to extend less than 5 feet deep.

5. REVIEW OF AERIAL PHOTOGRAPHS

Aerial photographs of the project site from Maricopa County (2023) dated between 1953 through 2021 were reviewed for this project. In 1953, the Carefree Highway alignment was established but the roadway was unpaved. By 1976, the roadway was paved and some residential development began near the northeast portion of the project area. By 1991, residential development north of the alignment, at the base of Black Mountain, had started. Residential develop continued for the next few years with significant development begin constructed south of the alignment by 1996. In general, the development just north and south of the alignment has not changed much since 1998.

6. FIELD EXPLORATION AND LABORATORY TESTING

The following sections summarize our field exploration and laboratory testing activities.

6.1. Existing Pavement Condition Survey

Ninyo & Moore conducted a visual pavement condition survey to evaluate the current severity of distresses along the Carefree Highway between Cave Creek Road and Scottsdale Road. The survey was conducted in conjunction with other field activities. Our pavement condition survey findings are as follows:

- Medium to high severity block cracking was observed along much of the roadway.
- Medium to high severity alligator cracking was observed, mainly in areas where natural washes crossed the roadway. These areas typically had poor drainage which allow water to pool after storm events.
- Medium severity patching due to repairs or utility installation were prevalent along much of the roadway.
- Potholes and areas where delamination of previous AC overlays were observed in isolated areas along the roadway.

In general, the destresses seem to be a combination of aging AC pavement, increased traffic loading, poor draining and/or poor subgrade conditions. Select photographs are provided in Appendix A.

6.2. Asphaltic Concrete Cores

Ninyo & Moore collected four AC cores from the roadway (denoted as C-1 through C-4) on March 7, 2023. The approximate core locations are shown on Figures 2A through 2D. The cores were collected using an electronic core machine in order to accurately measure the existing AC and aggregate base (AB) thickness. These cores were photographed. The core photographs and their measured thicknesses are provided in Appendix A.

6.3. Soil Borings

Ninyo & Moore conducted a subsurface exploration of the site on February 23rd and March 13, 2023. The site exploration was needed to evaluate the subsurface conditions and to collect soil samples for laboratory testing. Our exploration consisted of drilling, logging, and sampling 10 exploratory soil borings denoted as B-1 through B-10. The approximate boring locations are shown on Figures 2A through 2D. The borings were drilled using a CME-55 and

CME-75 truck-mounted drill rig equipped with hollow-stem augers. Our borings were drilled to depths between 8.8 and 25 feet bgs. Logs of the borings are included in Appendix B.

Soil samples were collected at selected intervals and were logged in general accordance with the ASTM D2488. Disturbed soil samples were collected during standard penetration testing using a split-spoon sampler. Relatively undisturbed soil samples were collected at regular intervals by using modified ring-lined split tube samplers. Bulk samples were also collected from the HSA cuttings and placed in large plastic bags. The selected intervals at which the bulk soil samples were collected are provided on the boring logs. Descriptions of the soils encountered in our borings are presented on the boring logs.

6.4. Hand Sampling

Ninyo & Moore collected near-surface hand samples along the wash and existing side slope banks (denoted as H-1 through H-4) at four locations during our field operations on February 24, 2023. The approximate hand sample locations are shown on Figures 2C and 2D. Bulk samples were collected using hand tools and placed in large plastic bags. These samples were transported to our laboratory for testing to aide in sediment transport and scour analysis to be conducted by others. The results of the laboratory tests are included in Appendix C.

6.5. Laboratory Testing

Selected samples were visually classified and tested in our laboratory to evaluate their engineering properties as a basis for providing geotechnical recommendations for design and construction considerations. Our geotechnical laboratory testing included in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, and chemical testing to evaluate corrosivity (including soil pH, resistivity, sulfate, and chloride). The results of the geotechnical laboratory tests are included in Appendix C.

7. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections describe the site geology and subsurface conditions.

7.1. Geologic Setting

The project site is located in the Sonoran Desert Section of the Basin and Range physiographic province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 18 million years ago during the mid- to late-Tertiary. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains.

The surficial geology at the site consists of unconsolidated to weakly consolidated alluvial fan, terrace, and basin-floor deposits with moderate to strong soil development. Fan and terrace deposits are primarily poorly sorted, moderately bedded gravel and sand, and basin-floor deposits are primarily sand, silt, and clay. (Richard, SM, et al., 2000).

7.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on the results of our exploratory borings and our understanding of the general geology of the area. The boring logs contain our field test results, as well as our interpretation of the conditions between actual samples retrieved. Therefore, the boring logs contain both factual and interpretive information. Lines delineating subsurface strata on the boring logs are intended to group soils having similar engineering properties and characteristics. They should be considered approximate, as the actual transition between soil types may be gradual. Detailed stratigraphic information as well as a key to the soil symbols and terms used on the boring logs is provided in Appendix B.

7.2.1. Asphalt Concrete / Aggregate Base

Our borings were not advanced through the pavement. However, four cores were collected to evaluate the pavement section. The pavement section at these core locations consists of an AC surface overlaying an aggregate base (AB) material. The AC pavement thickness ranged between 3.5 and 5 inches. The AB thickness ranged between 6 and 9 inches based on measurements taken in our core holes along the roadway.

7.2.2. Fill

Fill soils were encountered at the surface of some of our borings (B-7 and B-8). The fill soils consisted of medium to very dense coarse-grained soils. The fill soils were characterized as clayey sand (SC) with varying amounts of gravel in our borings.

7.2.3. Alluvium

Native alluvial soils were encountered at the surface or below the fill soil layer within our borings. The alluvium generally consisted of medium dense to dense coarse-grained soils. The alluvial soils were characterized as clayey sand (SC), silty clayey sand (SC-SM) silty sand (SM), sand (SP or SW), and sand with silt (SP-SM or SW-SM) with varying amounts of gravel in our borings. Caliche nodules were encounter in several of our borings. Cobbles were encounter in borings located in natural washes (B-3 and B-9).

7.3. Groundwater

Groundwater was not encountered in our borings at the time of drilling. Well data from the Arizona Department of Water Resources (ADWR) Groundwater Site Inventory database (2023) indicates that regional groundwater has been historically encountered at greater than 150 feet bgs within one mile of the site. However, seasonal variations could cause fluctuations in the surrounding groundwater depths. Perched water tables may be encountered, especially within the natural drainages or after flood events.

8. GEOLOGIC HAZARDS

The following sections describe regional geologic hazards, including land subsidence, earth fissures, faults, and expansive soils.

8.1. Land Subsidence and Earth Fissures

Based on our field reconnaissance and information accessed at the Arizona Geologic Survey website (2023), the site is not located in an area with documented earth fissures. The closest documented earth fissures are approximately 15 miles to the southeast of the site. The project site is also not within an area with a measured land subsidence based on information accessed at the ADWR e-Library (2023).

8.2. Faulting and Seismicity

The site lies within the Sonoran zone, which is a relatively stable tectonic region located in southwestern Arizona, southeastern California, southern Nevada, and northern Mexico (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations, review of pertinent geologic data, and analysis of aerial photographs, Quaternary faults are not located on or adjacent to the property.

The closest know Quaternary fault to the site is the Carefree Fault Zone, located approximately 6 miles to the east of the site (Pearthree, 1998). The fault is a low, fairly well defined, west- to southwest-facing fault scarps as much as 3 meters high formed on Precambrian granite and possibly on Quaternary deposits. Seismic design considerations are provided in Section 10.2.

9. CONCLUSIONS

Based on the results of our subsurface evaluation and our review of available geotechnical data, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations of this report are incorporated into the design and construction of the proposed project, as appropriate. Geotechnical considerations include the following:

- Based on our visual pavement condition survey and subsurface exploration, the distresses observed in the AC pavement are likely due to a combination of aging AC pavement, increased traffic loading, poor draining and/or poor subgrade conditions. Given the number of patches/repairs, relatively thin AC pavement thickness, and variable AB thickness, pavement preservation options are limited and should not be considered for this project.

- The near surface site soils can generally be excavated or ripped using heavy-duty earthmoving or excavation equipment. However; dense soils with gravel, caliche nodules and cobbles were encountered in our borings, which may be more difficult to excavate and/or will slow the rate of excavation.

- Imported soils, and soils generated from on-site excavation activities, that exhibit a relatively low plasticity index (PI) can generally be used for engineered fill. Based on the results of our study, many of the near surface on-site soils are suitable for re-use as engineered fill.

- The proposed structures can be founded on spread footings or mat foundations. Spread footings and mat foundations should be founded on a zone of adequately moisture-conditioned and compacted engineered fill.

- New pavements and concrete flatwork should be founded on a zone of adequately moisture-conditioned and compacted engineered fill.

- An earthwork (shrinkage) factor of approximately 10 to 15 percent is estimated if the on-site soils are re-used as fill.

- Groundwater was not observed in our boring. Based on ADWR well data, the regional groundwater table has been historically measured at depths on the order of 150 feet bgs. In general, groundwater is not expected to be a constraint to the design and construction of this project. However, perched water tables may be encountered, especially within the natural drainages or after flood events.

- No documented geologic hazards are present underlying or immediately adjacent to the site.

- Corrosivity test results indicate that on-site soils are corrosive to ferrous materials and the sulfate content of the soils presents a negligible sulfate exposure to concrete.

10. RECOMMENDATIONS

The following sections present our geotechnical recommendations and were developed based on our understanding of the proposed construction (Section 4), the observed subsurface conditions (Section 7), and our experience. Given the project location, recommendations and guidelines outlined by the Maricopa Association of Governments (MAG), and/or the City of Scottsdale supplement should be used unless recommended differently herein. If the proposed construction is changed from that discussed herein or subsurface conditions other than those shown on the boring logs (Appendix A) are observed at the time of construction, Ninyo & Moore should be retained to conduct a review of the new information and to evaluate the need for additional recommendations.

10.1. Earthwork

The following sections provide our earthwork recommendations for this project. If the site grade is planned to change by more than 3 feet vertically, Ninyo & Moore should be contacted for additional recommendations.

10.1.1. Pre-Construction Survey

Prior to construction activities, it may be desirable to recognize the condition of the existing utilities, underground structures, or other features that are near the planned construction and to survey or document (e.g., photographs, video, official documentation, etc.) their pre-construction condition. The findings of the survey could be used to document any damage that might result from this project.

10.1.2. Site Preparation

If earthwork activities are needed, vegetation, unsuitable materials, or debris from the clearing operation should be removed from the site and disposed of or placed in non-structural areas (e.g., landscaping). Obstructions that extend below finish grade, if present, should be removed and the resulting voids filled with moisture-conditioned and compacted engineered fill.

After rough grade has been achieved and prior to further earthwork, the exposed subgrade should be proof-rolled and visually observed for the presence of debris, organic matter and other unsuitable materials. If unsuitable soils are encountered at subgrade level during earthwork operations, these soils should be removed to their full depth, and be replaced with engineered fill.

The geotechnical consultant should carefully evaluate any areas of loose, soft, or wet soils prior to placement of fill or other construction. Drying or over-excavation of some materials may be appropriate.

10.1.3. Subgrade Preparation

We recommend that the new shallow foundations (with anticipated loading not in excess of 2,500 pounds per square foot [psf]) and mat foundations (with anticipated loading not in excess of 2,000 psf) be supported on a zone of engineered fill that extends 2 feet below the bottom of the foundations. The engineered fill should be placed as discussed in this report. This overexcavation zone should extend a horizontal distance from the edge of the new foundation that is equal to the depth of the overexcavation.

In addition, we recommend that the flatwork and pavements be supported on 8 inches of moisture-conditioned and compacted engineered fill. This can be achieved by in-place scarification and re-compaction. The fill thickness should be measured from the bottom of the AB layer, where applicable. This subgrade improvement should extend laterally 1 foot beyond the slab/pavement footprint.

Furthermore, engineered fill to be used as new roadway subgrade should meet the design R-value of 35 as recommended in this report. We recommend that the construction control R-value be not less than the design R-value of 35.

After the overexcavation described above is finished and prior to the placement of engineered fill, exposed surfaces from excavations should be carefully evaluated by the geotechnical consultant for the presence of soft, loose, or wet soils that were not removed as part of the improvement process. This evaluation should consist of probing and visual observation of the excavation bottom. Based on this evaluation, additional remediation may be needed. This could include further scarification, moisture-conditioning and compaction of the exposed surface. This additional remediation, if needed, should be addressed by the geotechnical consultant during the earthwork operations.

10.1.4. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, site observations, and experience with similar materials. Excavation of the materials can generally be accomplished with heavy-duty earthmoving equipment. However; dense soils with gravel, caliche nodules and cobbles were encountered in our borings, which may be more difficult to excavate and/or will slow the rate of excavation.

Sidewalls for temporary excavations should not be anticipated to stand near-vertical without sloughing. Therefore, the sides of excavations and trenches for this project should be stabilized in order to reduce damage to adjacent facilities resulting from vertical or lateral movement of the soil. The sides of the excavation may be stabilized by sloping back the sides and/or by using bracing. However, the trench sidewalls may be difficult to stabilize due to the presence of low cohesion soils, which could have a potential to caving and sloughing during excavation, especially if the soils are wet or saturated. Additionally, vibrations caused by nearby traffic or construction equipment could accelerate sloughing.

10.1.5. Temporary Slopes

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system in compliance with OSHA Regulations for employees working in an excavation that may expose them to the danger of moving ground. Based on the soil conditions at the site, we recommend that OSHA Soil "Type C" classification be used for excavations at the site. This corresponds to temporary slopes of 1.5:1 (horizontal: vertical). This side slope is for excavations that are less than 20 feet deep. If material is stored or equipment is operated near an excavation, stronger shoring should be used to resist the extra pressure due to superimposed loads. Excavations over 20 feet should be designed by the contractor's engineer based on alignment-specific geotechnical analysis.

Upon making the excavations, soil and/or rock classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with OSHA standards.

10.1.6. Permanent Slopes

Permanent cut slopes and constructed embankment fill slopes should be no steeper than 2:1 (horizontal to vertical). New embankment fills should be benched into existing embankments, where appropriate. Benches should be level and wide enough to allow operation of and compaction by, construction equipment. Fill slopes should be constructed in a manner (e.g., overfilling and cutting to grade) such that the recommended degree of compaction is achieved to the finished slope face. Cut and fill slopes should be protected from erosion. This should promote re-vegetation and a stable slope. Periodic maintenance of exposed slopes should be anticipated.

Unprotected slopes may rill and erode if exposed to running water. Silty soils and soils containing fine sand are more susceptible in this regard. While 2:1 (horizontal to vertical) slopes are acceptable from a stability standpoint, laying slopes back to 3:1 (horizontal to vertical) will decrease runoff velocity and decrease the likelihood of serious erosion. Steeper slopes will need additional maintenance. Adequate drainage and temporary erosion protection covering could minimize erosion problems and promote post-construction vegetation. Plating the slopes with gravelly material or riprap will reduce the impacts of precipitation and slow the rate of erosion. If riprap is placed in the channel it should be adequately sized to prevent erosion of the embankment. Along longer slopes, brow ditches should be considered to reduce the amount of surface flow on the slope face. Where feasible, the existing vegetation should be salvaged and replaced.

10.1.7. Temporary Shoring

In some instances, it may be preferable to temporarily brace or shore the excavations rather than using open cuts to the base of the excavations. Temporary earth retaining systems will be subjected to lateral loads resulting from earth pressures. Shored excavations may be designed using the parameters on Figure 3.

The earth pressure values presented on Figure 3 assume that spoils from the excavation or other surcharge loads will not be placed above the excavation within a 1:1 (H:V) plane extending up and back from the base of the excavation. If spoil piles are placed closer than this to the braced or shored excavation, the resulting surcharge loads should be considered in the bracing or shoring design. We recommend that an experienced structural engineer design the bracing or shoring system. The bracing and shoring parameters presented in this report should be considered as guidelines.

Trench boxes may also be a suitable alternative to laying back the side walls; however, due to the presence of granular soils, the excavations may not stand open long enough to install the trench boxes. The contractor should be prepared to deal with these soil conditions and plan accordingly. Once installed, some sloughing is possible at the ends of the trench box; therefore, any loose material should be removed prior to backfilling of the trench.

10.1.8. Engineered Fill

On-site and imported soils that exhibit low plasticity indices and very low expansive potential are suitable for use as engineered fill. Low plasticity indices, as evaluated by ASTM D4318, are defined as a PI of 15 or less for this project. Low expansive potential soils are defined as having a swell potential of less than 1.5 percent when evaluated in accordance with ASTM D4546 with a load of 144 psf.

In addition, suitable fill should not include construction debris, organic material, or other non-soil fill materials. Clay lumps and rock particles should not be larger than 4 inches in dimension within the upper 5 feet of finished grade. Unsuitable fill material should be disposed of off-site or in non-structural areas.

Imported fill, if used, should consist of soils with a relatively low PI (15 or less). Import material in contact with ferrous metals should preferably have low corrosion potential (minimum resistivity more than 2,000 ohm-cm, chloride content less than 25 parts per million). In lieu of this, corrosion protection techniques (e.g., cathodic protection, pipe wrapping, etc.) can be implemented. A corrosion specialist should be consulted for recommendations of an appropriate corrosion protection technique. Imported material in contact with concrete should have a soluble sulfate content of less than 0.1 percent. The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

10.1.9. Re-use of On-Site Soils

The Atterberg limits tests performed on soil samples obtained from the borings resulted in PI values ranging between non-plastic (0) and 12. Based on our test results, many of the on-site soils are considered suitable for re-use as engineered fill. However, the effects of corrosion will need to be considered in design. Additional field sampling and laboratory testing should be conducted by the contractor either prior to or during construction to better screen for any unsuitable materials.

10.1.10. Engineered Fill Placement and Compaction

Engineered fill as described in this report should be moisture-conditioned within the moisture range shown below in Table 1 and mechanically compacted to the percentage shown. Engineered fill should generally be placed in 8-inch-thick loose lifts such that each lift is firm and non-yielding under the weight of construction equipment.

Engineered fill used to raise grade will settle a portion of its height due to its own weight prior to construction of the roadway. The magnitude of this settlement will depend on the type of fill used. In general, the engineered fill recommended in this report is expected to settle about 1 percent of its height. If needed, areas where fill will be used to raise the roadway prism should be scarified to a depth of 8-inches, moisture conditioned and compacted in accordance with Table 1 prior to fill placement. Areas where the roadway prism is already at grade should be scarified to a depth of 8-inches, moisture conditioned and compacted in accordance with Table 1 prior to the placement of AB.

Table 1 – Compaction Recommendations

Fill Description	Minimum Compaction per ASTM D698	Moisture Content
Native surfaces prior to new roadway embankment placement	95 percent	±3 percent of optimum
Roadway embankment and Engineered fill	95 percent	±2 percent of optimum
Pavement subgrade	95 percent	±2 percent of optimum
AB course	100 percent	±3 percent of optimum

An earthwork (shrinkage) factor of 10 to 15 percent is estimated for the onsite soils. This shrinkage factor range represents an average of the material tested and assumes that materials excavated from the site will be placed as fill. Potential bidders should consider this in preparing estimates and should review the available data to make their own conclusions regarding excavation conditions.

10.1.11. Site Drainage

Surface drainage should be provided to divert water away from the structures and off of paved surfaces. Surface water should not be permitted to drain toward the structures or to pond on flatwork or pavement areas. Positive drainage for this project is defined as a slope of 2 or more percent for a distance of 5 or more feet away from the roadway surface, roadway embankment, or structures.

10.2. Seismic Design Parameters

The current International Building Code (IBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal PGA that corresponds to the MCE_R for the site was calculated as 0.102g. The MCE_G PGA, based on the geometric mean PGA with a 2 percent probability of exceedance in 50 years, with adjustment for site class effects (PGA_M) was calculated as 0.154g. Seismic design parameters were calculated using the ATC Hazards by Location website (2023) seismic design tool.

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 2 presents the seismic design parameters for the site in accordance with IBC guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the ATC Hazards by Location website (2023) seismic design tool. The soil properties in the upper 100 feet of the site are not known in sufficient detail to justify selecting a Site Class C or better. Therefore, the default Site Class D should be used for this site. For structural design, the following seismic parameters should be used:

Table 2 – IBC Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.4
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.229g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.070g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.367g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.167g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.245g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.112g

10.3. Foundations

Based on the results of the field and laboratory evaluations, it is our opinion that proposed box culvert structures can be founded mat foundations with thickened sections. Small retaining walls, wing walls, or other small structures can be founded on shallow spread or continuous footings. Recommendations for these foundation systems are presented in the following sections of this report.

10.3.1. Mat Foundations

A mat foundation may be utilized for proposed box culvert and should bear at a depth of 24 inches or more below the adjacent finished grade, on a layer of compacted engineered fill, as described in this report. Mat foundations should be reinforced in accordance with the recommendations of the structural engineer.

Mat foundations founded on engineered fill may be designed using a net allowable bearing capacity of 2,000 psf for static conditions. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Total and differential settlements beneath mat slab foundations will vary depending on slab configuration and applied load. A modulus of subgrade reaction of 350 kips per cubic foot (kcf) may be used for design of mats supported on compacted engineered fill to estimate the settlement. This value is an estimate based on published typical ranges, results of laboratory testing performed on samples taken from our borings, and our experience with similar materials. If more refined estimates are needed, field plate load testing should be performed in the areas proposed for the new mats.

Mat foundations bearing on compacted engineered fill and subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.40 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load).

10.3.2. Shallow Foundations

Spread or continuous footings, if utilized, should be supported at a depth of 18 inches or more below the adjacent finished grade. The footings should be supported on engineered fill, as described in this report. Continuous footings should have a width of 18 or more inches, and isolated spread footings should have a width of 24 or more inches. Spread or continuous footings should be reinforced in accordance with the recommendations of the structural engineer.

Based on the available soil boring information, spread footings associated with the perimeter walls, and light structures supported on engineered fill may be designed using a net allowable bearing capacity of 2,500 psf for static conditions.

Total and differential settlement of up to about 1 inch and ½ inch respectively, may occur.

These settlement estimates are based on the assumption that the foundations act as isolated foundations, that is, the clear spacing between the foundation elements are the width of the largest adjacent foundation or more, and the settlement associated with fill soils has already occurred.

Foundations bearing on moisture-conditioned, compacted engineered fill that are subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.40 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load). A passive resistance value of 360 psf per foot of depth for drained conditions. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided that the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces. The foundations should preferably be proportioned such that the resultant force from lateral loadings falls within the kern (i.e., middle one-third).

10.4. Retaining Walls

Retaining walls should be supported on shallow spread footings as discussed in this report. Wall backfill should consist of free-draining granular material and should be accompanied by weepholes through the wall or corrugated, perforated pipe placed parallel to the wall or abutment bottom, wrapped in a filter fabric, and surrounded by 6 inches or more of granular filter material (e.g., pea gravel). In lieu of the wrapped open-graded gravel, a geocomposite drainage mat attached to the wall and discharging to a drain pipe or weepholes may be considered. Retaining wall should be designed using the applicable lateral earth pressures. The earth pressures provided assume level backfill at the top of the wall.

10.4.1. Active Conditions

Active earth pressure occurs when the wall moves away from the soils and the soil mass stretches horizontally, sufficient to mobilize its shear strength, and a condition of plastic equilibrium is reached. For drained, level backfill, an equivalent fluid active earth pressure of 40 psf per foot (psf/ft) of wall height should be used for design of cantilevered, yielding walls. An outward lateral movement of about 0.001H (where H is the height of the wall) at the top of the wall is generally needed to mobilized the active earth pressure condition.

Unrestrained retaining walls should also be designed to resist a horizontal earth pressure of 0.33q. The value for “q” represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

10.4.2. At Rest Conditions

A soil mass that is neither stretched nor compressed is said to be in an at-rest state. If the wall is rigidly restrained, so that it does not rotate sufficiently to reach the active earth pressure condition, at-rest earth pressure condition will exist. An equivalent fluid at-rest earth pressure of 60 psf/ft for drained level backfill should be used.

Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.50q. The value for "q" represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

10.4.3. Passive Conditions

Passive earth pressure occurs when the wall or foundation moves into the soil and the soil mass is compressed horizontally, mobilizing its shear strength. For below-grade portions of the walls with granular backfill (derived from on-site soils) in front of the toe of the wall, an ultimate equivalent fluid passive earth pressure of 360 psf/ft can be used for drained conditions. This value assumes that the ground is horizontal for a distance of 10 feet or more in front of the wall or three times the height generating the passive pressure, whichever is more. We recommend that the upper 12 inches of soil not protected by pavement or a concrete slab, or any soil subject to possible future scour or excavation, be neglected when

10.5. Pavement Design Summary

The pavement sections recommended in this report are based on the pavement design procedures outlined in the *ADOT Pavement Design Manual* (2017) and referred to as the Arizona Department of Transportation (ADOT) Manual in the follow sections. The following section provides information on estimated traffic loading, subgrade support, and pavement design parameters.

10.5.1. Traffic Loading

Ninyo & Moore reviewed traffic counts provided by TY Lin as part of the *Project Assessment Report* (2023), that were collected in 2022, to develop a basis for the estimated traffic loading over the design life of the pavement. Traffic counts were collected for both eastbound and westbound traffic along Carefree Highway just east of Cave Creek Road and just west of Scottsdale Road over a 24-hour period. The traffic along Carefree Highway ranged between 15,546 and 16,975 vehicles per day. While not specifically stated in the traffic data, this information has been assumed to be similar to an Average Annual Daily Traffic (AADT) volume. An AADT of 16,975 vehicles per day was selected for our design. A growth rate of 3 percent was assumed.

The traffic data along the project segment contains about 17 percent trucks with the AADT for Single Trucks being 2,735 trucks and Combo Trucks being 226 trucks. Combo Trucks represent 8 percent of the daily truck traffic and were used to determine the overall distribution of the class of trucks. A Truck Traffic Classification of AZ-6 was selected from Table A-1 in the ADOT Manual based on the percent of Combo Trucks.

Based on the traffic data, a directional distribution factor (D_D) of 51 percent, or 0.51 was determined. The configuration of the roadway will generally remain two lanes in each direction. Therefore, a lane distribution factor (D_L) of 90 percent, or 0.90 was selected from Table A-2 in the ADOT Manual.

The Roadway Functional Classification for Carefree Highway is a Principal Arterial with respect to Equivalent Single Axle Load (ESAL) calculations and is located in an urban area based on Figure A-2 in the ADOT Manual. Based on this classification, Table A-4 in the ADOT Manual recommends Cluster Number 2 to be use for determining the truck load factors.

These inputs, along with an assumed build year of 2023, were used to calculate the anticipated ESALs over a 20-year design life. The resulting ESALs were calculated to be 9,221,302. The full traffic calculations are provided in Appendix D.

10.5.2. Design R-Values

Table 3 summarizes the correlated R-values measured on soil samples obtained within the upper 5 feet of the existing site grades from various borings within the project limits:

Table 3 – R-Value Summary

Boring No.	Sample Depth (ft.)	R_c[1]
B-1	0.0-5.0	79
B-3	0.0-5.0	90
B-5	0.0-5.0	92
B-6	3.5-5.0	85
B-7	0.0-5.0	47
B-8	3.5-5.0	59
B-9	0.0-5.0	98
B-10	3.5-5.0	87
		R_c Mean = 72

Note:
[1]Correlated R-value (R_c).

The mean correlated R-value summarized in Table 3 is 72, however, to account for variability that may be found along the roadway alignment as well as the potential need for import materials an R-value of 35 was used for design.

10.5.3. Resilient Modulus

A design R-value of 35 was utilized in our analysis of the new pavement section to be constructed over prepared subgrade. A seasonal variation factor (SVF) of 1.0 was selected for the design from Figure 2-1 in the ADOT Manual. Using these inputs, a resilient modulus of 21,050 pounds per square inch was calculated.

10.5.4. Drainage Coefficient

A drainage coefficient of 1.0 was selected for use in our design from Table 2-7 in the ADOT Manual based on the anticipated quality of drainage and the SVF.

10.5.5. Standard Deviation and Level of Reliability

ADOT recommends a combine standard error value (S_0) of 0.45 for flexible pavement design. Carefree Highway is characterized as a "Non-Divided, Non-Interstate Highway, 10,000+ ADT" with respect of pavement design and is needed to meet a level of reliability of 95 percent. A standard normal deviate (Z_R) value of -1.645 was selected from Table 2-1 in the ADOT Manual and utilized in our design.

10.5.6. Serviceability Index

As a "Non-Divided, Non-Interstate Highway, 10,000+ ADT", the Carefree Highway pavement is designed for a change in the present serviceability index (ΔPSI) of 1.4. Table 2-2 in the ADOT Manual recommends an initial serviceability (P_o) value of 4.2 and a terminal serviceability (P_t) value of 2.8.

10.5.7. Flexible Pavement - Structural Coefficients and Minimum Thicknesses

Table 2-6 in the ADOT Manual provides a list of structural coefficients based on material type. For our design, a structural coefficient of 0.44 was used for AC and a structural coefficient of 0.14 was used for AB.

Table 2-10 in the ADOT Manual provides minimum structural numbers (SNs) and minimum AC thicknesses base on roadway functional classifications. As a "Non-Divided, Non-Interstate Highway, 10,000+ ADT", the Carefree Highway pavement should have a SN of 3.50 or more with a minimum AC thickness of 5.0 inches.

10.5.8. Alternative Flexible Design– Geogrid Design Inputs

As an alternative, a bi-axial geogrid can be utilized to mechanically stabilize the AB layer. A bi-axial geogrid placed between a properly moisture conditioned and compacted subgrade and the AB layer can effectively increase the structural coefficient of the AB as much as 60 percent. For this alternative, Ninyo & Moore utilized a structural coefficient of 0.22 to evaluate an alternative pavement section using geogrid.

10.5.9. New Flexible Pavement Sections

Utilizing a design R-Value of 35 and the anticipated traffic over 20 years, three equivalent alternative asphalt pavement sections were developed for Carefree Highway between Cave Creek Road and Scottsdale Road. The sections are provided in Table 4:

Table 4 – Calculated Pavement Sections

Roadway Segment	Subgrade Improvement	AC (in)	AB (in)	Pavement Section Thickness (in)	SN
Carefree Highway Alternative No. 1	Engineered Fill[1]	6.0	8.0	14.0	3.76
Carefree Highway Alternative No. 2	Engineered Fill[1]	5.0	12.0	17.0	3.88
Carefree Highway Alternative No. 3	Engineered Fill with a geogrid[1]	5.0	8.0	13.0	3.96

Note:
[1] Includes minimum of 8-inches of improvement.

The pavement sections presented in Table 4 are based on the expected traffic and the existing / improved subgrade soil conditions. The full design life of 20 years is expected to be achieved with these pavement sections with periodic maintenance to include possible overlays, and as long as good drainage is provided and maintained. In Arizona, pavement sections often become brittle and experience cracking before the design life is attained, therefore, the asphalt surface needs to be maintained and sealed periodically over its life. If the subgrade soils experience a significant increase in moisture content, accelerated pavement deterioration and increased maintenance should be anticipated. Pavement and base course materials should not be placed when the subgrade is wet. Good surface drainage should be provided away from the edges of paved areas to minimize lateral moisture transmission into the subgrade soils.

10.5.10. Structural Number Check

In accordance with the procedure for flexible pavement design noted in the ADOT Manual, and using the above-mentioned parameters, we have checked that the SN for the proposed AC roadway sections are adequate, as presented below.

Table 5 – SN Summary

Roadway Segment	Minimum Required SN[1]	Calculated SN of New Pavement
Carefree Highway Alternative No. 1	3.75	3.76
Carefree Highway Alternative No. 2	3.75	3.88
Carefree Highway Alternative No. 3	3.75	3.96

Note:
[1] For 8 inches of improved subgrade, Mr = 12,155 psi.

10.5.11. Flexible Pavement Preservation

Given the nature of the pavement distresses presented as part of the pavement condition survey, along with the inconsistent AC and AB thickness measurements collected during our field exploration, a pavement preservation alternative would provide little benefit to the project. It is Ninyo & Moore's recommendation that pavements associated with this project consist of full-depth removal and replacement.

10.5.12. Rigid Pavement Design

In addition to flexible pavement, a rigid pavement design alternative was considered. The same traffic loading, level of service, and serviceability index were utilized to evaluate a Portland cement concrete pavement (PCCP). ADOT recommends a combine standard error value (S_0) of 0.25 for rigid pavement design.

In addition, the ADOT Manual recommends an average modulus of rupture for PCCP of 670 pounds per square inch (psi). An assumed average concrete compressive strength of 5,000 psi which yields a modulus of elasticity for PCCP of 4,000,000 psi.

Based on Table 2-8, a load transfer coefficient (J) of 3.9 was assigned for plain-jointed concrete without dowels.

A modulus of subgrade reaction (k) value of 1,085 pounds per cubic inch (pci) was determined based on the design resilient modulus of 21,050.

From Table 2-9 in the ADOT manual, a loss of soil support value of 0 was utilized based on the SVF of 1.0 and base material of AB. Using the loss of soil support value and resilient modulus values, a corrected, effective modulus of subgrade reaction (k) of 1,085 pci was determined.

Based on these inputs, Ninyo & Moore recommends a PCCP be not less than 10 inches thick. The PCCP should be support on a minimum of 4 inches of compacted AB material placed over compacted subgrade material.

10.6. Concrete Flatwork

Concrete flatwork should be supported on a zone of moisture-conditioned and compacted engineered fill as described in this report to reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil. We further recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer.

10.7. Corrosion

The corrosion potential of the on-site materials was tested to evaluate its potential effect on the foundations and structures. Our corrosion evaluation of the on-site soils is based on the results of our laboratory testing done for this project. A corrosion specialist should perform their own analysis.

Laboratory testing consisted of pH, minimum electrical resistivity, and chloride and soluble sulfate contents. The pH and minimum electrical resistivity tests were performed in general accordance with Arizona Test 236c, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 736, respectively. The results of these corrosivity tests are presented in Appendix C.

The soil pH values of the selected samples tested from our borings ranged between 7.5 and 9.5, which is considered to be alkaline. The minimum electrical resistivity of the samples tested ranged between 804 and 4,087 ohm-cm, which is considered corrosive to ferrous materials. The chloride content of the samples tested ranged between 7 and 178 ppm, which also indicates a corrosive environment for ferrous materials. The soluble sulfate content of the soil samples tested was 0.0003 to 0.001 percent by weight of soil, which is considered to represent negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are corrosive to ferrous materials. To reduce the corrosion potential of buried metallic utilities, we recommend that topsoil, organic soils, soils, and mixtures of sand and clay not be placed adjacent to buried metallic utilities. Rather, we suggest that sand or gravel be placed around buried metal piping. Also, buried utilities of different metallic construction or operating temperatures should be electrically isolated from each other to minimize galvanic corrosion problems. In addition, new piping should be electrically isolated from old piping, if any, so that the old metal will not increase the corrosion rate of the new metal. A corrosion specialist should be consulted for further recommendations.

10.8. Concrete

Laboratory chemical tests performed on on-site soil samples indicated sulfate contents was 0.0003 to 0.001 percent by dry weight of soil. Based on American Concrete Institute (ACI) criteria (Table 6), the on-site soils should be considered to present a negligible sulfate exposure to concrete.

Due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and regional practice, we recommend that "Type II" cement be used for the construction of concrete structures at this site. Due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, pozzolan or admixtures designed to increase sulfate resistance may be considered. Calcium chloride should not be used as an admixture in concrete.

Table 6 – ACI Requirements for Concrete Exposed to Sulfate-Containing Soil

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Cement Type	Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	f'c, Normal-Weight and Lightweight Aggregate Concrete, psi x 0.00689 for MPa
Negligible	0.00 - 0.10	--	--	--
Moderate[2]	0.10 - 0.20	II, IP(MS), IS (MS)	0.50 or less	4,000 or more
Severe	0.20 - 2.00	V	0.45 or less	4,500 or more
Very severe	Over 2.00	V plus pozzolan[3]	0.45 or less	4,500 or more

Notes:

1 A lower water-cementitious materials ratio or higher strength may be needed for low permeability or for protection against corrosion of embedded items or freezing and thawing (ACI Table 4.2.2).

2 Seawater.

3 Pozzolan that has been evaluated by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

We recommend that the structural concrete have a water-cementitious materials ratio no more than 0.50 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. Higher strength concrete may be selected for increased durability and resistance to slab curling and shrinkage cracking.

10.9. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect, or if the project characteristics are significantly changed.

10.10. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of over-excavation, to evaluate the suitability of the on-site materials for use as fill and to observe placement and test compaction of fill soils. If another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

11. LIMITATIONS

The field evaluation, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

12. REFERENCES

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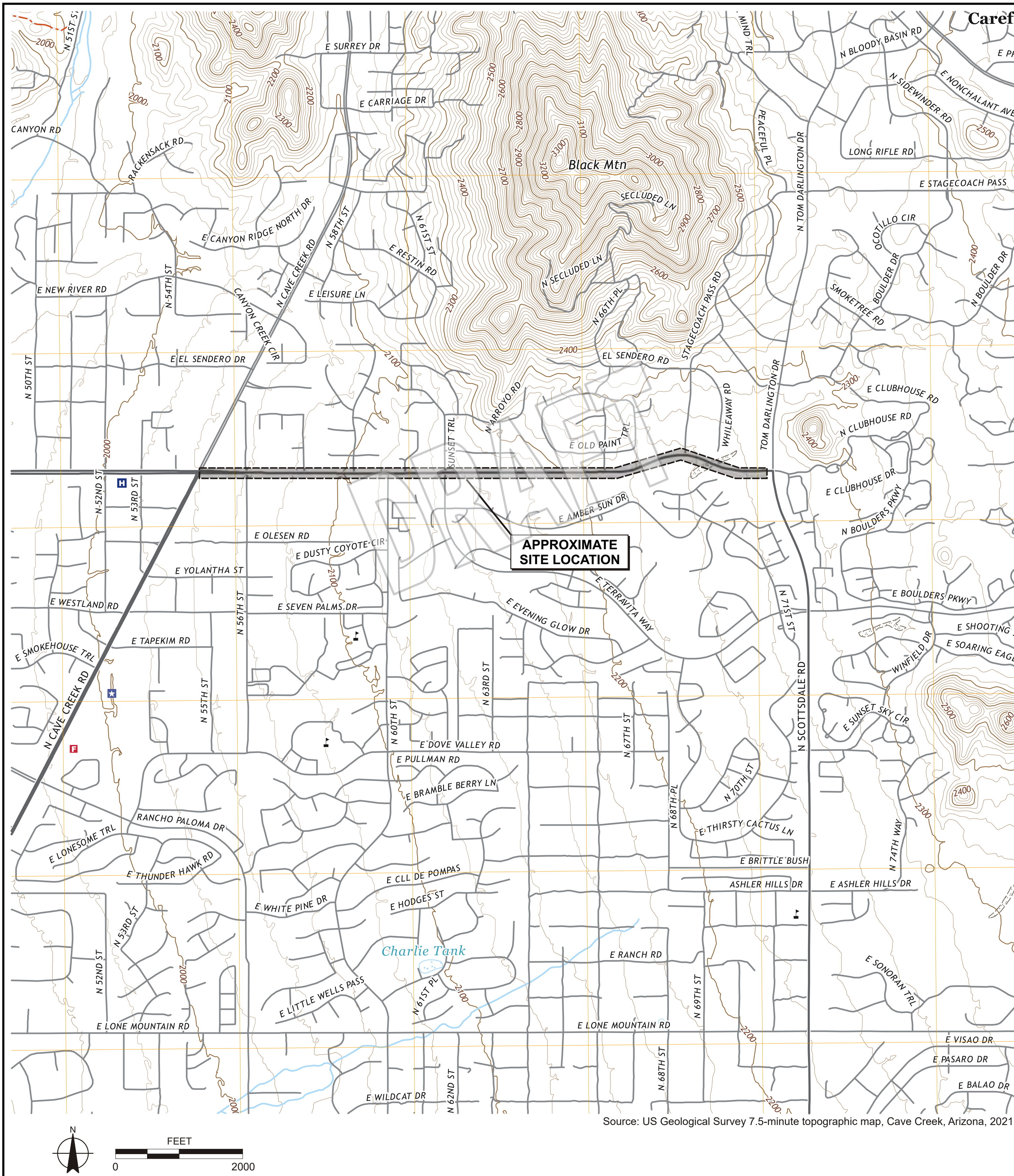
FIGURES



Source: US Geological Survey 7.5-minute topographic map, Cave Creek, Arizona, 2021.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 1

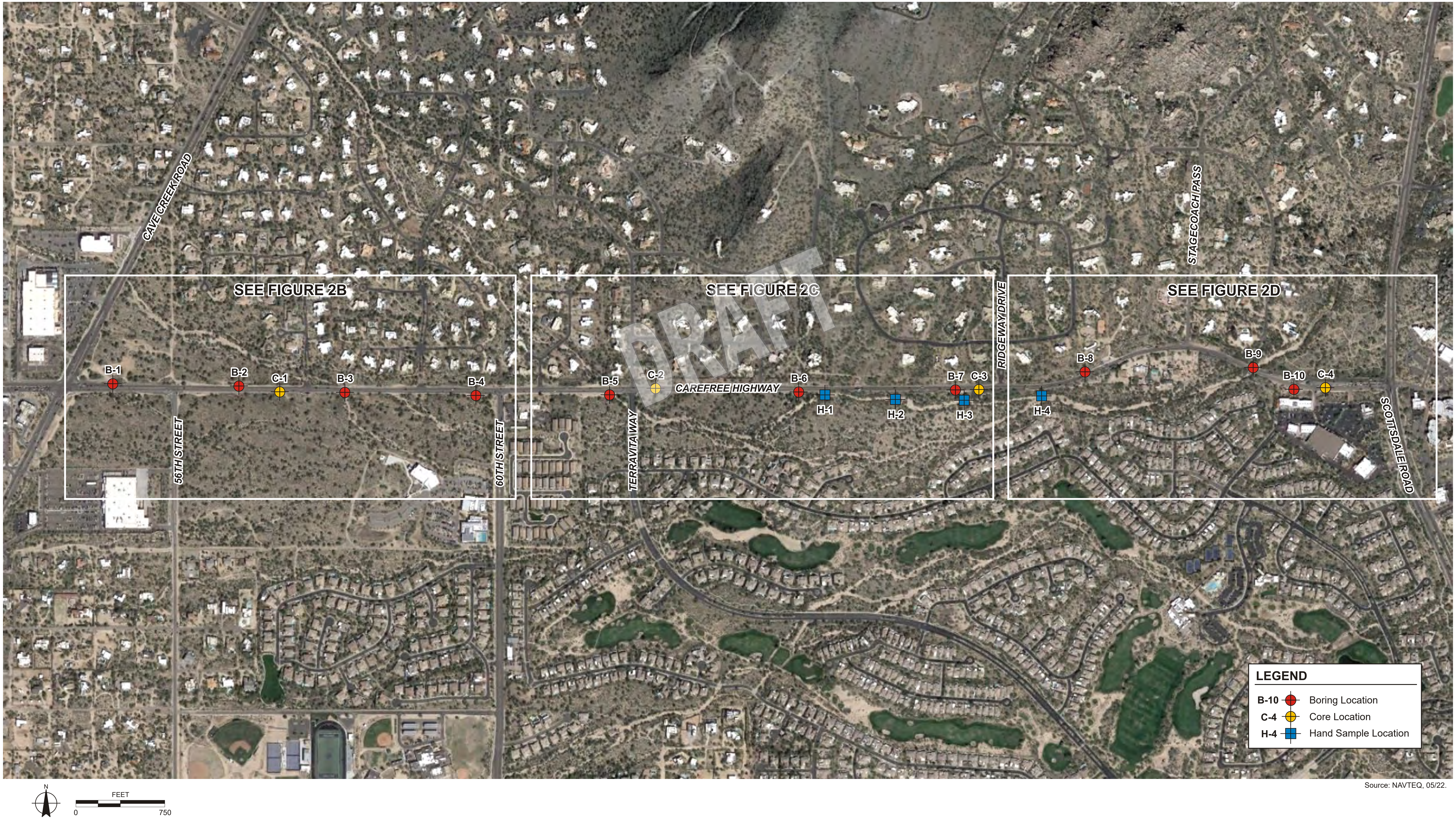
SITE LOCATION

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

bsm file no: 7272vmap0123

607272001 | 4/23



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 7272bim0123

FIGURE 2A

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OVERVIEW OF EXPLORATION LOCATIONS

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

607272001 | 4/23

Source: NAVTEQ, 05/22.

N

FEET

0 400

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 7272blm0123b

FIGURE 2B

BORING B-1 TO BORING B-4

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 7272blm0123c

FIGURE 2C

BORING B-5 TO CORE LOCATION C-3

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

607272001 | 4/23

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Source: NAVTEQ, 05/22.

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 7272blm0123d

FIGURE 2D

HAND SAMPLE H-4 TO CORE LOCATION C-4

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

Ninyo & Moore
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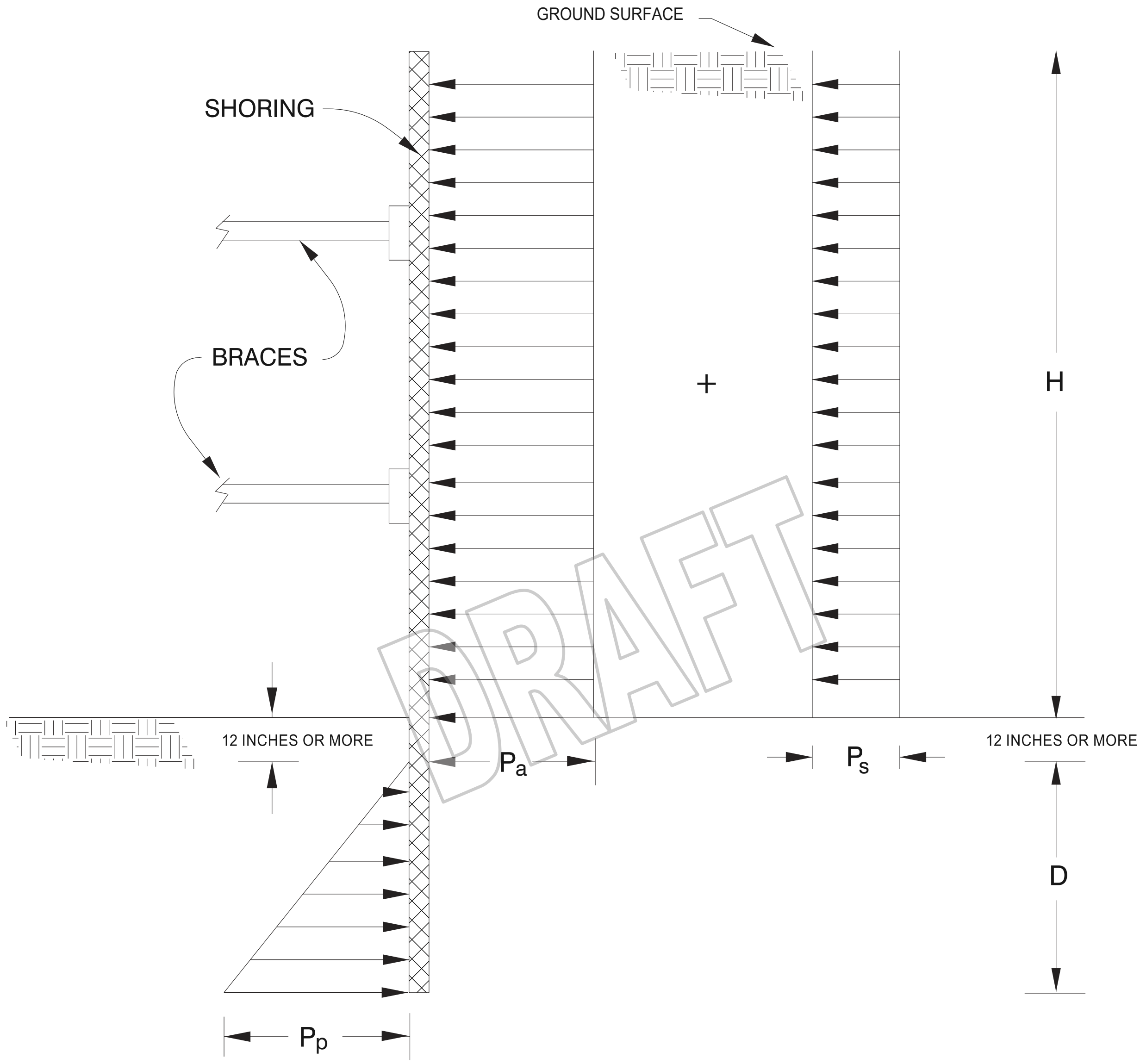
607272001 | 4/23



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   $P_a = 30H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   $P_p = 300D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 3

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)

CAREFREE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

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APPENDIX A

Photographs

Photograph 1: **Typical block cracking.**

Photograph 2: **Typical block cracking.**

Photograph 3: **Example of delamination of previous overlay**

Photograph 4: **Example of alligator cracking and patch in poor drainage area.**

Photograph 5: **Example of poor drainage.**

Photograph 6: **Example of utility trench patch.**

PHOTOGRAPHS

CAREFREE HIGHWAY – CAVE CREEK ROAD TO SCOTTSDALE ROAD
SCOTTSDALE, ARIZONA

Photograph 7: **Core Location C-1, asphaltic concrete thickness approximately 4.5 inches. Approximately 6 inches of aggregate base at C-1**

Photograph 8: **Core Location C-2, asphaltic concrete thickness approximately 5 inches. Approximately 6.5 inches of aggregate base at C-2.**

Photograph 9: Core Location C-3, asphaltic concrete thickness approximately 5 inches. Approximately 9 inches of aggregate base at C-3.

Photograph 10: Core Location C-4, asphaltic concrete thickness approximately 3.5 inches. Approximately 7.5 inches of aggregate base at C-4

APPENDIX B

Boring Logs

APPENDIX B

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples consisting of auger cuttings of representative earth materials were obtained from selected exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a SPT sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven up to 18 inches into the ground with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel (California) Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D1586. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

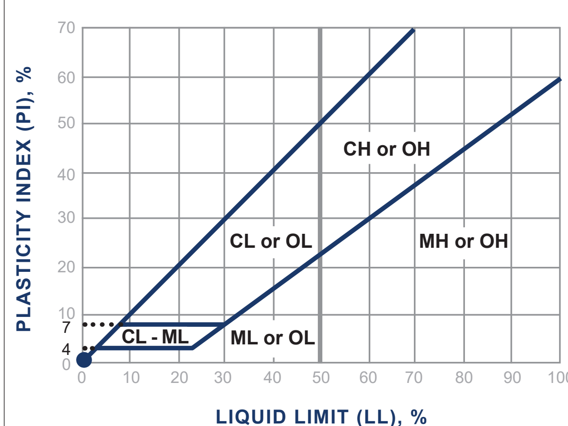
Soil Classification Chart Per ASTM D 2488

Primary Divisions		Secondary Divisions	Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

DATE DRILLED 3/13/23 BORING NO. B-1

GROUND ELEVATION 2,053' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Wildcat)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, dry to moist, loose, silty SAND with gravel.
			11	6.2	113.1			
			16					Medium dense.
5								
			34					Trace clay.
			15					
10								Total Depth = 10 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
20								

DATE DRILLED	3/13/23	BORING NO.	B-2
GROUND ELEVATION	2,076' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Wildcat)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	SDN		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	ALLUVIUM: Brown, dry, medium dense, silty SAND; few fine gravel.
	Driven	13					
	Driven	13	2.6	117.1			Loose.
5							
	Driven	6					Trace fine gravel
	Driven	50/3"					Very dense.
10							Total Depth = 8.8 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling.
15							
20							

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SW-SM	ALLUVIUM: Brown, dry, medium dense, well graded SAND with silt and gravel.
			19					
5			18					Trace coarse gravel.
			35	4.4	120.2			
			83				SM	Brown, dry, very dense, silty SAND; numerous caliche nodules; weakly cemented.
10								
15			77/11"	1.7	121.2			
			50/5"					Possible cobbles.
20								

DATE DRILLED 2/23/23 BORING NO. B-3

GROUND ELEVATION 2,090' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							SM	ALLUVIUM: (Continued) Brown, dry, very dense, silty SAND.
		■	50/4"					
								Total Depth = 23.8 feet. Groundwater not encountered during drilling. Backfilled on 2/23/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
25								
30								
35								
40								

DATE DRILLED 2/23/23 BORING NO. B-3

GROUND ELEVATION 2,090' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/23/23 BORING NO. B-4 GROUND ELEVATION 2,115' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN **DESCRIPTION/INTERPRETATION**
0							SM	ALLUVIUM: Brown, dry, medium dense, silty SAND; trace clay.
			9					
			15	6.5	107.9			
5								
			25					Dense; few fine gravel.
			50/5"					
10								Total Depth = 9.4 feet. Groundwater not encountered during drilling. Backfilled on 2/23/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED	2/23/23	BORING NO.	B-5
GROUND ELEVATION	2,138' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	SDN		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SW-SM	ALLUVIUM: Brown, moist, loose, well graded SAND with silt and gravel.
		50/1"					No ring recovery.
		3					
5							
		11	5.5	112.6		SM	Brown, moist, loose, silty SAND; few fine gravel.
		16					Medium dense.
10							
		9					Loose.
15							
		11				SC	Brown, moist, medium dense, clayey SAND.
20							

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							SC	ALLUVIUM: (Continued) Brown, moist, medium dense, clayey SAND.
		■	50/3"					
25								Total Depth = 24.3 feet. Groundwater not encountered during drilling. Backfilled on 2/23/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED 2/23/23 BORING NO. B-5

GROUND ELEVATION 2,138' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DATE DRILLED	2/23/23	BORING NO.	B-6				
GROUND ELEVATION	2,179' ± (MSL)	SHEET	1	OF	2		
METHOD OF DRILLING	CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)						
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"				
SAMPLED BY	DM	LOGGED BY	DM	REVIEWED BY	SDN		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, moist, loose, silty SAND with gravel.
			12					No recovery; possible coarse gravel.
			12					
5								
			13					Medium dense; trace coarse gravel.
			90/11"					Very dense.
10								
							SM	Brown, dry, very dense, silty SAND; few fine gravel; scattered caliche nodules.
			36					
15								
			90/11"					
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/23/23 BORING NO. B-6 GROUND ELEVATION 2,179' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN DESCRIPTION/INTERPRETATION
20							SM	ALLUVIUM: (Continued) Brown, dry, very dense, silty SAND; few fine gravel; scattered caliche nodules.
		■	33					
25								Total Depth = 25 feet. Groundwater not encountered during drilling. Backfilled on 2/23/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
35								
40								

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
								DATE DRILLED 3/13/23 BORING NO. B-7 GROUND ELEVATION 2,205' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Wildcat) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN
0							SC	FILL: Brown, dry, medium dense, clayey SAND with gravel.
			35	4.8	118.0			
			26					Dense.
5								
			47					
			53					Very dense; scattered caliche nodules.
10								Total Depth = 10 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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20								

DATE DRILLED	3/13/23	BORING NO.	B-8
GROUND ELEVATION	2,225' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Wildcat)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	SDN		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, dry, medium dense, clayey SAND; few fine gravel.
			10					
							SC-SM	ALLUVIUM: Brown, dry to moist, medium dense, silty gravel sand with gravel.
			30					
5								
							SC	Brown, dry to moist, medium dense, clayey SAND; trace fine gravel.
			11					
			50/3"					Very dense.
								Total Depth = 8.8 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling.
10								
15								
20								

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED 3/13/23 BORING NO. B-9

GROUND ELEVATION 2,245' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Wildcat)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SW	ALLUVIUM: Brown, moist, loose, well graded SAND with gravel.
			13					
			3					
5								
			50/5"	15.4	109.7		SC	Brown, moist, very dense, clayey SAND; few coarse gravel.
			63/10"					
10								
			50/2"					No recovery; coarse gravel; cobbles.
15								
			50/2"				GP	Brown, moist, very dense, poorly graded GRAVEL with sand.
20								

DATE DRILLED	3/13/23	BORING NO.	B-9
GROUND ELEVATION	2,245' ± (MSL)	SHEET	2 OF 2
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Wildcat)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	SDN		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							GP	ALLUVIUM: (Continued) Brown, moist, very dense, poorly graded GRAVEL with sand.
		▬	50/0"					Total Depth = 23.5 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
25								
30								
35								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 3/13/23 BORING NO. B-10 GROUND ELEVATION 2,252' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Wildcat) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN **DESCRIPTION/INTERPRETATION**
0							SP	ALLUVIUM: Brown, moist, loose, poorly graded SAND.
			3					
			14	4.1	109.4		SP-SM	Brown, moist, loose to medium dense, poorly graded sand with silt.
5								
			9					Medium dense; trace clay.
			50				SC	Brown, moist, dense, clayey SAND; trace fine to coarse gravel; decomposed granite.
10								Total Depth = 10 feet. Groundwater not encountered during drilling. Backfilled on 3/13/23 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
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APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix B.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory excavations in Appendix B.

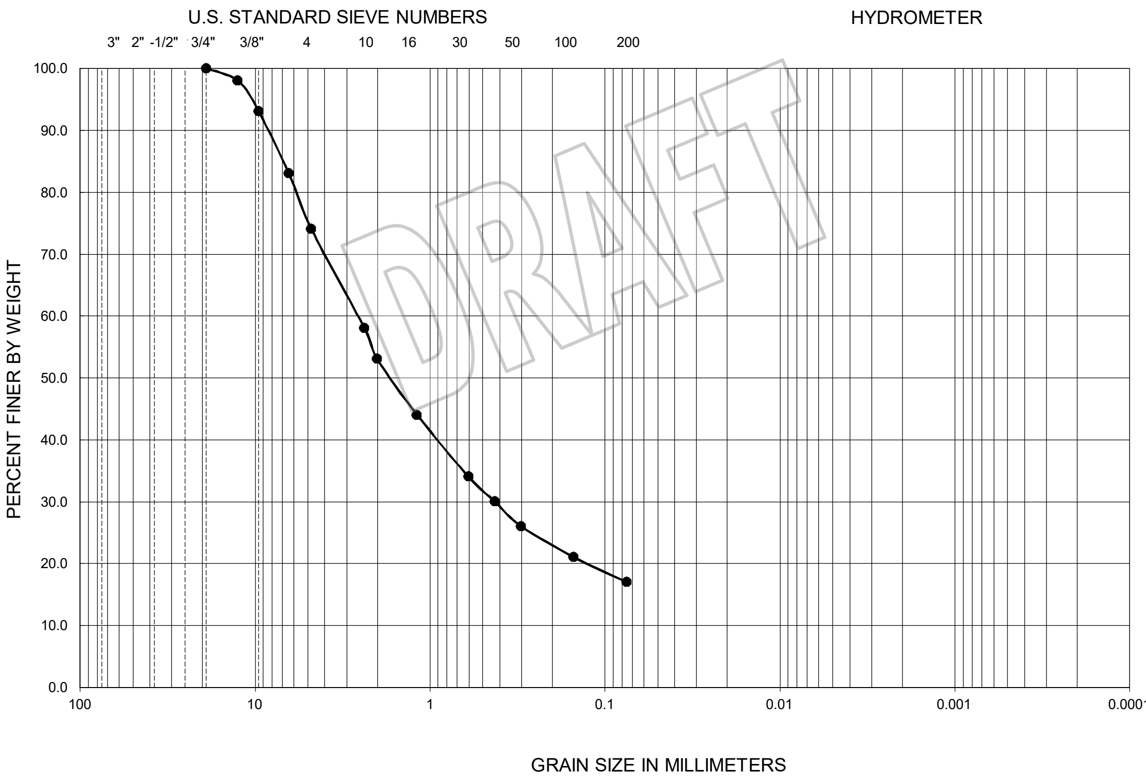
Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM C136 and D422. The grain-size distribution curves are shown on Figures C-1 through C-13. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figures C-14 and C-15.

Consolidation Tests
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures C-16 through C-19.

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on representative samples in general accordance with Arizona Test Method 236b. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with Arizona Test Method 733 Arizona Test Method 736, respectively. The test results are presented on Figure C-20.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	0.0-5.0	--	--	NP	--	0.432	2.61	--	--	17.0	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

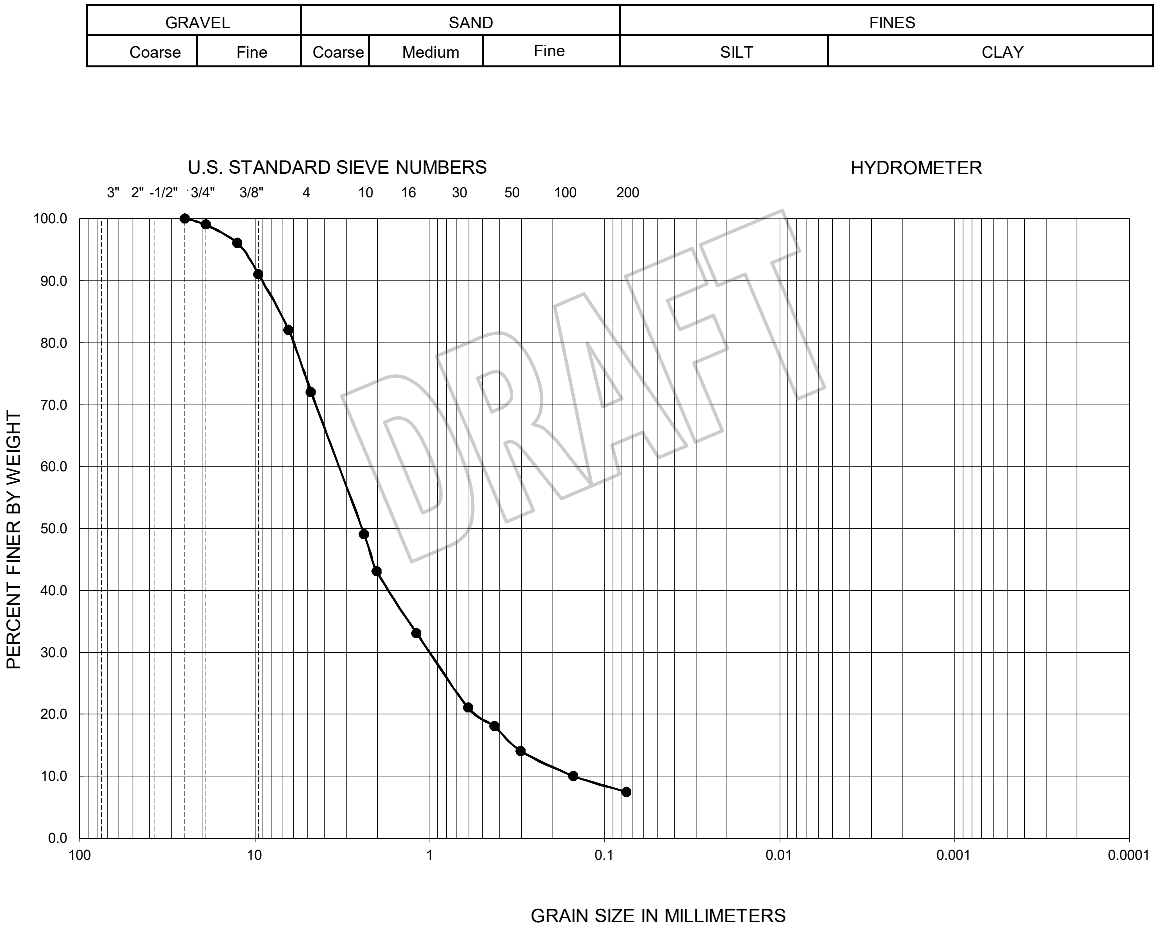
FIGURE C-1

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	0.0-5.0	--	--	NP	0.150	0.998	3.34	22.3	2.0	7.4	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-2

GRADATION TEST RESULTS

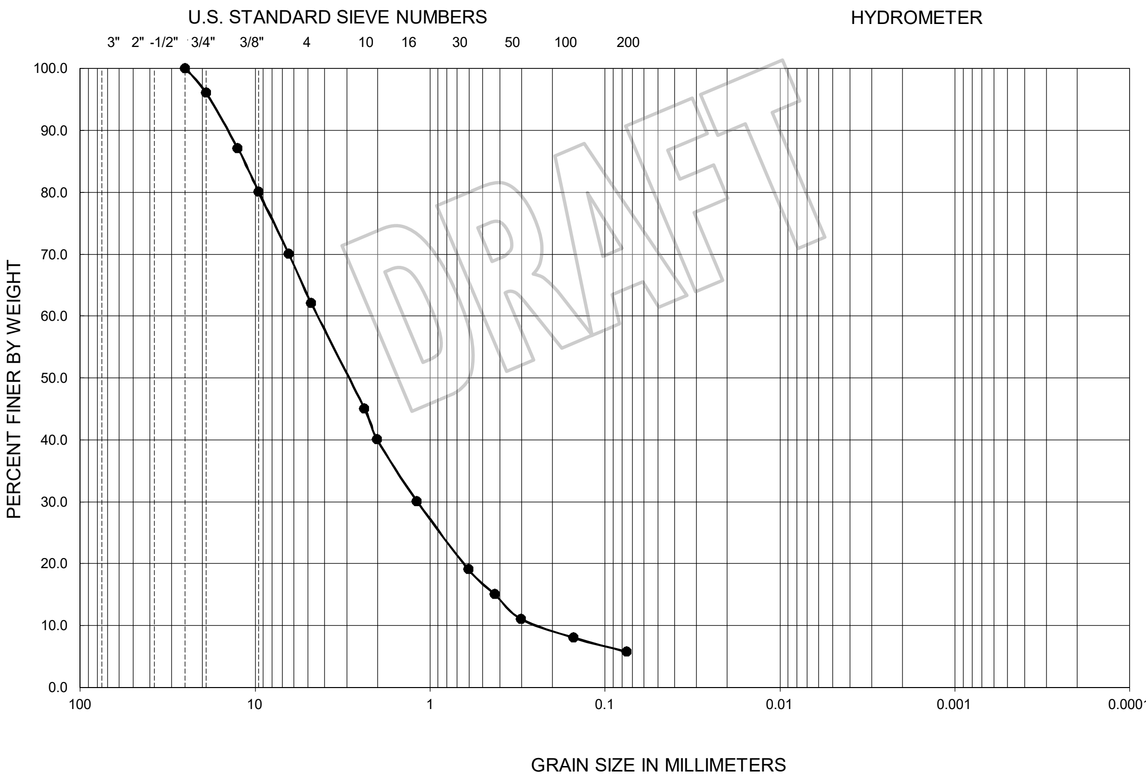
CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

607272001 | 4/23

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-5	0.0-5.0	--	--	NP	0.241	1.194	4.44	18.4	1.3	5.7	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-3

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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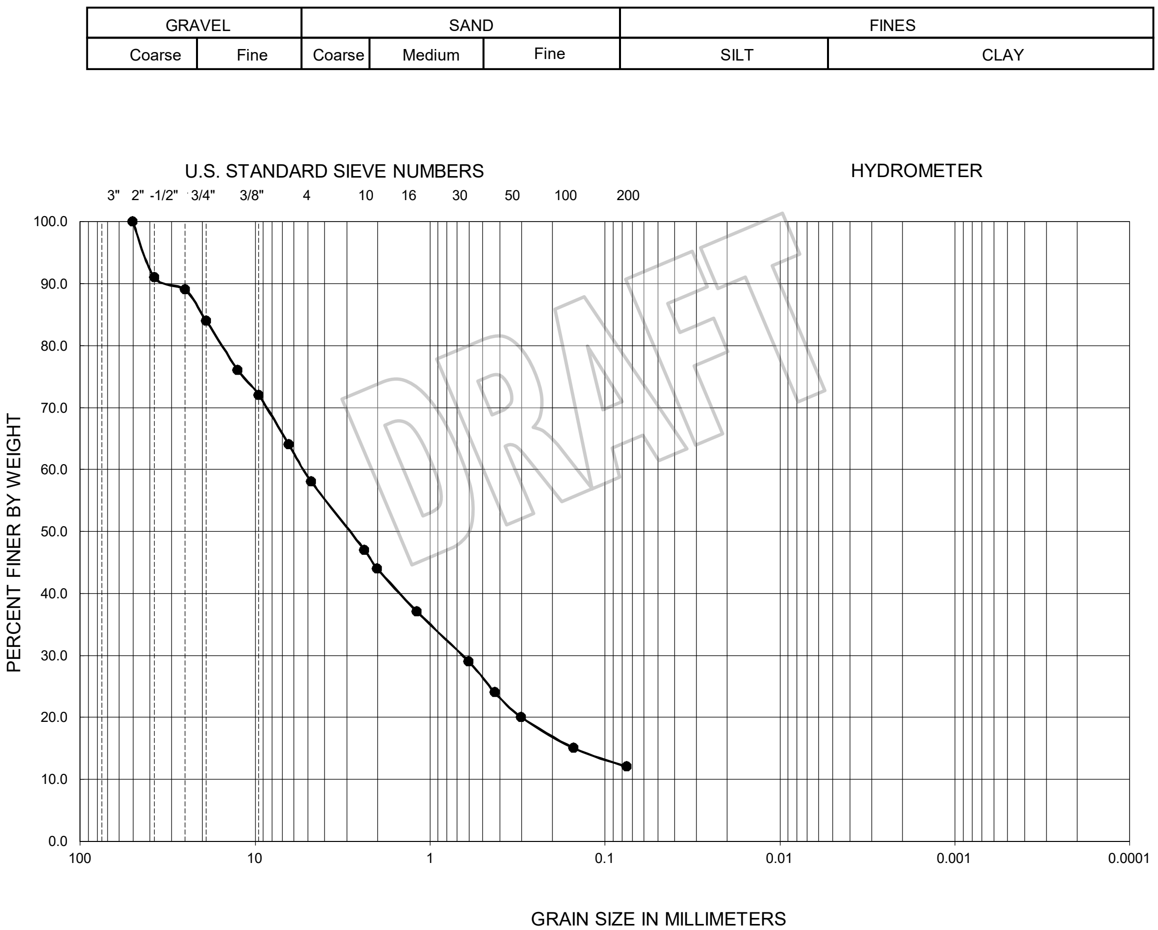
Geotechnical & Environmental Sciences Consultants



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	3.5-5.0	--	--	NP	--	0.639	5.29	--	--	12.0	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-4

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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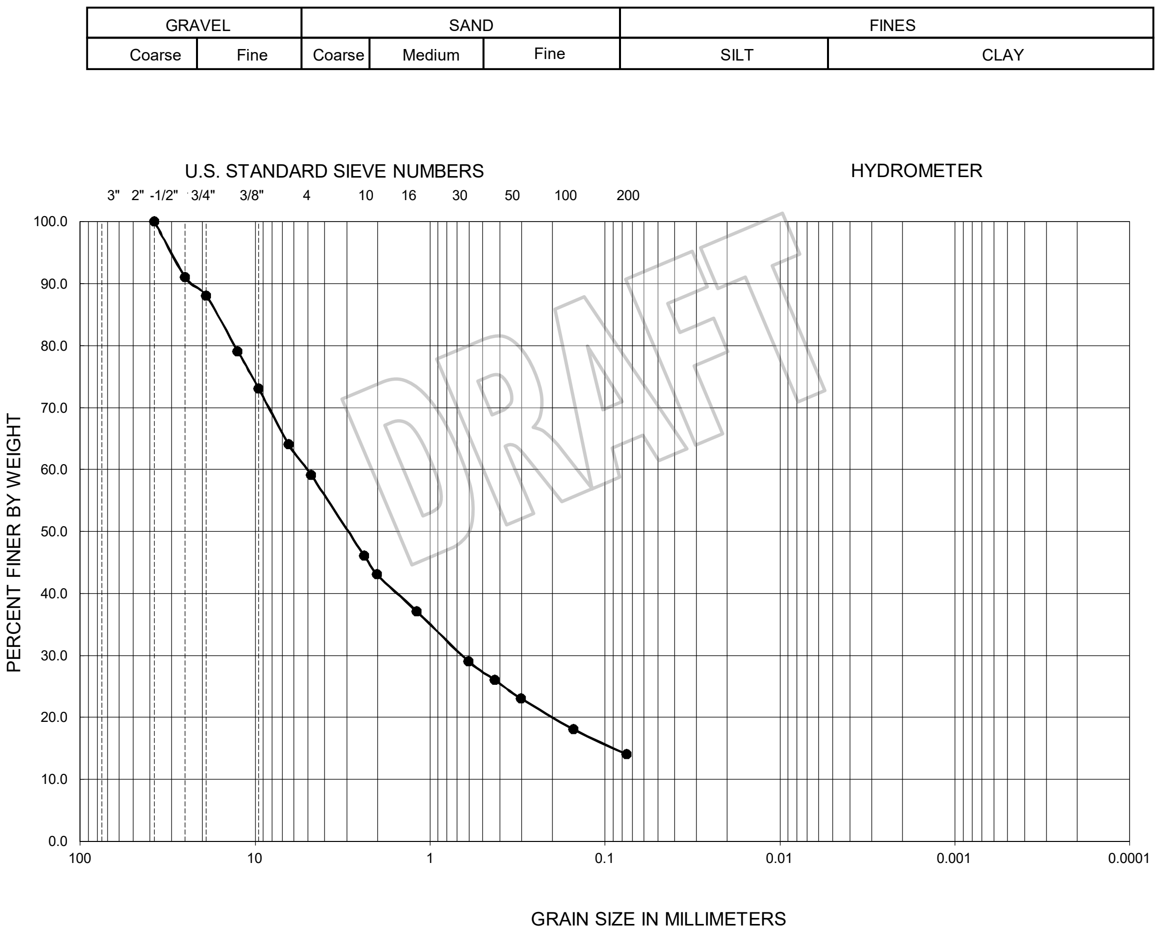


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	8.5-10.0	--	--	NP	--	0.639	5.10	--	--	14.0	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-5

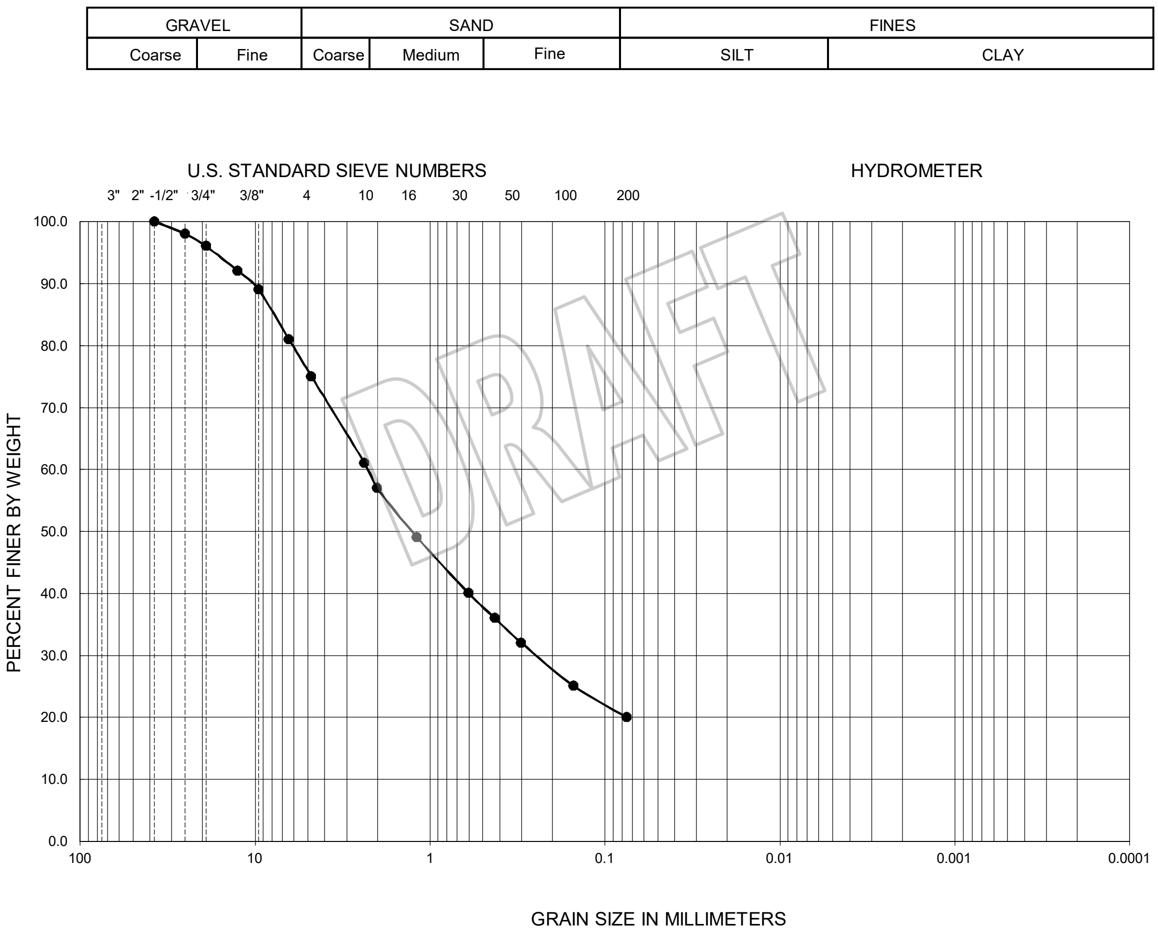
GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

607272001 | 4/23



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-7	0.0-5.0	26	14	12	--	0.249	2.29	--	--	20.0	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-6

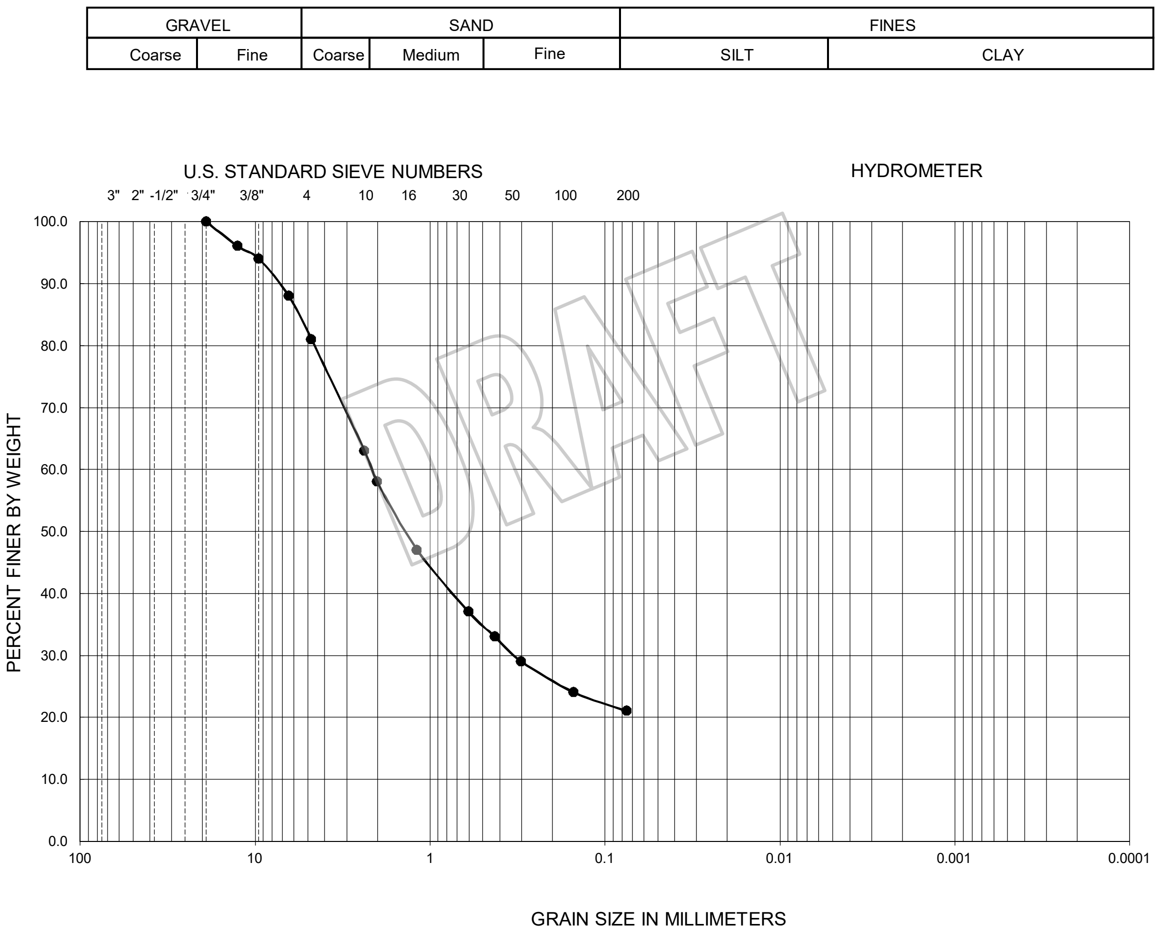
GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

607272001 | 4/23

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-8	3.5-5.0	28	22	6	--	0.333	2.15	--	--	21.0	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-7

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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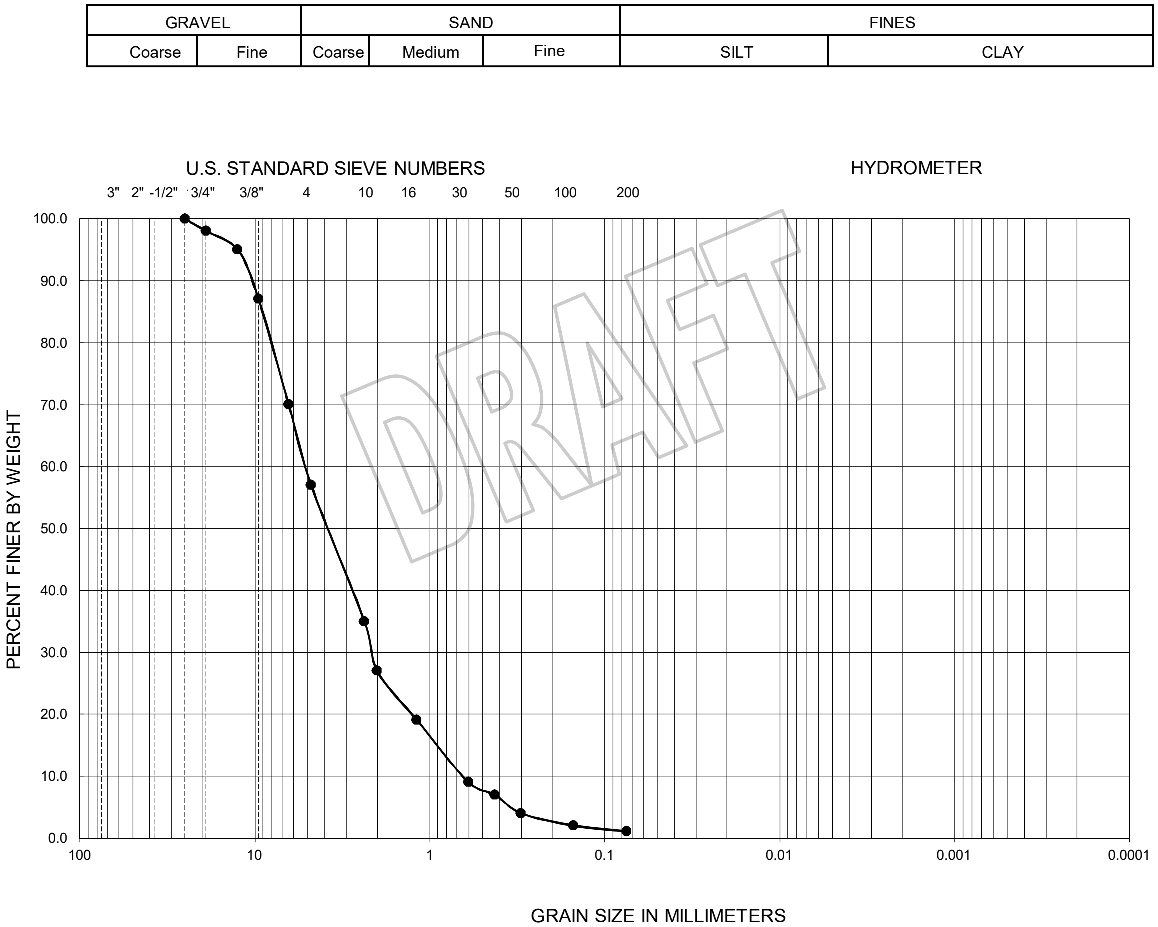

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-9	0.0-5.0	--	--	NP	0.627	2.142	5.14	8.2	1.4	1.1	SW

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-8

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
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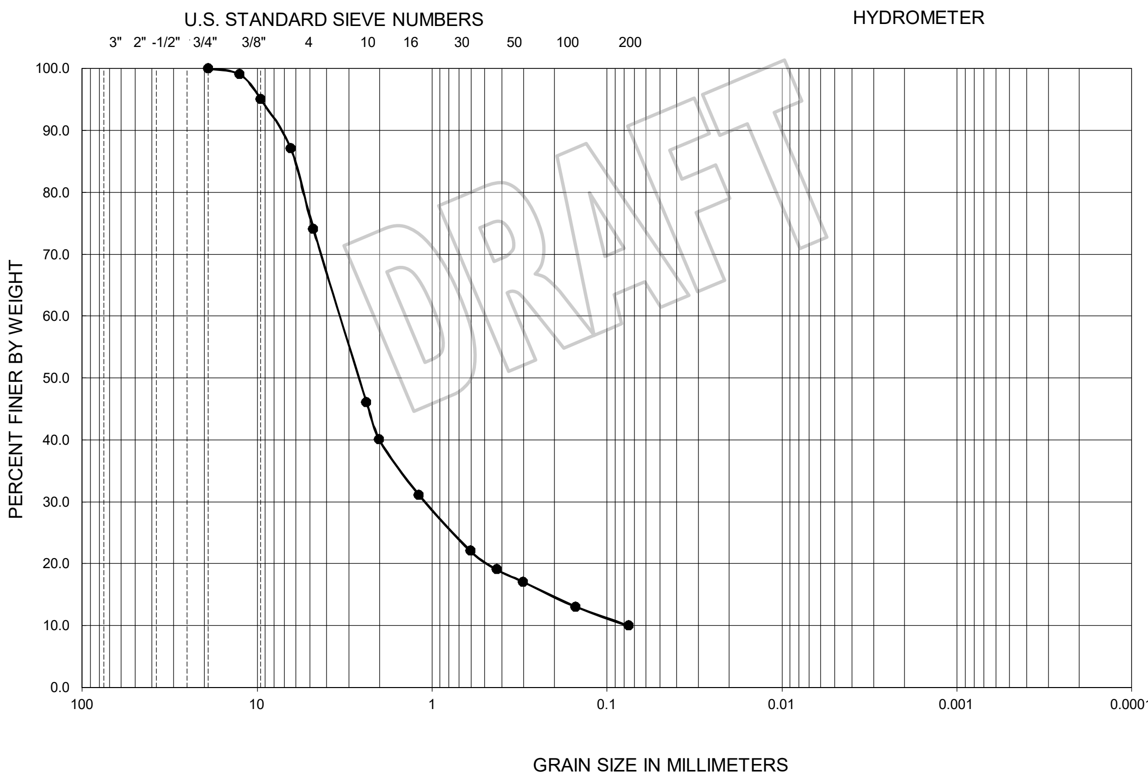
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-10	3.5-5.0	--	--	NP	0.075	1.103	3.39	45.3	4.8	9.9	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-9

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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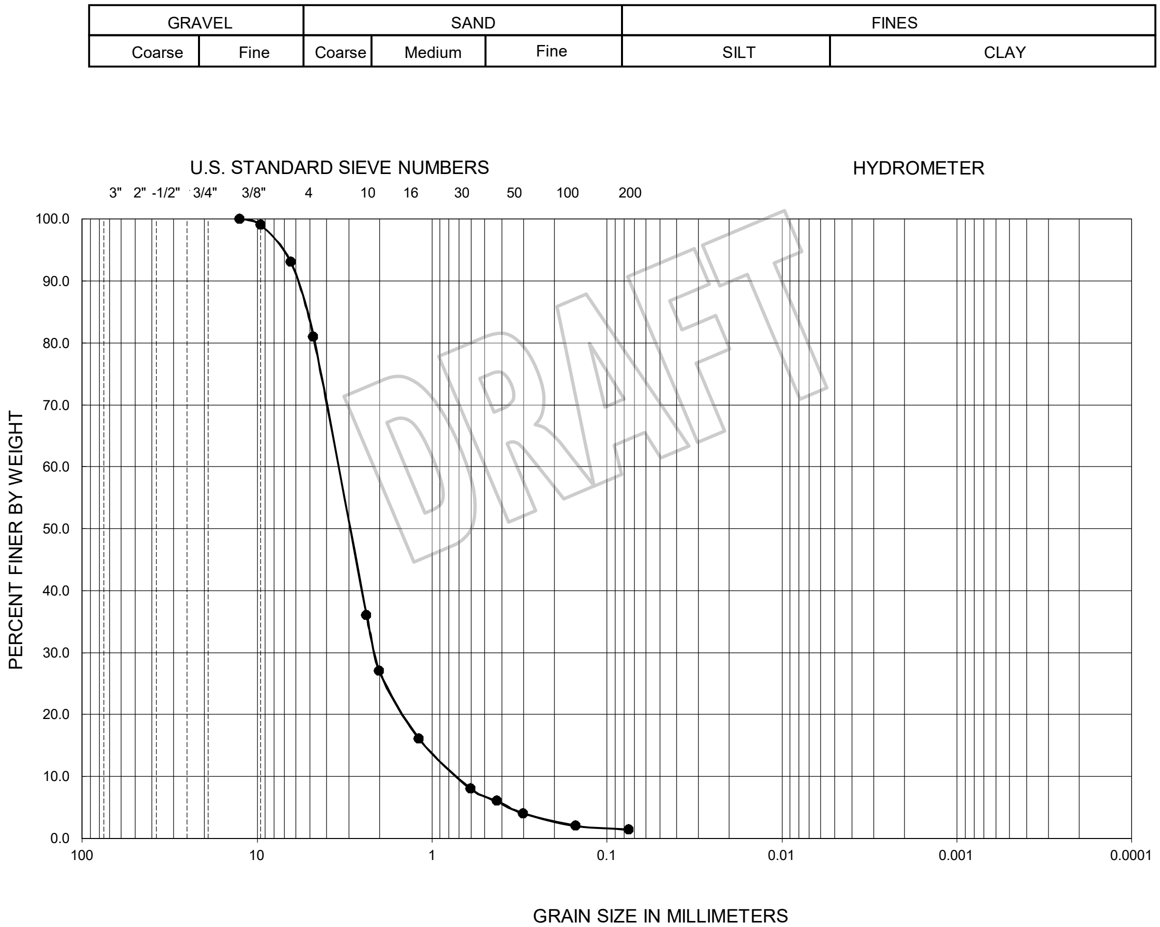

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	H-1	0.0-2.0	--	--	NP	0.698	2.126	3.48	5.0	1.9	1.4	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-10

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
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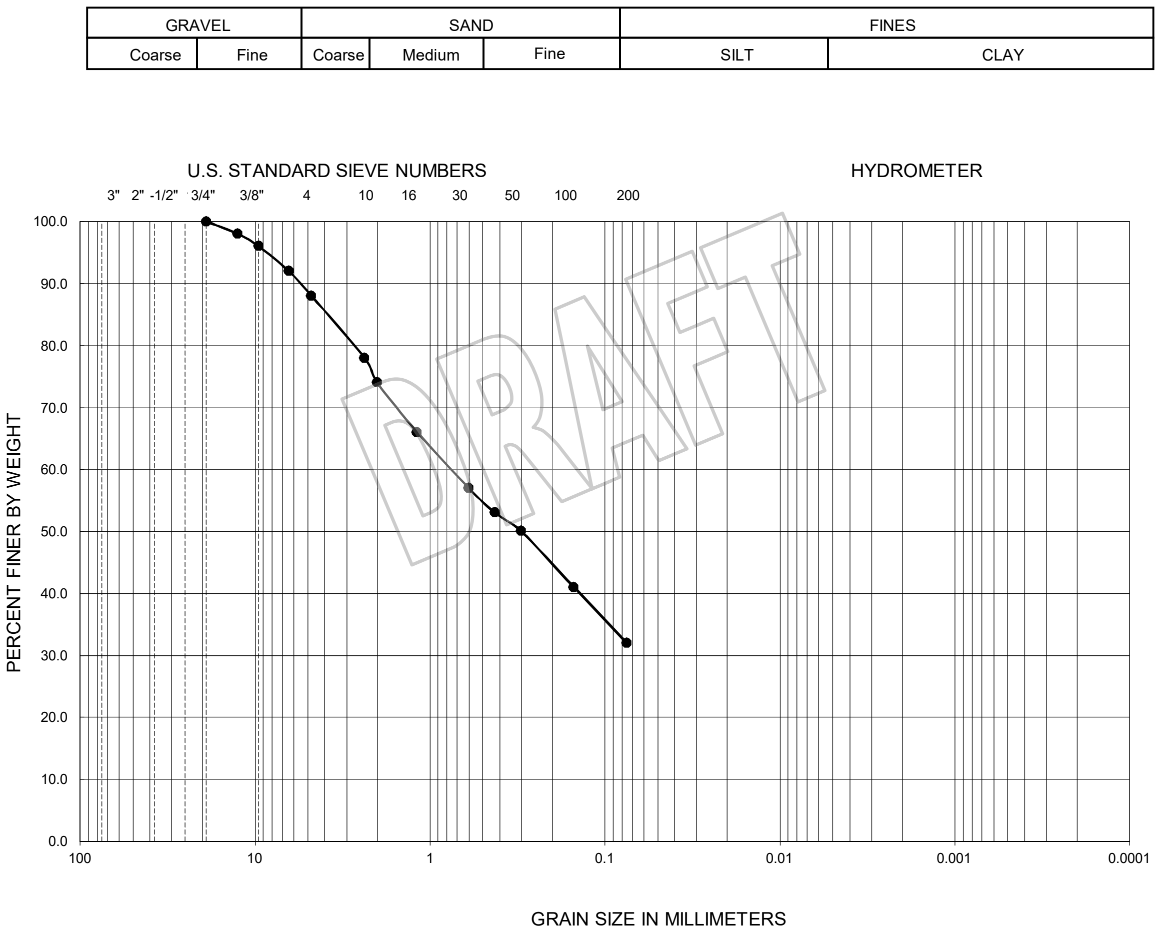


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	H-2	0.0-2.0	30	22	8	--	--	0.74	--	--	32.0	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

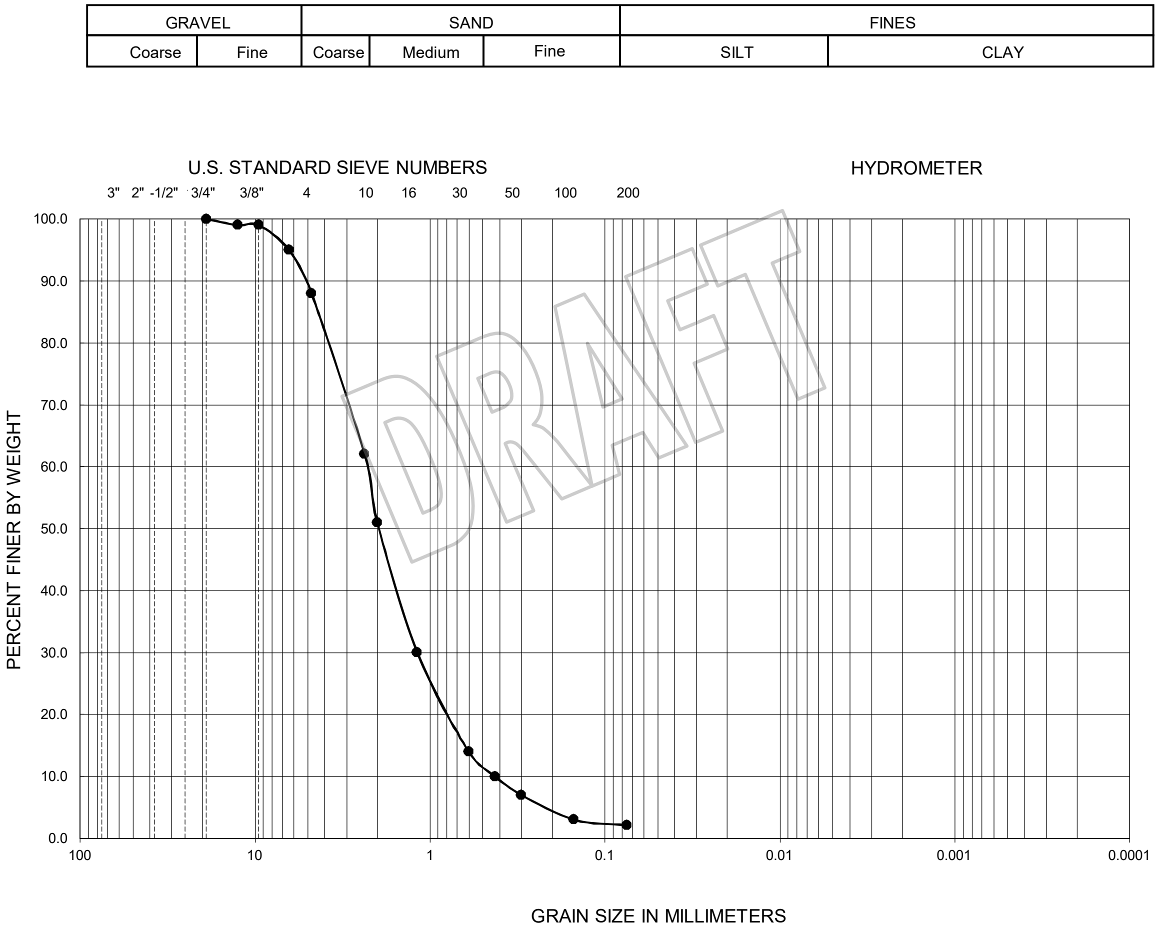
FIGURE C-11

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	H-3	0.0-2.0	--	--	NP	0.432	1.194	2.31	5.4	1.4	2.1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-12

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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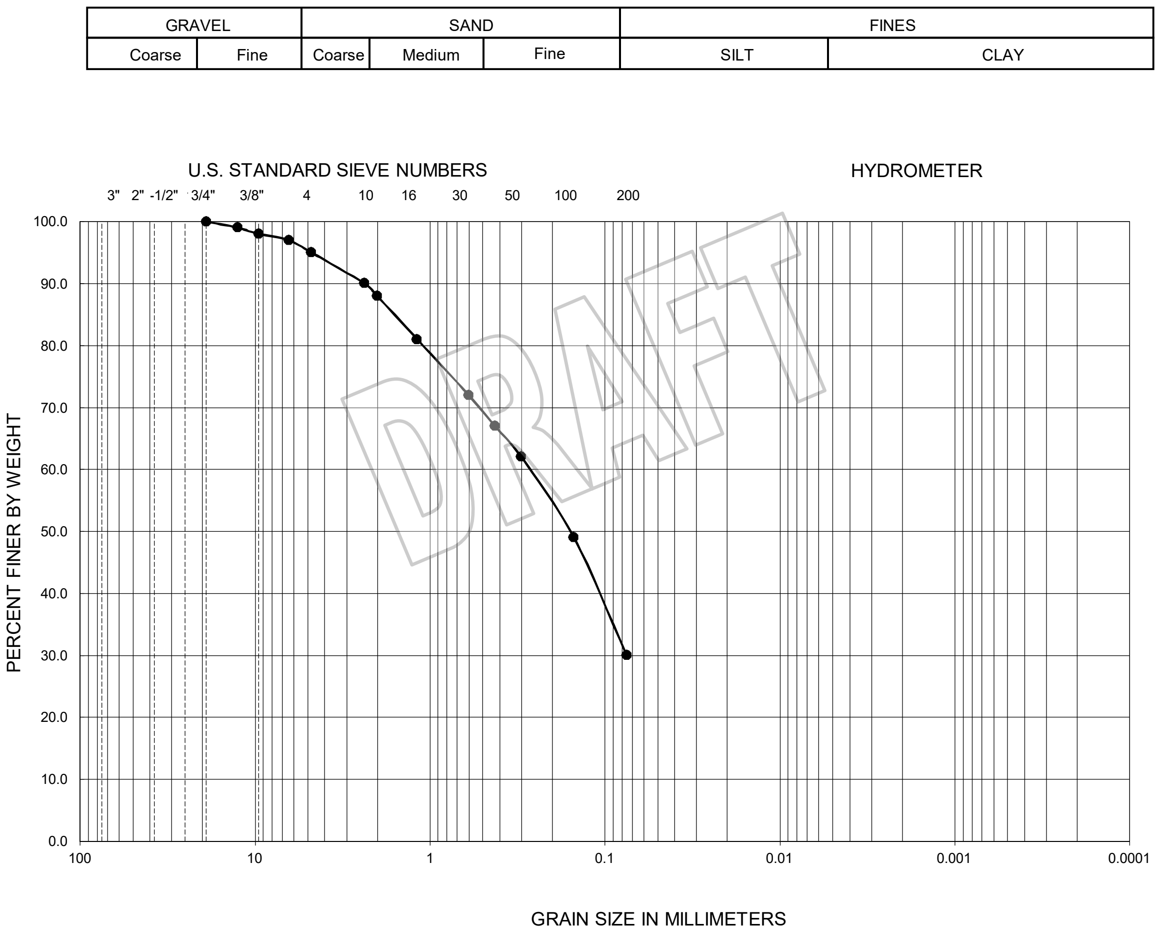

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	H-4	0.0-2.0	--	--	NP	--	0.074	0.27	--	--	30.0	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE C-13

GRADATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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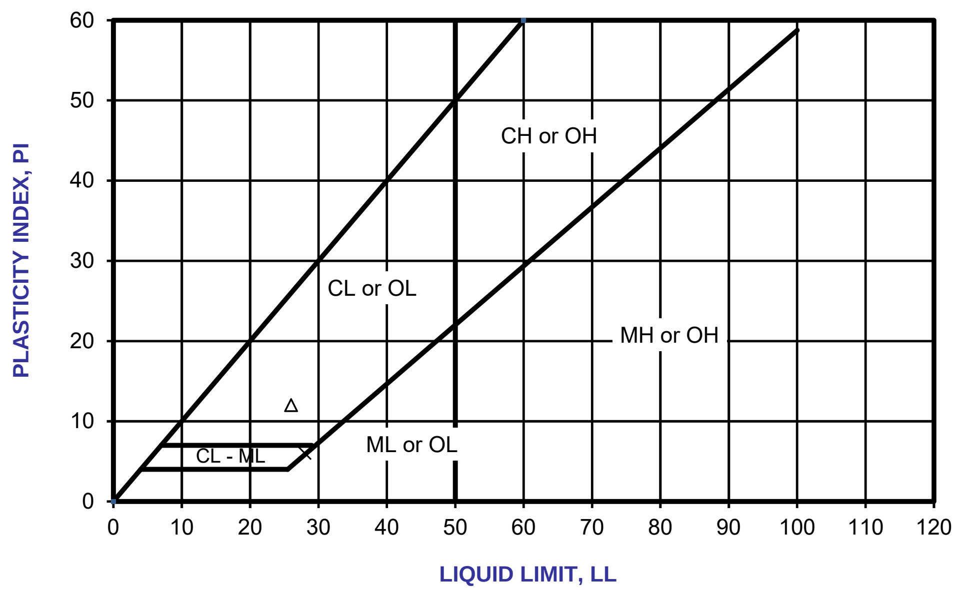
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	0.0-5.0	--	--	NP	ML	SM
■	B-3	0.0-5.0	--	--	NP	ML	SW-SM
◆	B-5	0.0-5.0	--	--	NP	ML	SW-SM
○	B-6	3.5-5.0	--	--	NP	ML	SM
□	B-6	8.5-10.0	--	--	NP	ML	SM
Δ	B-7	0.0-5.0	26	14	12	CL	SC
X	B-8	3.5-5.0	28	22	6	CL-ML	SC-SM
+	B-9	0.0-5.0	--	--	NP	ML	SW

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE C-14

ATTERBERG TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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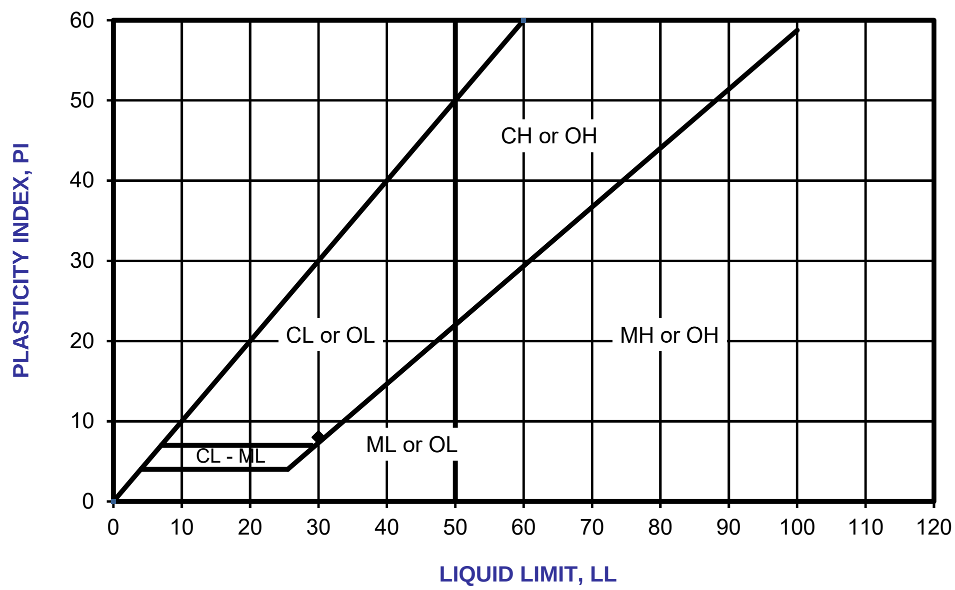
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-10	3.5-5.0	--	--	NP	ML	SP-SM
■	H-1	0.0-2.0	--	--	NP	ML	SP
◆	H-2	0.0-2.0	30	22	8	CL	SC
○	H-3	0.0-2.0	--	--	NP	ML	SP
□	H-4	0.0-2.0	--	--	NP	ML	SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE C-15

ATTERBERG TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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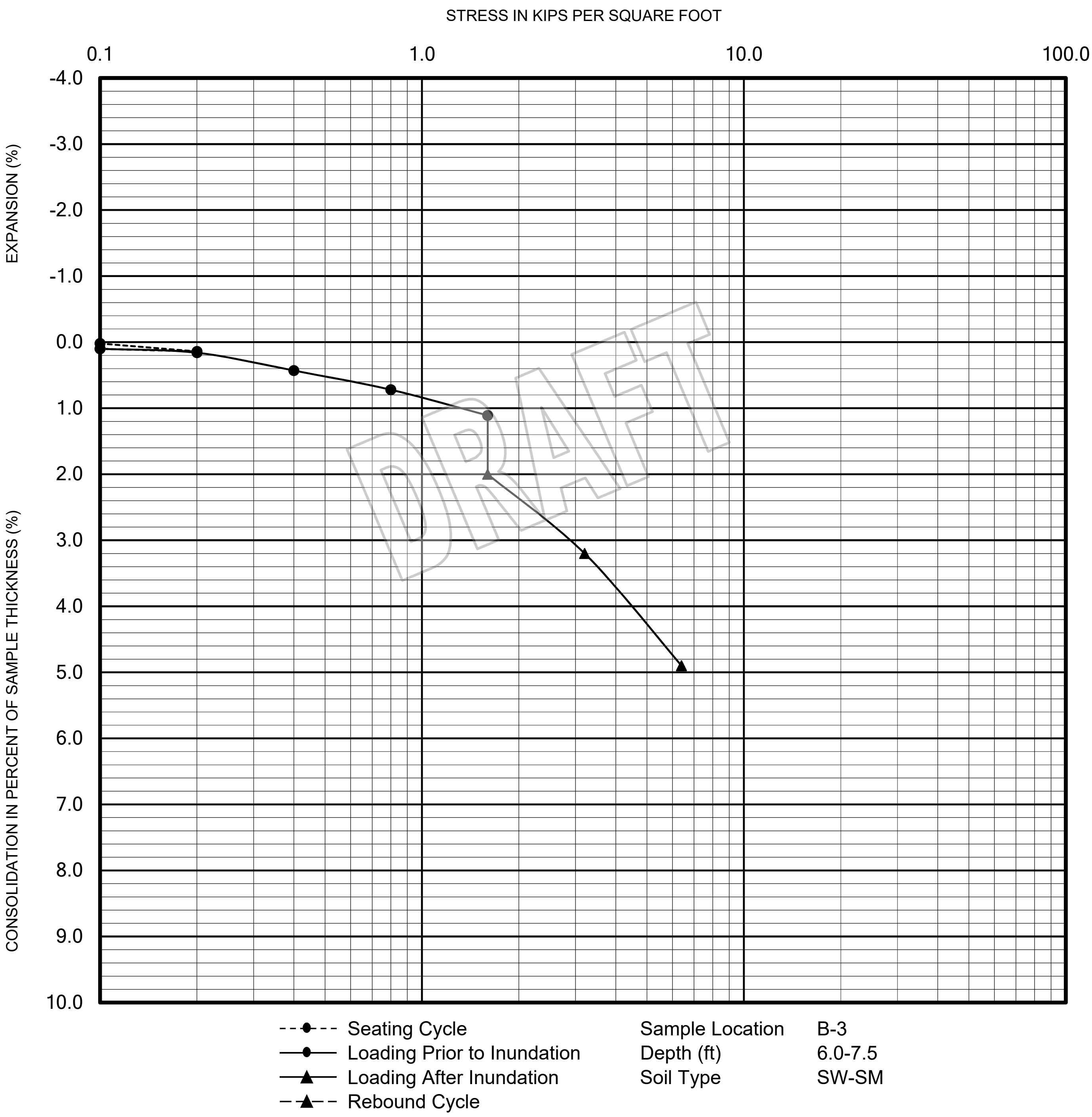

Sample Location	B-3
Depth (ft)	6.0-7.5
Soil Type	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-16

CONSOLIDATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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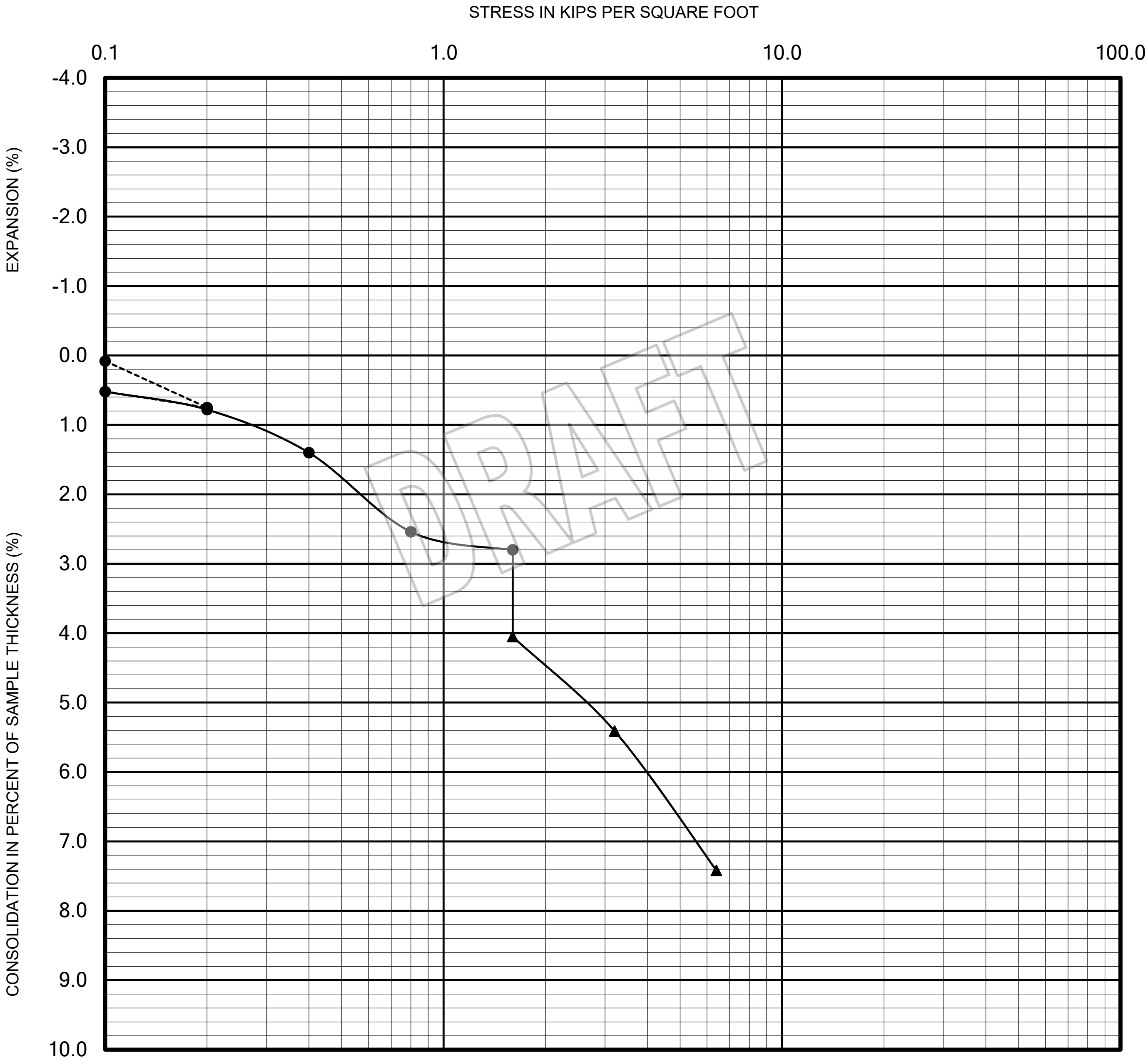


Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-5
Depth (ft)	6.0-7.5
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

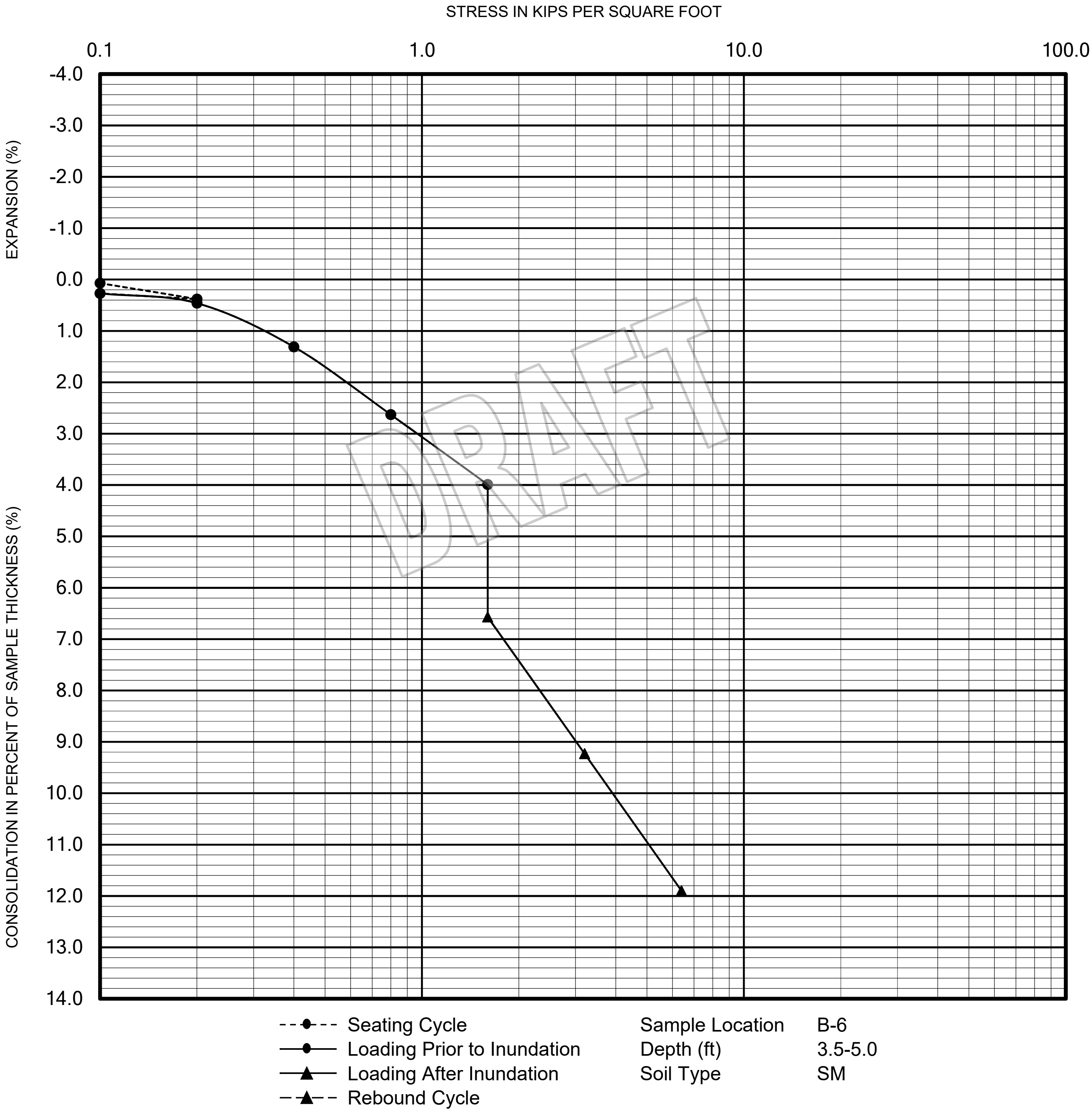
FIGURE C-17

CONSOLIDATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Sample Location	B-6
Depth (ft)	3.5-5.0
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-18

CONSOLIDATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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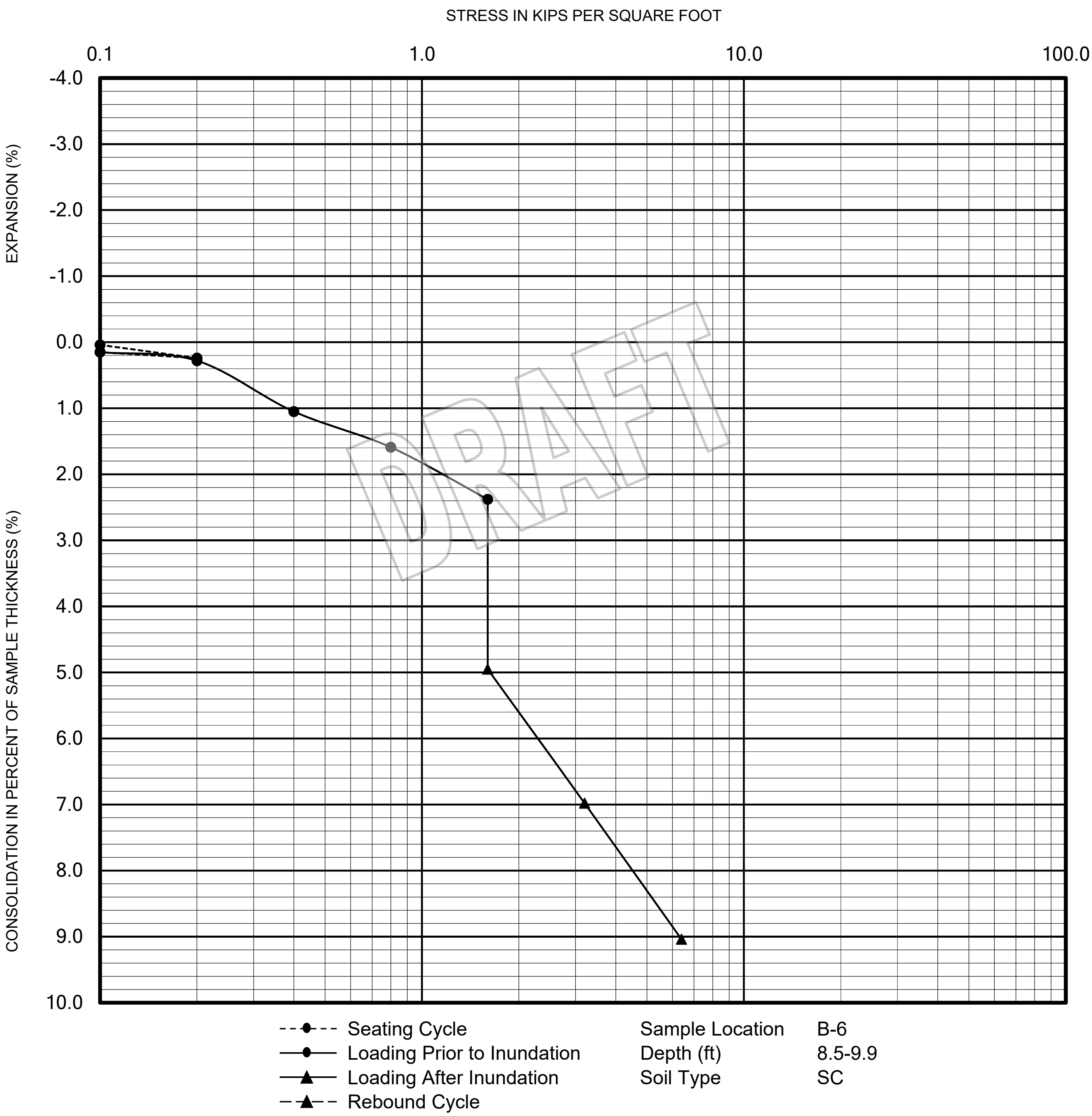

Sample Location	B-6
Depth (ft)	8.5-9.9
Soil Type	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-19

CONSOLIDATION TEST RESULTS

CAREFRE HIGHWAY - CAVE CREEK ROAD TO SCOTTSDALE ROAD

SCOTTSDALE, ARIZONA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	(%)	CHLORIDE CONTENT [3] (ppm)
B-3	0.0-5.0	7.5	4,087	3	0.0003	7
B-5	0.0-5.0	9.5	2,412	7	0.0007	9
B-9	0.0-5.0	7.6	804	5	0.0005	178

[1] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 236c

[2] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 733

[3] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 736

APPENDIX D

Pavement Design Calculations

Design ESAL Calculation Worksheet

TRACS:	NA		Project No.:	607272001		Project Name:	Carefree Highway Improvments		
Route:	Carefree Highway	BMP:	NA	Design Life (years):	20.0	Functional Class:	Urban Principal Arterial		
		EMP:	NA	Growth Rate (%):	3.0	Pavement Type:	Flexible & Rigid	Cluster Number:	Cluster 2

				Truck Volumes		Percent Trucks (% of total AADT)		Percent Trucks (% of total AADTT)		
				Singles	Combos	Singles	Combos	Singles	Combos	TTC
AADT	2022	16,975	Data Year	2,735	226	16.11%	1.33%	92%	8%	AZ-6
AADT	2025	18,549	Build Year							

Select appropriate TTC from Table A-1 using % Combos Above

Vehicle Class	1 - 3	4	5	6	7	8	9	10	11	12	13	Total
Distribution (%)	N/A	7.8	65.8	4.4	0.2	11.7	9.1	0.7	0.2	0.0	0.1	100.0
Class Volume (daily)	15,313	252	2,129	142	6	379	294	23	6	0	3	18,549
Growth Factor (G)	26.87	26.87	26.87	26.87	26.87	26.87	26.87	26.87	26.87	26.87	26.87	
Class Volume (total)	150,189,776	2,475,206	20,880,580	1,396,270	63,467	3,712,808	2,887,740	222,134	63,467	0	31,733	
Truck Load Factor (T_{LF})	0.0008	1.06	0.39	0.96	0.61	0.91	1.34	1.53	1.96	1.33	3.50	
Class ESAL's	120,152	2,623,718	8,143,426	1,340,419	38,715	3,378,656	3,869,571	339,865	124,395	0	111,067	20,089,983
											Total Number of Trucks	31,733,404
											Directional Distribution Factor (D_D)	0.51
											Lane Distribution Factor (D_L)	0.90
											Total ESAL's	9,221,302

ADOT Pavement Design - Carefree Highway Improvements - Alternative No. 1

Roadway Functional Classification	Non-Divided, Non-Interstate Highways, 10,000+ ADT
Facility Type	Mainline

Effective Roadbed Soil Resilient Modulus (Subgrade Support)

R-Test Values

Correlated R-Values

Plasticity Index	% Passing No. 200	Correlated R-Value	Plasticity Index	% Passing No. 200	Correlated R-Value
0	17	79			
0	7.4	90			
0	5.7	92			
0	12	85			
12	20	47			
6	21	59			
0	1.1	98			
0	9.9	87			

Total R-Value Tests	0	Total Correlated R- Value Tests	8	Mean R-Value	72
Mean of R-value tests		Mean of Correlated R-Value tests	72	Design R-Value	35
Std. Dev of R Value Tests		Std. Dev of correlated R-Value Tests	17.62	Seasonal Variation Factor	1.0

Calculated Resilient Modulus (M_R)	21,050	pounds per square inch (psi)
Design Resilient Modulus	21,050	psi **[Maximum M_R 26,000 psi]**

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Reliability

Level of Reliability	95%
Standard Normal Deviate (Z_R)	-1.645
Combined Standard Error (S_0)	0.45

Performance Criteria (Serviceability)

Initial Serviceability (P_0)	4.2
Terminal Serviceability (P_t)	2.8
Change in Serviceability (ΔPSI)	1.4

Traffic Loading

Equivalent Single Axle Loads	9,221,302
Percent of Mainline ESAL's	100%
Design ESAL's	9,221,302

Performance Criteria (Serviceability)

Structural Number (SN)	**3.75**

Roadway Section

	Coefficients			
Material Type	**Material**	**Drainage**	**Thickness**	**Structural Number**
Asphaltic Concrete	0.44	1.00	6	2.64
Recycled Asphaltic Concrete	0.28	1.00		
Cement or Bituminous Treated Base	0.28	1.00		
Cement or Lime Treated Subgrade	0.23	1.00		
Aggregate Base	0.14	1.00	8	1.12
Aggregate Subbase	0.11	1.00		
			Design Structural Number	**3.76**
			Required Structural Number	**3.75**

ADOT Pavement Design - Carefree Highway Improvements - Alternative No. 2

Roadway Functional Classification	Non-Divided, Non-Interstate Highways, 10,000+ ADT
Facility Type	Mainline

Effective Roadbed Soil Resilient Modulus (Subgrade Support)

R-Test Values

Correlated R-Values

Plasticity Index	% Passing No. 200	Correlated R-Value	Plasticity Index	% Passing No. 200	Correlated R-Value
0	17	79			
0	7.4	90			
0	5.7	92			
0	12	85			
12	20	47			
6	21	59			
0	1.1	98			
0	9.9	87			

Total R-Value Tests	0	Total Correlated R- Value Tests	8	Mean R-Value	72
Mean of R-value tests		Mean of Correlated R-Value tests	72	Design R-Value	35
Std. Dev of R Value Tests		Std. Dev of correlated R-Value Tests	17.62	Seasonal Variation Factor	1.0

Calculated Resilient Modulus (M_R)	21,050	pounds per square inch (psi)
Design Resilient Modulus	21,050	psi **[Maximum M_R 26,000 psi]**

Reliability

Level of Reliability	95%
Standard Normal Deviate (Z_R)	-1.645
Combined Standard Error (S_0)	0.45

Performance Criteria (Serviceability)

Initial Serviceability (P_0)	4.2
Terminal Serviceabilty (P_t)	2.8
Change in Serviceability (ΔPSI)	1.4

Traffic Loading

Equivalent Single Axle Loads	9,221,302
Percent of Mainline ESAL's	100%
Design ESAL's	9,221,302

Performance Criteria (Serviceability)

Strucutral Number (SN)	**3.75**

Roadway Section

	Coefficients			
Material Type	**Material**	**Drainage**	**Thickness**	**Structural Number**
Asphaltic Concrete	0.44	1.00	5	2.20
Recycled Asphaltic Concrete	0.28	1.00		
Cement or Bituminous Treated Base	0.28	1.00		
Cement or Lime Treated Subgrade	0.23	1.00		
Aggregate Base	0.14	1.00	12	1.68
Aggregate Subbase	0.11	1.00		
			Design Structural Number	**3.88**
			Required Structural Number	**3.75**

ADOT Pavement Design - Carefree Highway Improvements - Alternative No. 3

Roadway Functional Classification	Non-Divided, Non-Interstate Highways, 10,000+ ADT
Facility Type	Mainline

Effective Roadbed Soil Resilient Modulus (Subgrade Support)

R-Test Values

Correlated R-Values

Plasticity Index	% Passing No. 200	Correlated R-Value	Plasticity Index	% Passing No. 200	Correlated R-Value
0	17	79			
0	7.4	90			
0	5.7	92			
0	12	85			
12	20	47			
6	21	59			
0	1.1	98			
0	9.9	87			

Total R-Value Tests	0	Total Correlated R- Value Tests	8	Mean R-Value	72
Mean of R-value tests		Mean of Correlated R-Value tests	72	Design R-Value	35
Std. Dev of R Value Tests		Std. Dev of correlated R-Value Tests	17.62	Seasonal Variation Factor	1.0

Calculated Resilient Modulus (M_R)	21,050	pounds per square inch (psi)
Design Resilient Modulus	21,050	psi **[Maximum M_R 26,000 psi]**

Reliability

Level of Reliability	95%
Standard Normal Deviate (Z_R)	-1.645
Combined Standard Error (S_0)	0.45

Performance Criteria (Serviceability)

Initial Serviceability (P_0)	4.2
Terminal Serviceability (P_t)	2.8
Change in Serviceability (ΔPSI)	1.4

Traffic Loading

Equivalent Single Axle Loads	9,221,302
Percent of Mainline ESAL's	100%
Design ESAL's	9,221,302

Performance Criteria (Serviceability)

Structural Number (SN)	**3.75**

Roadway Section

	Coefficients			
Material Type	**Material**	**Drainage**	**Thickness**	**Structural Number**
Asphaltic Concrete	0.44	1.00	5	2.20
Recycled Asphaltic Concrete	0.28	1.00		
Cement or Bituminous Treated Base	0.28	1.00		
Cement or Lime Treated Subgrade	0.23	1.00		
Aggregate Base With Geogrid	0.22	1.00	8	1.76
Aggregate Subbase	0.11	1.00		
			Design Structural Number	**3.96**
			Required Structural Number	**3.75**

ADOT Pavement Design - Carefree Highway Improvements - Rigid Alternative

Roadway Functional Classification	Non-Divided, Non-Interstate Highways, 10,000+ ADT
Facility Type	Mainline

Effective Roadbed Soil Resilient Modulus (Subgrade Support)

R-Test Values

Correlated R-Values

Plasticity Index	% Passing No. 200	Correlated R-Value	Plasticity Index	% Passing No. 200	Correlated R-Value
0	17	79			
0	7.4	90			
0	5.7	92			
0	12	85			
12	20	47			
6	21	59			
0	1.1	98			
0	9.9	87			

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Total R-Value Tests	0	Total Correlated R- Value Tests	8	Mean R-Value	72
Mean of R-value tests		Mean of Correlated R-Value tests	72	Design R-Value	35
Std. Dev of R Value Tests		Std. Dev of correlated R-Value Tests	17.62	Seasonal Variation Factor	1.0

Average Modulus of Rupture (S'_c)	670	pounds per square inch (psi)
Modulus of Elasticity (E_c)	4,000,000	pounds per square inch (psi)
Drainage Coefficient (C_d)	1.00	Table 2-7
Load Transfer Coefficient (J)	3.9	Table 2-8
Calculated Resilient Modulus (M_R)	21,050	pounds per square inch (psi)
Design Resilient Modulus	21,050	psi **[Maximum M_R 26,000 psi]**
Modulus of Subgrade Reaction (k)	1,085	
Loss of Support (LS)	0	Table 2-9
Design Modulus of Subgrade Reaction (k_{corr})	1,085	Figure 2-6 (Chart to the right)

Reliability

Level of Reliability	95%
Standard Normal Random Variable (Z_R)	-1.645
Overall Standard Deviation (S_0)	0.25

Performance Criteria (Serviceability)

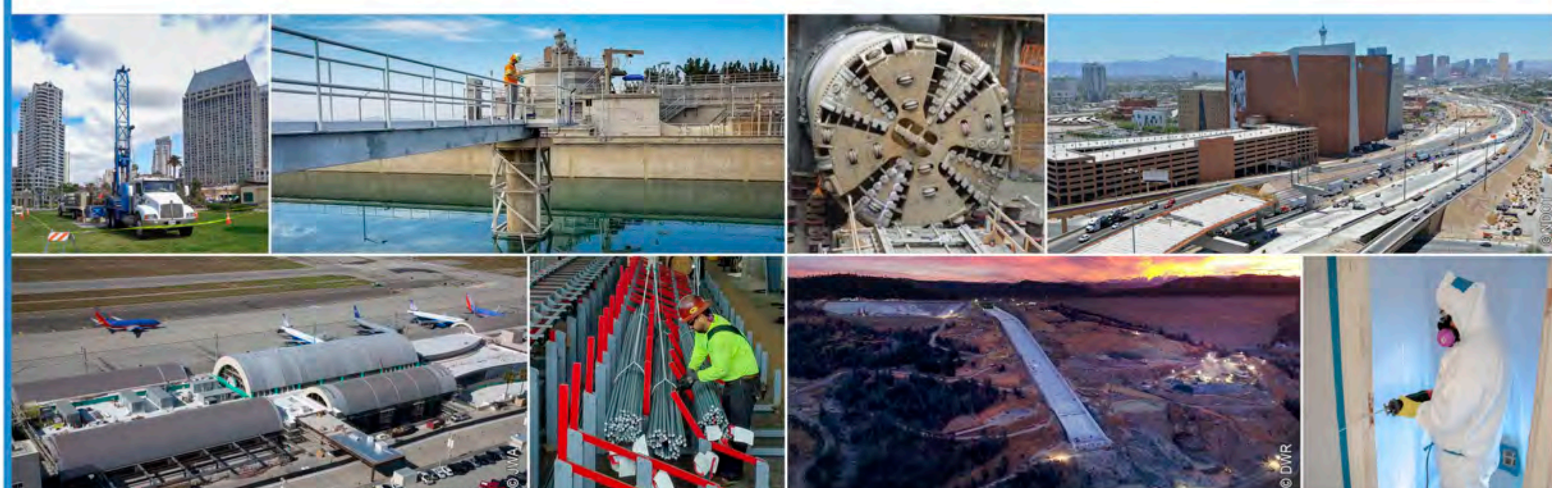
Initial Serviceability (P_0)	4.2
Terminal Serviceability (P_t)	2.8
Change in Serviceability (ΔPSI)	1.4

Traffic Loading

Equivalent Single Axle Loads	9,221,302
Percent of Mainline ESAL's	100%
Design ESAL's	9,221,302

Performance Criteria (Serviceability)

Thickness of the Pavement Slab (D)	**9.87**



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