GEOTECHNICAL EVALUATION
CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

PREPARED FOR:
Arizona Water Company
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PREPARED BY
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October 31, 2016
Project No. 603971001

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Mr. Fred Schneider
Arizona Water Company
PO Box 29006
Phoenix, Arizona 85038

Subject: Geotechnical Evaluation
Chapel Hills Water Campus
Mallard Drive and State Route 179
Sedona, Arizona

Dear Mr. Schneider:

In accordance with our proposal, and your authorization, Ninyo & Moore has performed a 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 to you during this phase of the project.

Respectfully submitted,
NINYO & MOORE

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1. INTRODUCTION

In accordance with our proposal and your authorization, we have performed a geotechnical evaluation for the proposed Arizona Water Company (AZWC) Chapel Hills Water Campus site located near the southwest corner of the intersection of Mallard Drive and State Route 179 in Sedona, Arizona. The site is comprised of Coconino County Assessor's Parcel Numbers (APNs) 401-33-029T, -029V, -029W, and -029Y. The purpose of our evaluation was to assess the subsurface conditions at the project site in order to formulate 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

The scope of services for this project included:

- Reviewing available topographic information, soil surveys, geologic literature, and aerial photographs of the project area.
- Conducting a field trip to the site for geologic reconnaissance and review of available equipment access.
- Conducting a field trip to mark out the exploration locations.
- Notifying Arizona 811 (Blue Stake) of the proposed test pit locations prior to excavating.
- Performing a geotechnical exploration, which included excavation of a total of three test pits via rubber-tired backhoe and total of two borings. The test pits extended to backhoe refusal on bedrock. The borings were drilled to a depth of 30 and 31 feet and were located within the proposed tank site. The borings were advanced with rock coring equipment. A Ninyo & Moore employee observed the excavation and drilling field work. Driven, bulk and rock core samples were recovered from each excavation. The test pits and borings were backfilled with excavation spoils upon completion.
- Performing six seismic refraction survey lines within the proposed tank footprint. These seismic refraction survey lines complement the test pit information, and provide information associated with the excavation characteristics of the underlying materials.
- Conducting laboratory testing of selected samples for index, strength and chemical parameters.

- Preparing this Geotechnical Evaluation that presents our field procedures and provides a discussion on the geologic setting and potential geologic hazards as well as recommendations for foundation design and construction.

3. SITE DESCRIPTION

The project site for the planned Chapel Hills Water Campus is situated in Section 19, Township 17 North, Range 6 East, of the Salt River and Baseline Meridian and is located southwest of the intersection of West Mallard Drive and State Route 179 in Sedona, Arizona (Figure 1). At the time of our evaluation, the project site was located within a residential subdivision overlooking State Route 179 located outside its eastern boundary. A rock outcrop situated atop a northwest to southeast trending ridgeline was the dominant topographic feature of the site, and was bounded on either side by natural drainage features. Slopes generally trended down from the southeast to the northwest. The site was relatively undeveloped with vegetation on the site that included juniper trees and grass. A utility easement traversed the east side of the site, which represented the site access at the time of our evaluation.

Based on the *Sedona, 7.5-Minute United States Geological Survey (USGS) Topographic Quadrangle Map (2014),* the site elevation ranges from approximately 4,170 to 4,195 feet relative to mean sea level. Based on the topographic map, the vicinity of the site generally slopes from the southeast down to the northwest.

Aerial photographs from Google Earth were reviewed for this project. An aerial photograph from 1997 depicted the site and surrounding areas as undeveloped, with the exception of the residence to the southwest of the site, near State Route 179 and Mallard Drive. A 2003 aerial photograph shows the utility easement cleared of vegetation and construction activities at surrounding properties. Additional aerial photographs indicate many of the residential construction surrounding the project site were constructed approximately between 2003 and 2007. An aerial photograph from 2010 depicted the site as similar to its current condition.

4. PROPOSED CONSTRUCTION

Per the Site Plan we received from your office, this project consists of the design and construction of a water campus on the site, including a 1.5 million gallon tank and the affiliated booster pumps, electrical equipment, and hydro tank.

At the time of our evaluation, grading and drainage plans for the site were not available. However, based on conversations with your office, the new water tank is to be partially buried at the site. Based on our understanding, the planned foundation elevation of the tank will be on the order of 10 feet or less below existing grade. Due to the site topography and proposed layout of the tank, the current grades within the tank foot print vary from approximately 4,174 to 4,192 feet. As such, we understand some grade raise fill may be needed.

Given that the proposed tank foundation may extend into the underlying bedrock formation , we have prepared and submitted separately a proposal to perform additional explorations to better evaluate the bedrock conditions.

5. FIELD EXPLORATION AND LABORATORY TESTING

Ninyo & Moore conducted a subsurface exploration at the site to evaluate the subsurface conditions and to collect soil samples for laboratory testing. The exploration consisted of excavating, drilling, logging, and sampling three exploratory test pits (denoted as TP-1, TP-2, and TP-3) using a rubber-tired backhoe, and two borings (denoted as B-1 and B-2) using rock coring equipment. The test pits extended to depth ranging from approximately 2 to 3.5 feet below ground surface and encountered excavation refusal on bedrock. The borings extended to depth of 30 to 31 feet below ground surface. The test pits and borings were situated in the approximate locations depicted on Figure 2.

Bulk and relatively undisturbed samples were collected at various depths within the test pits. Rock core samples were recovered for testing and analysis. Detailed descriptions of the soils/rocks encountered in our test pits and borings are presented in Appendix A. The samples obtained during the excavation and coring operations were visually classified, placed into appropriate containers, and transported to the Ninyo & Moore laboratory for testing and evaluation.

Laboratory testing was performed on select representative samples collected during our subsurface evaluation in order to evaluate the in-situ moisture content and dry density, gradation analysis, Atterberg limits, consolidation (response-to-wetting), unconfined compressive strength and corrosivity characteristics (pH, minimum electrical resistivity, and chloride and soluble sulfate contents). Detailed descriptions of our laboratory test methods are presented on the test pit logs and/or in Appendix B.

Ninyo and Moore personnel conducted seismic refraction surveys at the site on November 15, 2012 and December 13, 2012, to evaluate the approximate depth to bedrock and rippability characteristics of the near surface materials. The seismic refraction data were collected with a SmartSeis S12, high performance exploration seismograph and 12 vertical component geophones. A 16-pound hammer and metal plate were used as the seismic wave source. A total of 6 seismic refraction traverses were performed, and the approximate locations of the traverses are depicted on Figure 2. The results of our seismic refraction surveys are presented in Appendix C.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

6.1. Geologic Setting

The project site is located in the Transition Zone physiographic province, an approximately 50-mile wide mountain belt that traverses northwest-southeast through central Arizona. This province is the physiographic transition between the Colorado Plateau province and Basin and Range province in Arizona. The province is characterized by the presence of Proterozoic (1.7 billion years) granitic and metamorphic rocks that were exposed as the sedimentary Paleozoic (510 million years) and Mesozoic (245 million years) rocks were eroded away.

High-angle normal faults and block faulting occurred during the Mid-Tertiary which also coincided with an increase in volcanic activity during this time (Kamilli and Richard, 1998).

The surficial geology of the site is described as the Lower Permian-Age (approximately 290 million years) Hermit Formation. The formation generally consists of thin-bedded deposits of sandstone and siltstone (Dewitt, Langenheim, Force, and Lindberg, 2008). Based on the United States Department of Agriculture Web Soil Survey, the site soils are mapped as the Sedona and Turist Soils.

6.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on our field explorations and laboratory testing, and our understanding of the general geology of the area.

Weathered in-place bedrock (classified as Schnebly Hill Formation sandstone) was encountered in our test pits and borings. The Schnebly Hill Formation is a dark red sandstone, approximately 800 to 1,000 feet thick that is the main component of the "Red Rocks" of Sedona. The Schnebly Hill Formation sandstone is laid down in flat-bedded horizontal layers, interspersed with multiple thin white layers of limestone conglomerate. The weathered in-place sandstone extended up to 31 feet bgs in our borings. More detailed descriptions are presented on the test pit and core logs in Appendix A.

6.3. Groundwater

Groundwater was not encountered in our borings during our field exploration. Based on well data presented by the Arizona Department of Water Resources Web Groundwater Data Map, the average depth to groundwater has been estimated at approximately 310 to 360 feet bgs near the site. However, shallow or perched groundwater conditions could be encountered during construction based on the proximity of the unnamed drainages at the southern and northern edges of the site and should be considered in the design of the project. It should be noted that groundwater levels may change and can vary with seasonal rainfall patterns, long-term climate fluctuations and with the influence of site conditions such as drainage patterns, joint and fracture geometry in the bedrock, and variable weathering in the bedrock. As such, subterranean seeps or springs should be anticipated during the construction of the project.

6.4. Surface Water

Based on information presented on Federal Emergency Management Agency (FEMA) Floodplain Maps, the project site was in flood Zone X indicating a 0.2 percent annual chance of a flood.

7. GEOLOGIC HAZARDS – FAULTING

The site lies within the Transition Zone, which is a relatively stable tectonic region that traverses northwest-southeast through central Arizona (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations and on our review of readily available published geological maps and literature, there are no known active faults underlying the subject site or adjacent area. The closest documented Quaternary age fault to the site is the Verde Fault Zone, located approximately 20 miles to the southwest of the site (Richard, Reynolds, et. al 2000). The Verde fault zone is the main (master) fault on the southwestern margin of the Verde Valley, which is a large, asymmetric, southwest-tilted graben in the Basin and Range province near the margin of the Colorado Plateaus. This steeply northeast-dipping fault zone follows the base of a high, relatively linear, steep, northeast-facing mountain front. Documented evidence of Quaternary movement along the fault exists along a short section of the southern part of the fault zone, where fault scarps as much as about 23 feet high are formed on high, dissected alluvial fans of probable early to middle Pleistocene age. Recent movement on the fault has been documented as Late Quaternary (<130,000 years), and the estimated slip rate of the fault is about 0.2 mm per year. Seismic parameters recommended for the design of the proposed improvements are presented in Section 9.4.

8. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, 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:

- In general, the surface materials at the project site are considered to be rippable with conventional heavy-duty excavation equipment in good working condition. However, shallow sound bedrock was encountered during our field evaluation, and will call for more aggressive excavation techniques during construction (possibly blasting).

- Imported soils and soils generated from on-site excavation activities that exhibit a relatively low plasticity and very low to low expansive potential can generally be used as engineered fill. Particles larger than 4 inches in dimension should not be used as backfill material. Based on the results of our laboratory testing, some of the onsite soils may be suitable for use as fill; however, sorting and/or crushing should be anticipated.

- Based on the results of the field and laboratory evaluations, it is our opinion that the proposed structures for this project can be founded on spread foundations either proportioned for low to moderate bearing pressures on the order of 2,500 pounds per square foot (psf) when supported on a zone of moisture-conditioned and compacted engineered fill or proportioned for higher bearing pressures on the order of 5,000 psf when supported on sound bedrock.

- Groundwater was not encountered during the time of our field exploration. Based on well data from nearby wells, the regional groundwater table is on the order of approximately 310 to 360 feet bgs. However, shallow or perched groundwater conditions could be encountered during construction based on the proximity of the unnamed drainages at the southern and northern edges of the site and should be considered in the design of the project. In addition, subterranean seeps or springs should be anticipated in bedrock formations during the construction of the project.

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

- The site soils exhibited a potential to be corrosive to ferrous metals. The sulfate content of the soils presented a negligible sulfate exposure for concrete.

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction.

These recommendations are based on our understanding of the project. Ninyo & Moore should be contacted for additional recommendations and/or evaluation if the proposed construction is changed from that discussed in this report.

9.1. Earthwork

The following sections provide our earthwork recommendations for this project. In general, the earthwork specifications contained in the Maricopa Association of Governments (MAG), *Uniform Standard Specifications and Details for Public Works Construction* (including any amendments from the City of Sedona and/or Central Yavapai County Governments [YAG]), should apply, except as noted in this report.

9.1.1. Excavations

Based on the results of our field exploration, the surface materials should generally be excavatable with heavy-duty excavation equipment. However, bedrock was encountered in our explorations as shallow as approximately 2 feet bgs. This bedrock will slow the rate of excavation and be more difficult to excavate. Specialized bedrock excavation equipment and/or blasting may be needed to facilitate excavations to the designed depths. More discussions regarding rippability at the site, as well as our seismic refraction survey results, are presented in Appendix C. The above recommendations should be used with discretion, and contractors should make their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. Nevertheless, the contractor should anticipate difficult excavation during construction.

Given that the proposed tank foundation may extend into the underlying bedrock formation that was physically explored.

The proposed excavations will generate oversize material (particles larger than 4 inches), such as weathered or newly fractured rock. Screening, disposal, and/or crushing of this material should be anticipated if re-use of this material is considered.

It may be desirable to note 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 to the existing utilities that might result from this work. We also recommend vibration and noise monitoring during initiation of rock removal to evaluate disturbances, if any, resulting from the selected construction techniques.

9.1.2. Temporary Slope Stability

Excavations that are 20 feet deep or less could be constructed using a sloped excavation in accordance with Occupational Safety and Health Administration (OSHA) Standards. OSHA standards provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Although we anticipate the site excavation will be within a zone of relatively horizontal layered bedrock, fracture planes or dipping rock bedding planes could result in unstable excavations. This is especially important if fracture planes or bedding planes are found that dip into the excavation during construction.

However, for planning purposes, we recommend that the OSHA soil “Type C” be used for the colluvial soils and/or decomposed fractured rock and a temporary side slope of 1.5:1 (Horizontal:Vertical), or flatter, be considered for sloped excavations that are 20 feet deep or less. For portions of excavations extending into sound bedrock, we recommend a temporary side slope of 1:1, or flatter. For any temporary bedrock slopes, the contractor should provide an area at the toe of the slope to collect rockfall. Workers should be protected from rockfall as needed by one or more methods such as scaling loose rock, providing mesh on the slope, providing a restricted rockfall collection area, etc.

Temporary excavations that encounter surface seepage may need shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone.

Excavations encountering seepage, if any, should be evaluated on a case-by-case basis. Additional considerations regarding bottom stability and dewatering are provided in Section 9.1.4.

9.1.3. Shoring

Shoring should be used in areas where slopes would exceed the inclinations recommended by OSHA. Shoring systems should be designed by a qualified civil engineer registered in the State of Arizona and be inspected daily during construction by qualified contractor personnel. If excessive movement or slippage is observed, the shoring engineer or contractor should be notified and the shoring system should be strengthened before personnel are allowed to enter the excavation.

Temporary shoring may be needed in areas where cuts are relatively high, and will be open during construction. Temporary earth retention systems may include braced systems, such as trench boxes or shields with internal supports. Cantilever systems like soldier piles and lagging could be considered; however, driving these systems into underlying rock may call for core drilling. Rock bolting, nailing, and/or anchors could also be considered. The choice of shoring method should be left to the discretion of the contractor since economic considerations and/or individual contractor’s experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice and rigidity of support.

9.1.4. Bottom Stability and Dewatering

Some of the proposed below-grade excavations may encounter perched groundwater or saturated units. Groundwater (or surface water accumulation where groundwater is not encountered) may cause the bearing surface to weaken. The base of the excavation should, therefore, be sloped to drain towards a sump or other dewatering equipment. Heavily saturated units or perched groundwater zones, if encountered, may call for more aggressive means of dewatering and consultation with a qualified expert.

Discharge of water from the excavations to natural drainage channels may entail securing a special permit.

9.1.5. Grading, Fill Placement, and Compaction

Vegetation, debris and other unsuitable materials from the clearing operation should be removed from the site and disposed of at a legal dumpsite. Demolition debris, if any, should also be removed from the site and disposed of at a legal dumpsite, and any resulting excavations should be backfilled with engineered fill or soil-cement slurry.

On-site and imported soils that exhibit relatively low plasticity indices and very low to low expansive potential are generally suitable for re-use as engineered fill. Relatively low plasticity indices are defined as a Plasticity Index ([PI] by the American Society for Testing and Materials [ASTM] D 4318) value of 20 or less. Very low to low expansive potential soils are defined as having an Expansion Index ([EI] by ASTM D 4829) of 50 or less. The Atterberg limits test performed on a selected sample resulted in a PI value of 0 (non-plastic). As such, it is our opinion that some of the on-site soils might be suitable for re-use as engineered fill during construction. Additional field sampling and laboratory testing should be conducted by the contractor prior to construction to better define whether unsuitable soils are present at the project site.

In addition, suitable fill should not include construction debris, organic material, or other non-soil fill materials. Rock particles and clay lumps should be less than 4 inches in dimension. Given the nature of the bedrock material, oversized excavation material should be expected. As such, sorting and/or crushing may be needed. Unsuitable material should be disposed of off site or in non-structural areas.

It is our understanding the tank foundation elevation will be such that it is supported on bedrock. However, due to the site topography and our seismic refraction surveys, the underlying competent bedrock surface elevation may vary within the tank footprint. The tank foundation should not bear on any transitional materials (i.e. soil/bedrock contact), and should bear either on a layer of engineered fill or bedrock. If colluvium is encountered at the bearing elevation, it should be removed to competent bedrock and replaced with engineered fill or controlled low strength material (CLSM). It is acceptable to have the proposed tank foundation, and retaining walls, if any, be founded entirely on competent bedrock material. For this case, the bedrock should be evaluated by Ninyo & Moore prior to placement of the structural concrete.

Results of our consolidation (response-to-wetting) testing performed on a shallow soil sample indicated significant collapse upon inundation with water at an approximated light footing load. As such, we recommend an overexcavation of 2 feet below existing grade for any shallow non-tank footings. The overexcavation should extend laterally 2 feet horizontally beyond the edge of footings.

In any areas that will receive fill, the surface should be prepared by scarifying the upper 6, or more inches or until bedrock is encountered, whichever is shallower. Care should be used in applying fill soils to existing slopes. In general, existing slopes or excavated bedrock surfaces under new fills steeper than approximately 5:1 (Horizontal:Vertical) should be “benched” such that a weak plane is not formed and to promote consistent compactive effort across each lift of fill. This zone of scarified material should then be moisture-conditioned and compacted as detailed below, prior to the placement of grade-raise fill.

Grade raise fill should be placed in horizontal lifts no more than approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to a relative compaction of 95 percent, (as evaluated by ASTM D 698), and at a moisture content generally near the laboratory optimum.

We recommend that new grade slabs, and pavements be supported on 12 or more inches of adequately moisture-conditioned and compacted engineered fill or until bedrock is encountered, whichever is shallower. This improved zone can either be improved by overexcavation or scarification. The fill thickness should be measured from the bottom of any base material and should be compacted by appropriate mechanical methods, to 95 percent, relative compaction, in accordance with ASTM D 698 at a moisture content generally above its optimum. The overexcavation below these areas should extend laterally 12 or more inches horizontally beyond the slab/pavement footprint.

Following the overexcavations detailed above, the resulting surface should be carefully evaluated by Ninyo & Moore by visual observations, proof rolling, and/or probing. Ninyo & Moore should also evaluate any areas of soft or wet soils prior to placement of grade-raise fill or other construction. Based on this evaluation, additional excavation and/or remediation may be needed. This additional remediation, if needed, should be addressed by Ninyo & Moore during the earthwork operations and could consist of additional overexcavation or reworking of the exposed surface.

The fill thickness should be compacted by appropriate mechanical methods to 95 percent relative compaction, in accordance with ASTM D 698 at a moisture content generally near optimum.

For estimating purposes, an earthwork (shrinkage) factor of about 10 to 20 percent for the on-site colluvium and a bulking factor (swell) of about 5 to 10 percent for the sound bedrock is anticipated.

9.1.6. Imported Fill Material

Imported fill, if utilized, should consist of granular material with a very low or low expansion potential. Import material in contact with ferrous metals or concrete should preferably have low corrosion potential (minimum resistivity more than 2,000 ohm-cm, chloride content less than 25 parts per million [ppm]). Soluble sulfate content should preferably be less than 0.1 percent. Ninyo & Moore should evaluate such materials and details of their placement prior to importation.

9.1.7. Permanent Fill Slopes

Permanent fill slopes that are protected from erosion (by riprap, shotcrete, gabions, etc.) for this project can be sloped at an angle of 2:1 (horizontal:vertical) for fills placed behind the new tank. Generally, slopes of 3:1 (horizontal:vertical) or flatter are used for vegetated slopes to help promote vegetation growth and for ease of maintenance.

9.2. Pipeline Considerations

We recommend that the new underground utilities be supported on 4 inches or more of granular bedding material such as sand and gravel, or crushed rock meeting the requirements in MAG. This bedding/pipe-zone backfill should extend 1 foot above the pipe crown (Figure 3). Care should be taken not to allow voids to form beneath the pipe, (i.e., the pipe haunches should be continuously supported), to avoid damaging the pipelines. This may involve fill placement by hand or small compaction equipment. The bedding/pipe-zone should be placed in horizontal lifts no more than approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to a relative compaction of 95 percent, (as evaluated by ASTM D 698), and at a moisture content generally near the laboratory optimum.

We recommend that trench backfill should not include construction debris, organic material, or other non-soil fill materials. Rock particles and clay lumps should be less than 4 inches in dimension. Unsuitable material should be disposed of off site or in non-structural areas. Ninyo & Moore should evaluate such materials and details of their placement.

The trench backfill soils should be compacted to 95 percent of the maximum dry density at a moisture content generally near optimum moisture, as evaluated by ASTM D 698. The lift thickness may vary depending on the type of compaction equipment used. To reduce potential settlements resulting from consolidation of the backfill, we recommend that backfill should generally be placed in lifts not exceeding 8 inches in loose thickness. We recommend using hand-operated compaction equipment and 4-inch thick loose lifts adjacent to concrete walls and in confined areas.

When backfilling, care should be taken to fill voids with compacted material so that excessive settlement of the backfill will not occur. Settlement can be mitigated by backfilling with granular material that is easy to compact or by using a Controlled Low Strength Material (CLSM), sometimes referred to as Controlled Density Fill. More detailed recommendations regarding the use of CLSM are provided in Section 9.3.

The modulus of soil reaction (E´) is used to characterize the stiffness of soil backfill placed at the sides of buried pipelines for the purpose of evaluating deflection caused by the weight of the backfill over the pipe. For pipelines at a depth of up to 10 feet bgs, we recommend using an E´ value of 1,200 pounds per square inch (psi). This value assumes that the bedding and trench backfill materials are selected and compacted according to the recommendations provided herein.

9.3. Controlled Low Strength Material

It is our opinion that the pipe bedding material may be composed of controlled low strength material (CLSM). Also, the trench backfill zone can be filled with either CLSM or acceptable on-site soils. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. The use of CLSM has some advantages:

- A narrower trench can be used, thereby minimizing the quantity of soil to be excavated and possibly reducing disturbance adjacent facilities;
- The support given to the pipe in the bedding zone is generally better, and increased values of modulus of soil reaction (E'=3000 psi) can be used to design the pipe;
- Because little compaction is needed to place CLSM, there is less risk of damaging the pipe; and
- CLSM can be batched to flow into irregularities in the trench bottom and walls.

The CLSM design mix should be in accordance with the MAG. Additional mix design information can be provided upon request. The 28-day strength of the material should be no less than 50 pounds per square inch (psi) and no more than 120 psi.

Buoyant or uplift forces on the piping should be considered when using CLSM and prudent construction techniques may result in multiple pours to avoid inducing excessive uplift forces. The construction methods should not allow the pipe to displace laterally or vertically during placement of CLSM. Sufficient time should be provided to allow the CLSM to cure before placing additional lifts of CLSM or trench backfill.

9.4. Seismic Design Considerations

Based on a Probabilistic Seismic Hazard Assessment for the conterminous United States, issued by the USGS (2002 data), the site is located in a zone where the peak ground accelerations having 10 and 2 percent probability of being exceeded in 50 years are 0.141g, and 0.328 g, respectively. These ground motion values are calculated for “rock” sites, which correspond to a shear-wave velocity of approximately 2,500 to 5,000 feet per second in approximately the top 100 feet bgs. Different soil or rock types may amplify or de-amplify these values. The proposed improvements should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with International Building Code (ICC, 2009) guidelines and mapped spectral acceleration parameters (USGS, 2011).

Table 1 – 2009 International Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	B
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.0
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.329g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.099 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.329 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.099 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.219 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.066 g

9.5. Foundations

We understand that the below grade tank structure will consist of retaining walls supported on spread or continuous footings and a slab-on-grade. Additionally, we understand that the other structures will be supported by slabs-on-grade or shallow footings near the ground surface. Accordingly, spread or continuous footings may be used and should be supported on either a zone of engineered fill extending 2 or more feet below existing grade, or on undisturbed sound bedrock, as described in Section 9.1.5. Continuous footings should have a width of 16 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. For the design of the footings, an allowable bearing pressure of up to of 2,500 psf can be used for foundations bearing on engineered fill and 5,000 psf for foundations bearing on sound bedrock. Total and differential settlement of up to about 1 inch and ½-inch or less, respectively, may occur. Distortions of about 1/2- inch (vertical) over 20 feet (horizontal) or less are possible.

Foundations subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.35 (total frictional resistance equals the coefficient of friction multiplied by the dead load). A passive resistance value of 300 psf per foot of depth up to 3,000 psf may be used. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided that the passive resistance does not exceed two-thirds 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).

9.6. Floor Slabs

The design of the floor slabs is the responsibility of the structural engineer. However, from a geotechnical standpoint, we recommend that concrete floor slabs have a thickness of 4 inches or more and be reinforced with steel rebar. Placement of the reinforcement in the slabs is vital for satisfactory performance. The slabs should be underlain by 4 or more inches of moist clean sand and/or minus 3/4-inch gravel. Below the sand should be 12 inches of engineered fill as described in Section 9.1.5.

The floor slab should either be constructed so that it “floats” independent of the foundations or be designed to be structurally connected to the foundations as evaluated by the structural engineer. Soils underlying the slabs should be moisture-conditioned and compacted in accordance with the recommendations contained in Section 9.1.5. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.7. Lateral Earth Pressures

Walls that are not restrained from movement at the top and have a level backfill behind the wall may be designed using an “active” equivalent fluid unit weight of 35 pounds per cubic foot (pcf) for drained conditions. This value assumes compaction within about 5 feet of the wall will be accomplished with relatively light compaction equipment, and that very low to low expansive backfill will be placed behind the wall. This value also assumes that the retaining walls will have a height less than 12 feet. Retaining walls should also be designed to resist a surcharge pressure of 0.35q. The value for “q” represents the pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

The “at-rest” earth pressure against walls that are restrained at the top or braced so that they cannot yield, and with level backfill, may be taken as equivalent to the pressure exerted by a fluid weighing 55 pcf for drained conditions. Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.5q. The value for “q” represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

For “passive” resistance to lateral loads, we recommend that an equivalent fluid weight of 300 pcf be used up to a value of 3,000 psf for drained conditions. This value assumes that the ground is horizontal for a distance of 10 feet or more behind 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 be neglected when calculating passive resistance.

Measures should be taken so that moisture does not build up behind retaining walls. Back drainage measures should include free-draining backfill material and perforated drainpipes or weep holes. Drainpipes should outlet away from structures, and retaining walls should be waterproofed in accordance with the recommendations of the project civil engineer or architect. Retaining wall drainage guidelines are presented on Figure 4. To reduce the potential for water- and sulfate/salt-related damage to the retaining walls, particular care should be taken in the selection of the appropriate type of waterproofing material to be utilized and in the application of this material.

9.8. Pavements

No traffic information was provided during the writing of this report. However, we assume that traffic will consist of maintenance vehicles and occasional heavy trucks. For the paved areas, we assume that asphalt concrete (AC) will be utilized. The pavement section given below is assumed to bear on imported or on-site soils with an average soil R-value of 20 or more.

An asphalt pavement section consisting of 3 inches of plant-mix asphalt (per MAG Section 710) over 6 inches of graded aggregate base (AB) (per MAG Section 702) can be considered in the standard duty pavement areas. However, for areas that will experience heavy truck traffic, an asphalt pavement section consisting of 4 inches of plant-mix asphalt (per MAG Section 710) over 9 inches of graded AB can be utilized.

Portland cement concrete pavements (PCCP) are recommended for areas that will experience regular truck traffic, main ingress and egress areas, and in areas where vehicles will be turning or loading (e.g., adjacent to site infrastructure). PCCP in heavy traffic areas should have a thickness of 8 inches or more, with edges thickened to 10 inches. In parking areas (if any are planned) not subject to truck traffic, the concrete pavement thickness can be reduced to 6 inches, with edges thickened to 8 inches.

Concrete pavements should have longitudinal and transverse joints that meet the applicable requirements of the jurisdictional specifications. Concrete pavements should be underlain by 4 inches or more of AB that meets jurisdictional specifications, or if jurisdictional specification are not applicable, Section 702 of the MAG specifications.

The minimal reinforcement for the concrete pavement areas should be No. 4 reinforcing bars placed 18 inches on-center (each way) in the middle one-third of slab height. The structural engineer may decide that additional reinforcement is needed.

For both the PCC and asphalt pavements given above, we recommend the underlying subgrade soils be prepared as described in Section 9.1.5. AB material should be compacted to a relative compaction of 100 percent of the maximum dry density, as evaluated by ASTM D 698, at a moisture content near optimum.

9.9. Concrete Flatwork

To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints and/or reinforcement steel at appropriate spacing as designed by the structural engineer. We recommend that exterior concrete flatwork be supported on 12 or more inches of moisture-conditioned and compacted engineered fill as described in Section 9.1.5 of this report. Positive drainage should be established and maintained adjacent to flatwork.

9.10. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of samples obtained during our subsurface evaluation that were considered representative of soils at the subject site.

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 236b, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 736, respectively. The results of the corrosivity tests are presented in Appendix B.

The soil pH value of the selected representative sample was 8.4, which is considered to be slightly alkaline. The minimum electrical resistivity measured in the laboratory was 3,352 ohm-cm, which is not considered corrosive to ferrous materials. The chloride content of the samples tested was 10 ppm, which does not represent a corrosive environment to ferrous materials. The soluble sulfate content of the soil sample tested was 0.001 percent by weight, which is considered to represent negligible sulfate exposure for concrete.

Notwithstanding the results of the laboratory testing mentioned above, we recommend special consideration be given to the use of heavy gauge, corrosion protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further assistance.

9.11. Concrete

Laboratory chemical tests performed on selected samples of on-site soils indicated a sulfate content up to 0.001 percent by weight. Based on the following American Concrete Institute (ACI) table, the on-site soils are considered to have a negligible sulfate exposure to concrete.

Table 2 – 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.

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.

Notwithstanding the sulfate test results, and given the type of facility at the site, we recommend the use of Type II or Type V cement for construction of concrete structures at this site. Additionally, pozzolan or admixtures designed to increase sulfate resistance may be considered.

The concrete should 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.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that for slabs-on-grade, the concrete be placed with a slump in accordance with ACI Table 5.2.1 of Section 302.1R of “Guidelines for Floor and Slab Construction,” or ACI Table 2.2 of Section 332R in “Guidelines for Residential Cast-in-Place Concrete

Construction." If a higher slump is needed for screeding and leveling, a super plasticizer is recommended to achieve the higher slump without changing the recommended water to cement ratio. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in slabs in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for slabs-on-grade and foundations be in accordance with IBC 1907.7.1. The structural engineer should be consulted for additional concrete specifications.

9.12. Site Drainage

Surface drainage should be provided to divert water away from the structures (below- and above-ground) and off of paved surfaces. Surface water should not be permitted to drain toward the structures or to pond adjacent to footings or on pavement areas. Positive drainage is defined as a slope of 2 or more percent for a distance of 5 or more feet away from the structures. Roof gutters should be installed on buildings. Downspouts should discharge to drainage systems away from structures, pavements, and flatwork. Soil improvements below the new grade slabs and pavement sections should be sloped to drain beyond the edges of these areas.

9.13. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, the civil engineer, Ninyo & Moore, 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.

9.14. 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 overexcavation, to evaluate the suitability of proposed borrow 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 that 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.

10. LIMITATIONS

The field evaluation, laboratory testing, 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 re-use of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11. REFERENCES

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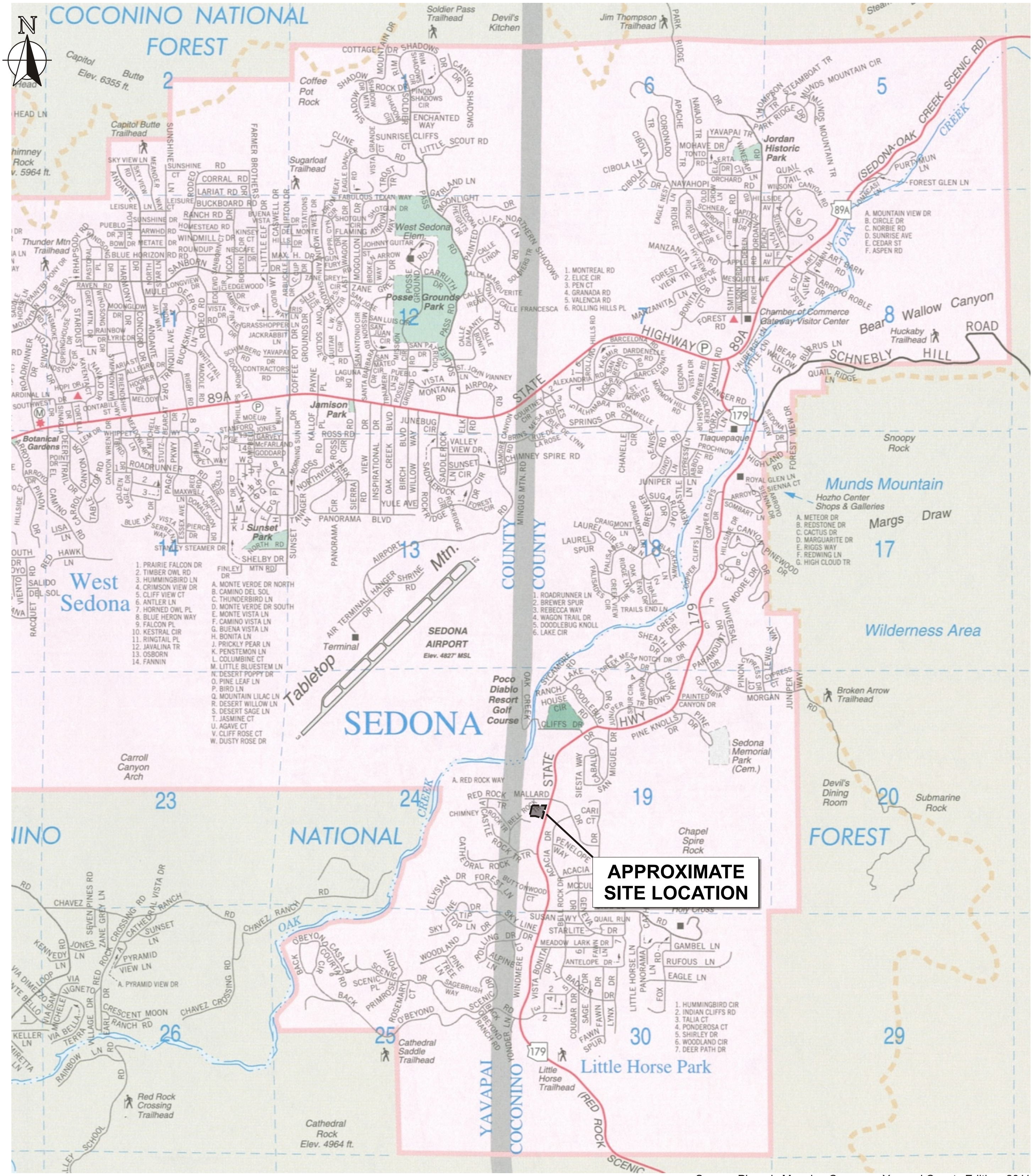


Source: Phoenix Mapping Company, Yavapai County Edition, 2011.

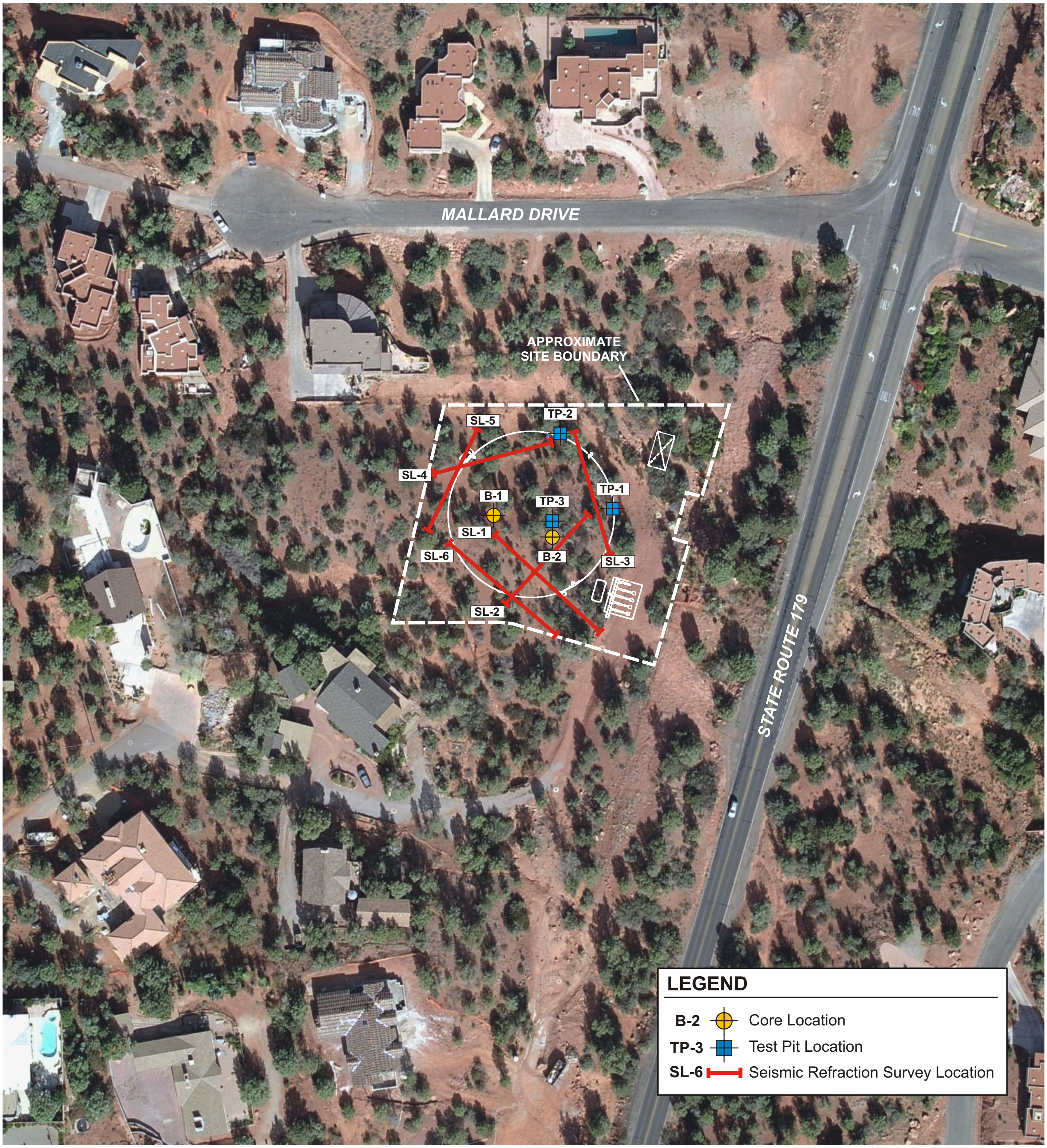
0 3300

Approximate Scale:
1 inch = 3300 feet

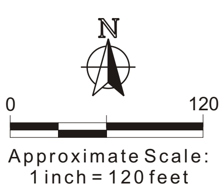
Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore		SITE LOCATION	FIGURE 1
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	

file no: 3971vmap1016



Source: Pictometry, 05/11.

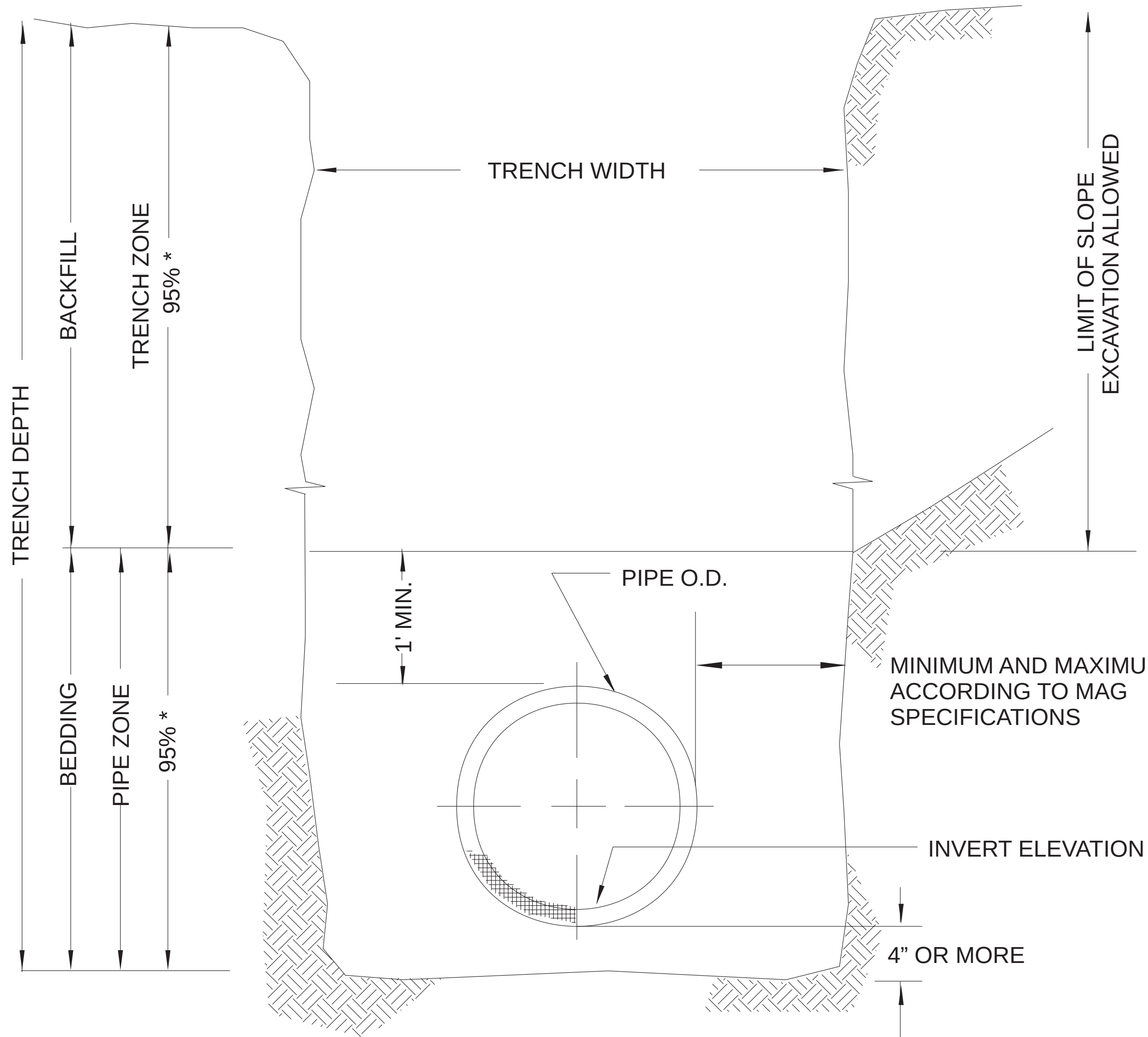


Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore

		EXPLORATION LOCATIONS	FIGURE
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	2

file no: 3971srl1016



SECTION

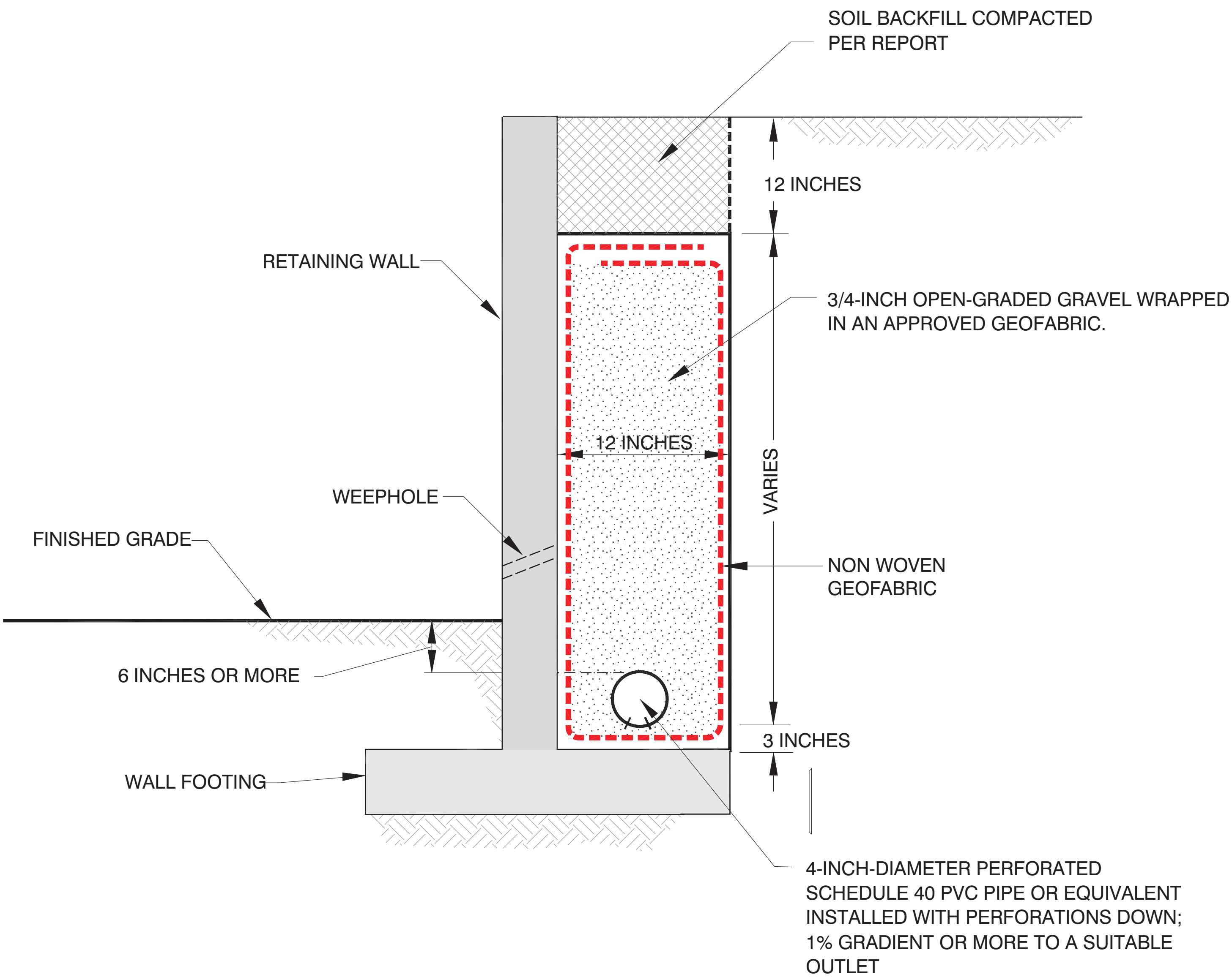
NOTE:

* INDICATES MINIMUM RELATIVE COMPACTION
(SEE REPORT FOR DETAILS).

NOT TO SCALE

Ninyo & Moore	PIPE BEDDING GUIDELINES		FIGURE
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	3

file no: 3971pbd1112a



NOTE:

IN LIEU OF THE WRAPPED OPEN-GRADED GRAVEL, A GEOCOMPOSITE DRAINAGE MAT ATTACHED TO THE WALL AND DISCHARGING INTO THE DRAIN PIPE OR WEEP HOLES MAY BE CONSIDERED.

NOT TO SCALE

Ninyo & Moore	RETAINING WALL DRAINAGE GUIDELINES	FIGURE
PROJECT NO: 603971001 / DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	4

file no: 3971rwd1112a

APPENDIX A

TEST PIT AND CORE 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 of representative earth materials were obtained from the exploratory test pits. The samples were bagged 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 methods.

The Modified Split-Barrel 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 the weight of a hammer or the Kelly bar of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the test pit logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

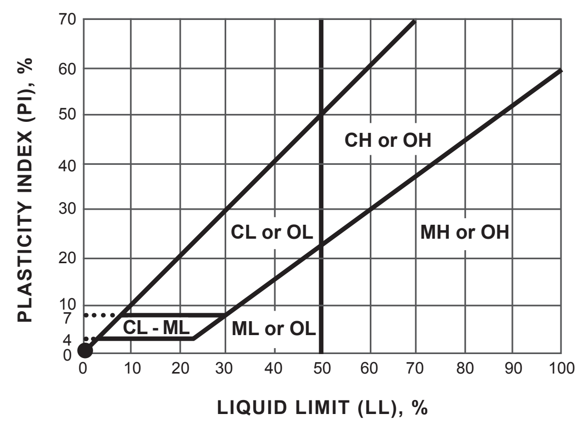
SOIL CLASSIFICATION CHART PER ASTM D 2488

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 clay
		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: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: 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: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: 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

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

Ninyo & Moore

Explanation of Test Pit, Core, Trench and Hand Auger Log Symbols

PROJECT NO.	DATE

EXCAVATION LOG EXPLANATION SHEET

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	
0						SM	FILL:
							Bulk sample.
						ML	Dashed line denotes material change.
							Drive sample.
1							Sand cone performed.
							Seepage
							Groundwater encountered during excavation.
							No recovery with drive sampler.
2							Groundwater encountered after excavation.
							Sample retained by others.
				xx/xx			Shelby tube sample. Distance pushed in inches/length of sample recovered in inches
3							No recovery with Shelby tube sampler.
						SM	ALLUVIUM:
							Solid line denotes unit change.
							Attitude: Strike/Dip
4							b: Bedding
							c: Contact
							j: Joint
							f: Fracture
							F: Fault
							cs: Clay Seam
5							s: Shear
							bss: Basal Slide Surface
							sf: Shear Fracture
							sz: Shear Zone
							sbs: Sheared Bedding Surface
							The total depth line is a solid line that is drawn at the bottom of the excavation log.

SCALE: 1 inch = 1 foot

FIGURE

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TEST PIT LOG

CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

PROJECT NO.	DATE
603971001	10/16

DATE EXCAVATED 11/13/12 TEST PIT NO. TP-1

GROUND ELEVATION -- LOGGED BY DM

METHOD OF EXCAVATION JD 310 SG Backhoe, 14" Bucket

LOCATION Sedona, Arizona

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0–2	X			3.0	87.5	GM	COLLUVIUM: Red, dry, very dense, fine to coarse silty GRAVEL. Backhoe refusal on bedrock.
2							Total Depth = 2 feet. (Refusal) Groundwater not encountered during excavation. Backfilled on 11/13/12 promptly after completion of excavating. Note: Groundwater, though not encountered at the time of excavating, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
4							
6							
8							
10							

SCALE = 1 in./2 ft.

FIGURE A-1

Ninyo & Moore

TEST PIT LOG

CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

PROJECT NO.	DATE
603971001	10/16

DATE EXCAVATED 11/13/12 TEST PIT NO. TP-2

GROUND ELEVATION -- LOGGED BY DM

METHOD OF EXCAVATION JD 310 SG Backhoe, 14" Bucket

LOCATION Sedona, Arizona

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	COLLUVIUM: Red, dry to damp, medium dense, silty SAND; few fine to coarse gravel. Backhoe refusal on bedrock.
2							Total Depth = 2 feet. (Refusal) Groundwater not encountered during excavation. Backfilled on 11/13/12 promptly after completion of excavating. Note: Groundwater, though not encountered at the time of excavating, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
4							
6							
8							
10							

SCALE = 1 in./2 ft.

FIGURE A-2

Ninyo & Moore

TEST PIT LOG

CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

PROJECT NO.	DATE
603971001	10/16

DATE EXCAVATED 11/13/12 TEST PIT NO. TP-3
GROUND ELEVATION -- LOGGED BY DM
METHOD OF EXCAVATION JD 310 SG Backhoe, 14" Bucket
LOCATION Sedona, Arizona

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■ (0–3 ft)					GM	COLLUVIUM: Red, dry to damp, very dense, fine to coarse silty GRAVEL.
1–2		■ (1–2 ft)		2.7	112.1		
3.5							Backhoe refusal on bedrock.
							Total Depth = 3.5 feet. (Refusal) Groundwater not encountered during excavation. Backfilled on 11/13/12 promptly after completion of excavating.
							Note: Groundwater, though not encountered at the time of excavating, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

Depth scale: 0, 2, 4, 6, 8, 10

SCALE = 1 in./2 ft.

DATE DRILLED 10/06/16	CORE NO. B-1	
GROUND ELEVATION 4,194'± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING Core Rig D-120	DRILLER D&S Drilling, Inc.	
BASELINE STATION N/A	LATERAL OFFSET N/A	
SAMPLED BY DCW	LOGGED BY DCW	REVIEWED BY FAR

DEPTH (feet)	Samples: Driven	BLOWS/FOOT	INTERVAL CORED (feet)	RECOVERY (%)	RQD (%)	DRILLING RATE (minutes per ft.)	FRACTURE DIP (degrees)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0									GP	COLLUVIUM: Dry, dense, fine to coarse GRAVEL with sand; trace silty.
										SCHNEBLY HILL FORMATION: Reddish brown; moist; soft; highly weathered SANDSTONE; very intensely fractured. No recovery; sandstone clast in shoe.
10										
			5	80	63	11				@ 11' to 16': Reddish brown; moist; moderately soft; moderately weathered; intensely to moderately fractured.
			5	95	92	10				@ 16' to 21':Reddish brown; moist; moderately hard; moderately weathered; moderately to slightly fractured.
20										
			5	100	95	7				@ 21' to 26': Reddish brown; moist; moderately hard; moderately weathered; slightly fractured.
			5	90	70	5				@ 26' to 31': Reddish brown, moist, moderately hard; slightly weathered; slightly fractured.
30										
										Total Depth = 31 feet. Groundwater not encountered during drilling. Backfilled on 10/06/16 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.

Ninyo & Moore

CORE LOG

MALLARD DRIVE AND HIGHWAY 179
SEDONA, ARIZONA

PROJECT NO.	DATE	FIGURE
603971001	10/16	A-4

DATE DRILLED	6	CORE NO.	B-2
GROUND ELEVATION	4,194'± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	Core Rig D-120	DRILLER	4,194'± (MSL)
BASELINE STATION	N/A	LATERAL OFFSET	N/A
SAMPLED BY	DCW	LOGGED BY DCW	REVIEWED BY FAR

DEPTH (feet)	Samples: Driven	BLOWS/FOOT	INTERVAL CORED (feet)	RECOVERY (%)	RQD (%)	DRILLING RATE (minutes per ft.)	FRACTURE DIP (degrees)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0									GP	COLLUVIUM: Reddish brown, dry, dense, fine to coarse GRAVEL with sand.
										SCHNEBLY HILL FORMATION: Reddish brown; moist; soft; high weathered SANDSTONE; very intensely fractured.
10										No recovery; sandstone clast in shoe.
										@ 16' to 22': Reddish brown; moist; moderately hard; slightly weathered; moderately to slightly fractured.
20			5	40	47	3				
										@ 22' to 27':
			5	90	85	5				
										@ 27' to 30': Reddish brown, moist, moderately hard; moderately weathered; intensely to moderately fractured.
			3	20	17	4				
30										Total Depth = 30 feet. Groundwater not encountered during drilling. Backfilled on 10/06/16 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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CORE LOG

MALLARD DRIVE AND HIGHWAY 179
SEDONA, ARIZONA

PROJECT NO.	DATE	FIGURE
603971001	10/16	A-5

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory test pits in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory test pits were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory test pits in Appendix A.

Gradation Analysis

Gradation analysis tests were performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (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 D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure B-2.

Consolidation Test

One consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. 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 test are summarized on Figure B-3.

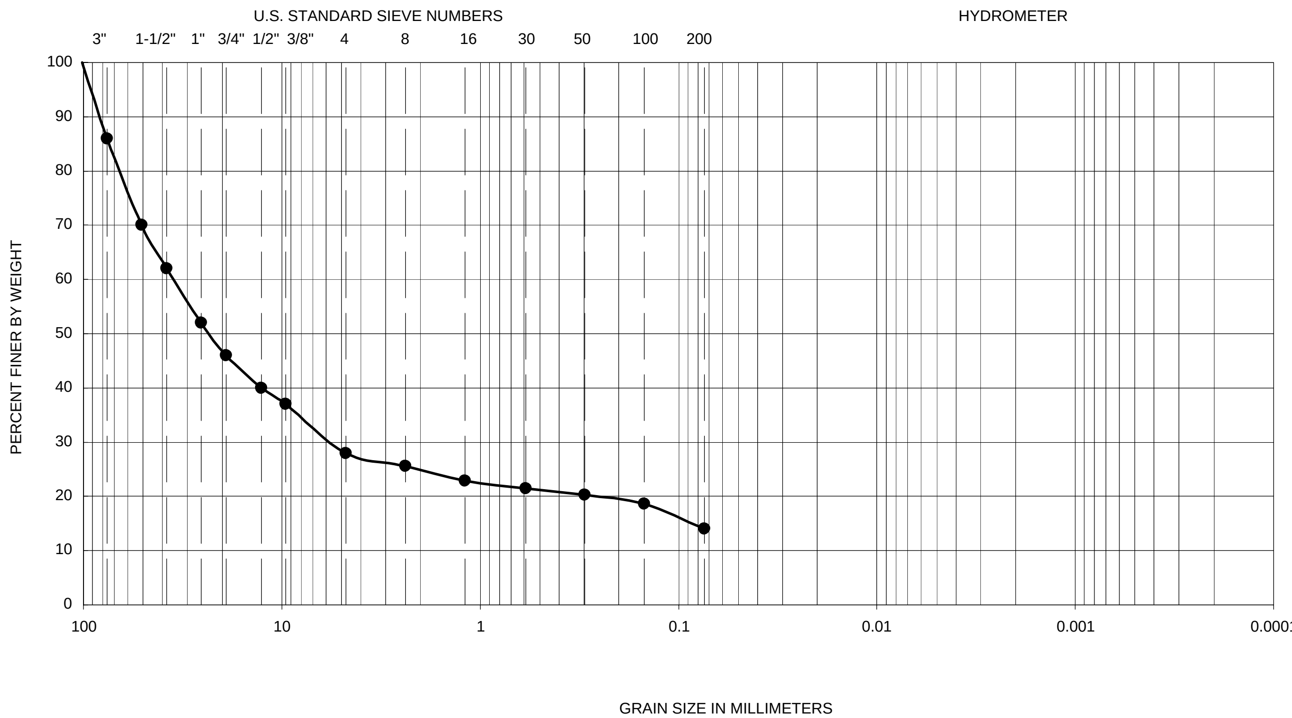
Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative sample in general accordance with Arizona Test Method 236b. The soluble sulfate and chloride content of a selected sample was evaluated in general accordance with Arizona Test Method 733 and Arizona Test Method 736, respectively. The test results are presented on Figure B-4.

Unconfined Compressive Tests

The analysis includes unconfined compressive strength testing of selected rock core samples in general accordance with ASTM D 2166. The test results are presented on Figure B-5.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



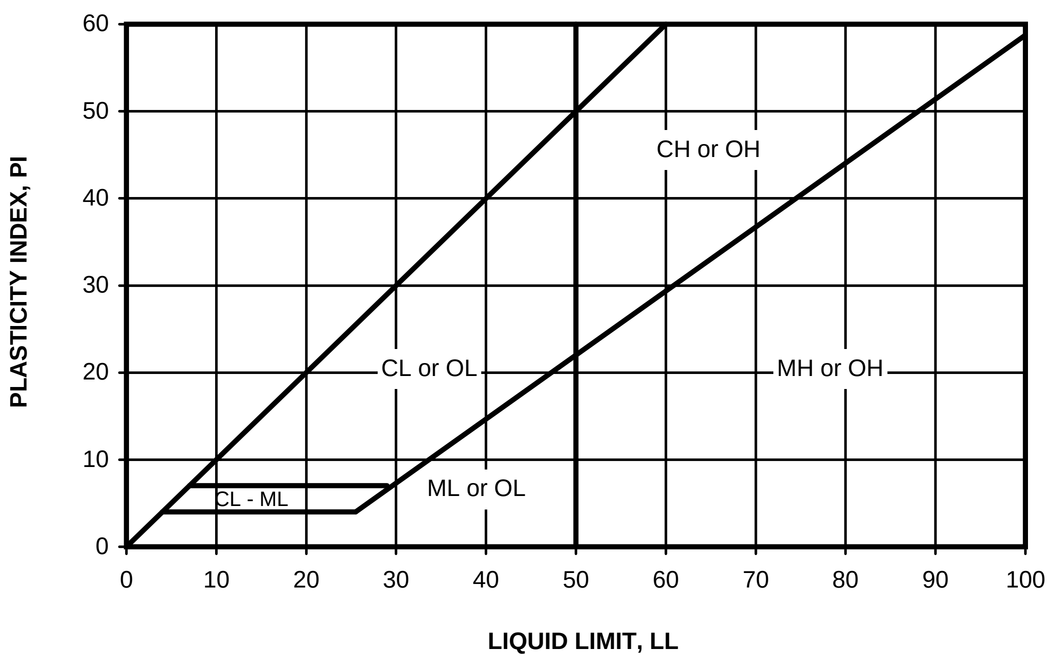
Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	TP-1	0-2	--	--	NP	--	--	--	--	--	14	GM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHAPEL HILLS WATER CAMPUS MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	B-1
603971001	10/16		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
	TP-1	0-2	--	--	NP	ML	GM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore

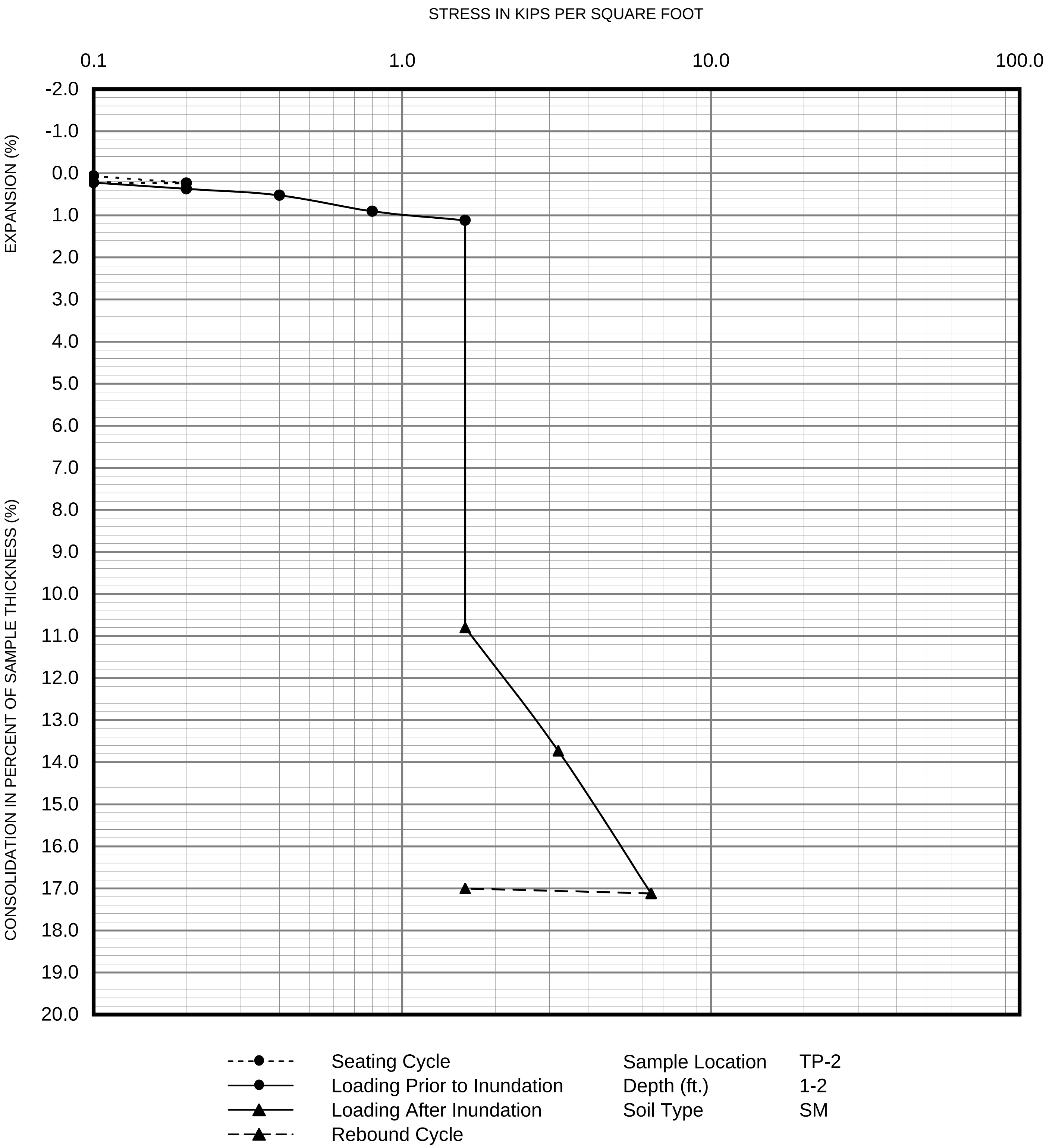
ATTERBERG LIMITS TEST RESULTS

PROJECT NO.	DATE
603971001	10/16

CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

FIGURE

B-2



Sample Location	TP-2
Depth (ft.)	1-2
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore

CONSOLIDATION TEST RESULTS

PROJECT NO.	DATE
603971001	10/16

CHAPEL HILLS WATER CAMPUS
MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

FIGURE

B-3

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
TP-1	0-2	8.4	3,352	12	0.001	10

[1] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 236b
[2] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 733
[3] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 736

PROJECT NAME: MALLARD DRIVE AND STATE ROUTE 179
PROJECT LOCATION: SEDONA, ARIZONA
PROJECT NUMBER: 603971001

DATE SAMPLED : 10/6/2016
SAMPLED BY: FAR
DATE TESTED: 10/18/2016

ASTM D 2166

CORE #	LENGTH (in)	DIAMETER (in)	LENGTH/DIAMETER CORRECTION	LOAD (lbs)	COMPRESSIVE STRENGTH (psi)
B-1 (12-13')	3.76	1.88	1.000	22700	8180
B-1 (20-21')	3.77	1.88	1.000	16730	6030
B-1 (30-31')	3.76	1.88	1.000	41950	15110
B-2 (18-19')	3.76	1.88	1.000	11240	4050
B-2 (22-23')	3.75	1.88	0.999	19540	7030

file no: 3971cht1016a

Ninyo & Moore

COMPRESSIVE STRENGTH OF DRILLED CORES

PROJECT NO: 603971001

DATE: 10/16

GEOTECHNICAL EVALUATION
CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

FIGURE
B-5

APPENDIX C

GEOPHYSICAL SEISMIC REFRACTION SURVEY

APPENDIX C

GEOPHYSICAL SEISMIC REFRACTION SURVEY

Ninyo and Moore personnel conducted seismic refraction surveys at the site on November 15, 2012 and December 13, 2012, to evaluate the approximate depth to bedrock and rippability characteristics of the near surface materials. The seismic refraction data were collected with a SmartSeis S12, high performance exploration seismograph and 12 vertical component geophones. A 16-pound hammer and metal plate were used as the seismic wave source. A total of six seismic refraction traverses were performed, and the approximate locations of the traverses are depicted on Figure 2.

The seismic refraction method uses first-arrival times of refracted seismic waves in units of milliseconds to evaluate the thicknesses and seismic velocities of subsurface layers. Seismic waves generated by hammer at the ground surface at a given "shot" point are refracted at boundaries separating materials of contrasting material velocities. These refracted seismic waves are then detected by a series of surface geophones and recorded with a seismograph. Each hammer shot is recorded as time zero, and the elapsed time in milliseconds that the seismic compressional wave (P-wave) signals take to travel to each geophone are recorded. This information is used in conjunction with the known shot-to-geophone horizontal distances, to obtain thickness and velocity information about the subsurface materials. Horizontal distances between the geophones are calculated using topographic corrected distances.

The refraction method requires that subsurface velocities (and therefore material density) increase with depth. A layer having a velocity lower than that of the layer which overlies it will not be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. This is known as a "velocity inversion" problem. In addition, relatively significant lateral variations in velocity, such as those which occur at buried discontinuous caliche deposits, cemented soils that are surrounded by lower velocity soils, or high velocity rock layers overlying low velocity rock layers (such as higher velocity sandstone layers overlying relatively lower velocity siltstone or shale layers) can also result in the misinterpretation of the subsurface conditions when using this method. Relatively near surface accumulations of significant amounts of nested cobbles or boulders, caliche deposits, and/or cemented rock layers, or cemented soils can create velocity inversion problems as these materials generally have a higher velocity than the surrounding materials, which will often mimic bedrock velocities.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass for each detected layer. Localized areas of differing composition, texture, or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The following rippability chart (Table C-1) is based on our experience with similar materials. It assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that soil and rock characteristics can play a significant role in determining excavation rates and rippability. In addition, where excavations encounter or penetrate weathered or fresh bedrock or cemented bedrock, rock characteristics, such as depth of and degree of weathering, degree of cementation (if any), the presence or absence of fractures and/or joints, and fracture/joint spacing and orientation, also play a significant role in determining rock rippability. These soil and rock characteristics may also vary with location and depth.

Table C-1 – Qualitative Rippability Classification

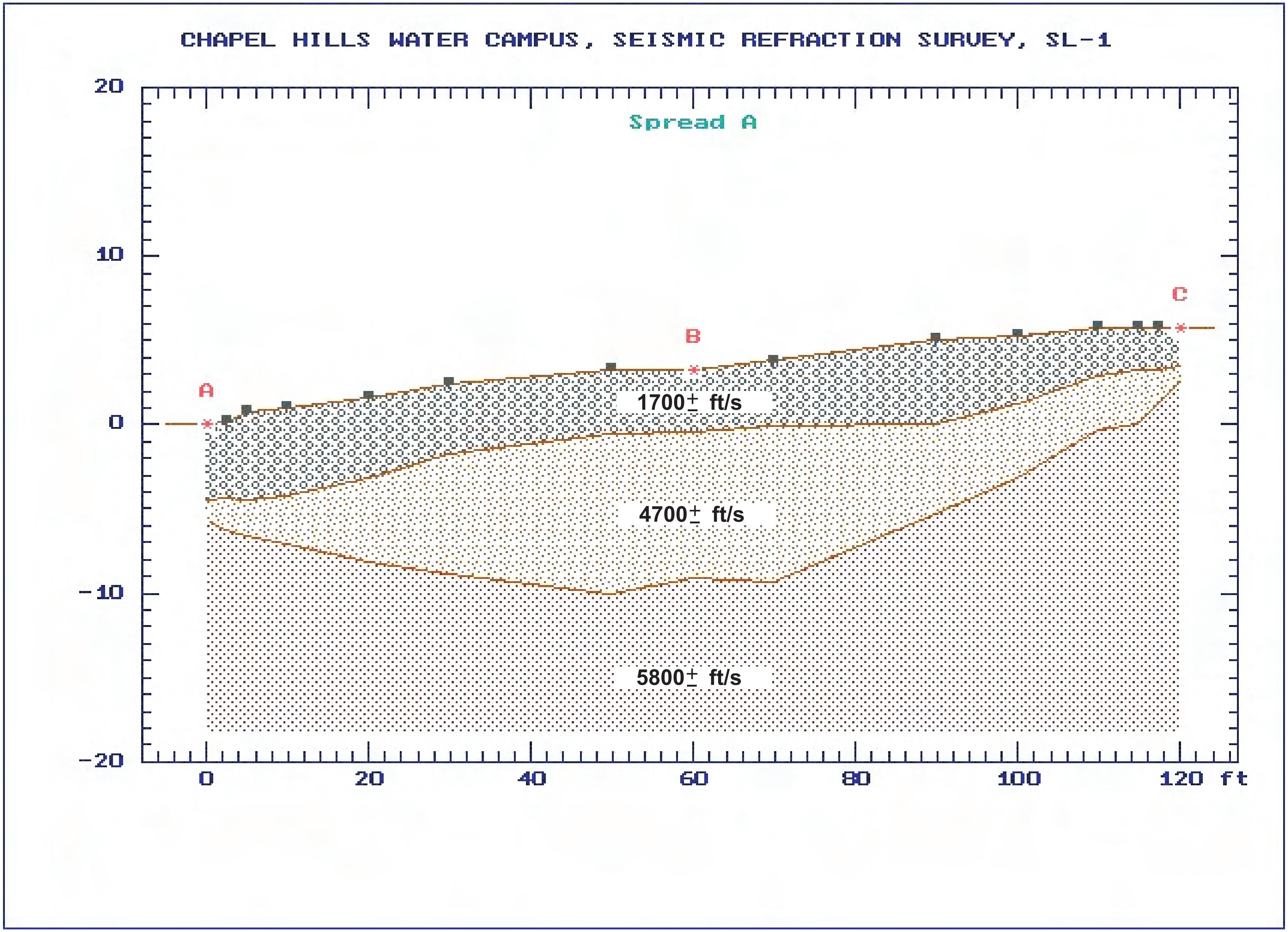
0 to 2000 ft/s	Easy Ripping
2000 to 4000 ft/s	Moderate Ripping
4000 to 5500 ft/s	Difficult Ripping, Possible Blasting
5500 to 7000 ft/s	Very Difficult Ripping, Probable Blasting
Greater than 7000 ft/s	Blasting Generally Required

This classification does not assess the use of alternative or unconventional excavation equipment or techniques, such as drilled piers, shafts, tunneling, etc. For trenching and other relatively narrow excavation operations, the rippability figures should be scaled downward. For example, velocities as low as 3,200 feet per second might indicate difficult ripping or possible blasting during trench excavation or drilled shaft excavation operations. In addition, the presence of cobbles and boulders, which can be troublesome in trench and shaft excavations, should be anticipated. It should be noted that variations in erosion rates and fracture density and spacing may have caused variable depths to bedrock and/or cemented soils that might not be detected by our methods. It is also possible that a spatially varying presence of cemented soils, sandstone, shale, siltstone, and limestone bedrock, in addition to boulders and cobbles, might be encountered in areas of the site. The above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. Table D-2 lists the average velocities and depths calculated from the seismic refraction traverses conducted during this evaluation. Our seismic refraction layer profiles are presented in Figures D-1 through D-6.

It should also be noted that, as a general rule of thumb, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line. The lengths of our six seismic refraction survey lines are listed, with our interpretations, in Table D-2.

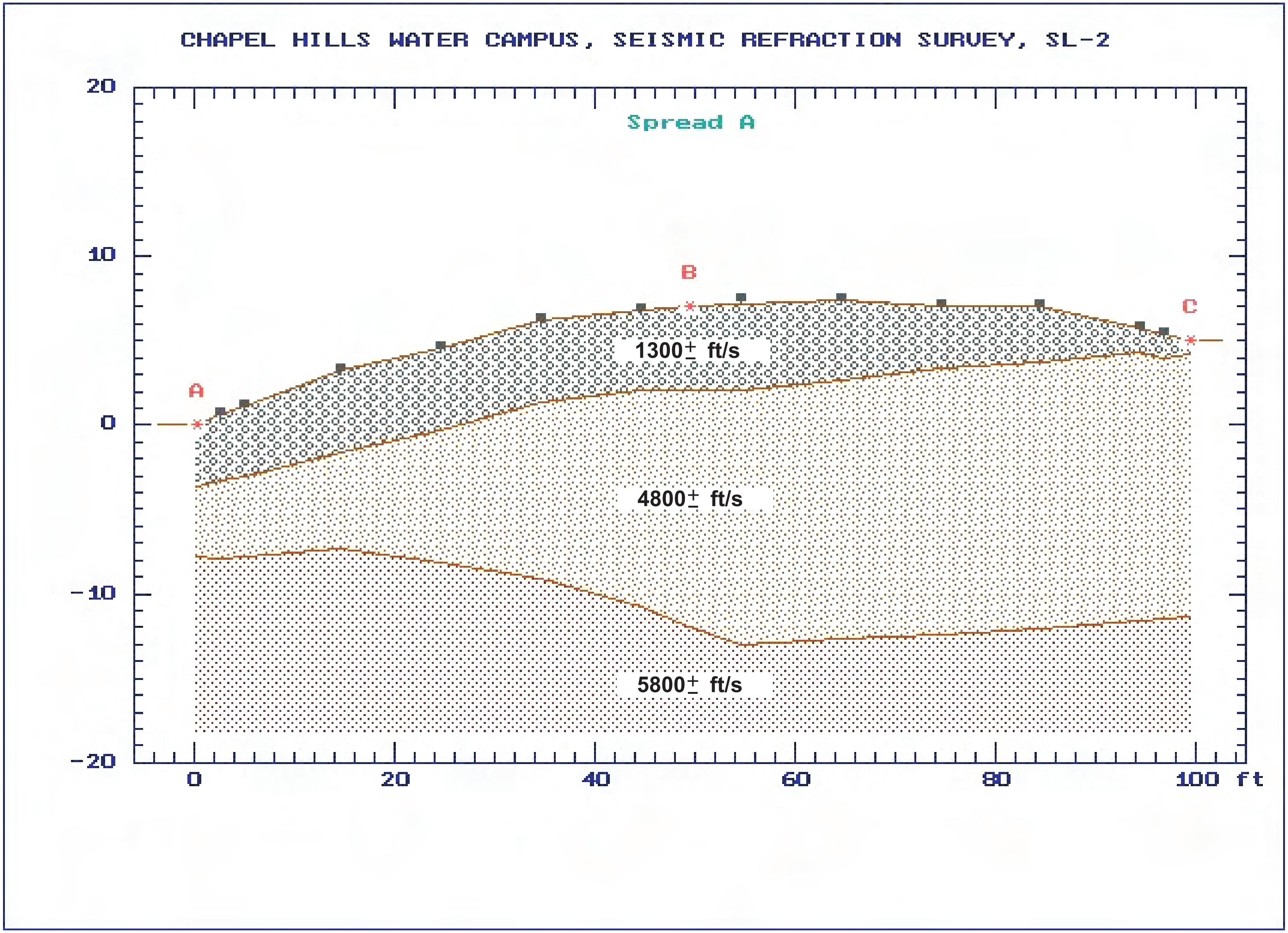
Table C-2 – Seismic Refraction Results

Traverse No. And Length	Approximate Velocity Feet/Second	Approximate Depth to Bottom of Layer (range in feet below ground surface)	Qualitative Rippability
SL-1 120 feet	V1 = 1,700 V2 = 4,700 V3 = 5,800	2-6 6-13 --	Easy Ripping Difficult Ripping, Possible Blasting Very Difficult Ripping, Probable Blasting
SL-2 100 feet	V1 = 1,300 V2 = 4,800 V3 = 5,800	<1-5 7-20 --	Easy Ripping Difficult Ripping, Possible Blasting Very Difficult Ripping, Probable Blasting
SL-3 100 feet	V1 = 1,500 V2 = 3,600 V3 = 5,300	2-4 13-18 --	Easy Ripping Moderate Ripping Very Difficult Ripping, Probable Blasting
SL-4 120 feet	V1 = 1,100 V2 = 4,100 V3 = 5,900	2-4 20-25 --	Easy Ripping Difficult Ripping, Possible Blasting Very Difficult Ripping, Probable Blasting
SL-5 90 feet	V1 = 1,500 V2 = 4,500 V3 = 5,500	1-5 9-14 --	Easy Ripping Difficult Ripping, Possible Blasting Very Difficult Ripping, Probable Blasting
SL-6 120 feet	V1 = 1,200 V2 = 3,100 V3 = 5,800	1-7 8-14 --	Easy Ripping Moderate Ripping Very Difficult Ripping, Probable Blasting



Ninyo & Moore		SEISMIC REFRACTION SURVEY	FIGURE
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	C-1

file no: 3971srs1212a



Ninyo & Moore		SEISMIC REFRACTION SURVEY	FIGURE C-2
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	

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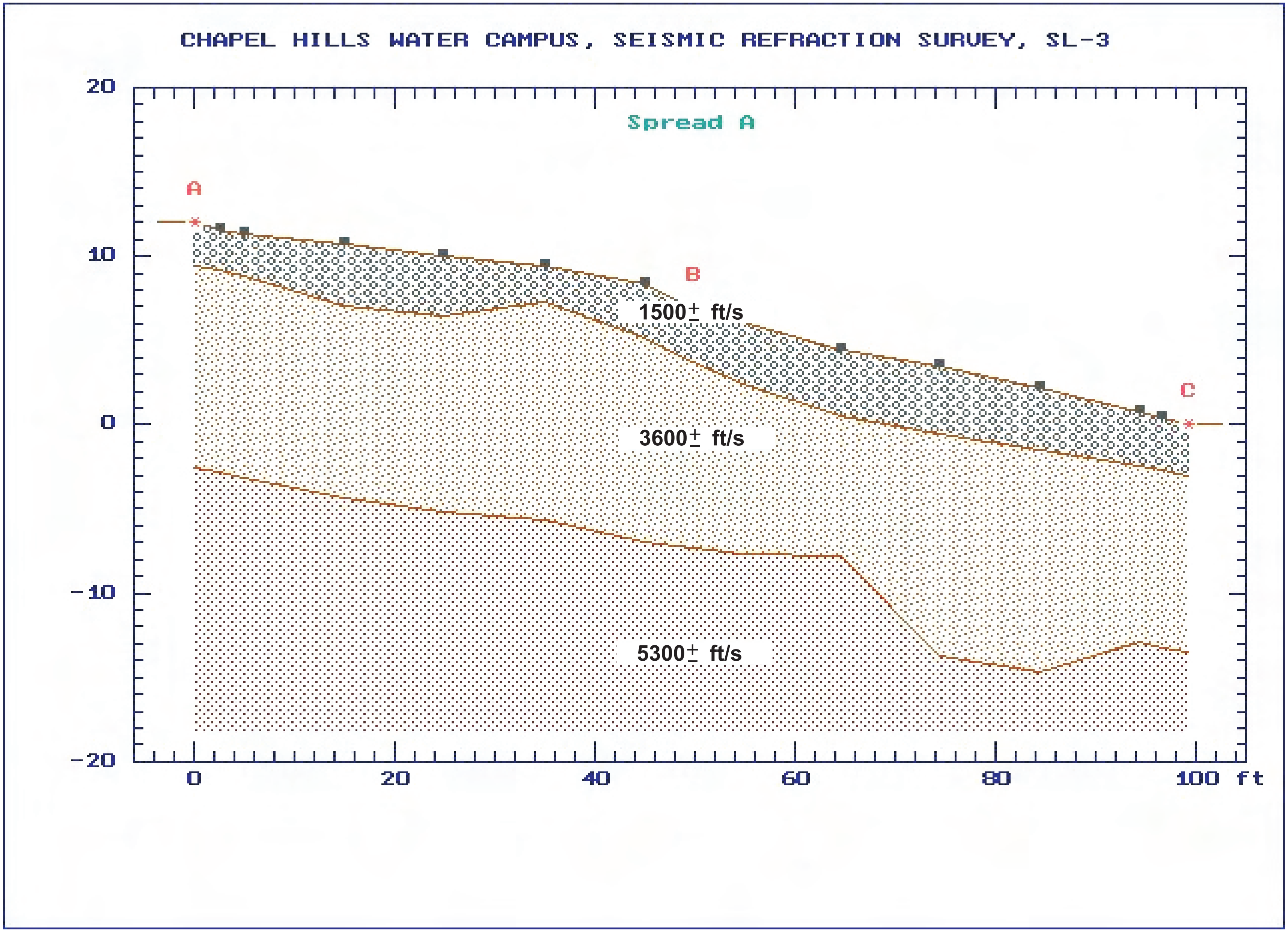
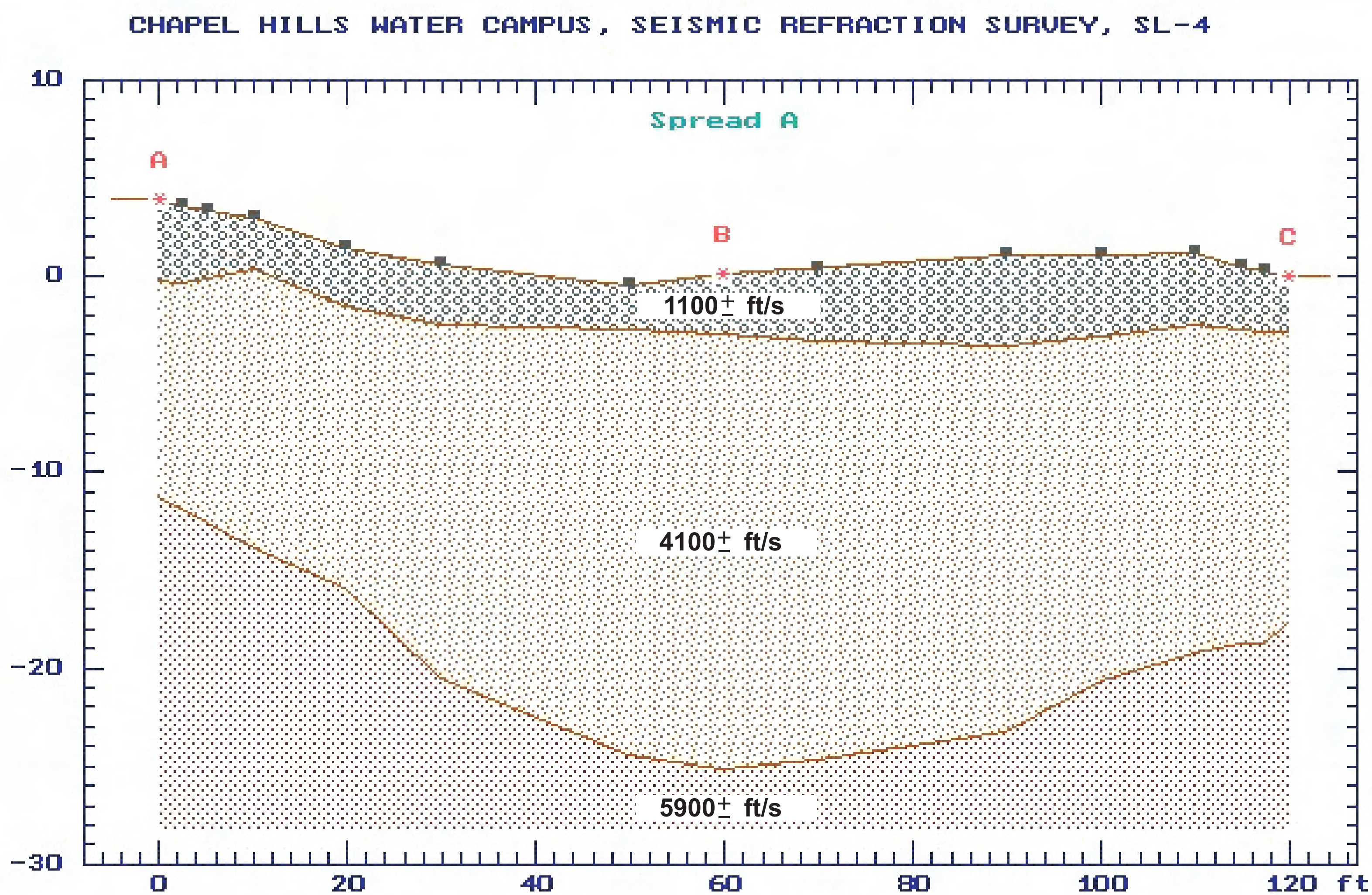


Ninyo & Moore		SEISMIC REFRACTION SURVEY	FIGURE C-3
PROJECT NO: 603971001	DATE: 10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA	

file no: 3971srs1212c



Ninyo & Moore

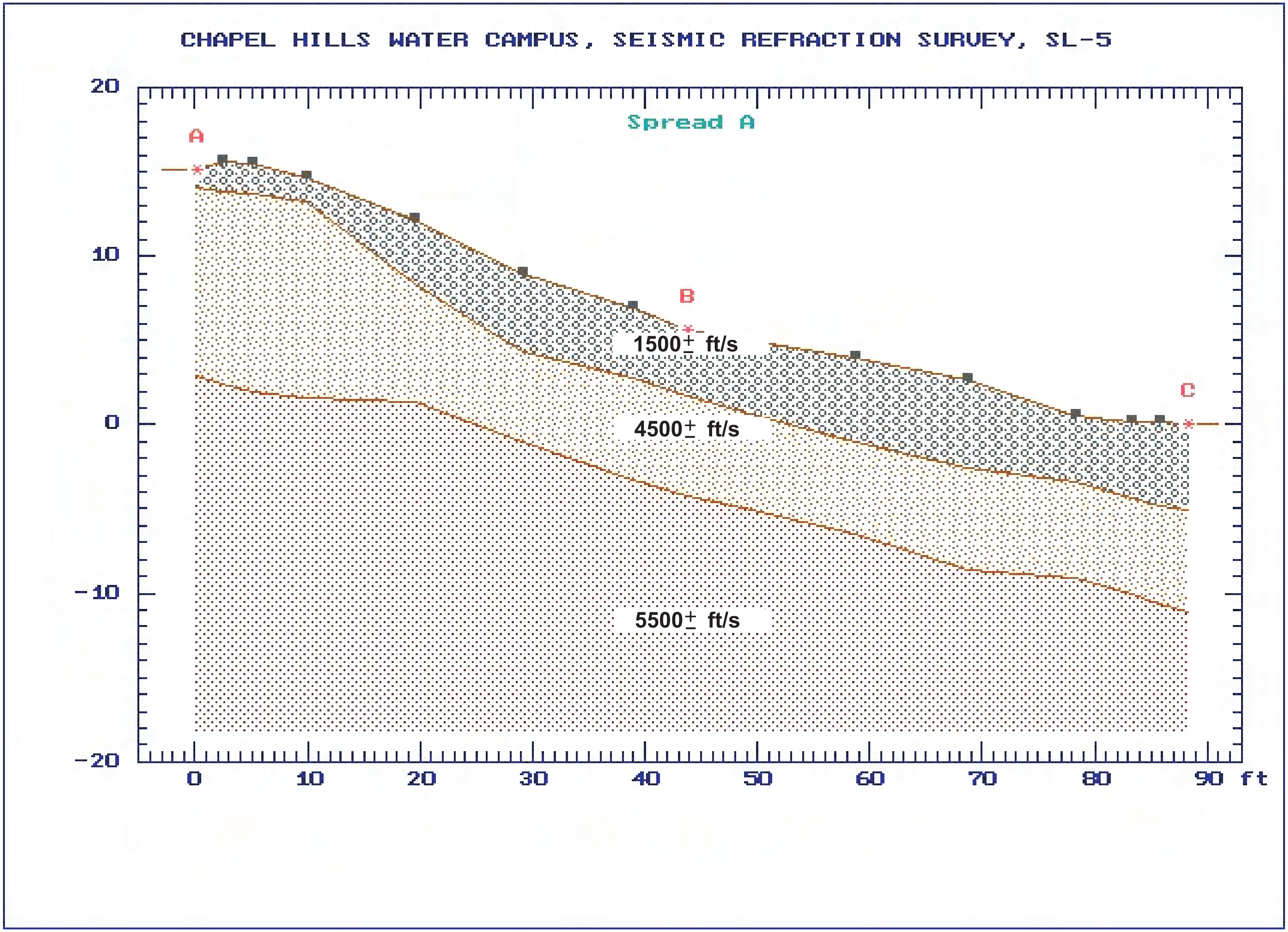
SEISMIC REFRACTION SURVEY

PROJECT NO:	DATE:	
603971001	10/16	GEOTECHNICAL EVALUATION CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179 SEDONA, ARIZONA

FIGURE

C-4

file no: 3971srs1212d



Ninyo & Moore

SEISMIC REFRACTION SURVEY

FIGURE

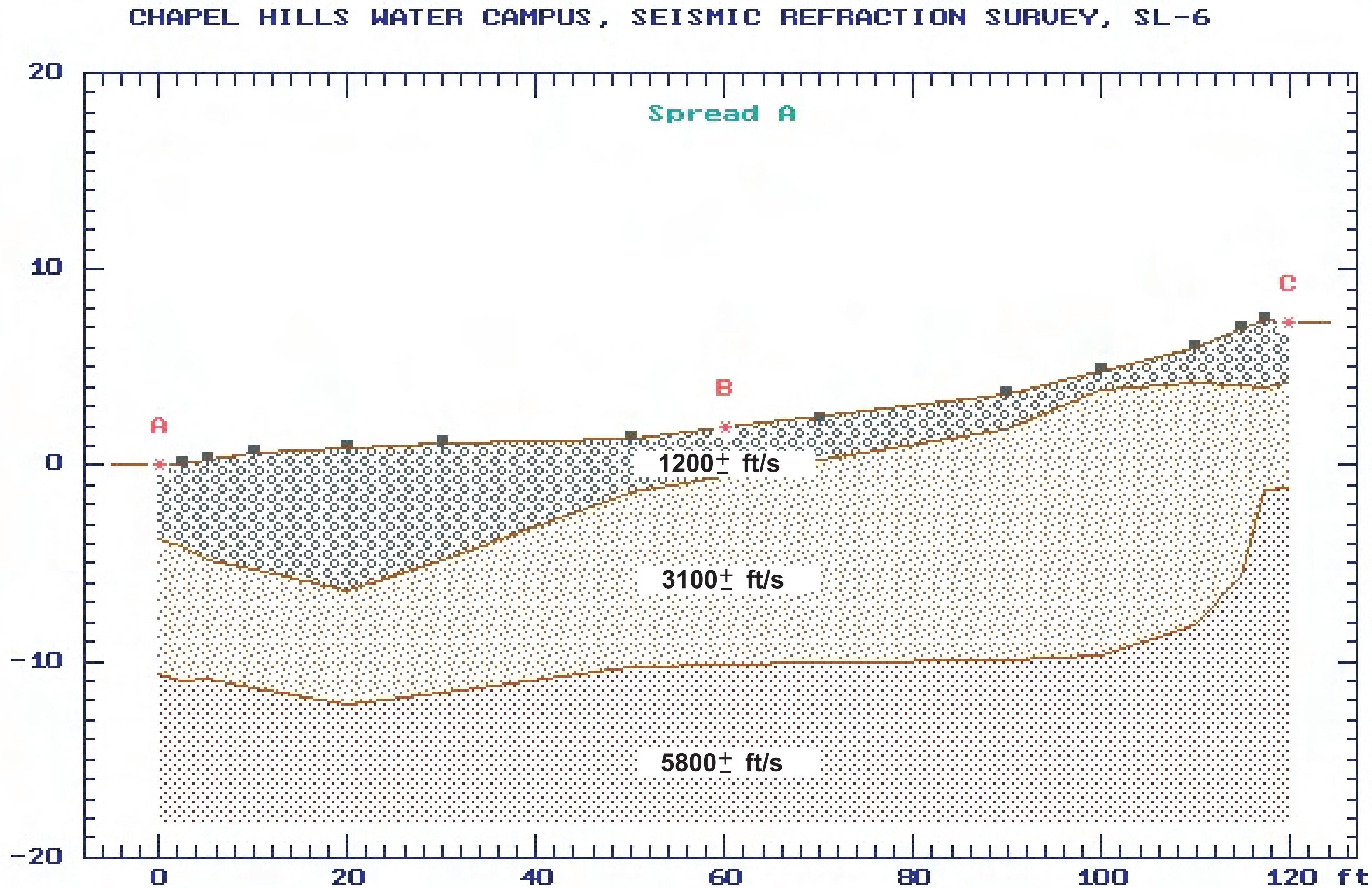
C-5

PROJECT NO:
603971001

DATE:
10/16

GEOTECHNICAL EVALUATION
CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

file no: 3971srs1212e



Ninyo & Moore

SEISMIC REFRACTION SURVEY

FIGURE

C-6

PROJECT NO:
603971001

DATE:
10/16

GEOTECHNICAL EVALUATION
CHAPEL HILLS WATER CAMPUS - MALLARD DRIVE AND STATE ROUTE 179
SEDONA, ARIZONA

file no: 3971srs1212f

APPENDIX D

PHOTOGRAPHIC DOCUMENTATION

Photograph 1: **Sample cores B-1, 11 through 16 feet.**

Photograph 2: **Sample cores B-1, 16 through 21 feet.**

Photograph 3: **Sample cores B-1, 21 through 26 feet.**

Photograph 4: **Sample cores B-1, 26 through 31 feet.**

Photograph 5: **Sample cores B-2, 17 through 22 feet.**

Photograph 6: **Sample cores B-2, 22 through 27 feet.**

Photograph 7: **Sample cores B-2, 27 through 30 feet.**