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GEOTECHNICAL INVESTIGATION
Sage Memorial Hospital
Junction of Hwy 268 and Hwy 191
Ganado, Arizona

IGES Job No. 01276-002

July 10, 2009

Prepared for:

Navajo Health Foundation
c/o Jason Erichsen

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TABLE OF CONTENTS

APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed Sage Memorial Hospital to be located on the west side of the town of Ganado, Arizona. The property is located just north of the junction of U.S. Highway 268 and U.S. Highway 191 in Ganado, Arizona. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slabs-on-grade, pavements, surface drainage, and identify potential geotechnical and geologic hazards associated with the soils conditions at the site and provide recommendations for mitigation.

As a part of this investigation, subsurface soil conditions were explored by completing 6 test pits excavated to depths ranging from 14 to 18 feet below the existing site grade. The upper 6 feet of soil encountered generally consists of a medium stiff, low plasticity Sandy Lean CLAY (CL). From approximately 6 to 10 feet we encountered medium dense Clayey SAND (SC) and Silty SAND (SM) deposits. Thinly bedded clay and silt were commonly observed in the sandy stratum. Below 10 feet was encountered a medium stiff to stiff Silty CLAY (CL-ML) that extended to the maximum depth of exploration in three of the test pits (TP-3, TP-4, and TP-5). Bedrock was encountered in TP-1, TP-2, and TP-6 and consisted of weathered sandstone from the Chinle Formation.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project. All undocumented fill and identified hydro-collapsible soils identified below planned improvements should be over-excavated to bedrock and replaced with properly compacted low permeability structural fill if conventional footings are used. Due to the amount of material that would need to be over-excavated to remove the collapsible soils if conventional footings are used, and the varying depths at which bedrock was found, the owner may wish to consider ground improvement using Geopier® rammed aggregate piers, cast-in-place concrete piles, drilled shafts, helical piers, or driven steel piles or H piles may be considered.

NOTE: The scope of services provided within this report is limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed Sage Memorial Hospital to be located on the west side of the town of Ganado, Arizona. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slabs-on-grade, pavements, surface drainage, and identify potential geotechnical and geologic hazards associated with the soils conditions at the site and provide recommendations for mitigation.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal, dated April 9, 2009 and your signed authorization.

The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

The property is located just north of the junction of U.S. Highway 268 and U.S. Highway 191 in Ganado, Arizona (see Plate A-1, *Site Vicinity Map*). The subject site is bounded on the east by the existing Sage Memorial Hospital and surrounding buildings. The site is bounded on the north by the Pueblo Colorado Wash, on the west by open fields, and on the south by small buildings that are along Highway 264. The site has a total area of approximately 26 acres.

We understand that the Hospital is still in the planning process and details of the development are not finalized. We understand the structure could range from a small single story slab-on-grade structure to a large multi-story facility with a basement.

3.0 METHODS OF STUDY

3.1 LITERATURE REVIEW

In preparation of this report, we have reviewed the Arizona Geologic Map, aerial photos, and other relevant published data.

3.2 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by completing 6 test pits excavated to depths ranging from 14 to 18 feet below the existing site grade. Plate A-2 in Appendix A shows the approximate locations of the test pits completed for this investigation. Exploration points were chosen by the client and it is our understanding that the locations chosen were based on avoiding utility locations and trying to maximize coverage of the entire subject site. Subsurface soils conditions as encountered in the explorations were logged at the time of our investigation by a member of our technical staff and are presented on the enclosed test pit logs, Plates A-3 through A-8 in Appendix A. A *Key to Soil Symbols and Terminology* is presented on Plate A-9.

The test pits were excavated with the aid of a DEERE 120C trackhoe. Both bulk and relatively "undisturbed" soil samples were obtained in the test pit explorations. Relatively "undisturbed" soil samples were obtained with the use of a hand sampler attached to a 6-inch long brass tube driven into the soil with a 2 pound sledge. All samples were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils observed in the explorations were logged and classified in general accordance with the Unified Soil Classification System (USCS). Classifications for the individual soil units are shown on the attached Test Pit Logs (Plates A-3 through A-8).

3.3 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- In situ moisture content and dry density (ASTM D2937 and D2216)

- Atterberg limits (ASTM D4318)
- No. 200 Sieve Wash (ASTM D1140)
- Collapse (ASTM D4546 and/or D5333)
- Grain Size Distribution (ASTM D422)
- Standard Proctor (ASTM D698C)
- California Bearing Ratio (ASTM D1883)
- Unconsolidated Unconfined Triaxial Test (ASTM D2850)
- Water-soluble sulfate concentration for cement type recommendations
- Resistivity and pH to evaluate corrosion potential of ferrous metals in contact with site soils

Laboratory test results are shown on the test pit logs (Appendix A), the test result plates presented in Appendix B (Plates B-1 through B-5) and in the Summary of Laboratory Test Results Table (Plate B-6).

3.4 ENGINEERING ANALYSIS

Based on the proposed construction at the site, the following engineering analyses were performed:

- Excavatibility
- Excavation stability
- Preliminary bearing capacity of foundation soils
- An estimate of foundation settlement
- Pavement Design
- Surface Drainage
- Preliminary corrosion assessment

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

Excavatibility and excavation stability were evaluated based on the excavation conditions encountered and the laboratory test results. For excavation stability, OSHA minimum requirements are typically followed unless conditions warrant further flattening of slopes.

Bearing capacity values were estimated using Hansen's modifications to Terzaghi's original bearing capacity formula. Strength parameters for the bearing soils were assigned based on the laboratory test data, field data and observations. A factor of safety of 3 was used in developing a range of allowable bearing values. Bearing capacities were also limited to minimize settlement of foundation elements. The ultimate bearing pressure for foundations and slabs bearing on the subsurface soils was determined using the undrained strength values from the unconsolidated-undrained triaxial test.

A preliminary corrosion assessment was completed based on the laboratory test results obtained from the Soluble Sulfate, pH and Resistivity tests performed on a representative sample.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our field investigation the site was occupied by approximately 26 structures which consisted of residential structures, a barn, and a metal storage facility. Majority of the western side of the property is empty fields. Vegetation consists predominantly of grasses and sage brush. The site is relatively flat.

4.2 SUBSURFACE CONDITIONS

The subsurface soil conditions were explored at the site by completing 6 test pits at representative locations across the site. The test pits were advanced to depths ranging from 14 to 18 feet below existing site grade. The soils encountered in the borings were visually classified and logged by a member of our technical staff and are included in the boring logs in Appendix A (Plates A-3 through A-8). The subsurface conditions encountered during our investigation are discussed below.

4.2.1 Earth Materials

Based on our observations, the site is underlain by Quaternary-aged colluvium and alluvium deposits derived mostly from the weathered sandstone of the Chinle Formation.

The upper 6 feet of soil encountered generally consists of a medium stiff, low plasticity Sandy Lean CLAY (CL). From approximately 6 to 10 feet we encountered medium dense Clayey SAND (SC) and Silty SAND (SM) deposits. Thinly bedded clay and silt were commonly observed in the sandy stratum. Below 10 feet was encountered a medium stiff to stiff Silty CLAY (CL-ML) that extended to the maximum depth of exploration in three of the test pits (TP-3, TP-4, and TP-5). Bedrock was encountered in TP-1, TP-2, and TP-6 and consisted of weathered sandstone from the Chinle Formation.

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types (Plates A-3 through A-8). The actual in-situ transition may be gradual. Due to the nature and depositional characteristics of the native soils, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater

Groundwater was not encountered in any of the explorations completed for this investigation. Due to the season of our investigation (just after spring run-off), we anticipate groundwater levels to be near their seasonal high. However, it is our experience that during snowmelt, runoff, irrigation on the property and surrounding properties, high precipitation events, and other events, the groundwater level can rise several feet. Fluctuations in the groundwater level should be expected over time.

4.2.3 Collapsible Soils

Collapse (often referred to as "hydro-collapse") is a phenomena where undisturbed soils exhibit volumetric strain and consolidation upon wetting and loading. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and can be mitigated by over-excavating porous, potentially collapsible soils and replacing with engineered fill and by controlling surface drainage and runoff. Based on the results of laboratory testing and our observations at the site, the onsite native surficial soils are expected to have a **moderate to severe** potential for collapse upon wetting. Our laboratory testing indicated collapse percentages from 1 to 6 percent, with a single test having 15.6 percent (volumetric strain). A summary of the test results are presented on Plate B-3.

4.2.4 Strength of Earth Materials

Two Unconsolidated-Undrained (UU) triaxial tests were performed on samples that classifies as Clayey SAND (SC) and Sandy Lean CLAY (CL). The tests indicated that the samples tested had a compressive strength of 730 psf and 1,381 psf (this value approximates the undrained shear strength), respectively. A summary of the test results are presented on Plate B-4.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The site is located at an elevation of approximately 6,345 feet above mean sea level, west of the town of Ganado, Arizona in the Colorado Plateau region. The Colorado Plateau region covers an area of 130,000 square miles and is found in four states (Arizona, Utah, New Mexico, and Colorado). The Colorado Plateau is bounded by the Rocky Mountains (north and east), the Great Basin (west) and the Sonoran Desert (south). Uplift of the Colorado Plateau began approximately 15 million years ago and subsequent erosion has formed the landscape of today (Eldredge, 1996).

5.2 STRATIGRAPHY

Geologic units in the study area are mapped as Holocene to Upper Triassic (210-230 Ma). The bedrock consists of pale-red to brown, medium- to coarse-grained sandstone, silty sandstone, and claystone of the Chinle Formation. The bedrock is covered by Quaternary alluvial deposits (*Qal*), which consist of unconsolidated sand, silt, and clay deposits in active stream channels and floodplains.

5.3 SEISMICITY AND FAULTING

Review of the "Arizona Geologic Map" published by the Arizona Geological Survey (AZGS, 2000), indicates that there are no known active faults that pass under or immediately adjacent to the site. An active fault is defined as a fault displaying evidence of movement during Holocene time (eleven thousand years ago to the present). The site is located approximately 140 kilometers north of the Concho fault and 150 kilometers east of the Leupp faults.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2006). Spectral responses for the Maximum Considered Earthquake (MCE) are shown in the table below. These values generally correspond to a two percent probability of exceedance in 50 years (2PE50) for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field exploration, it is our opinion that this location is best described as a Site Class C. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site's approximate latitude and longitude of 35.7113° and -109.5483° respectively. Based on IBC, the site coefficients are F_a=1.20 and F_v= 1.70. From this procedure the peak ground acceleration (PGA) is estimated to be 0.100g. The MCE PGA and Design response spectrum are presented in Appendix C on Plate C-1.

MCE Seismic Response Spectrum Spectral Acceleration Values for IBC Site Class C [a]

Site Location: **Latitude = 35.7113 N** **Longitude = -109.5483 W**	**Site Class C Site Coefficients:** **Fa = 1.2** **Fv = 1.7**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	0.208xFa = 0.250
1.0	0.049xFv = 0.083

[a] IBC 1615.1.3 recommends scaling the MCE values by 2/3 to obtain the design spectral response acceleration values.

5.4 OTHER GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of the site. There are several hazards in addition to seismicity and faulting that if present at the site, should be considered in the design of roads and critical facilities such as water tanks and structures designed for human occupancy. The other identified geologic hazards considered for this site are shallow stream flooding, bedrock, and collapsible soils. A complete list of potential geologic hazards is included in the *Summary of Geologic Hazards Table* in Appendix C (Plate C-2).

5.4.1 Stream flooding

Stream flooding is a hazard related to spring snowmelt, run-off and flash–flooding from summer rainstorms. Flood hazards should be considered when planning for development for critical facilities located in areas having a potential flood risk.

The Pueblo Colorado Wash runs approximately northeast-southwest just north of the subject site. This wash is largely controlled by the Ganado Lake, located approximately 3.5 kilometers upstream. However, this creek may potentially flood following major rainfall events, rapid to extreme snowmelt, or other major runoff events that could require increasing the outflow from the dam. The Ganado Dam receives run off through the Pueblo Colorado Wash that is a tributary of the Little Colorado River and has an active capacity of 2,880 acre-feet (Ganado Chapter website, 2009). The dam was also built in the late 1800's and was reconstructed under the Safety Dams Program in 1995. The design engineer should assess the flooding potential for the stream, as well as the potential high water level for maximum dam releases.

5.4.2 Shallow Bedrock

Shallow bedrock is a potential hazard that exists when bedrock is found just below the surface when excavation is planned at the site. It is generally expensive and time consuming to remove. Shallow bedrock should be considered when planning the development of the reservoir and road located within the area subjected to this hazard.

Shallow bedrock consisting of the Chinle Formation was encountered in test pits (TP-1, TP-2, and TP-6). Based on the depth encountered, it is unlikely that the removal or excavation of the bedrock will be required. However, the contractor should be aware that if any deep excavations are planned, it is unlikely that the bedrock will be able to be removed below the first several feet with traditional excavation equipment. If deep excavations into the bedrock are planned, alternative means of removal should be considered which include blasting.

5.4.3 Hydro-collapsible soils

Hyrdo-collapsible soils were observed in each of the 6 test pits excavated as part of our investigation. Soils that have a potential to collapse under increased loading and moisture conditions are characterized by a pinhole structure and are therefore hydro-collapsible. In general, hydro-collapsible soils were observed in fine-grained soils that include SILT (ML), Silty CLAY (CL-ML), and Lean CLAY (CL), although hydro-collapsible soils may also include sandy soils. Mitigation measures for these soils are discussed in the Conclusions and Recommendations Section of the report (Section 6.0).

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project. All undocumented fill and identified hydro-collapsible soils identified below planned improvements should be over-excavated to bedrock and replaced with properly compacted low permeability structural fill if conventional footings are used. Excavated native soils, once properly moisture conditioned, may be used as low permeability replacement structural fill. The foundation for the proposed on-grade structure may then consist of conventional shallow spread footings founded entirely on structural fill.

Due to the amount of material that would need to be over-excavated to remove the collapsible soils if conventional footings are used, and the varying depths at which bedrock was found, the owner may wish to consider ground improvement using Geopier® rammed aggregate piers, cast-in-place concrete piles, drilled shafts, helical piers, or driven steel piles or H piles may be considered. If subsurface conditions other than those described herein are encountered during construction or if design and layout changes are initiated IGES must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

The following sub-sections present our recommendations for general site grading, design of foundations, slabs-on-grade, pavement design, lateral earth pressures, and soil corrosion.

6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, and concrete slabs-on-grade. Proper site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade moisture conditions.

6.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris, moisture sensitive soils (for the replacement structural fill option) and undocumented fill (if any) should be removed. Any existing utilities should be re-routed or protected in-place. Tree roots should be grubbed-out and replaced with engineered fill. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill as recommended in Section 6.2.4.

6.2.2 Excavations

Since over-excavation is required, the excavations should extend a minimum of 1 foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond slabs-on-grade and pavements. Structural fill should be placed and compacted in accordance with the recommendations contained in this report (Section 6.2.4).

Prior to placing structural fill, all excavation bottoms should be scarified to at least 6 inches, moisture conditioned as necessary to at or slightly above optimum moisture content (OMC), and compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (Modified Proctor). Soft, wet, or 'pumping' soil conditions may preclude scarifying the excavation bottom.

6.2.3 Excavation Stability

The contractor is responsible for site safety, including all temporary trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health Administration (OSHA) standards to evaluate soil conditions. Soil types are expected to include Type C soils (sandy soils with an unconfined compressive strength less than 1,000 psf). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Sloping the sides at 1½ horizontal to 1 vertical

(1.5H:1V) (34 degrees) in accordance with OSHA Type C soils may be used as an alternative to shoring or shielding.

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of excavated native soils if the recommendations contained in this report are followed. In general, pinholes were observed in the fine-grained SILT (ML) and Lean CLAY (CL) soils observed during our investigation. If the fine-grained soils are to be used as structural fill they should be thoroughly processed to remove all of the pinhole structure in order to reduce the potential for hydro-collapse induced settlement. Where structural fill is required over the potentially collapsible fine-grained soils, the imported material should have a minimum fines content of 30 percent to create a relatively low permeability barrier. Using a lower permeability, fine grained soil as structural fill such as the onsite material, will provide a barrier to help reduce the potential for the underlying collapsible soils from becoming wet. In contrast, a higher permeability, granular soil will provide an area where excess moisture can accumulate, as well as a means of moisture conveyance to other areas of the site, increasing the potential for settlement resulting from the collapsible soils. In areas where structural fill is required above native gravel soils, the structural fill may consist of on-site or imported granular soil since there is no need to create a low permeability barrier.

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 8-inch loose lifts if compacted by light-duty rollers, and maximum 12-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. The moisture content should be within approximately 2 percent of the OMC for all structural fill. Prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials have been removed and/or the excavation bottom has been properly stabilized. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

In addition, all utility trenches backfilled below pavement sections, curb and gutter and concrete flatwork, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557).

Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

6.3 FOUNDATIONS

Inasmuch as this is a preliminary geotechnical investigation, and based on the nature of the site soils, we have presented several alternatives to consider for foundations. We recommend that the owner and other design engineers take into consideration the risk associated with the hydro-collapsible soils. Due to the depth of collapsible soils, the owner may want to give consideration to constructing a basement so that the footings are established closer to the bedrock and mitigation of collapsible soils may be more cost effective. We also recommend that a final geotechnical investigation be completed prior to construction to adequately assess the collapsible soils. We recommend that borings be completed if necessary to collect additional representative samples at depth or that test pits be excavated and benched back so that the geotechnical engineer can collect a relatively undisturbed sample from the test pit at depth (10 to 17 feet).

It should be noted that many of the foundation alternatives mentioned below will leave some collapsible soils in place and that a certain level of risk may still exist for these soils so collapse and affect the structure. If this risk is not acceptable to the owner, consideration should be given to removing all of the collapsible soils and backfilling with structural fill as recommended in Section 6.2.4 of this report.

6.3.1 Geopier® Deep Foundation Systems

Due to the amount of material that would need to be over-excavated to remove the collapsible soils, and the varying depths at which bedrock was observed, the owner may wish to consider ground improvement using rammed aggregate piers, or Geopier® elements, installed on a grid pattern. This option would help reduce the potential for collapse induced settlement, as the Geopier® "shafts" would be founded directly upon the underlying bedrock. In addition, a Geopier® system would allow for the placement of new fill and/or floor slab directly atop the Geopier®-reinforced subgrade. Geopier® deep foundation systems are generally used in conjunction with conventional spread footings as discussed in Section 6.3.2.

The use of Geopiers® would significantly increase the net allowable bearing capacity of the subgrade such that foundation dimensions could be adjusted to minimize the cost of concrete for the project. Geopier® is a proprietary technology and is generally provided on a turnkey or design-build basis. Final analysis and cost estimate for this approach would be provided directly by the Geopier® Foundation Company.

6.3.2 Conventional Spread Footings

Based on our field observations and considering the presence of highly collapsible soils throughout the subject site, we recommend that conventional footings only be used for this project if a complete excavation and replacement to bedrock is performed or in conjunction with Geopier® mitigation, that the footings for the proposed structure be founded entirely on improved site conditions. All structural fill beneath the foundations should be placed and compacted in accordance with our recommendations contained in Section 6.2.4 of this report. The building pad should be over-excavated until bedrock is reached and the collapsible soils completely removed and replaced with structural fill, such that the footings bear entirely on a uniform zone of structural fill. Due to the depth of collapsible soils, the owner may want to give consideration to constructing a basement so that the footings are established closer to the bedrock and mitigation of collapsible soils may be more cost effective. We recommend that IGES evaluate the bottom of the excavation prior to placement of structural fill.

Shallow spread or continuous wall footings constructed on structural fill over suitable soil improvements may be proportioned utilizing a maximum net allowable bearing pressure of **2,000 to 3,000 pounds per square foot (psf)** for dead load plus live load conditions. The bearing capacity will depend on the thickness of the zone of structural fill and elevation of the footings. The foundation should not be constructed partially on native soils and partially on structural fill. Bearing capacity was limited due to settlement considerations. Settlement was limited to a maximum of one inch. If increased bearing capacities are required we recommend consideration be given to using a deep foundation system discussed in the following sections.

All foundations exposed to the full effects of frost should be established at a minimum depth of 30 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., a continuously heated structure), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

If conventional footings are used, it is imperative that the recommendations in the moisture protection and surface drainage section (Section 6.7) be strictly adhered to. Additionally, it should be noted that if footings are placed on a zone of structural fill over native soils, the potential for hydro-collapse still exists; if these soils become wetted, significant settlement as well as differential settlement could occur. If the owner is not willing to accept this risk, then all of the soils beneath structural elements such as footings that have a hydro-collapse potential will need to be removed.

Due to the amount of material that would need to be over-excavated to remove the collapsible soils and the varying depths at which bedrock was found, the owner may wish to consider an alternative to conventional footings.

6.3.3 Cast-In-Place Concrete Piles

Two types of cast-in-place concrete piles can be considered at this site; conventional drilled piers/shafts or Augered Cast-in-Place piles (ACIP). Conventional drilled piers/drilled shafts (also referred to as *caissons*) are constructed using large-diameter flight augers, either cased or uncased, drilled to a specified depth and bearing layer. Reinforcement is placed within the open shaft and concrete is subsequently pumped into the bottom of the shaft. ACIP piles are constructed by first drilling a pile shaft with a conventional continuous flight auger. As the flight auger is withdrawn from the ground, concrete is injected under pressure at the tip of the auger. Once the auger has been removed, a steel reinforcement assembly (or “cage”) is placed within the concrete, thereby creating a reinforced concrete shaft. ACIP piles are generally more economical when a relatively few piles will be constructed (i.e., on the order of 100 piles or less). If this alternative is selected, cast-in-place concrete pile design can be provided by IGES upon request.

6.3.4 Helical Piers

Helical piers are another potentially economical alternative to the aforementioned over-ex and replace and deep foundation systems. Lateral capacity needs should be specifically considered when using helical piers (battered piers are typically used to resist lateral loads). Allowable load capacity for helical piers is typically on the order of 25 to 50 kips per pier, depending on the size of the pier. However, it should be noted that due to the presence of moderately corrosive soil, cathodic protection will likely be required with the use of helical piers. The local helical-pier supplier/installer can provide more information on the design of these piers.

6.3.5 Driven Steel Piles or H Piles

Driven piles may also be considered as an alternative deep foundation system. Driven piles will have higher allowable lateral capacities but may be more costly due to required specialty equipment and the relatively shallow bearing layer (20-30 feet). If this alternative is selected, actual pile design can be provided by IGES upon request.

6.4 SETTLEMENT

6.4.1 Static Settlement

Static settlement of properly designed and constructed conventional foundations, founded as described above, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

6.5 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.35 for clayey native soils or structural fill should be used.

Ultimate lateral earth pressures from natural soils and *granular* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Condition	**Level Backfill**	
	Lateral Pressure Coefficient	**Equivalent Fluid Density (pcf)**
Active (Ka)	0.36	43
At-rest (Ko)	0.53	64
Passive (Kp)	2.77	330

These coefficients and densities assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of either native granular soil or sandy imported material with an Expansion Index (EI) less than 20.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

6.6 PAVEMENT DESIGN

IGES has prepared a preliminary pavement section design. When the final plans for the development are completed with anticipated traffic volumes, number of structures, various types and number of businesses that will be using the area, etc. the pavement section should be reevaluated. Based on the soil conditions observed across the subject site, a representative sample was obtained to get a CBR value in the laboratory. The representative soil sample was obtained at a depth of approximately 1 foot below existing site grade and had a CBR value of 7.0. Based on soil classification and laboratory obtained CBR values, the near surface soils are expected to provide moderate pavement support. Anticipated traffic volumes were not available at the time this report was prepared. However, based on our understanding of the project development we assumed traffic on roadways would consist of approximately 500 passenger vehicles trips per day with 10 percent trucks to account for delivery vehicles, moving vans and construction traffic over a 20 year design life. The following pavement designs have been developed for a 20-year design life assuming a 2 percent annual growth rate, and our assumed equivalent single axle load (ESAL) of 300,000 ESALs for these main arteries through the community. Based on the information obtained and the above mentioned assumptions, we recommend the following pavement section be constructed on properly prepared subgrade:

Preliminary Pavement Section Recommendations*

	Asphalt Concrete (in.)	Untreated Base Course (in.)	Subbase (in.)
Major Through Streets, Hospital	**3**	**5**	**6**

*Ganado City or the State of Arizona may have specific pavement requirements for the types of roadways planned that may exceed the minimum section recommendations presented above

During construction, a significant amount of heavy construction traffic occurs. Some distress may occur on the pavement during this initial construction time period. Maintenance may need to be performed after completion of construction. Alternatively, placement of asphalt surfacing may be deferred to near completion of construction or limiting heavily loaded traffic to thickened restricted unpaved areas.

Asphalt has been assumed to be a high stability plant mix and base course material composed of crushed stone with a minimum CBR of 70. Road base should be compacted to a minimum density of 95 percent as determined by ASTM D-1557 (Modified Proctor). Asphalt should be compacted to a minimum of 96 percent of the Marshall maximum density. Asphalt and aggregate base material should conform to local requirements.

Where Portland Cement Concrete (PCC) pavements are planned, such as near trash enclosures or other areas expected to support heavy truck traffic, the pavement is recommended to be a minimum of 6 inches in thickness. Concrete pavement should be underlain by a minimum 6 inches of aggregate base course.

If conditions vary significantly from our stated assumptions, IGES should be contacted so we can modify our pavement design parameters accordingly. The County or other governing authority may have pavement requirements over and above those listed and these should be adhered to where applicable.

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the proposed facility should be implemented. We recommend that hand watering, desert landscaping or Xeriscape be considered within 10 feet of the foundations. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures or to storm water runoff areas. Additionally, the ground surface within 10 feet of the structures should be constructed so as to slope a minimum of five percent away. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement. We anticipate that sewer utilities will be in place for new structures and that no septic tanks or leachate fields will be used.

6.8 PRELIMINARY SOIL CORROSION POTENTIAL

Laboratory test results indicate that near surface native soils tested have a sulfate content 490 ppm. Based on this result, the soils are classified as having a moderate potential for sulfate attack to concrete. We anticipate that conventional Type II cement can be used for all of the concrete.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, a representative soil sample was tested in our soils laboratory for soil resistivity (AASHTO T288), soluble chloride content, and pH. The tests indicated that the onsite soil tested has a minimum soil resistivity of 2,100 OHM-cm, a soluble chloride content of 56 ppm, and a pH of 9.3. Based on this result, the onsite native soil is considered **corrosive to severely corrosive** to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal in contact with existing site soils, particularly ancillary water lines and reinforcing steel, and valves.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

7.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction. Further, because of the early stage of design and planning for this project, this investigation and the recommendations presented in this report should be considered preliminary. As such, as design concepts are firmed, IGES should be given the opportunity to review any significant changes proposed to determine if additional site investigations may be required.

IGES staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Observation of foundation soils to assess their suitability for footing placement.

- Observation of soft/loose soils over-excavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

8.0 REFERENCES CITED

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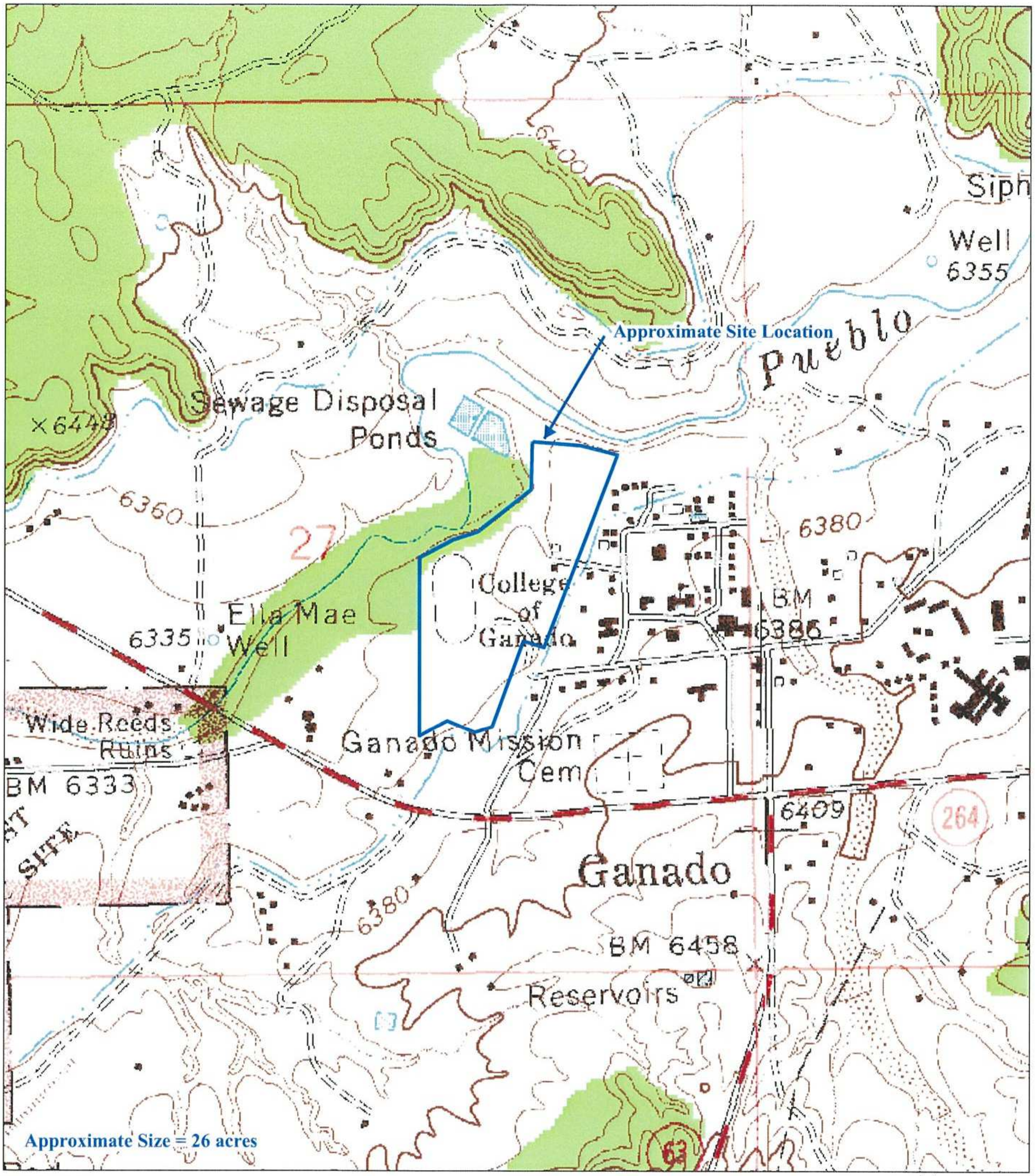
APPENDIX A



BASE MAP:
GANADO, ARIZONA
U.S.G.S. 7.5 MINUTE QUADRANGLES

0 550 1,100 2,200 Feet

1:10,000

N

IGES

Project Number - 01276-002

Geotechnical Investigation
Sage Memorial Hospital
Junction of Hwy 268 and Hwy 191
Ganado, Arizona

SITE VICINITY MAP

Plate

A-1

DATE
STARTED: 4/14/09
COMPLETED: 4/14/09
BACKFILLED: 4/14/09

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number 01276-002

IGES Rep: BMJ
Rig Type: DEERE 120C Trackhoe

TEST PIT NO:
TP - 1
Sheet 1 of 1

LOCATION
NORTHING 631,371 EASTING 3,952,762 ELEVATION 6,382

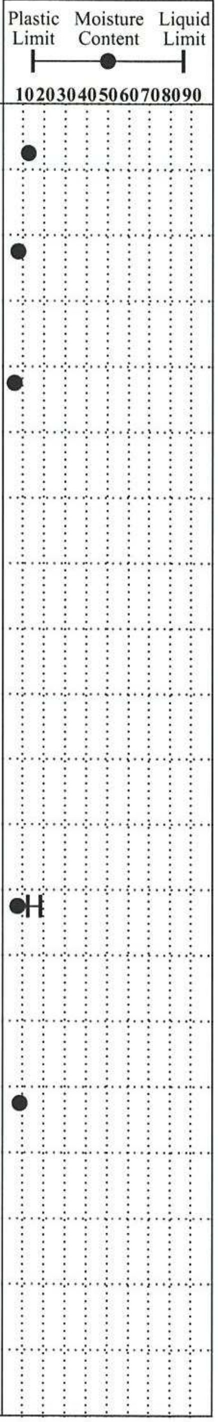
Depth (feet)	Samples	Unified Soil Classification	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		CL	@0' - TOPSOIL - Lean CLAY with sand, medium stiff, slightly moist, dark brown, roots in upper few inches					
1	▯				12.2			
1		SC	@1' - Clayey SAND, medium dense, slightly moist, tan-brown, some pinholes, sand is fine-grained, small interbedded layers of Silty SAND (SM) that range from 2-inches to 6-inches in thickness					
2.5	⧗			106.8	7.3			
4	▯				5.6	27.4		
6		CL-ML	@6' -Silty CLAY with sand, medium stiff, slightly moist, brown, pinholes observed throughout					
12	▯				7.1		18	6
15	⧗			86.0	8.2			
16	▯		@16' - BEDROCK - Chinle Formation					

No Groundwater Encountered

Bottom of Test Pit @ 16.5 Feet

GRAPHICAL LOG

Moisture Content and Atterberg Limits



IGES

SAMPLE TYPE
▯ - GRAB SAMPLE
⧗ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate
A - 3

DATE	
STARTED:	4/14/09
COMPLETED:	4/14/09
BACKFILLED:	4/14/09

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona

Project Number 01276-002

IGES Rep: BMJ

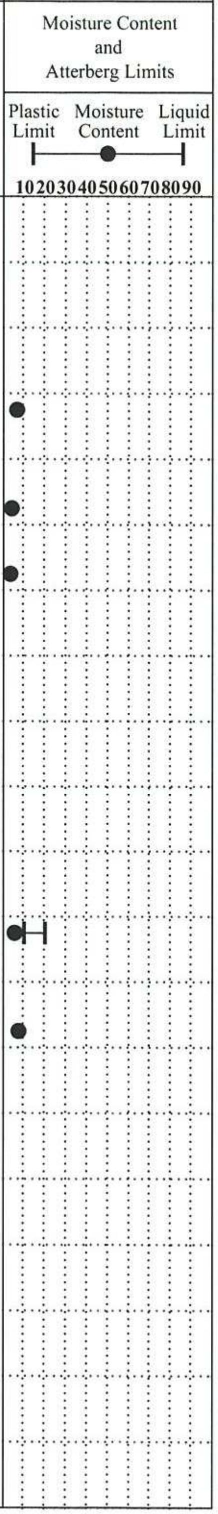
Rig Type: DEERE 120C Trackhoe

TEST PIT NO:
TP - 2
Sheet 1 of 1

LOCATION

NORTHING 631,320 EASTING 3,952,877 ELEVATION 6,370

DEPTH METERS	DEPTH FEET	SAMPLES	WATER LEVEL	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0			CL	@0' - TOPSOIL - Lean CLAY with sand, medium stiff, slightly moist, dark brown, roots in upper few inches					
				CL	@½' - Lean CLAY with sand, medium stiff, slightly moist, brown-tan, some pinholes observed					
1		⧗				101.6	6.6			
				SM	@4' - Silty SAND, loose to medium dense, slighlty moist, tan, sand is fine-grained, small interbedded layers of Lean CLAY with sand that are approximately 2 to 6-inches thick					
	5	⧗				94.0	4.1			
		▯					3.5	15.3		
2										
				CL	@9' - Lean CLAY with sand, medium stiff to stiff, slightly moist, orange-brown, pinholes throughout					
3	10									
		▯					5.5		20	10
		▯					7.4			
4		▯			@13½' - BEDROCK - Chinle Formation					
					No Groundwater Encountered					
	15				Bottom of Test Pit @ 14 Feet					
5										
6										



SAMPLE TYPE
- ▯ - GRAB SAMPLE
- ⧗ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- ▼- MEASURED
- ▽- ESTIMATED

NOTES:

IGES

Plate
A - 4

DATE	
STARTED:	4/14/09
COMPLETED:	4/14/09
BACKFILLED:	4/14/09

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number 01276-002

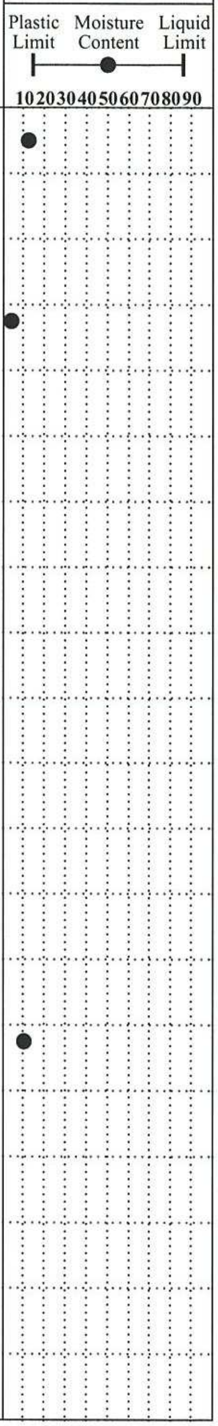
IGES Rep: BMJ
Rig Type: DEERE 120C Trackhoe

TEST PIT NO:
TP - 3
Sheet 1 of 1

LOCATION
NORTHING 631,249 EASTING 3,953,068 ELEVATION 6,374

DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	CL	@0' - TOPSOIL - Lean CLAY with sand, medium stiff, slightly moist, dark brown, roots in upper few inches		12.1			
1	CL	@1' - Lean CLAY with sand, medium stiff, slighlty moist, tan-brown, pinholes throughout					
			96.1	3.9			
5		@5' - sand content increasing with depth					
7½		@7½' - smal linterbedded lenses of Clayey SAND (SC) approximately 2 to 4-inches thick					
14	SC	@14' -Clayey SAND, dense, slightly moist, brown		10.0	48.5		
		No Groundwater Encountered					
		Bottom of Test Pit @ 18 Feet					

Moisture Content and Atterberg Limits



SAMPLE TYPE
▯ - GRAB SAMPLE
⧗ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate
A - 5

IGES

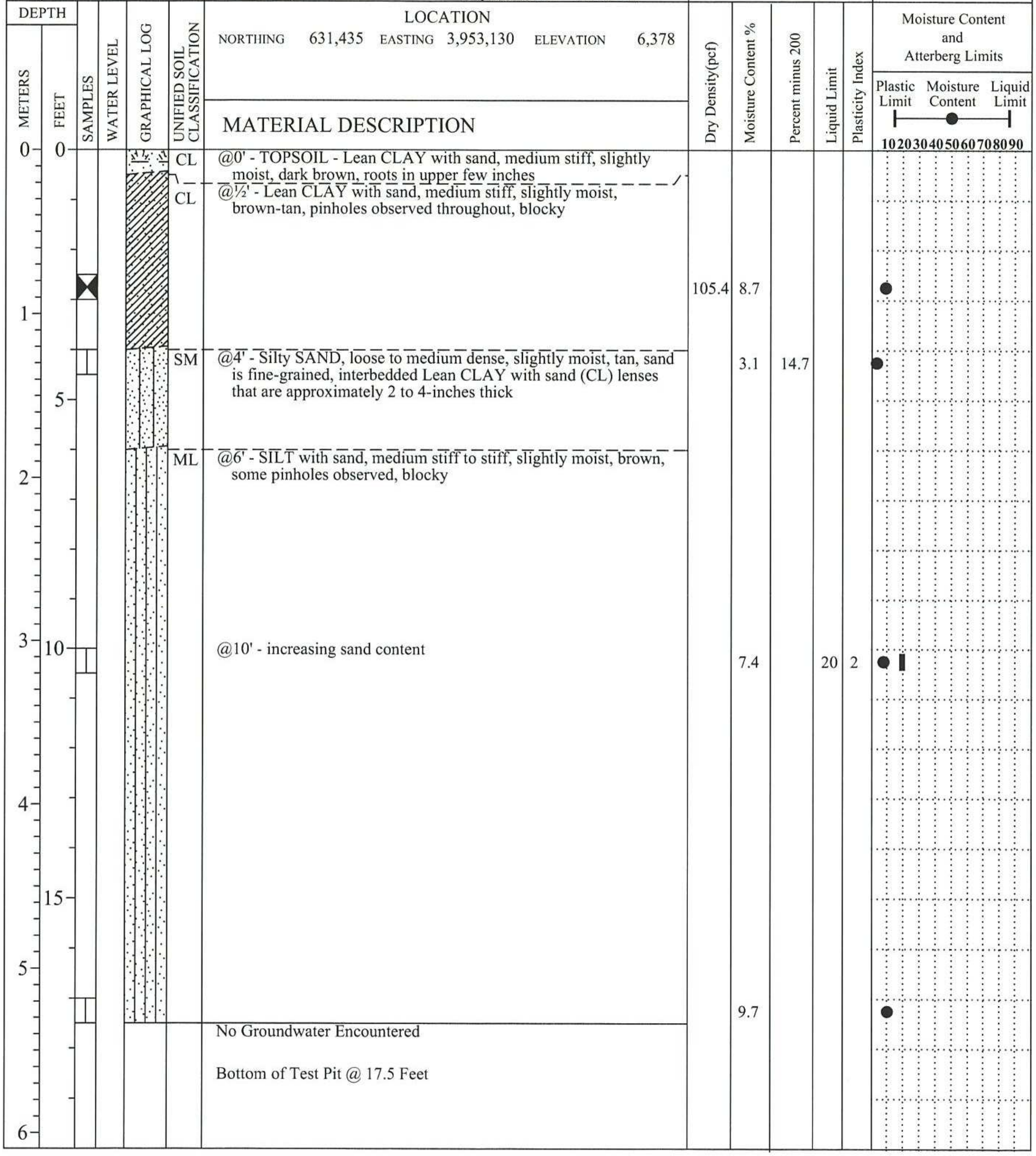
DATE		
STARTED:	4/14/09	
COMPLETED:	4/14/09	
BACKFILLED:	4/14/09	

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number 01276-002

IGES Rep: BMJ
Rig Type: DEERE 120C Trackhoe

TEST PIT NO:
TP - 4
Sheet 1 of 1

LOCATION
NORTHING 631,435 EASTING 3,953,130 ELEVATION 6,378



Depth (feet)	Unified Soil Classification	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	CL	@0' - TOPSOIL - Lean CLAY with sand, medium stiff, slightly moist, dark brown, roots in upper few inches					
½	CL	@½' - Lean CLAY with sand, medium stiff, slightly moist, brown-tan, pinholes observed throughout, blocky					
			105.4	8.7			
4	SM	@4' - Silty SAND, loose to medium dense, slightly moist, tan, sand is fine-grained, interbedded Lean CLAY with sand (CL) lenses that are approximately 2 to 4-inches thick		3.1	14.7		
6	ML	@6' - SILT with sand, medium stiff to stiff, slightly moist, brown, some pinholes observed, blocky					
10		@10' - increasing sand content		7.4		20	2
				9.7			

No Groundwater Encountered

Bottom of Test Pit @ 17.5 Feet

SAMPLE TYPE
- ▯ - GRAB SAMPLE
- ☒ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- ▼- MEASURED
- ▽- ESTIMATED

NOTES:

Plate

A - 6

DATE

STARTED: 4/14/09

COMPLETED: 4/14/09

BACKFILLED: 4/14/09

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number 01276-002

IGES Rep: BMJ

Rig Type: DEERE 120C Trackhoe

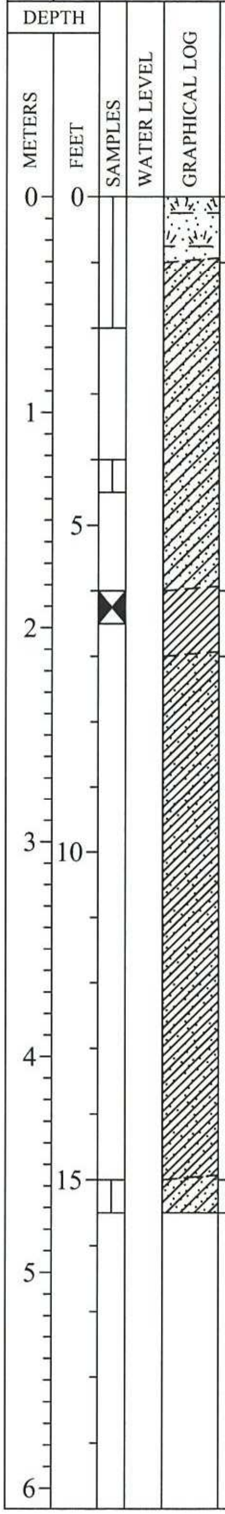
TEST PIT NO:

TP - 5

Sheet 1 of 1

LOCATION

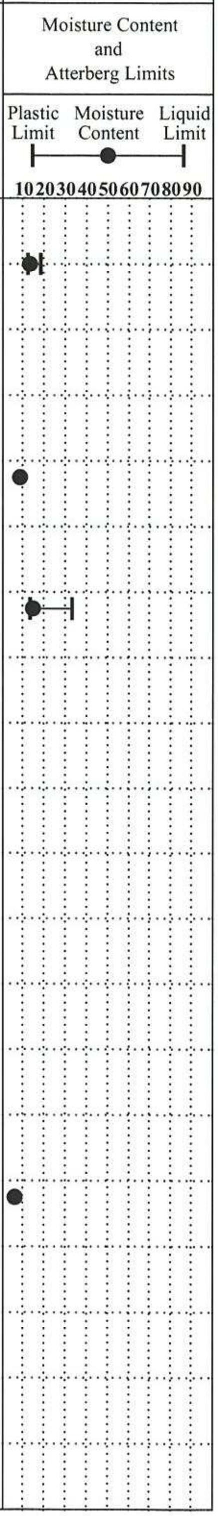
NORTHING 631,492 EASTING 3,953,199 ELEVATION 6,364



UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
SC-SM	@0' - TOPSOIL -Silty Clayey SAND, medium dense, slightly moist, dark brown					
SC	@1' - Clayey SAND, medium dense, slightly moist, brown, sand is fine-grained		12.9	29.9	18	6
			8.3	21.6		
CL	@6' - Lean CLAY, medium stiff, moist, brown to orange-brown, pinholes observed throughout	93.9	14.4		33	20
CL	@7' - Lean CLAY with sand, medium stiff, slightly moist, brown, pinholes observed throughout					
SC	@15' - Clayey SAND, medium dense to dense, slightly moist, brown, pinholes observed		5.7	22.5		

No Groundwater Encountered

Bottom of Test Pit @ 15.5 Feet



IGES

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SAMPLE TYPE
▯ - GRAB SAMPLE
☒ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate A - 7

DATE	
STARTED:	4/14/09
COMPLETED:	4/14/09
BACKFILLED:	4/14/09

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number 01276-002

IGES Rep: BMJ
Rig Type: DEERE 120C Trackhoe

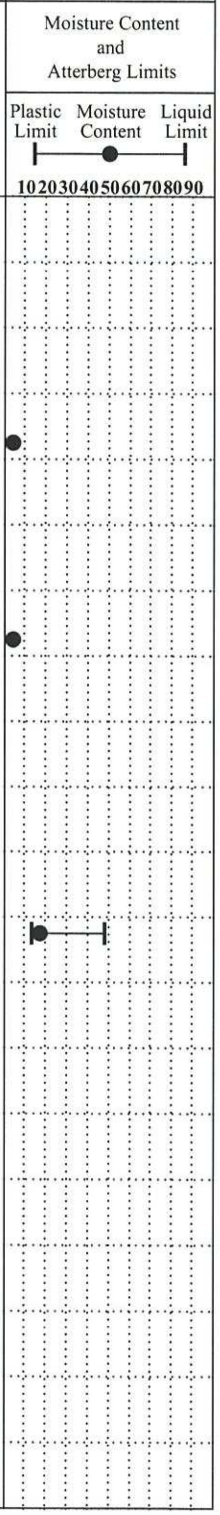
TEST PIT NO:
TP - 6
Sheet 1 of 1

LOCATION
NORTHING 631,331 EASTING 3,952,985 ELEVATION 6,369

SAMPLES	WATER LEVEL	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
		CL	@0' - TOPSOIL - Lean CLAY with sand, medium stiff, slightly moist, dark brown, blocky					
		CL	@1' - Lean CLAY with sand, medium stiff, slighlty moist, tan-brown, pinholes throughout					
▯								
⧗				107.2	4.1			
		SM	@5' - Silty SAND, loose to medium dense, slightly moist, tan, sand is fine-grained, interbedded lenses of Lean CLAY with sand (CL)					
▯					4.1	22.6		
			@9' - sand is medium-grained					
▯		CL	@11' - Lean CLAY with sand, medium stiff, moist, brown, some small gravel clasts generally less than ½-inch in diameter, clasts are well-rounded		17.1		48	35
			@17½' - BEDROCK - Chinle Formation					

No Groundwater Encountered

Bottom of Test Pit @ 18 Feet



IGES

SAMPLE TYPE
▯ - GRAB SAMPLE
⧗ - 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate
A - 8

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2 - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

LOG KEY SYMBOLS

⊕	BORING SAMPLE LOCATION	⧗	TEST-PIT SAMPLE LOCATION
▼	WATER LEVEL (level after completion)	▽	WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

		TORVANE	POCKET PENETROMETER	FIELD TEST
CONSISTENCY	SPT (blows/ft)	UNTRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

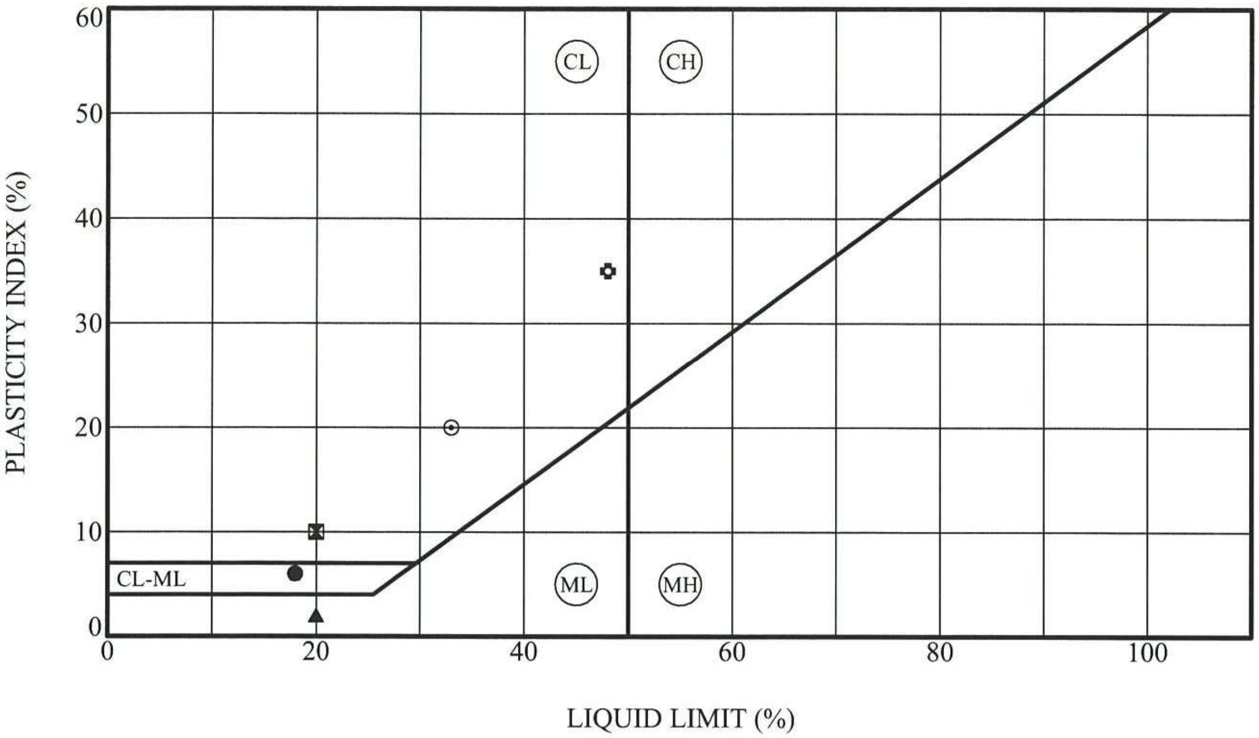
IGES

Key to Soil Symbols and Terminology

Plate
A-9

Project Number 01276-002

APPENDIX B



Sample Location		Depth (ft)	LL (%)	PL (%)	PI (%)	Classification
●	**TP - 1**	**12.0**	**18**	**12**	**6**	**Silty CLAY with sand (CL-ML)**
⊠	**TP - 2**	**11.0**	**20**	**10**	**10**	**Lean CLAY with sand (CL)**
▲	**TP - 4**	**10.0**	**20**	**18**	**2**	**SILT with sand (ML)**
★	**TP - 5**	**0.0**	**18**	**12**	**6**	**Clayey SAND (SC)**
⊙	**TP - 5**	**6.0**	**33**	**13**	**20**	**Lean CLAY (CL)**
✿	**TP - 6**	**11.0**	**48**	**13**	**35**	**Lean CLAY with sand (CL)**

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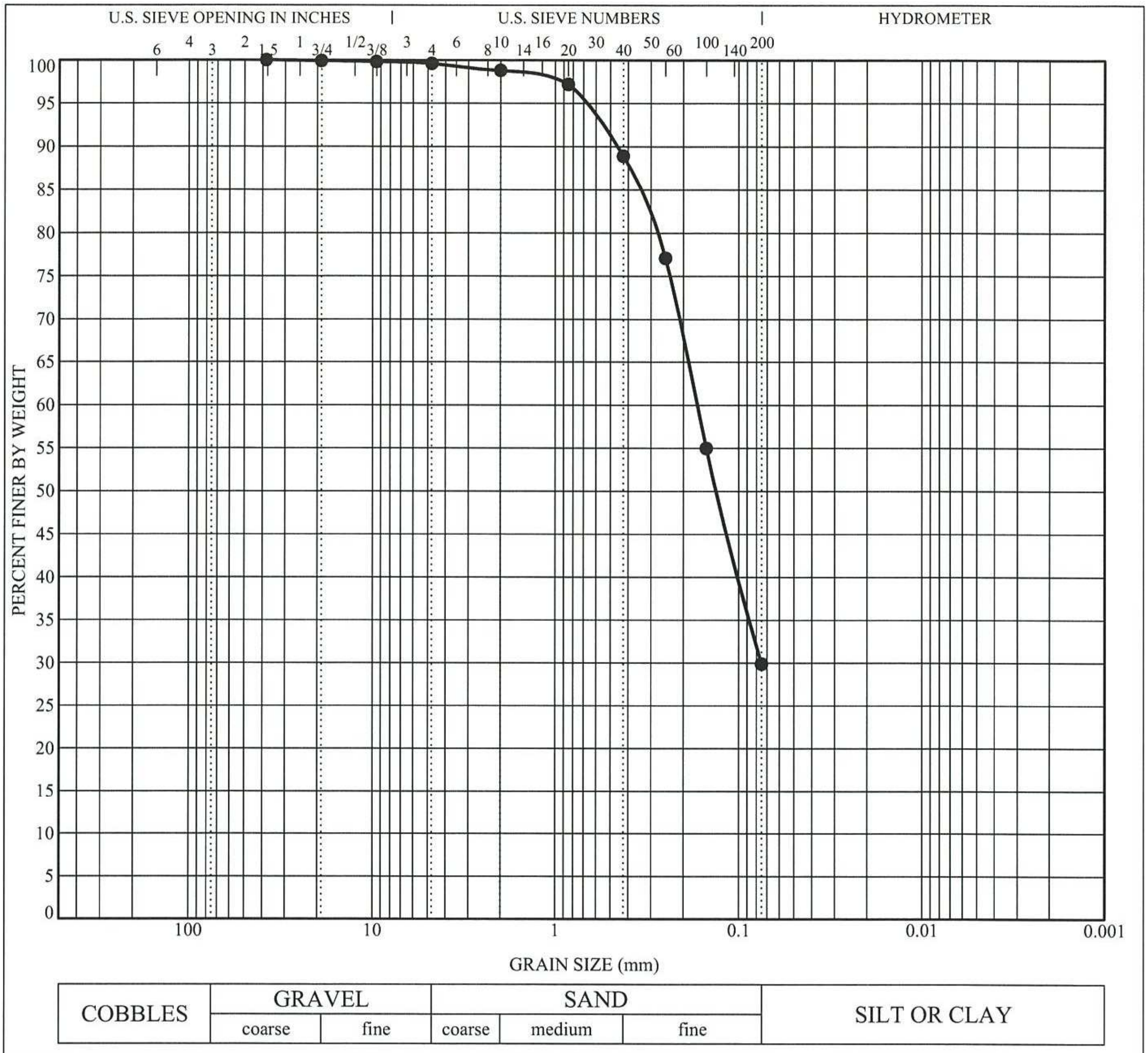
ATTERBERG LIMITS' RESULTS

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number: 01276-002

Plate
B - 1



Sample Location		Depth	Classification	LL	PL	PI	Cc	Cu
●	**TP - 5**	**0.0**	**Silty Clayey SAND (SC-SM)**	**18**	**12**	**6**		

Sample Loctaion		Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP - 5**	**0.0**	**38.1**	**0.168**	**0.075**		**0.4**	**69.7**	**29.9**	

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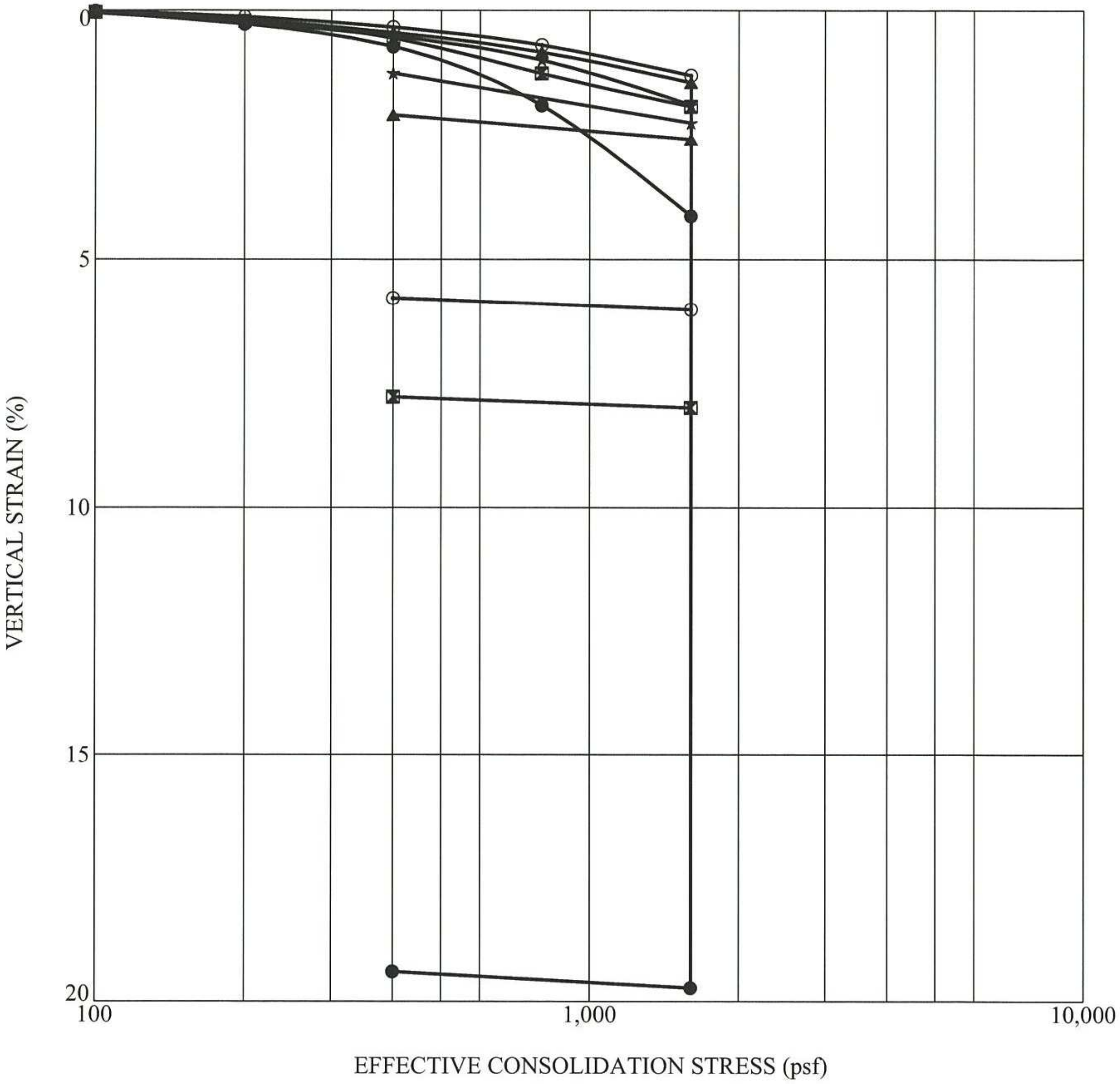
GRAIN SIZE DISTRIBUTION

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number: 01276-002

Plate

B - 2



Sample Location		Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP - 1**	**15.0**	**Silty CLAY with sand (CL-ML)**	**86**	**8**	**1600**	**0.00**	**15.60**
☒	**TP - 2**	**4.5**	**Silty SAND (SM)**	**100**	**4**	**1600**	**0.00**	**6.06**
▲	**TP - 4**	**2.5**	**Lean CLAY with sand (CL)**	**105**	**9**	**1600**	**0.00**	**1.14**
★	**TP - 5**	**6.0**	**Lean CLAY (CL)**	**94**	**21**	**1600**	**0.00**	**0.36**
⊙	**TP - 6**	**3.5**	**Lean CLAY with sand (CL)**	**107**	**4**	**1600**	**0.00**	**4.70**

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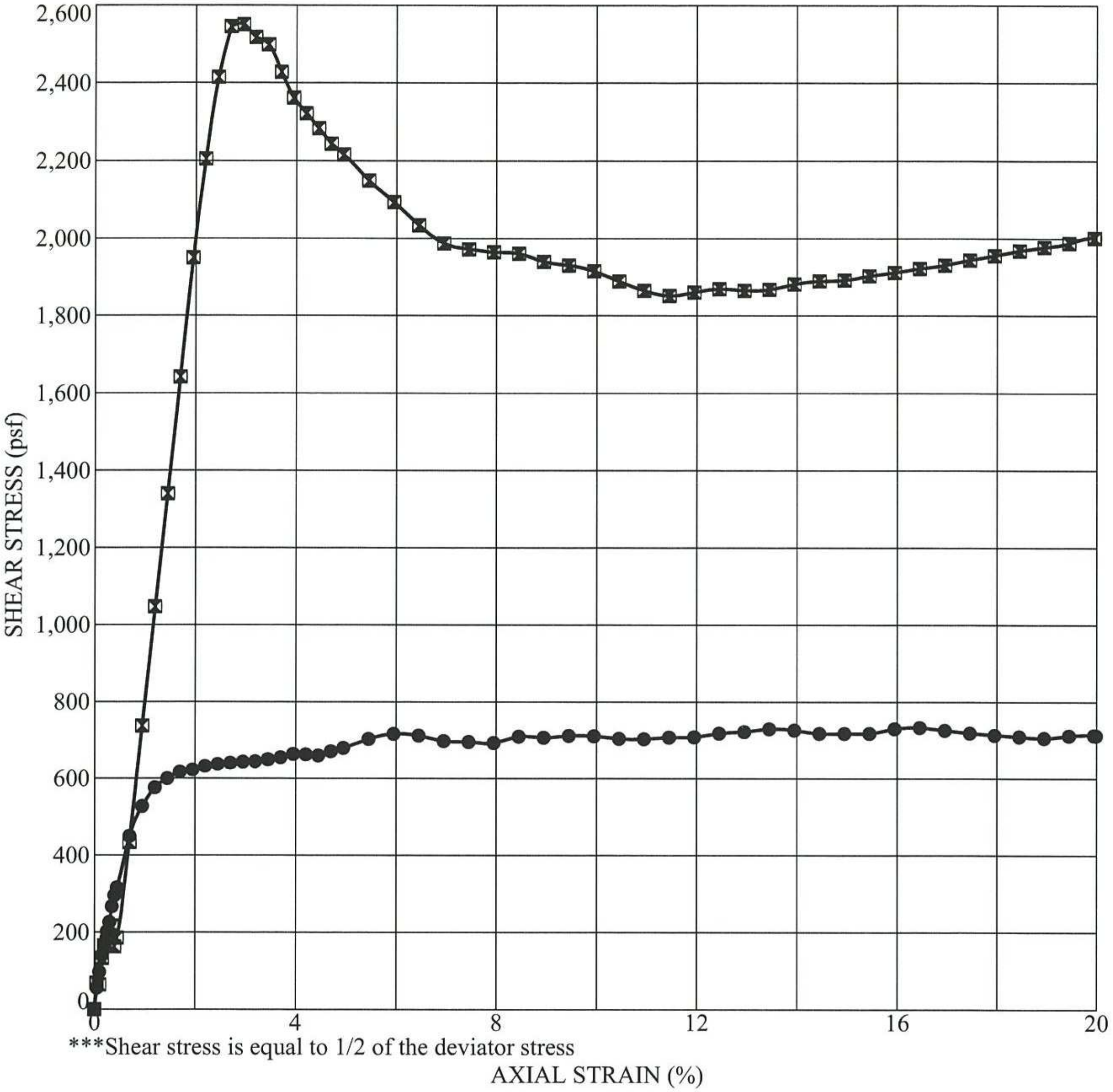
1-D SWELL/COLLAPSE TEST

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number: 01276-002

Plate

B - 3



***Shear stress is equal to 1/2 of the deviator stress

	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Maximum Shear Stress (psf)
●	**TP - 1**	**2.0**	**Clayey SAND (SC)**	**107**	**7**	**730**
☒	**TP - 2**	**3.0**	**Lean CLAY with sand (CL)**	**102**	**7**	**2551**

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UU TRIAXIAL TEST

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number: 01276-002

Plate

B - 4

Source of Material **TP - 3 0.5 ft.**

Material Description **Lean CLAY with sand (CL)**

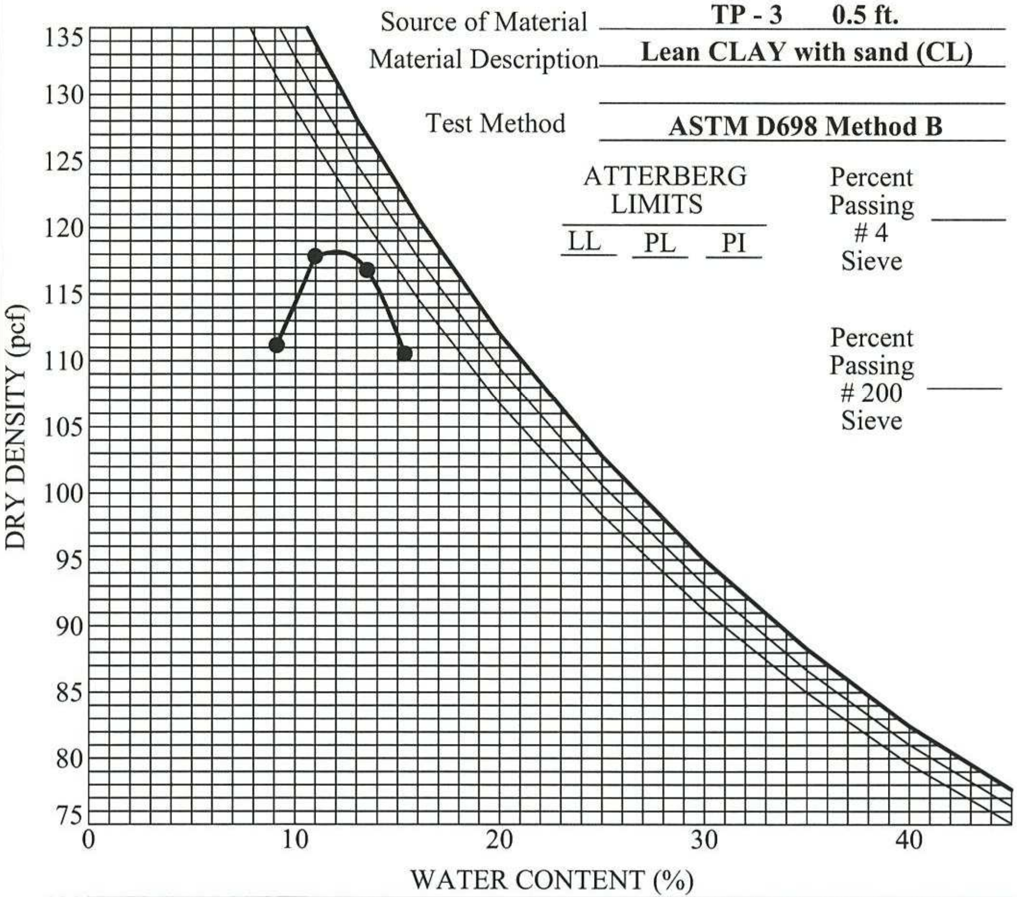
Test Method **ASTM D698 Method B**

ATTERBERG LIMITS

LL ___ PL ___ PI ___

Percent Passing # 4 Sieve ______

Percent Passing # 200 Sieve ______



TEST RESULTS

Maximum Dry Density **119.2** (pcf)

Optimum Water Content **12.2** (%)

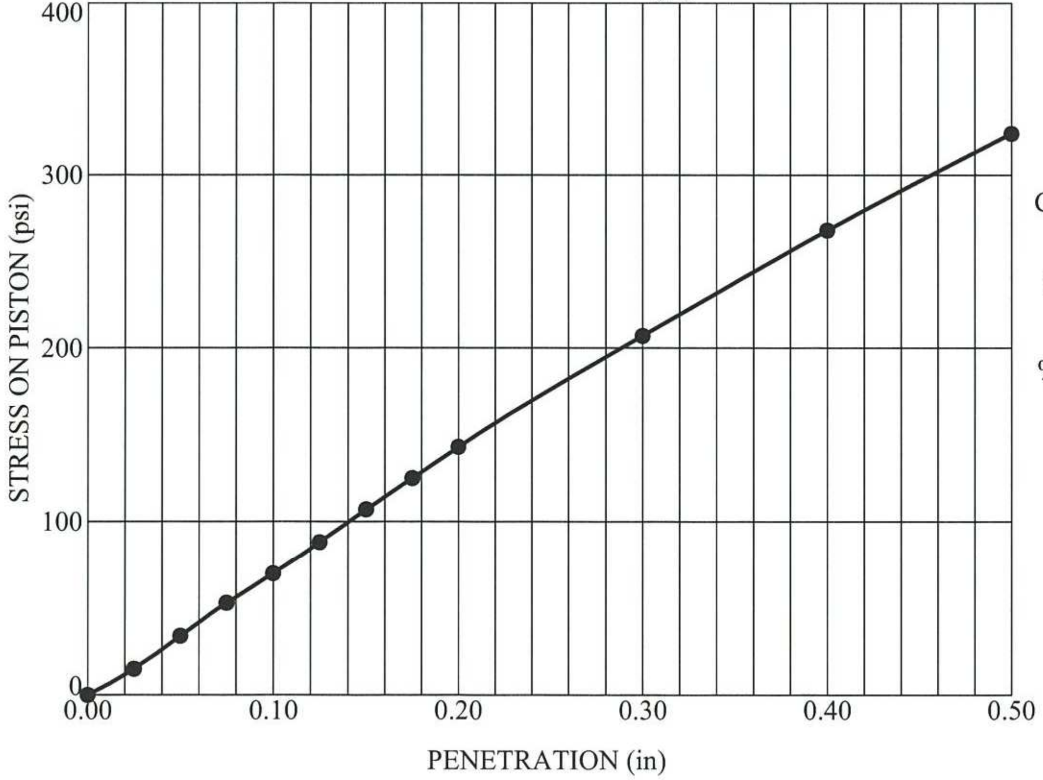
Percent Rock ______(%)

Corrected Maximum Dry Density ______(pcf)

Corrected Optimum Water Content ______(%)

Curves of 100% Saturation for Specific Gravity Equal to:

2.60, 2.70, 2.80



Dry Density **119.2** (pcf)

Relative Compaction **100** (%)

Surcharge **50** (psf)

% Standard CBR **7.00**

Swell **0.13** (%)

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COMPACTION AND CBR TEST

Geotechnical Investigation
Navajo Health Foundation - Sage Memorial
Junction of Hwy 264 and Hwy 191
Ganado, Arizona
Project Number: 01276-002

Plate

B - 5

SUMMARY OF LABORATORY TEST RESULTS TABLE

Sage Memorial Hospital | **Junction of Hwy 264 and Hwy 191, Ganado, Arizona** | **Project Number 01276-002**

SAMPLE LOCATION		NATURAL DRY DENSITY (pcf)	NATURAL MOISTURE CONTENT %	GRADATION (%)			ATTERBERG LIMITS		UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST (psf)	COLLAPSE POTENTIAL		PROCTOR		CBR (%)	CHEMICAL TESTS			UNIFIED SOILS CLASSIFICATION
Point No.	Depth (ft)			Gravel >#4	Sand	Silt and Clay <#200	Liquid Limit	Plasticity Index		Collapse (%)	Pressure (psf)	Maximum Dry Density (pcf) STD	Optimum Moisture (%) *STD		Soluable Sulfate (ppm)	Resistivity (Minimum ohm-cm)	pH	
TP - 1	0.5		12.2															Lean CLAY with sand (CL)
	2	106.8	7.3						730									Clayey SAND (SC)
	4		5.6			27.4												Clayey SAND (SC)
	12		7.1				18	6										Silty CLAY with sand (CL-ML)
	15	86	8.2							15.6	1600							Silty CLAY with sand (CL-ML)
TP - 2	3	101.6	6.6						2551									Lean CLAY with sand (CL)
	4.5	94	4.1							6.06	1600							Silty SAND (SM)
	5.5		3.5			15.3												Silty SAND (SM)
	11		5.5				20	10										Lean CLAY with sand (CL)
	12.5		7.4															Lean CLAY with sand (CL)
TP - 3	0		12.1															Lean CLAY with sand (CL)
	0.5											119.2	12.2	7	490	2100	9.3	Lean CLAY with sand (CL)
	3	96.1	3.9															Lean CLAY with sand (CL)
	14		10			48.5												Clayey SAND (SC)
TP - 4	2.5	105.4	8.7							1.14	1600							Lean CLAY with sand (CL)
	4		3.1			14.7												Silty SAND (SM)
	10		7.4				20	2										SILT with sand (ML)
	17		9.7															SILT with sand (ML)
TP - 5	0		12.9	0.4	69.7	29.9	18	6										Silty Clayey SAND (SC-SM)
	4		8.3			21.6												Clayey SAND (SC)
	6	93.9	14.4				33	20		0.36	1600							Lean CLAY (CL)
	15		5.7			22.5												Clayey SAND (SC)
TP - 6	3.5	107.2	4.1							4.7	1600							Lean CLAY with sand (CL)
	6.5		4.1			22.6												Silty SAND (SM)
	11		17.1				48	35										Lean CLAY with sand (CL)

Plate B-6

APPENDIX C

SITE GROUND MOTION [IBC SECTION 1613]

Project: Sage Memorial Hospital
Latitude = 35.7113
Logitude = -109.5483

Number: 01276-002
Date: 4/29/09
By: BMJ

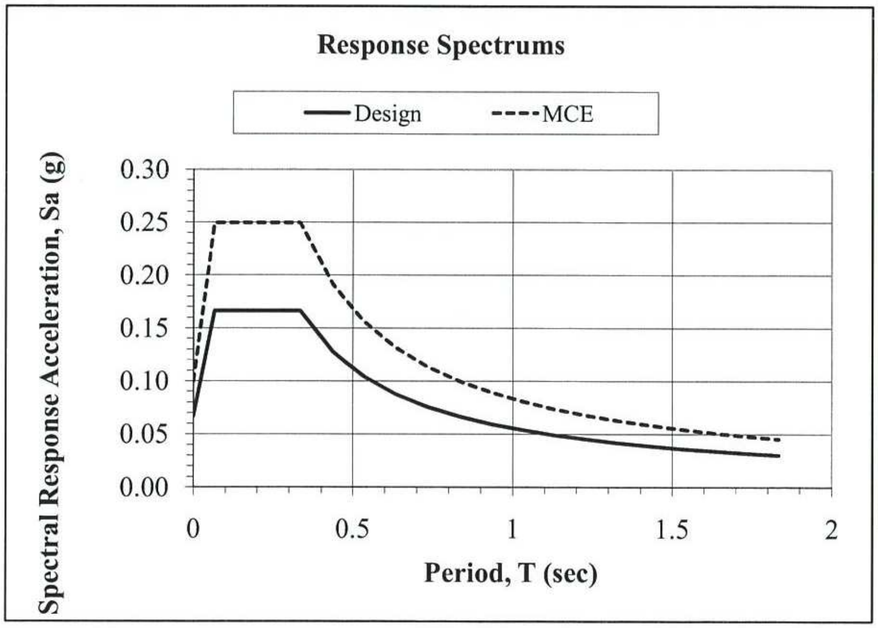
Ss = 0.208 (g) The mapped spectral accleration for short periods [1613.5]
S_1 = 0.049 (g) The mapped spectral accleration for a 1-second period

Site Class = C Table 16.13.5.2
Fa = 1.20 Table 1613.5.3(1)
Fv = 1.70 Table 1613.5.3(2)

S_{MS} = 0.250 S_{MS} = Fa*Ss *The maximum considered E.Q. spectral resonse accelerations
S_{M1} = 0.083 S_{M1} = Fv*S_1 for short and 1-second periods [1613.5.3]
MCE/PGA = 0.100 0.4*S_{MS} [In accordance with 1802.2.7]

S_{DS} = 0.166 S_{DS} = 2/3*S_{MS} *The design spectral response acceleration
S_{D1} = 0.056 S_{D1} = 2/3*S_{M1} at short and 1-second periods

T_0 = 0.067 T_0 = 0.2*S_{D1}/S_{DS}
T_s = 0.334 T_s = S_{D1}/S_{DS}
ΔT = 0.1 Time step for diagram



T (sec)	Sa (g)	Sa (MCE) (g)
0	0.07	0.10
0.07	0.17	0.25
0.33	0.17	0.25
0.43	0.13	0.19
0.53	0.10	0.16
0.63	0.09	0.13
0.73	0.08	0.11
0.83	0.07	0.10
0.93	0.06	0.09
1.03	0.05	0.08
1.13	0.05	0.07
1.23	0.05	0.07
1.33	0.04	0.06
1.43	0.04	0.06
1.53	0.04	0.05
1.63	0.03	0.05
1.73	0.03	0.05
1.83	0.03	0.05

SUMMARY OF GEOLOGIC HAZARDS

Navajo Health Foundation, Granato, Arizona **Project Number 01256-002**

Hazard	Hazard Rating*				Further Study Recommended**
	Not Assessed	Probable	Possible	Unlikely	
Earthquake					
Ground Shaking			X		See Geotechnical Report
Surface Faulting				X	
Tectonic Subsidence				X	
Liquefaction				X	
Slope Stability				X	
Flooding (Including Seiche)				X	
Slope Failure					
Rock Fall				X	
Landslide				X	
Debris Flow				X	
Avalanche				X	
Problem Soils					
Collapsible		X			See Geotechnical Report
Soluble				X	
Expansive				X	
Organic				X	
Piping				X	
Non-Engineered Fill				X	
Erosion				X	
Active Sand Dune				X	
Mine Subsidence				X	
Shallow Bedrock		X			See Geotechnical Report
Shallow Groundwater				X	
Flooding					
Streams			X		See Geotechnical Report
Alluvial Fans				X	
Lakes				X	
Dam Failure				X	
Canals/Ditches				X	
Radon	X				

* Hazard Rating :
Not assessed - report does not consider this hazard and no inference is made as to the presence or absence of the hazard at the site
Probable -Evidence is strong that the hazard exists and mitigation measures should be taken
Possible - hazard may exist, but the evidence is equivocal, based only on theoretical studies, or was not observed and furthes study is necessary as noted
Unlikely - no evidence was found to indicate that the hazard is present, hazard not known or suspected to be present

Further Study :
E - geotechnical/engineering, H - hydrologic, A - Avalanche, G - Additional detailed geologic hazard study out of the scope of this study

IGES

Project Number - 01276-002

Geotechnical Investigation
Sage Memorial Hospital
Junction of Hwy 268 and Hwy 191
Ganado, Arizona

GEOTECHNICAL MAP

Plate
A-2