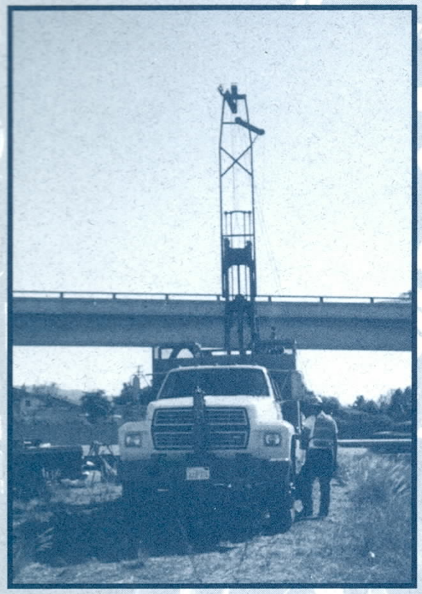
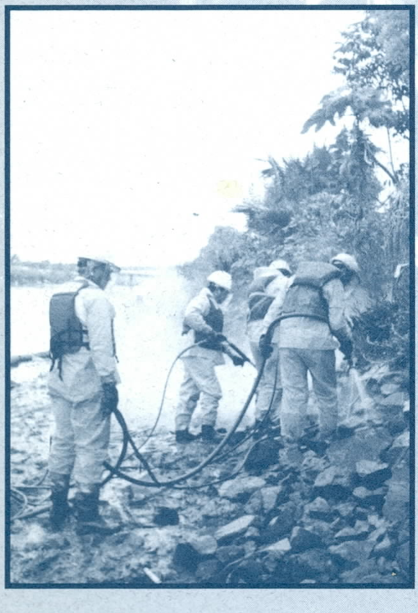
GEOTECHNICAL EVALUATION
MCDOWELL ROAD BASIN
AND STORM DRAIN
MESA, ARIZONA





Geotechnical

and

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Sciences

Consultants

Ninyo & Moore

GEOTECHNICAL EVALUATION
MCDOWELL ROAD BASIN
AND STORM DRAIN
MESA, ARIZONA

PREPARED FOR:
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January 11, 2006
Project No. 601052001

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Mr. Bob Eichinger, P.E., CFM
Kimley Horn and Associates, Inc.
7878 North 16th Street, Suite 300
Phoenix, Arizona 85020

Subject: Geotechnical Evaluation
McDowell Road Basin and Storm Drain
Mesa, Arizona

Dear Mr. Eichinger:

In accordance with our Agreement for Services dated July 11, 2005, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced project. The attached report describes our methodology, and presents our findings, conclusions, and recommendations regarding the geologic and geotechnical conditions in the basin area and along the project alignment.

We appreciate the opportunity to be of service to you during this phase of the project. If you have any questions or comments regarding this report, please call at your convenience.

Sincerely,
NINYO & MOORE

A. Tom MacDougall

Registered Professional Engineer (Civil) CERTIFICATE NO. 43700 A. TOM MacDOUGALL Date Signed 1/11/06 ARIZONA, U.S.A.

A. Tom MacDougall, P.E.
Project Engineer

Steven D. Nowaczyk

Registered Professional Engineer (Civil) CERTIFICATE NO. 34866 STEVEN D. NOWACZYK Date Signed 1.11.06 ARIZONA, U.S.A.

Steven D. Nowaczyk, P.E.
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Distribution: (6) Addressee
(2) Addressee on CD-ROM

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1. INTRODUCTION

In accordance with the Agreement for Services dated July 11, 2005, we have performed a geotechnical evaluation for proposed storm drain improvements along McDowell Road, between Hawes Road and Sossaman Road, in Mesa, Arizona. The project also includes the construction of a detention basin at the southwest corner of the intersection of Sossaman Road and McDowell Road. The purpose of our evaluation was to observe existing subsurface conditions along the project alignment and to formulate recommendations relative to the design and construction of the planned improvements.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Reviewing readily available geotechnical reports, geologic maps, as-built data, and aerial photographs.
- Performing a site reconnaissance, notifying Arizona Blue Stake of proposed subsurface work, and coordinating layout of the proposed boring locations with utility companies prior to drilling.
- Drilling, logging, and sampling 15 exploratory test borings within the proposed detention basin and along the storm drain alignment, each extending to depths ranging from about 4.5 to 20 feet below the ground surface (bgs). The boring logs are presented in Appendix A.
- Performing five seismic refraction surveys to evaluate excavation characteristics along the project alignment.
- Excavating five test trenches using a backhoe to evaluate excavation characteristics, observe soil conditions, and correlate geophysical testing.
- Performing pavement cores at two locations along McDowell Road in areas above the proposed storm drain. Photographs of the cores are presented in Appendix E.
- Testing selected soil samples in our laboratory to evaluate in-situ moisture content and dry density, grain-size distribution, Atterberg limits, expansion index, response to wetting behavior (hydro-consolidation), standard Proctor moisture-density relationships, R-values, unconfined compression strength of the cemented soils, and corrosion characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the logs in Appendix A and/or in Appendix B.
- Performing agronomic soil testing to assist in the landscaping of the detention basin.

- Preparing this report to present our findings, conclusions, and recommendations regarding the design and construction of the planned improvements.

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

3. SITE DESCRIPTION

The project alignment is located within Township 1 North, Range 6 East, Sections 5, 6 and Township 2 North, Range 7 East, Section 32. The alignment extends along McDowell Road, from just west of Sossaman Road to Hawes Road, and about two miles south of the Salt River in Mesa, Arizona. The general location of the project area is depicted on the Project Location Map (Figure 1). At the time of our evaluation, the site consisted of an asphalt paved roadway bordered by residences and undeveloped desert.

According to the *Buckhorn, Arizona-Maricopa Co., 7.5-Minute United States Geological Survey (USGS) Topographic Quadrangle Map, (1982),* the average elevation in the detention basin area is approximately 1,640 feet relative to mean sea level (MSL). The ground surface elevations along McDowell Road range from roughly 1,645 feet MSL at the west end to roughly 1,750 feet MSL at the east end. Based on the information obtained from this map, the topography in the project vicinity slopes from the northeast down to the southwest.

Four aerial photographs were reviewed for this project. A 1967 United States Department of Agriculture (USDA) aerial photograph depicts the site as an unpaved road surrounded by farmland and undeveloped desert. A 1992 USGS aerial photograph depicts the site as a paved roadway primarily surrounded by undeveloped desert and scattered drainages. The aerial photograph also shows a few residences surrounding the roadway. The detention basin area was depicted as undeveloped desert. A 1999 aerial photograph from *Landiscor's Phoenix Real Estate Photo Book* and a 2004 aerial photograph from the Maricopa County Assessor's website also show the basin area as undeveloped desert. The storm drain alignment is depicted as a paved roadway surrounded by residences and scattered parcels of undeveloped desert, similar to the current conditions. Our evaluation of the aerial photographs and visual reconnaissance did not indicate any large disturbed areas that might be indicative of past undocumented development or areas of large-scale earthwork.

4. PROPOSED CONSTRUCTION

The proposed improvements associated with this project include:

- Construction of a new, landscaped detention basin to the southwest of the intersection of McDowell Road and Sossaman Road;
- Installation of storm drain piping (diameters ranging from 60 inches to 102 inches) and appurtenances using cut and cover techniques. Stormwater from an existing private drainage system at Hawes Road and various inlets along McDowell Road will be diverted through the storm drain to the existing Las Sendas Wash or, in high flow situations, the new detention basin; and
- Restoring the pavement sections for roadways which overlie the storm drain alignment.

The new detention basin will occupy approximately 15,000 square feet and the base elevation will be approximately 10 to 15 feet lower than the surrounding ground surface elevations. During low flow events, storm water will be diverted to the Las Sendas Wash. During high-flow events, a subsurface weir/splitter will divert runoff into the landscaped detention basin.

We have assumed that the conveyance pipe will be placed below other existing utilities and invert elevations will be up to approximately 17 feet bgs. It is our understanding that reinforced concrete pipe (RCP) will be used for the stormwater lines and will be installed using cut-and-cover techniques. We understand that Controlled Low Strength Material (CLSM) will be used as backfill from the invert elevation to the spring line. According to the proposed design concept, various pipe diameters are planned along various sections of this storm drain segment ranging from 54 inches at the inlet to 102 inches at the outlet.

5. FIELD EXPLORATION AND LABORATORY TESTING

Ninyo & Moore utilized a phased approach within the proposed detention basin area and along the proposed storm drain alignment in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Four phases were utilized and consisted of: hollow-stem auger borings, seismic refraction surveys, test trenches, and sonic borings. Each phase is discussed below.

On August 2 and 3, 2005, Ninyo & Moore conducted the first phase of the subsurface exploration, which consisted of the drilling, logging, and sampling of 13 small-diameter borings and coring through the existing pavement section. The borings were drilled using a CME-75 truck-mounted drill rig equipped with hollow-stem augers. The borings, denoted as B-1 through B-13, were planned to extend to about 20 feet bgs. However, auger refusal was encountered in borings B-2 through B-13 at depths shallower than planned. As such, those borings were terminated at depths ranging from 4.5 to 18 feet bgs. Boring B-1 extended about 19 feet bgs. Bulk and relatively undisturbed soil samples were collected at selected intervals. Detailed descriptions of the soils encountered at each boring location are presented on the boring logs in Appendix A. The pavement section was cored to measure the thickness of the asphaltic concrete (AC) and the underlying aggregate base. Approximately 5.5 inches of AC over 6 inches of AB was measured in PC-1 and approximately 5 inches of AC over 5 inches of AB was measured at PC-2. The approximate locations of the borings and pavement cores are shown on Figure 2.

Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and American Society for Testing and Materials (ASTM) D 2488) by observing cuttings and drive samples. Collected ring samples were trimmed in the field, wrapped in plastic bags, and placed in cylindrical plastic containers to retain in-place moisture conditions. Similarly, the Standard Penetration Test and bulk samples were sealed in plastic bags to retain their approximate in-place moisture.

The second phase of the field exploration included seismic refraction surveys. The surveys were performed on September 28, 2005 to evaluate rippability characteristics. A SmartSeis S12 seismograph and 12 geophones were utilized to collect generalized and approximate velocities of seismic waves transmitted through subsurface soils. Correlations between the seismic wave velocities and excavatability, and additional discussion on the seismic refraction surveys are provided in Appendix C. The locations of the surveys are shown on Figure 2.

Five test trenches were excavated on September 29, 2005 to evaluate excavatability, to collect samples, and to compare our observations with the seismic refraction results. Test trenches were excavated near the locations of the seismic refraction surveys and are also shown on Figure 2. A Case 580 backhoe, with a reach of approximately 11 feet, was used to excavate the trenches. A Ninyo & Moore geologist was on-site to log the excavated soils and to collect bulk and chunk samples at selected intervals. Logs of the trenches are included in Appendix A.

The fourth phase of exploration included advancing two borings to approximately 20 feet bgs using sonic drilling techniques. Sonic drilling employs the use of high frequency mechanical vibration and rotation to advance a steel casing into the subsurface materials. It can penetrate many soil or rock strata on which conventional hollow-stem auger would refuse. The sonic borings were advanced on October 7, 2005 at the locations shown on Figure 2. Ninyo & Moore personnel logged the observed soils and collected samples at selected intervals. Detailed descriptions of the soils encountered at the two boring locations are presented on the boring logs in Appendix A.

The soil samples collected from our field activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona for geotechnical laboratory analysis. The laboratory testing included evaluation of the following:

- In-situ moisture content and dry density;
- Grain-size distribution;
- Atterberg limits;
- Standard Proctor moisture-density relationships;
- Response to wetting behavior (hydro-consolidation)
- Expansion Index;
- R-value; and
- Corrosion characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides).

The results of the laboratory tests are presented on the logs in Appendix A and/or in Appendix B. Also, Appendix B contains additional descriptions of each laboratory test performed. Agronomic soil testing was performed on selected samples of the basin soils by Fruit Growers Laboratory of Santa Paula, CA, and the test results 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 Sonoran Desert Section of the Basin and Range Physiographic Province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 13 million years ago during the mid- to late-Tertiary age. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains, as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains. The surficial geology of the site is comprised of 3 units. These units consist of late Pleistocene (10,000 to 250,000 years) alluvial fan and terrace deposits, a combination of late Pleistocene and Holocene deposits (< 250,000 years), and middle Pleistocene (250,000 to 750,000 years) alluvial fan and terrace deposits. Particle sizes in the late Pleistocene deposits range from sand to cobbles and boulders. These soils have moderate soil development with argillic horizons and calcic horizons (stage I to III). The second unit is a combination of both late Pleistocene and Holocene alluvial deposits. This unit has a variety of young and older soils with grain sizes ranging from silt to boulders. The middle Pleistocene deposits consist of particle sizes ranging from sand to boulders, fining downstream. These deposits have strong soil development characterized by argillic horizons and calcic horizons (stage II to IV) (Pearthree and Huckleberry, 1994). Descriptions of the soils encountered during our evaluation are presented in the following section.

6.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing and our understanding of the general geology of the area. The following sections provide a generalized description of the materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1. Fill

Fill soils were encountered at the surface of borings B-6 and B-13, extending to depths ranging from approximately 1.5 to 3.5 feet bgs. The fill generally consisted of silty sand.

6.2.2. Alluvium

Alluvium was encountered at the surface of borings B-1 through B-5, B-7 through B-12, B-1A and B-2A, and below the fill soils in B-6 and B-13. Generally, the alluvium extended to the total depth explored. This material generally consisted of silty or clayey sand with gravel. Scattered caliche filaments and weakly to strongly cemented soils were observed within the alluvium. In some cases, auger and backhoe refusal was encountered and therefore the explorations did not reach the target depths as explained in Section 5 of this report. Table 1 summarizes the depth to auger refusal encountered in the borings (if applicable). Table 2 summarizes the depth to backhoe refusal for the test trenches. The depths to auger and backhoe refusal may not correlate with field rippability.

Table 1 – Summary of Depths to Auger Refusal

Boring Number	Depth of Auger Refusal (feet)	Boring Number	Depth of Auger Refusal (feet)
B-2	4.5	B-8	6
B-5	16	B-9	18
B-4	16	B-10	6
B-5	16	B-11	11
B-6	16	B-12	9.5
B-7	6	B-13	16

Table 2 – Summary of Depths to Backhoe Refusal

Test Trench ID	Depth of Backhoe Refusal (feet)
L-2	6.9
L-3	7.5

6.3. Groundwater

Groundwater was not encountered in our borings. Based on well data from the Arizona Department of Water Resources, the approximate depth to groundwater is on average over 100 feet bgs. In general, groundwater does not need to be considered for the design and the construction of the project. However, groundwater levels can fluctuate due to seasonal variations, irrigation, groundwater withdrawal or injection, and other factors.

7. GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site, including land subsidence and earth fissures, faulting and seismicity, surface rupture, and liquefaction.

7.1. Land Subsidence and Earth Fissures

Groundwater depletion due to groundwater pumping has resulted in land subsidence and earth fissures in numerous alluvial basins in southern Arizona. It has been estimated that subsidence has affected more than 3,000 square miles and has caused damage to a variety of engineered structures and agricultural land (Schumann and Genualdi, 1986). From 1948 to 1983, excessive groundwater withdrawal has been documented in several alluvial valleys where groundwater levels have been reportedly lowered by up to 500 feet. With such large depletions of groundwater, the alluvium has undergone consolidation resulting in large areas of land subsidence.

In Arizona, earth fissures are generally associated with land subsidence and pose an on-going geologic hazard. Earth fissures generally form near the margins of geomorphic basins where significant amounts of groundwater depletion have occurred. Reportedly, earth fissures have also formed due to tensional stress caused by differential subsidence of the unconsolidated alluvial materials over buried bedrock ridges and irregular bedrock surfaces (Schumann and Genualdi, 1986).

Based on our field reconnaissance and review of the referenced material, there are currently no known earth-fissures underlying the subject alignment. Based on our research, the closest earth fissure to the site is located approximately 5 miles to the southeast of the project site, where water levels have dropped by approximately 300 feet or more. While the future occurrence of land subsidence and earth fissures cannot accurately be predicted, continued groundwater withdrawal in the area may result in subsidence and the formation of new fissures or the extension of existing fissures. Continued subsidence may increase the storm drain grade and may cause some areas of pipe failure.

7.2. Faulting and Seismicity

The site lies within the Sonoran Zone, which is a relatively stable tectonic region located in southwestern Arizona, southeastern California, southern Nevada, and northern Mexico (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations, review of pertinent geologic data and analysis of aerial photographs, faults are not located on or adjacent to the project. The closest fault to the site is the Sugarloaf fault, located approximately 18 miles to the northeast of the site (Pearthree, 1998). Up to 5 meters of displacement has occurred along this fault within upper and uppermost Pleistocene deposits, but middle Holocene deposits are not displaced.

Based on a Probabilistic Seismic Hazard Assessment for the Western United States, issued by the USGS (1999), the site is located in a zone where the peak ground accelerations that have a 10 percent, 5 percent, and 2 percent probability of being exceeded in 50 years are 0.05g, 0.07g and 0.11g, respectively. Due to the relatively low ground motions, seismic hazards (e.g., liquefaction, ground shaking, etc.) are considered to be negligible. Seismic design parameters according to the 2003 International Building Code (IBC) are presented in the following table.

Table 3 – Seismic Design Parameters

Parameter	Value	2003 IBC Reference
Site Class Definition	C	Table 1615.1.1
Site Coefficient F_a	1.2	Table 1615.1.2 (1)
Site Coefficient F_v	1.7	Table 1615.1.2 (2)

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 design and construction of the proposed project, as appropriate. Geotechnical considerations include the following:

- Refusal was encountered in 12 of 13 auger borings and in two of five test trenches. Borings and test trenches exposed strata with strong caliche cementation. It should be anticipated that the on-site soils will be difficult to excavate and will require specialized excavation equipment and techniques (e.g., hoe-ram, rock saw, blasting, etc.).
- Although cemented soils were encountered along the proposed alignment, due to interbedded layers of uncemented sandy material, the likely vibrations that will exist near open trenches (due to the adjacent roadway and construction activity), and the potential consequence of slope instability (road closure, structural damage), an OSHA soil-type "C" should be used for planning excavation side slopes. Due to the diameter of the pipe, and according to OSHA requirements, shoring will likely be needed during construction.
- We estimate an earthwork (shrinkage) factor of 5 to 15 percent for this project.
- Soils generated from on-site excavation activities that exhibit a very low to low expansion potential can generally be used as engineered fill. The on-site soils that we tested met this criterion. Cobbles and soil particles larger than 3 inches should not be used as backfill material unless appropriately processed.

- Groundwater was not observed in our borings. The groundwater table in the area on average is more than 100 feet bgs based on the nearby well data. In general, groundwater is not anticipated to be a design or construction consideration. However, groundwater levels can fluctuate due to seasonal factors. If considerable rainfall occurs or is anticipated during or near the time of construction, the contractor may wish to advance test holes prior to excavation to see if perched water or groundwater is present in the excavation zone.

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

- Corrosivity test results indicate that subgrade soils at the site may be corrosive to ferrous metals, and the sulfate content of the soils present a negligible sulfate exposure to concrete.

9. RECOMMENDATIONS

Based on our understanding of the project, the following recommendations are provided for the design and construction of the proposed storm drain. If the proposed construction is changed from that discussed in this report, Ninyo & Moore should be contacted for additional recommendations.

9.1. Storm Drain Considerations

The following sections provide our recommendations relating to the storm drain construction and design. In general, the specifications contained in Maricopa Association of Governments (MAG), *Uniform Standard Specifications and Details for Public Works Construction (2002)* are expected to apply unless noted.

9.1.1. Site Preparation

Construction areas should be cleared of unsuitable materials, including grass, weeds, asphalt pavement, concrete, old construction debris, and any other material that might interfere with the performance or progress of the work.

Within the limits of clearing and below the ground surface, roots, deleterious, or other objectionable material should be grubbed. Old pipes, channel lining, underground structures, vegetation, and debris, or waste should be removed if found along the storm drain alignment and disposed of at a legal dumpsite. Obstructions that extend below finish grade, if present, should be removed and resulting voids filled with compacted soil.

If the storm drain is to be installed near or beneath the foundation of an existing structure or utility, the existing structure or utility should be supported to reduce the potential for damage, and, if necessary, the drain pipe encased in concrete to accommodate imposed structural loads.

It may be desirable to identify structures or critical features that are very 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 of existing improvements that might result from this work. For other facilities (e.g., churches, homes, etc.), where excavation-induced settlement may be a concern, baseline elevations and horizontal control data should be recorded.

9.1.2. Trench Excavations

It is our opinion that the excavation of the on-site materials can generally be accomplished to the assumed earthwork depths (up to about 18 feet deep) with heavy earthmoving equipment and specialized excavation equipment in good operating condition. However, during the excavation, there is a potential for encountering very strongly cemented soils that could require rock breaking equipment or blasting. Contractors should make their own evaluations of excavatability and plan means and methods in accordance with their evaluation as well as project specifications. Approximate velocities from seismic refraction testing are provided in Appendix C.

Depending on the excavation method used, the proposed excavations may generate oversize material (particles larger than 3 inches) that will not be suitable for reuse as trench backfill. Screening, disposal, and/or crushing of this material should be anticipated if reuse is considered.

Excavations in soils with cemented material may tend to have rugged or irregular bottoms or sidewalls. In order to provide more consisted support and grade control to the pipe, we recommend that the proposed storm drains be supported on 12 inches or more of moisture-conditioned and compacted material such as sand, gravel, or aggregate base, with a particle size of 3/4-inch or less. If gravel or aggregate base is used for bedding material, a 4-inch layer of compacted sand should be used as a cushion between the pipe and foundation material. On-site materials with a particle size of 3/4-inch or less may be considered for pipe bedding if appropriately processed, moisture-conditioned, and compacted. Care should be exercised by the contractor to avoid damaging the corrosion protection on the CMP. Uniform pea gravel or crushed chips are not acceptable for use as foundation material. A pipe bedding detail is presented on Figure 4.

Depending on the gradation of the backfill materials used, it may be appropriate to line the trenches with a geotextile at some locations. Such locations may include wash crossings or areas prone to ponding or other standing water.

It may be difficult to place backfill against these irregular surfaces. When backfilling, care should be taken to fill voids with compacted material so that excessive settlement of the backfill will not occur.

We anticipate that the soil conditions and stability of the excavation sidewalls will vary along the storm drain alignment. Soils with higher fines content may stand vertically for a short time (less than 12 hours) with little sloughing. However, as the soil dries after excavation or as the excavations are exposed to rainfall, sloughing may occur. Soils with low cohesion (e.g., predominately sandy or gravelly material), will likely slough or cave during excavation, especially if wet or saturated. Additionally, vibrations caused by nearby traffic or construction equipment will accelerate sloughing.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration (OSHA) regulations, for employees working in excavations that may expose them to the danger of moving ground. Reducing the inclination of the sidewalls of the excavations, where feasible, may increase the stability of the excavations. If construction or earth material is stored or equipment is operated near an excavation, flatter slope geometry or stronger shoring should be used during construction. The OSHA regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches over 20 feet deep should be designed by the contractor's engineer based on alignment-specific geotechnical analyses. Although cemented layers were observed, for planning purposes and according to OSHA soil classifications, a "Type C" soil should be considered due to the presence of interbedded layers of uncemented soils and the anticipated roadway vibrations. Upon making the excavations, soil classification and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. This evaluation may result in re-classifying the soil type to "Type B" in some areas. Trench side walls can be sloped at a ratio of 1.5 horizontal (H) to 1 vertical (V) for "Type C" soils and at a ratio of 1 (H) to 1(V) for "Type B" soils.

In general, temporary slopes should be inclined no steeper than 1.5 (H):1(V) up to a depth of 20 feet below the surface. Due to the diameter of the pipe and MAG specifications, temporary excavations will likely need shoring. Lateral earth pressures recommended for braced excavations are presented on Figure 3. The earth pressure values in Figure 3 were derived by assuming an internal angle of friction of 34 degrees and an average total unit weight of 110 pcf for the depth of the excavation. If construction or earth material is stored or equipment is operated near an excavation, flatter slope geometry or stronger shoring should be used during construction. Temporary excavations that encounter 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 dewatering are provided in Section 9.1.3.

9.1.3. Construction Dewatering

Generally, we anticipate that significant groundwater will not be encountered along the proposed storm drain alignment. However, because the project excavations will be associated with existing drainage channels, the trench soils might capture surface water and become saturated and unstable. The contractor should divert surface water away from the trench or be made responsible for the design, timing, construction, operation, maintenance, and removal of a dewatering system(s), if needed. The system should prevent migration and pumping of soil fines with the discharge water. It is anticipated that any dewatering can likely occur by pumping from the trenches or sumps located outside of, and below the limits of the main excavation.

9.1.4. Trench Widths

The minimum trench width should be the pipe diameter plus 6 inches on each side. The maximum trench widths should be the pipe diameter plus 36 inches. In general, trench widths should be in accordance with MAG Section 601. The trench width should be taken as the clear distance between trench walls or the inside face-to-face distance between the ground support systems. This distance is intended to allow space to place the CLSM using techniques that lessen the opportunity for voids to form in the pipe zone.

9.1.5. Controlled Low Strength Material

We understand that CLSM will likely be used for backfill and extend from the pipe invert to approximately the pipe's spring line. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. The use of CLSM has some advantages:

1. A narrower trench can be used, thereby minimizing the quantity of soil to be excavated and possibly reducing disturbance to the near-by traffic;

2. The support given to the pipe is generally better, and greater values of modulus of soil reaction (E') can be used to design the pipe;

3. Because little compaction is needed to place CLSM, there is less risk of damaging the pipe;

4. If native soils are used to formulate the CLSM, less imported material will be needed; and

5. 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 (2004) or Standard Specifications for Public Works Construction (American Public Works Association, 1991) and applicable City of Mesa specifications. 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. If on-site materials are used for the aggregate mixture, test batches may be needed to observe conformity with strength requirements. If desired, a non-cement flowable backfill (e.g., fly ash) may be considered in lieu of CLSM, but should be carefully reviewed by the geotechnical engineer and approved by the engineer of record.

Buoyant or uplift forces on the piping should be considered when using CLSM and prudent construction techniques may require multiple pours to avoid inducing excessive uplift forces. The construction methods should not allow for the storm drain 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.

9.1.6. Trench Backfill

Trench backfill material above the spring line of the storm drain (above the CLSM) should be moisture conditioned to within 2 percent of its laboratory optimum and mechanically compacted to a relative compaction of 95 percent or more as evaluated by ASTM D 698-00. The trench backfill in the upper 2-foot zone (2 feet below pavement/flatwork sections) should also be moisture conditioned to within 2 percent of its laboratory optimum; however, in this zone the material should be mechanically compacted to a relative compaction of 100 percent or more as evaluated by ASTM D 698-00.

Lift thickness for backfill will be dependent upon the type of compaction equipment utilized, but should generally be placed in uniform lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe or other structures during the compaction of the backfill. Backfilling should generally be accomplished in a manner consistent with the standards provided by MAG (2002) and applicable City of Mesa specifications and/or amendments.

Soils generated from on-site excavation activities (excluding cobbles and large diameter particles) or imported soils that exhibit very low to low expansion potential are generally suitable for use as engineered fill. Very low to low expansion potential soils are defined as having an Expansion Index (by UBC Standard No. 18-2) of 50 or less and a Plasticity Index (PI) less than 15. Laboratory tests performed on near-surface soil samples obtained from our exploratory borings indicated Expansion Index values of 2 and 0, demonstrating a very low expansion potential. Furthermore, Atterburg test results indicated PIs of 7, 6, 0, and 9. Therefore, the soils encountered along the trench alignments, as well as processed materials generated during construction, should generally be suitable for reuse as trench backfill provided they are free of organic material, clay lumps, debris, and rocks or chunks greater than 3 inches in diameter. Additionally, suitable fill should not include deleterious or organic material, clay lumps, construction debris, rock particles, and other non-soil fill materials larger than 3 inches in diameter. This material should be disposed of off-site or in non-structural areas. Some screening of the on site soils may be needed. The content of rock in the backfill greater than 1-1/2 inches in diameter should not exceed 40 percent by weight.

We recommend that additional observation, soil sampling, and possible laboratory testing be conducted during construction to evaluate the presence of any unsuitable soils not encountered in our borings. Based on our observations and laboratory testing, we estimate an earthwork (shrinkage) factor of 5 to 15 percent for the on-site soils.

Imported fill, if utilized, should consist of clean, 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 greater than 2,000 ohm-cm, chloride content less than 25 parts per million [ppm], and soluble sulfate content of less than 0.1 percent). The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

9.1.7. Soil Parameters for Pipeline Design

Based on our field observations, our experience with similar materials, and our laboratory testing, a unit weight of 125 pcf can be estimated for engineered fill derived from on-site excavations. If import fill is used for trench backfill, a unit weight of 130 pcf may be estimated for use in design.

The modulus of soil reaction (E') is used to characterize the stiffness of the backfill placed on the sides of a buried pipe for the purpose of evaluating deflection caused by the weight of the backfill over the pipe. As mentioned previously, CLSM will be used and it is our understanding that the depth of cover will range from about 5 feet to 12 feet. We therefore recommend a general E' value of 1,800 psi.

The coefficient of friction between the soil and the pipe (or in this case the CLSM) depends upon the type of each material in the interaction. We understand that RCP will be utilized as the storm drain pipe. For planning purposes, we suggest a coefficient of friction, μ, of 0.35. The manufacturer of the pipe should be consulted for this parameter once the exact pipe material has been chosen.

9.2. Pavement Restoration

The following sections present our assumptions and recommendations for the flexible pavement sections to be restored following the storm drain installation. We understand that the affected reach of McDowell Road will not be improved (i.e., redesigned with new traffic data and pavement thicknesses), but restored. We assumed that the subgrade would be prepared according to the trench zone backfill described in Section 9.1.6.

9.2.1. Existing Pavement Section

During our field exploration activities, Ninyo & Moore advanced two pavement cores to evaluate the thickness of the roadway section. For pavement core PC-1, the AC was approximately 5.5 inches thick was underlain by about 6 inches of aggregate base (AB). For pavement core PC-2, the AC was approximately 5 inches thick which was underlain by approximately 5 inches of AB. Although some minor pavement distress was noted, a pavement evaluation was not part of this study. Based on our understanding that MCDOT was not planning on improving McDowell Road, we have assumed that the county is generally satisfied with the current pavement performance.

9.2.2. R-value

The surface soils encountered in the borings typically consisted of silty sand. Table 3 summarizes the laboratory and correlated R-values from the borings.

Table 4 – R-Value Summary

Boring No.	Sample Depth (ft.)	Correlated R-Value	Laboratory R-Value
B-1	0-5	--	72
B-5	1-2.5	64	--
B-5	0-5	--	69

9.2.3. Recommended Asphalt Pavement Sections

We recommend that the pavement sections provided in Table 4 be used for the pavement restoration associated with this project.

Table 5 – Recommended Asphalt Pavement Sections

Street	Layer	Thickness (Inches)
McDowell Road from Hawes Road to Sossaman Road	Bituminous Surface Course (MAG 12.5 mm)	3.0
	Bituminous Base Course (MAG 19 mm)	3.0
	Aggregate Base Course (MAG Section 702)	6.0

The recommended pavement thickness assumes that the above pavement section is founded on improved soil as needed, as outlined in Section 9.1.6. AB material should be compacted to a relative compaction of 100 percent or more of the maximum dry density, as evaluated by ASTM D 698-00, at a moisture content within approximately 2 percent of optimum.

9.3. Concrete Flatwork

To reduce the potential manifestation of distress to exterior concrete flatwork (such as curbs and sidewalks) due to movement of the underlying soil, we recommend that such flatwork (if utilized for this project) be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Additionally, we recommend that concrete flatwork be supported on 9 or more inches of adequately moisture-conditioned and compacted fill (in accordance with Section 9.1.6 of this report). Positive drainage should be established and maintained adjacent to flatwork.

9.4. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the storm drain pipe and structures. Corrosion potential was evaluated using the results of laboratory testing of a near-surface soil sample obtained during our subsurface evaluation that was 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 near-surface sample tested from exploratory borings B-1, B-5, and B-12. The pH results are 7.7, 8.8, and 8.6 respectively, which is considered to be alkaline. The minimum electrical resistivity measured for the near-surface samples from the exploratory borings B-1, B-5, and B-12 are 4,514 ohm-cm, 2,736 ohm-cm, and 1,642 ohm-cm respectively, which represents a moderately corrosive environment to ferrous metals. The chloride content of the samples tested from exploratory borings B-1, B-5, and B-12 was measured to be 41 ppm, 10 ppm, and 40 ppm respectively, which also may be corrosive to ferrous metals. The soluble sulfate content of the soil samples for exploratory borings B-1, B-5, and B-12 were measured to be 0.010 percent, 0.001 percent, and 0.004 percent respectively, which is considered to represent negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are likely corrosive to ferrous metals. Therefore, special consideration should be given to the use of heavy gauge, corrosion protected steel for use if there is potential for contact (or close proximity) to soil.

9.5. Concrete

Laboratory chemical tests performed on selected samples of on-site soils indicated sulfate contents of 0.010, 0.001, and 0.004 percent by weight. Based on the following IBC table, the on-site soils should be considered to have a negligible sulfate exposure to concrete.

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

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Cement Type	Maximum Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	Minimum 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	4,000
Severe	0.20 - 2.00	V	0.45	4,500
Very severe	Over 2.00	V plus pozzolan[3]	0.45	4,500

[1] A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 19-A-2).
[2] Seawater.
[3] Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Notwithstanding the sulfate test results and due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and local practice, we recommend the use of Type II cement for construction of concrete structures at this site. Due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, pozzalon or admixtures designed to increase sulfate resistance may be considered.

The concrete should have a water-cementitious materials ratio no greater than 0.45 by weight for normal weight aggregate concrete. The structural engineer should select the concrete design strength based on the project specific loading conditions.

9.6. Site Drainage

Surface drainage should be provided to divert water off of paved surfaces. Surface water should also not be permitted to pond on or below pavement areas. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or greater away from the pavements. To deter accumulation of water below the new pavement sections, the bottom of the overexcavated zone below the new pavement should be sloped toward the edges of the roadway.

9.7. Pre-Construction Conference

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

9.8. 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 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 reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11. SELECTED REFERENCES

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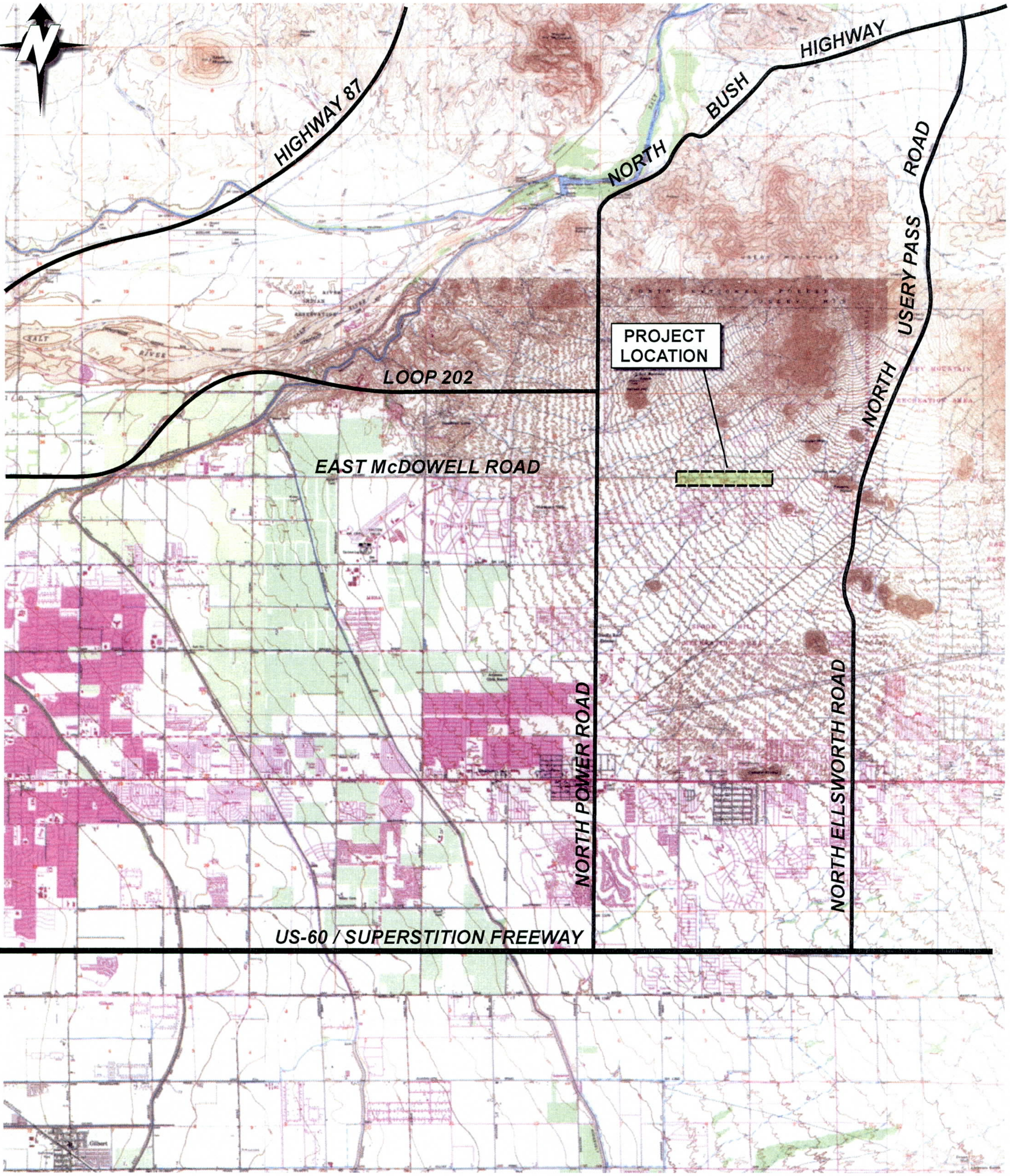

0 8300

Approximate Scale:
1:100,000
1 inch = 8300 feet

SOURCE: U.S. Geologic Service 7.5 minute topographic map, East Mesa, Arizona.

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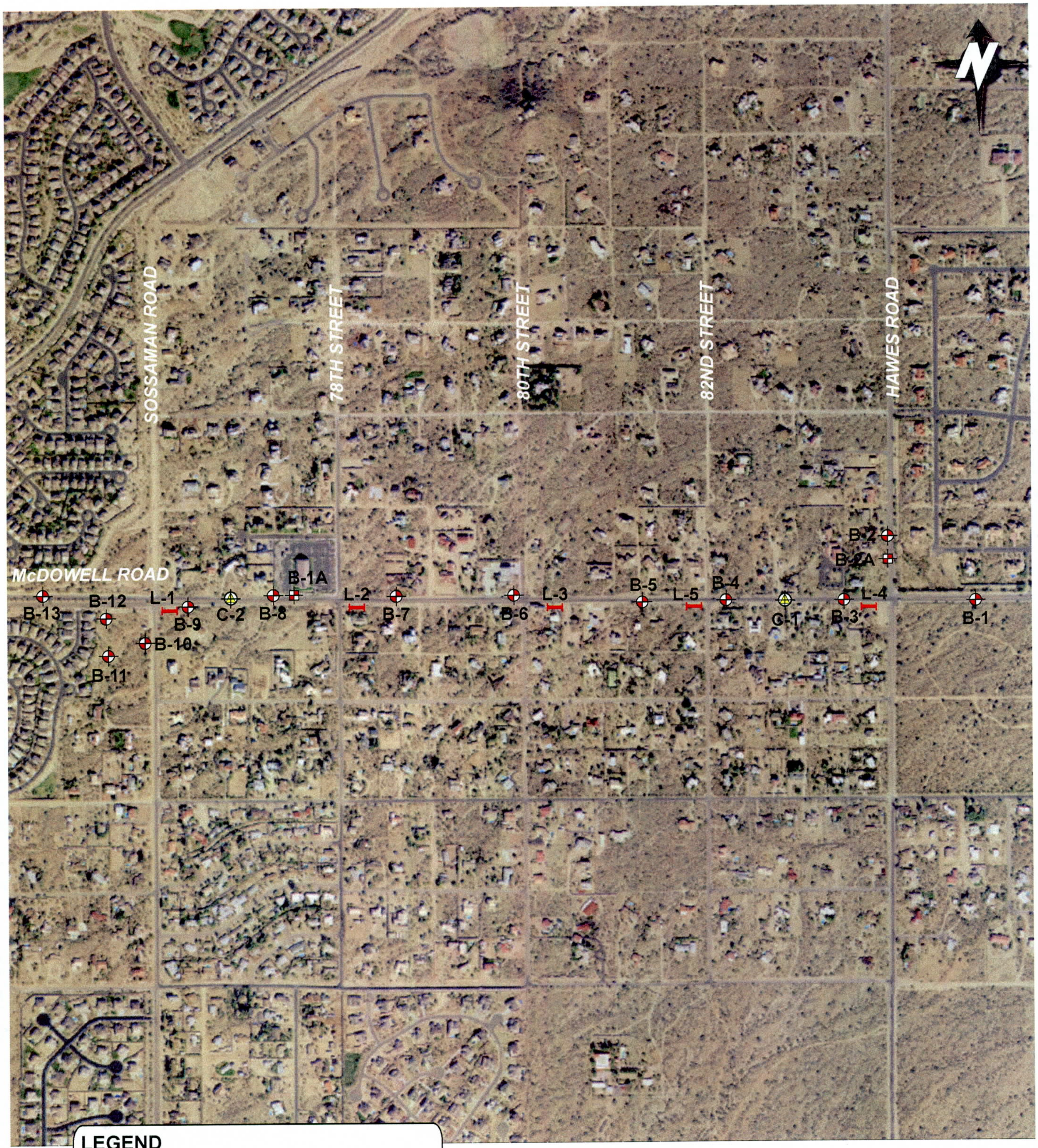
PROJECT LOCATION MAP

McDOWELL ROAD BASIN AND STORM DRAIN
MESA, ARIZONA

FIGURE 1

PROJECT No:	FILE No:	DATE:
601052001	1052vmap1005	01/06



LEGEND

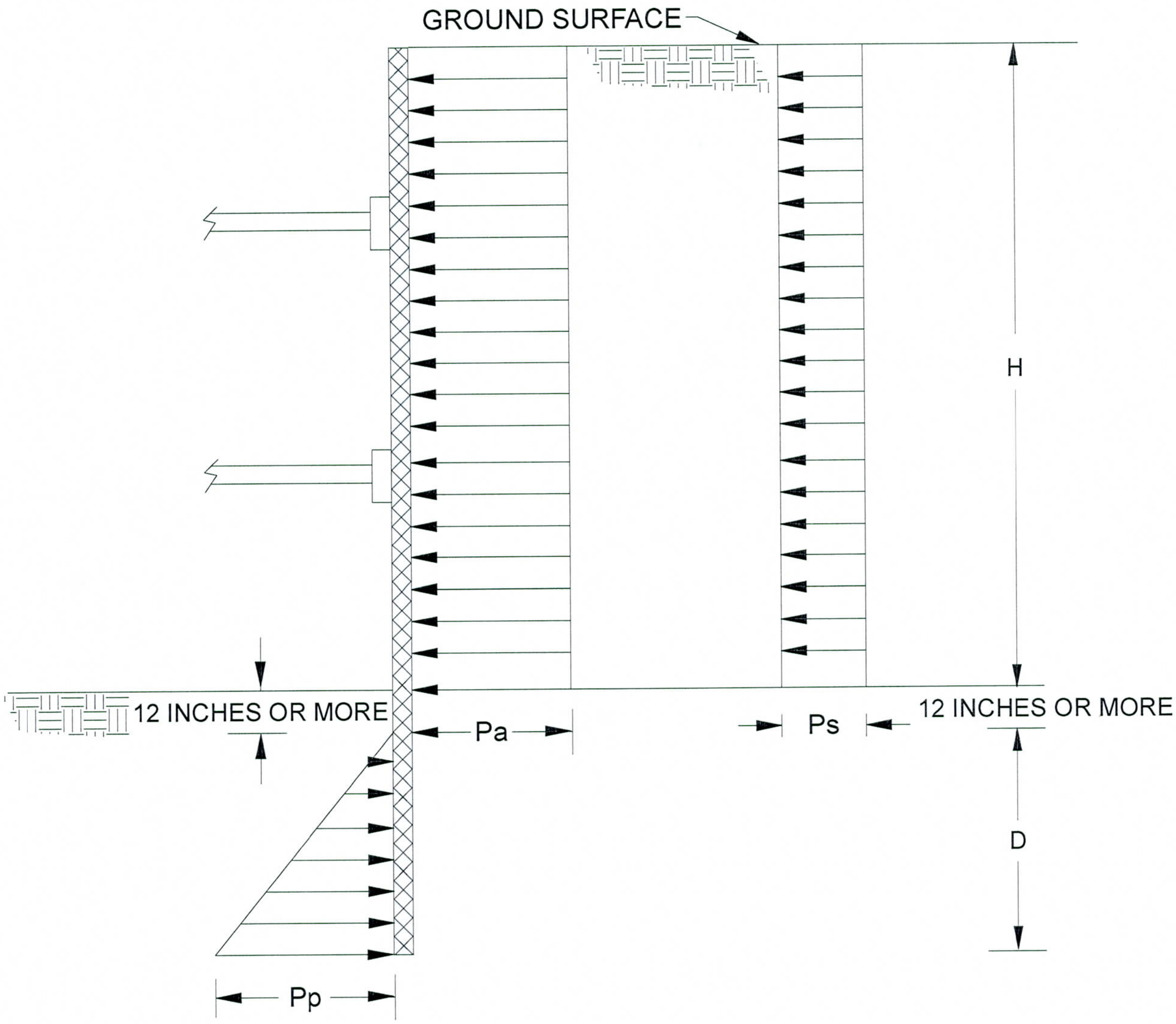
- B-13 Hollow-Stem Auger Boring Location
- C-2 Pavement Core Location
- L-5 Seismic Line and Test Trench Location
- B-2A Sonic Boring Location

0 1000

Approximate Scale:
1 inch = 1000 feet

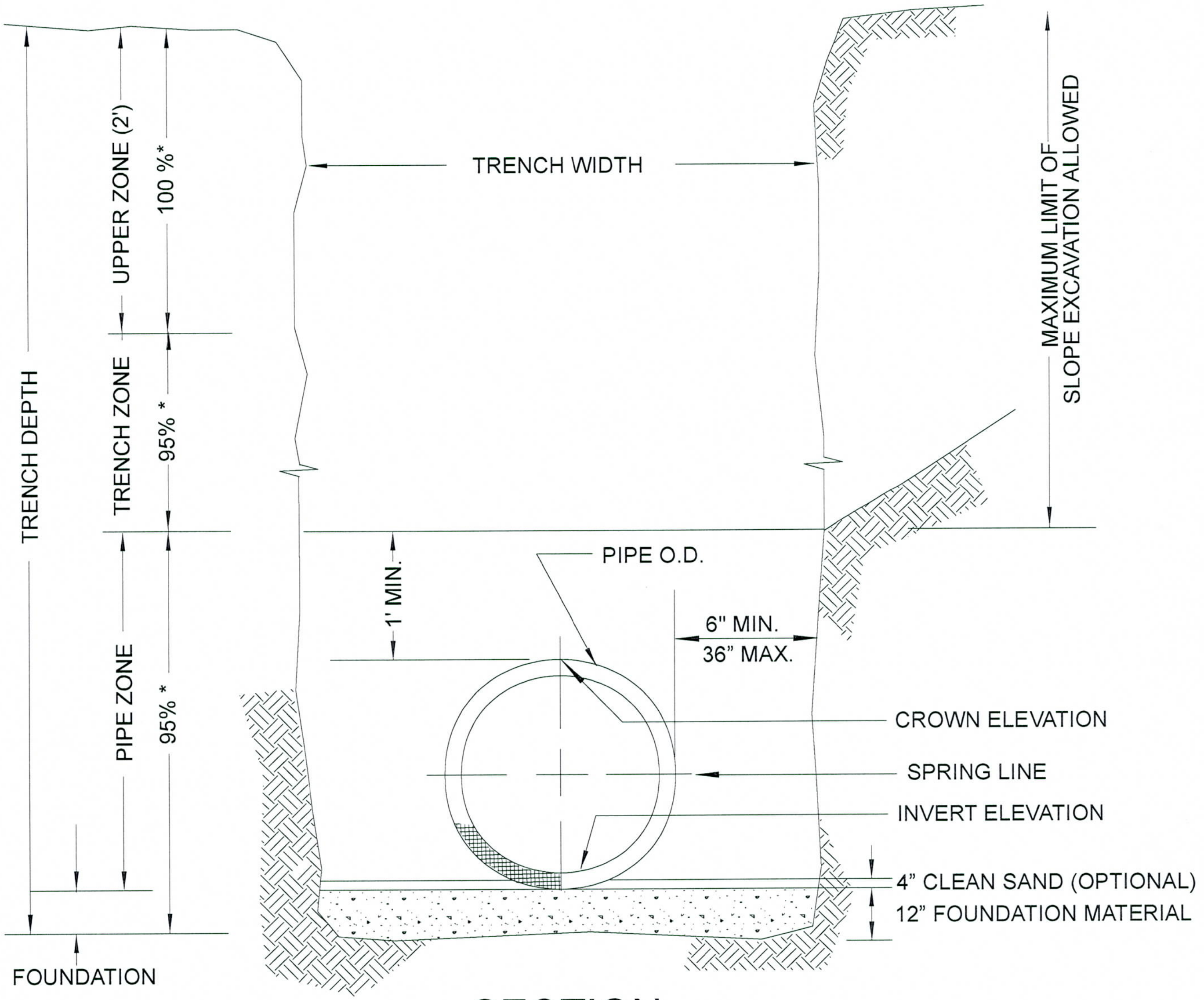
SOURCE: Maricopa County Assessor's GIS Dept, 2005.l

Ninyo & Moore	EXPLORATION LOCATION MAP	
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA		FIGURE 2
PROJECT No: 601052001	FILE No: 1052blm0905	DATE: 01/06



1. APPARENT LATERAL EARTH PRESSURE, Pa
   Pa = 20 H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, Ps
   Ps = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, Pp
   Pp = 380 D psf
4. ASSUMES GROUNDWATER NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

Ninyo & Moore	LATERAL EARTH PRESSURES FOR BRACED EXCAVATION	
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA		FIGURE 3
PROJECT No: 601052001	FILE No: 1052lepdtl0905	DATE: 01/06



SECTION

NOTE

* Indicates minimum relative compaction (see report for details).

Upper zone required for pavement areas only.

Diagram not drawn to scale.

NOT TO SCALE

Ninyo & Moore	PIPE BEDDING DETAIL		
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA			FIGURE 4
PROJECT No: 601052001	FILE No: 1052pipedetial	DATE: 01/06	

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APPENDIX A

BORING AND TEST TRENCH 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 borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test Spoon

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test spoon sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The spoon was driven up to 18 inches into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-84. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the spoon, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-84. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Chunk Samples

Chunk samples consisting of coherent blocks of relatively undisturbed material were collected from the excavations. These samples were sealed tightly in plastic bags and transported to the laboratory for testing.

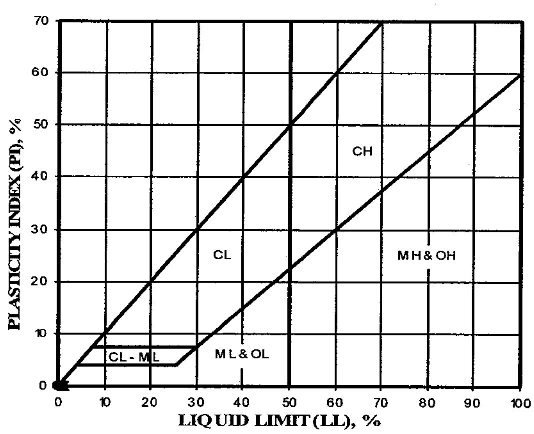
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL Coarse Fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



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U.S.C.S. METHOD OF SOIL CLASSIFICATION

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

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

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.
4							Attitude: Strike/Dip 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

Testpit explanation.xls

DATE DRILLED 08/03/05 BORING NO. B-1

GROUND ELEVATION 1766' MSL SHEET 1 OF 1

METHOD OF DRILLING CME-75, 6.5" Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, medium dense, silty fine to coarse SAND; few fine gravel.
			11					
			75	5.6	111.9			Very dense; scattered caliche filaments.
5								
			30				SP	Brown, damp, dense, fine to coarse SAND; trace silt.
10								
			50/3"				SC-SM	Brown, damp, very dense, clayey to silty fine to coarse SAND; little fine gravel.
15								
			48				SP	Brown, damp, very dense, fine to coarse SAND; trace gravel.
			50/6"	3.8	113.0			
								Total Depth = 19.0 feet. Groundwater not encountered. Backfilled 08/03/05.
20								

Ninyo & Moore

BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-1

DATE DRILLED 10/07/05 BORING NO. B-1A

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Mini-sonic

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY ESZ LOGGED BY ESZ REVIEWED BY ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, medium dense, silty fine to coarse SAND; little gravel; weakly cemented by caliche.
		X	110					Light brown; very dense; few clay; moderately to strongly cemented by caliche.
5								
							SC	Light brown, damp, very dense, clayey fine to coarse SAND; low to medium plasticity; few fine to coarse gravel; moderately to strongly cemented by caliche.
10								Brown; cementation not observed; scattered caliche filaments.
	X							Increase in plasticity; moderately to strongly cemented by caliche.
15								
	X	X	50/3"					Decrease in plasticity.
								Total depth = 19.3 feet. Groundwater not encountered. Backfilled on 10/07/05.
20								

Ninyo & Moore

BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-14

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/03/05 BORING NO. B-2 GROUND ELEVATION 1766' MSL SHEET 1 OF 1 METHOD OF DRILLING CME-75, 6.5" Hollow-Stem Auger DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY ESZ **DESCRIPTION/INTERPRETATION**
0							SP	ALLUVIUM: Brown, damp, very dense, gravelly fine to coarse SAND.
			82					
								Difficult drilling; coarse gravel; cobbles and possible boulders.
5								Total Depth = 4.5 feet. (Refusal) Groundwater not encountered. Backfilled 08/03/05.
10								
15								
20								

Ninyo & Moore

BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-2

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, medium dense, silty fine to coarse SAND; to little gravel.
								Light brown; very dense; weakly to moderately cemented.
5								
							SC	Light grayish brown, damp, very dense, clayey fine to coarse SAND; low to medium plasticity; few fine gravel; weakly to moderately cemented.
10								Reddish brown; moderately to strongly cemented.
15								Weakly to moderately cemented by caliche.
							SM	Light brown, damp, very dense, silty fine to coarse SAND.
							SC	Light brown, damp, medium dense, clayey fine to medium SAND; weak to moderately cemented caliche.
			50/5"					Very dense.
20								

DATE DRILLED 10/07/05 BORING NO. B-2A

GROUND ELEVATION -- SHEET 1 OF 2

METHOD OF DRILLING Mini-sonic

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY JRD LOGGED BY JRD REVIEWED BY ESZ

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-15

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 10/07/05 BORING NO. B-2A GROUND ELEVATION -- SHEET 2 OF 2 METHOD OF DRILLING Mini-sonic DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY JRD LOGGED BY JRD REVIEWED BY ESZ DESCRIPTION/INTERPRETATION
20								Total depth = 20.0 feet. Groundwater not encountered. Backfilled on 10/07/05.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-16

DATE DRILLED	08/03/05	BORING NO.	B-3
GROUND ELEVATION	1746' MSL	SHEET 1 OF 1	
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY DM	LOGGED BY DM	REVIEWED BY	ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, medium dense, silty fine to coarse SAND; trace fine gravel.
			25					
			28					Dense.
5								
			50/6"					Very dense; scattered caliche filaments.
10								
			50/2"					Few fine to coarse gravel.
15								
								Cobbles and possible boulders.
								Total Depth = 16.0 feet. (Refusal) Groundwater not encountered. Backfilled 08/03/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-3

DATE DRILLED 08/02/05 BORING NO. B-4

GROUND ELEVATION 1730' MSL SHEET 1 OF 1

METHOD OF DRILLING CME-75, 6.5" Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; few fine gravel.
			50/6"					
			50/4"	8.0	100.7			
5								
							SP	Brown, damp, medium dense, fine to coarse SAND.
			13					
10								
			50/6"	3.9	119.3			Very dense.
15								
								Cobbles and possible boulders.
								Total Depth = 16.0 feet. (Refusal) Groundwater not encountered. Backfilled 08/02/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/02/05 BORING NO. B-5 GROUND ELEVATION 1718' MSL SHEET 1 OF 1 METHOD OF DRILLING CME-75, 6.5" Hollow-Stem Auger DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY ESZ **DESCRIPTION/INTERPRETATION**
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND with gravel.
			50/6"					
			50/4"					
5								
							SP	Brown, damp, dense, fine to coarse SAND; few gravel.
			59	3.6	126.1			
10								
			50/6"					Very dense.
15								
								Cobbles and possible boulders.
								Total depth = 16.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-5

DATE DRILLED	08/03/05	BORING NO.	B-6
GROUND ELEVATION	1708' MSL	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	ESZ		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 6" thick.
							SM	FILL: Brown, damp, dense, silty fine to coarse SAND; few gravel.
			24				SM	ALLUVIUM: Brown, damp, dense, silty fine to coarse SAND; few gravel; scattered caliche filaments.
			50/5"					Very dense.
5								
			50/5"					
10								
			50/5"	5.4	106.0			
15								Cobbles and possible boulders.
								Total depth = 16.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/03/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-6

DATE DRILLED	08/02/05	BORING NO.	B-7	
GROUND ELEVATION	1679' MSL	SHEET	1	OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger			
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"	
SAMPLED BY	DM	LOGGED BY	DM	REVIEWED BY ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; trace fine gravel.
			50/5"					
			50/5"	4.9	107.2			
5								
								Cobbles and possible boulders.
								Total depth = 6.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
10								
15								
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-7

DATE DRILLED	08/02/05
BORING NO.	B-8
GROUND ELEVATION	1665' MSL
SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger
DRIVE WEIGHT	140 lbs. (Automatic)
DROP	30"
SAMPLED BY	DM
LOGGED BY	DM
REVIEWED BY	ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; few fine to coarse gravel; scattered caliche filaments.
		■	50/5"					
		■	50/2"					
5								
								Cobbles and possible boulders.
								Total depth = 6.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
10								
15								
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-8

DATE DRILLED	08/02/05	BORING NO.	B-9
GROUND ELEVATION	1653' MSL	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	ESZ		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; few fine to coarse gravel.
			48					
			50/3"	5.8	108.8			
5								
			45					Scattered caliche filaments.
10								
			50/1"					
15								
			50/3"					
								Total depth = 18.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-9

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/02/05 BORING NO. B-10 GROUND ELEVATION 1649' MSL SHEET 1 OF 1 METHOD OF DRILLING CME-75, 6.5" Hollow-Stem Auger DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY ESZ **DESCRIPTION/INTERPRETATION**
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; few gravel; scattered caliche filaments.
			50/6"	5.7	109.2			
			75/10"					
5								
								Total depth = 6.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
10								
15								
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-10

DATE DRILLED	08/02/05	BORING NO.	B-11
GROUND ELEVATION	1643' MSL	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	ESZ		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; scattered caliche filament.
			58					
			75/11"					Few fine to coarse gravel.
5								
			50/4"					
10								
								Total depth = 11.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
15								
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-11

DATE DRILLED	08/02/05
BORING NO.	B-12
GROUND ELEVATION	1645' MSL
SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger
DRIVE WEIGHT	140 lbs. (Automatic)
DROP	30"
SAMPLED BY	DM
LOGGED BY	DM
REVIEWED BY	ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Brown, damp, medium dense, silty fine to coarse SAND; few fine gravel.
			18					
			50/5"					Very dense; scattered caliche filament.
5								
			50/5"					Coarse gravel; cobbles and possible boulders.
10								Total depth = 9.5 feet. (Refusal) Groundwater not encountered. Backfilled on 08/02/05.
15								
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-12

DATE DRILLED	08/03/05	BORING NO.	B-13
GROUND ELEVATION	1633' MSL	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 6.5" Hollow-Stem Auger		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY DM	REVIEWED BY ESZ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 6" thick.
							SM	FILL: Brown, damp, medium dense, silty fine to coarse SAND; few gravel.
			29					
			50/6"				SM	ALLUVIUM: Brown, damp, very dense, silty fine to coarse SAND; few gravel; scattered caliche filaments.
5								
			50/6"				SC	Brown, damp, very dense, clayey fine to coarse SAND; little fine to coarse gravel.
10								
			50/5"					Cobbles and possible boulders.
15								
								Total depth = 16.0 feet. (Refusal) Groundwater not encountered. Backfilled on 08/03/05.
20								

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BORING LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	A-13

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TEST PIT LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

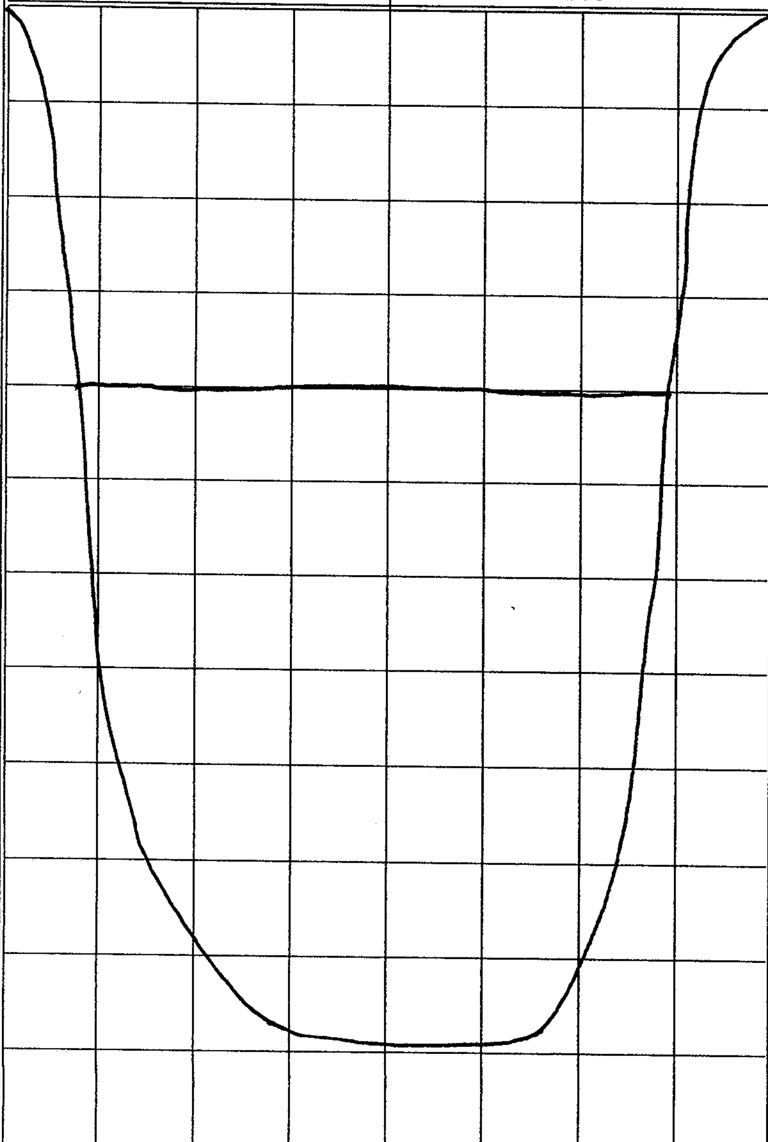
PROJECT NO.	DATE
601052001	1/06

DATE EXCAVATED 09/29/05 TEST PIT NO. L-1

GROUND ELEVATION -- LOGGED BY JSR

METHOD OF EXCAVATION Case 580 Backhoe

LOCATION South side of McDowell Road near Sossaman Road



DEPTH (FEET)	SAMPLES: Bulk	Driven	Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SC	FILL: Light brown, damp, medium dense, clayey fine to coarse SAND; low to medium plasticity; trace silt; scattered reworked caliche nodules.
2							
							Few silt.
4							Pieces of glass.
	■					SC	ALLUVIUM: Light brownish gray, damp, very dense, clayey fine to coarse SAND; low to medium plasticity; few fine to coarse gravel; trace silt; numerous caliche filaments and nodules; weakly to moderately cemented by caliche; cobbles and possible boulders.
6							
8							
10							
							Total depth = 10.9 feet. Groundwater not encountered. Backfilled on 09/29/05.
12							

SCALE = 1 in./2 ft.

FIGURE A-14

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TEST PIT LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

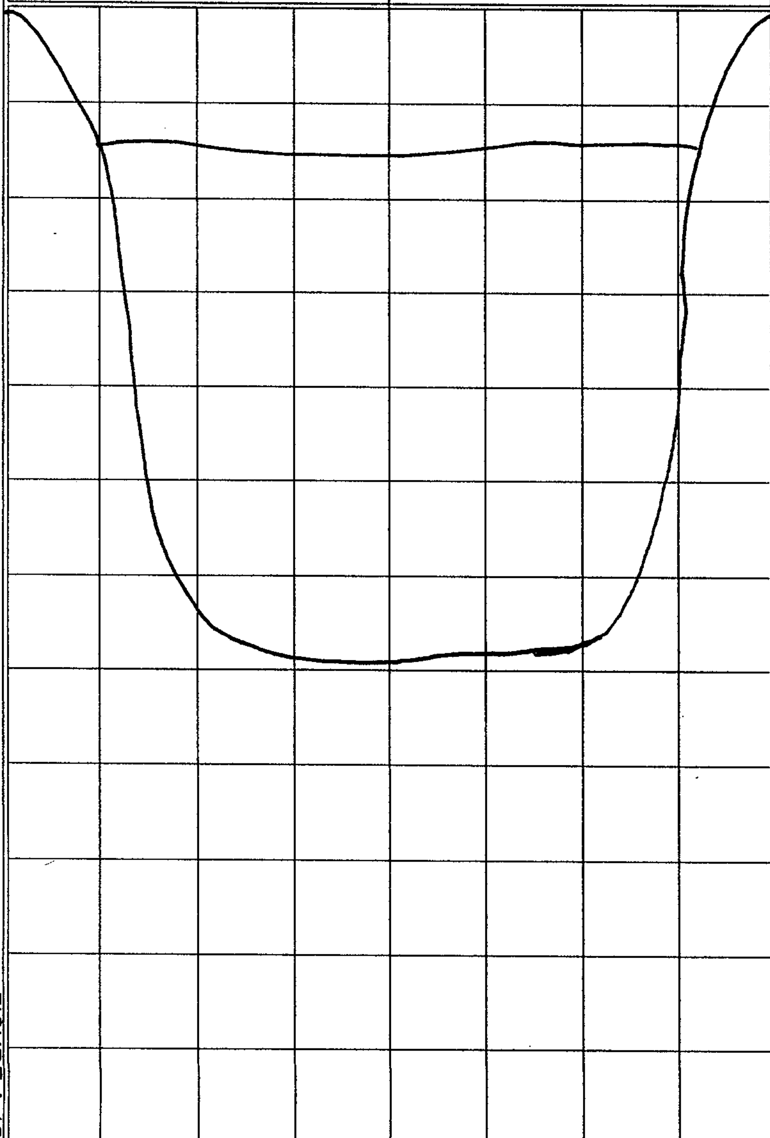
PROJECT NO.	DATE
601052001	1/06

DATE EXCAVATED 09/29/05 TEST PIT NO. L-2

GROUND ELEVATION -- LOGGED BY JSR

METHOD OF EXCAVATION Case 580 Backhoe

LOCATION South side of McDowell Road, east of 78th Street



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SC	FILL: Light brown, damp, clayey fine to coarse SAND; low to medium plasticity; few fine to coarse gravel; trace silt; numerous caliche filaments and nodules; weak to moderate cementation.
2						SC	ALLUVIUM: Light brownish gray, damp, very dense, clayey fine to coarse SAND; low to medium plasticity; few fine to coarse gravel and silt; numerous caliche filaments and nodules; weakly to moderately cemented by caliche.
4							Strongly cemented by caliche.
6	■						4C = 220 psi
							Refusal on caliche.
							Total depth = 6.9 feet. (Refusal) Groundwater not encountered. Backfilled on 09/29/05.
8							
10							
12							

SCALE = 1 in./2 ft.

FIGURE A-15

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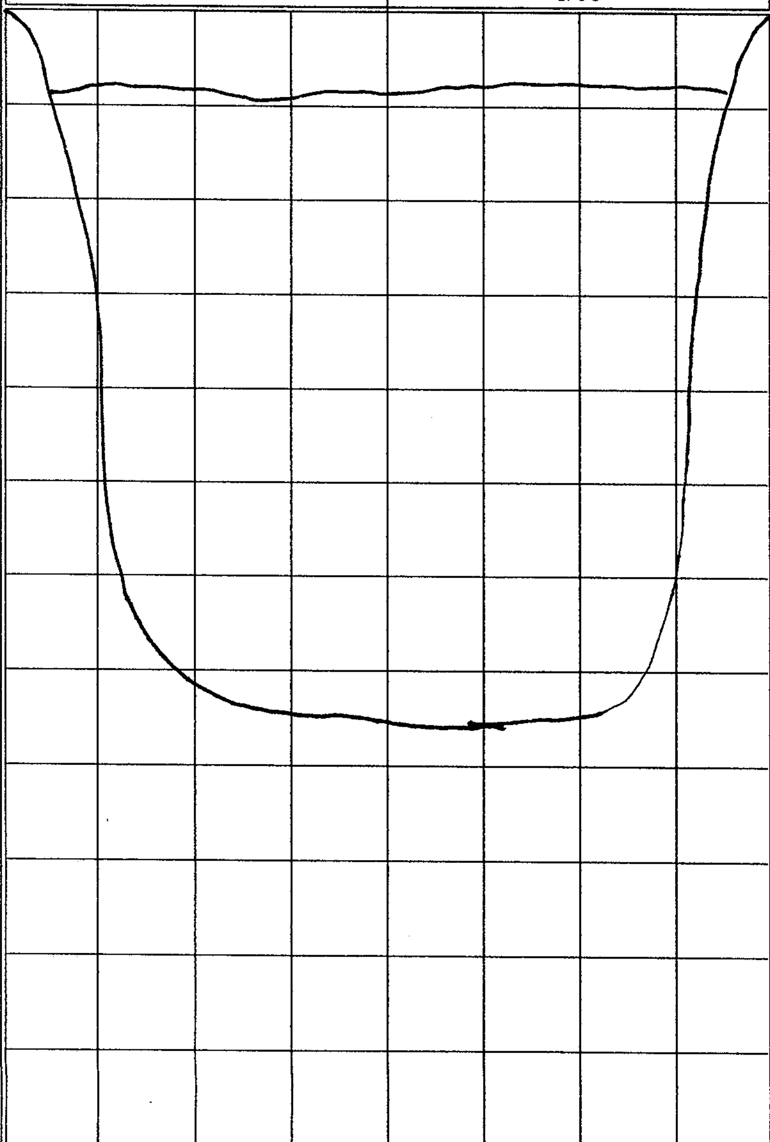
TEST PIT LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

PROJECT NO.	DATE
601052001	1/06

DATE EXCAVATED	09/29/05	TEST PIT NO.	L-3
GROUND ELEVATION	--	LOGGED BY	JSR
METHOD OF EXCAVATION	Case 580 Backhoe		
LOCATION	South side of McDowell Road, east of 80th Street		



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	FILL: Brown, damp, medium dense, silty fine to coarse SAND; few fine gravel; trace clay.
						SC	ALLUVIUM: Light brownish gray, damp, very dense, clayey fine to coarse SAND; low to medium plasticity; few fine gravel; trace silt; numerous caliche filaments and nodules; moderately to strongly cemented by caliche.
2							
4							
6							
							Refusal on caliche.
8							Total depth = 7.5 feet. (Refusal) Groundwater not encountered. Backfilled on 09/29/05.
10							
12							

SCALE = 1 in./2 ft.

FIGURE A-16

TEST PIT LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

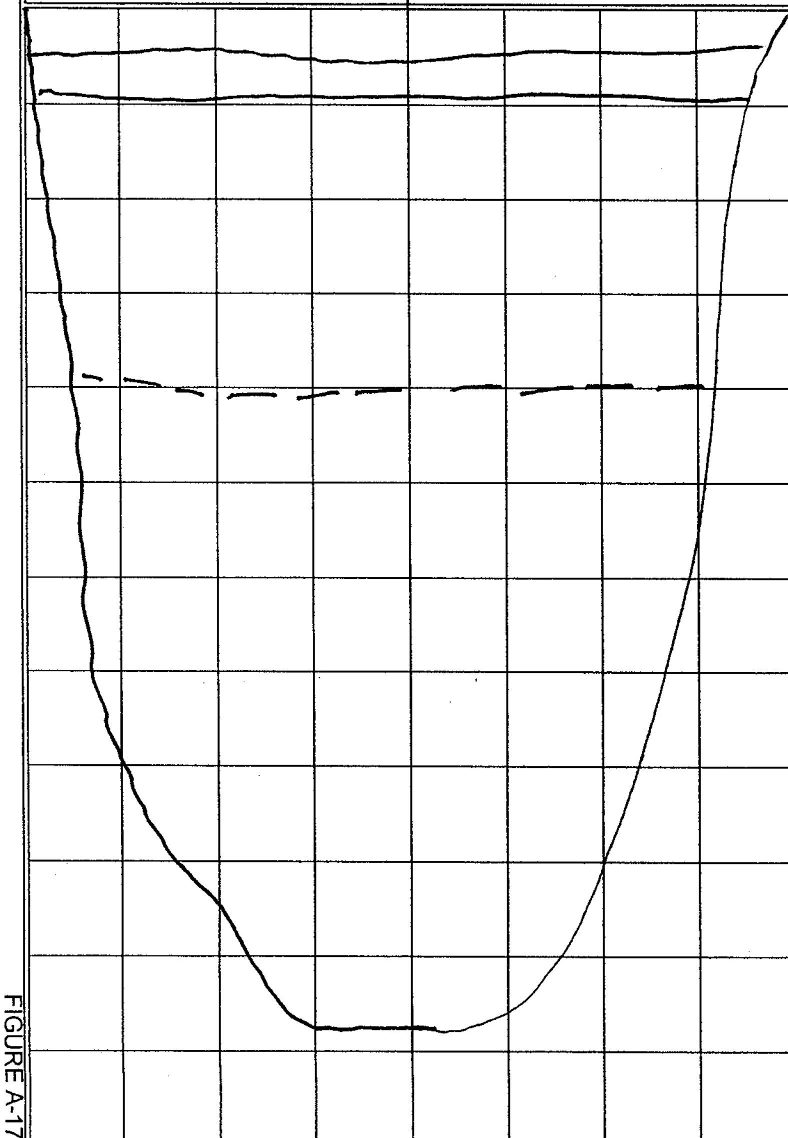
PROJECT NO.	DATE
601052001	1/06

DATE EXCAVATED 09/29/05 TEST PIT NO. L-4

GROUND ELEVATION -- LOGGED BY JSR

METHOD OF EXCAVATION Case 580 Backhoe

LOCATION South side of McDowell Road and west of Hawes Road



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0							ASPHALTIC CONCRETE: Approximately 5" thick.
						GP	AGGREGATE BASE: Approximately 6" thick. Brown, damp, dense, fine to coarse GRAVEL; few fine sand.
						SP	ALLUVIUM: Light brownish gray, damp, very dense, fine to coarse SAND; few fine gravel; trace silt and clay; numerous caliche filaments and nodules; moderately to strongly cemented by caliche.
2							
4						SM	Light brown, damp, very dense, silty fine to coarse SAND; few fine to coarse gravel; numerous caliche filaments and nodules; moderately to strongly cemented by caliche.
6							
8							
10							
							Total depth = 10.9 feet. Groundwater not encountered. Backfilled on 09/29/05.
12							

SCALE = 1 in./2 ft.

FIGURE A-17

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TEST PIT LOG

MCDOWELL ROAD BASIN AND STORM DRAIN
SOSSAMAN ROAD TO HAWES ROAD - MESA, ARIZONA

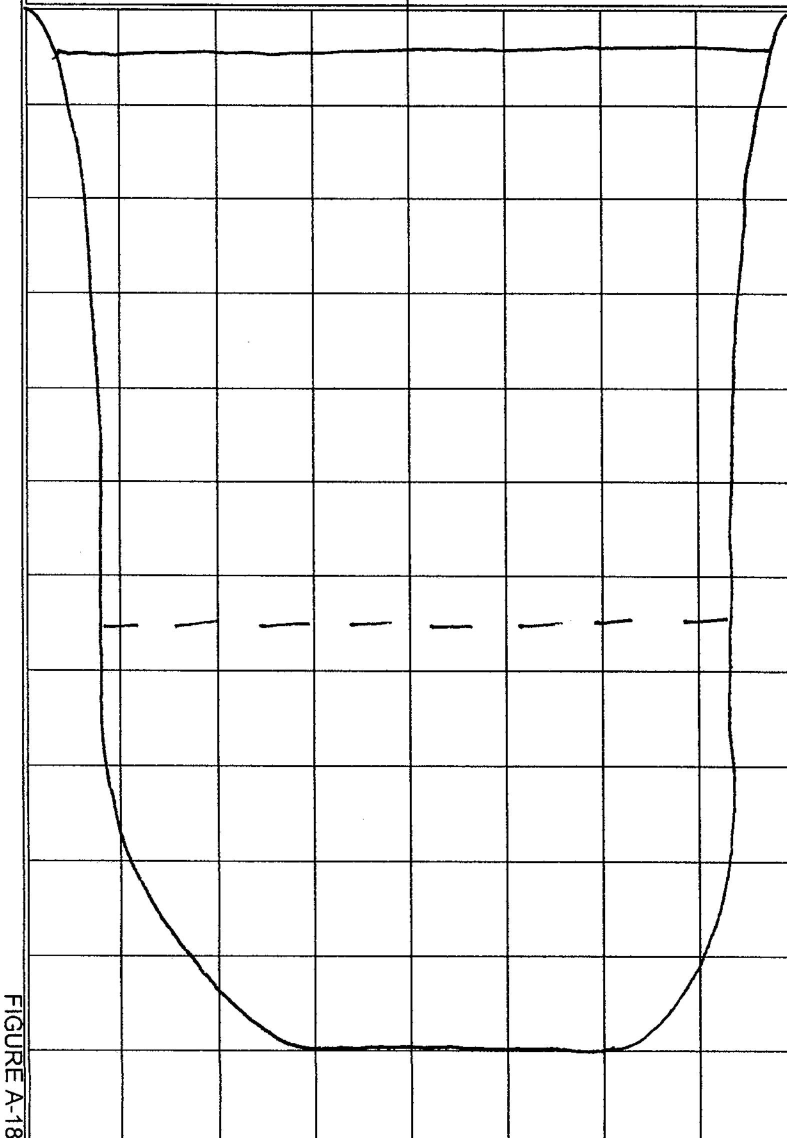
PROJECT NO.	DATE
601052001	1/06

DATE EXCAVATED 09/29/05 TEST PIT NO. L-5

GROUND ELEVATION -- LOGGED BY JSR

METHOD OF EXCAVATION Case 580 Backhoe

LOCATION South side of McDowell Road, west of 82nd Street



SCALE = 1 in./2 ft.

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SP	FILL: Brown, damp, medium dense, gravelly fine to coarse SAND; trace silt.
						SC	ALLUVIUM: Light brownish gray, damp, clayey fine to coarse SAND; low to medium plasticity; few fine to coarse gravel; numerous caliche filaments and nodules; moderately to strongly cemented by caliche.
2							
4							
6						SM	Light brownish gray, damp, very dense, silty fine to coarse SAND; few fine to coarse gravel; trace clay; numerous caliche filaments and nodules; moderately to strongly cemented by caliche.
8	■						
10							
11							Total depth = 11.0 feet. Groundwater not encountered. Backfilled on 09/29/05.
12							

FIGURE A-18

APPENDIX B

LABORATORY TEST RESULTS

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-93. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-94. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422-63. The grain-size distribution curves are shown on Figures B-1 through B-4. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

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 4318-00. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figure B-5.

Hydroconsolidation (Settlement Potential) Tests
Hydroconsolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 4546-03. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-6 andB-7.

Expansion Index Tests
The expansion index of selected materials was evaluated in general accordance ASTM D 4829-95. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water.

Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-8.

Maximum Dry Density and Optimum Moisture Content Tests

The maximum dry density and optimum moisture content of selected representative soil samples were evaluated in general accordance with ASTM D 698-00. The results of these tests are summarized on Figures B-9 and B-10.

R-Value

The resistance value, or R-value, of alluvial soils was evaluated in general accordance with ASTM D 2844-94. Samples were prepared and each was tested for exudation pressure and R-value. The graphically evaluated R-value at an exudation pressure of 300 pounds per square inch is reported. The test results are shown on Figure B-11

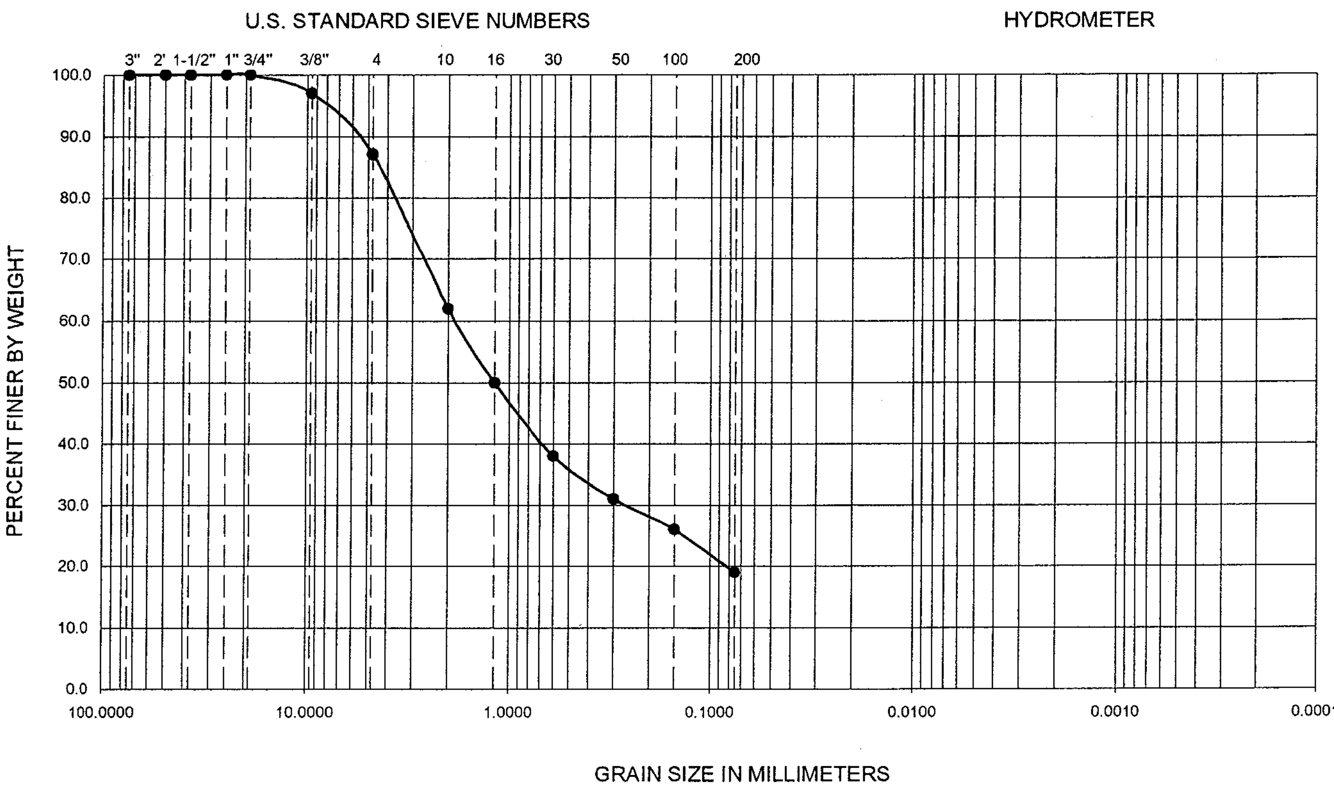
Unconfined Compression Tests

An unconfined compression tests was performed on a chunk sample in general accordance with ASTM D 2166-00. The test result is shown on the test trench log in Appendix A.

Soil Corrosivity Tests

Soil pH and minimum resistivity tests were performed on a representative soil sample in general accordance with Arizona Test 236b. The sulfate content was evaluated in general accordance with Arizona Test 733. The chloride content was evaluated in general accordance with Arizona Test 736. The test results are presented on Figure B-12.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-1	13.5-13.7	25	18	7	--	--	--	--	--	19	SC-SM

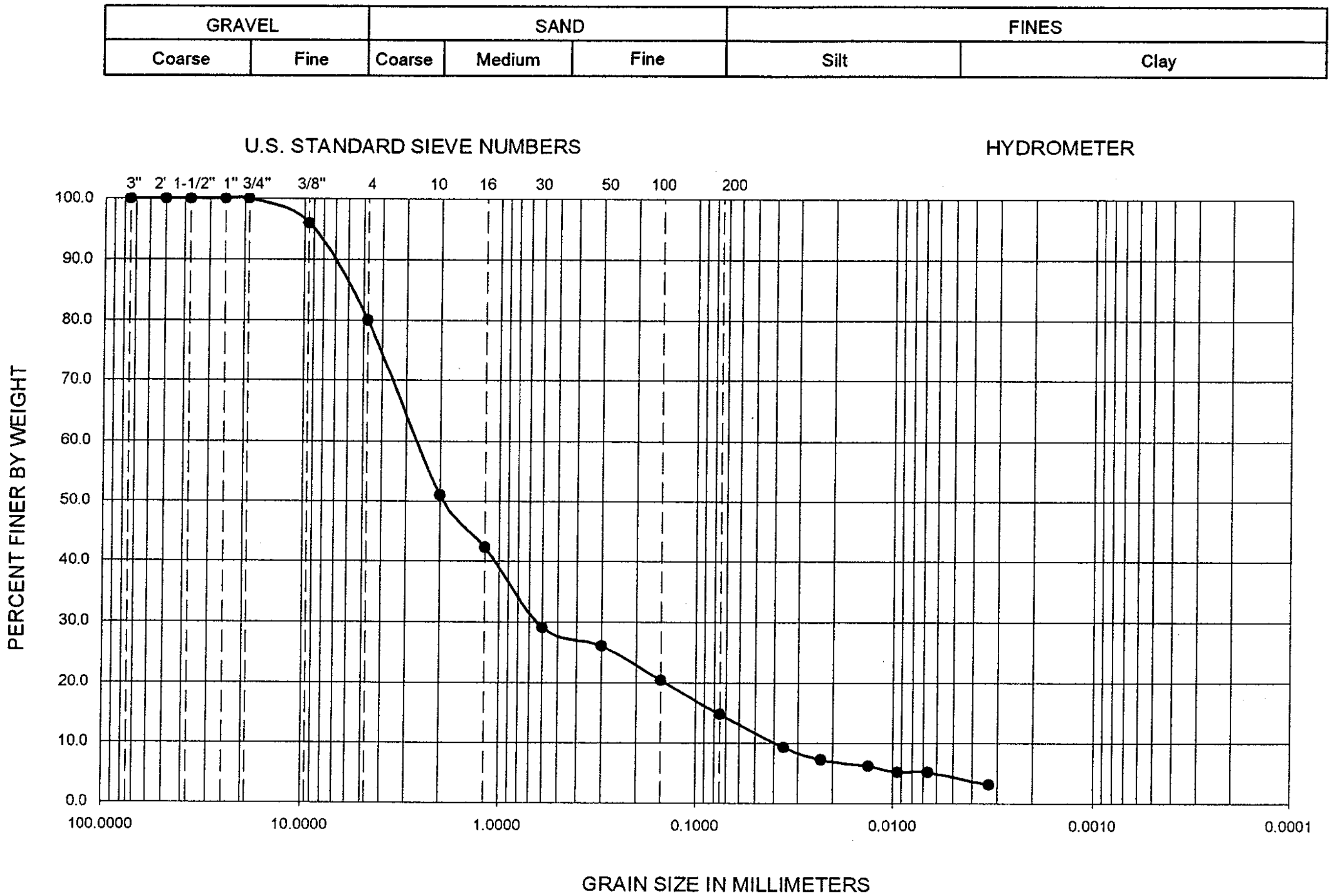
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-02

Ninyo & Moore

GRADATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-1



Symbol	Sample Location	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
●	B-5	1-1.5	38	32	6	--	--	--	--	--	14.8	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-02

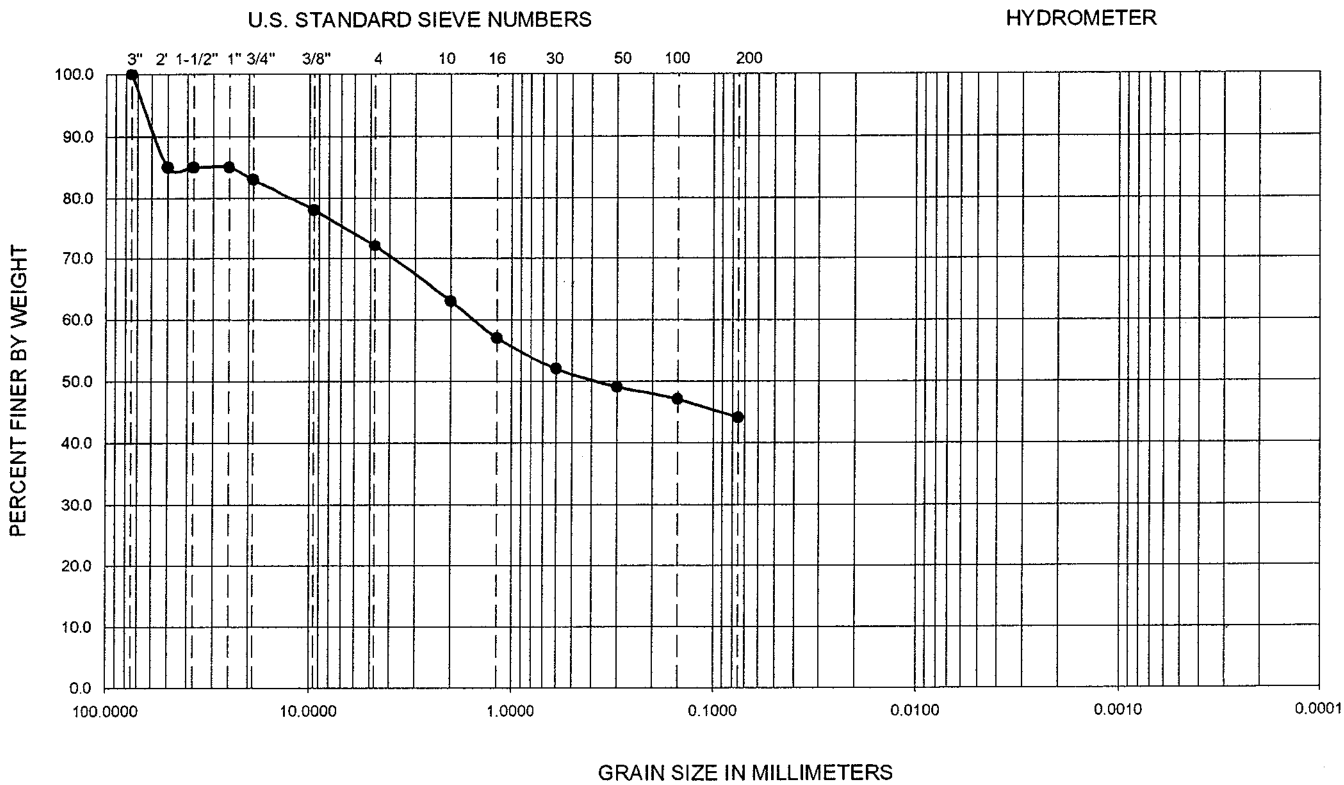
Ninyo & Moore

GRADATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-2

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-9	13.5-15	NP	NP	NP	--	--	--	--	--	44	SM

NP-INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-02

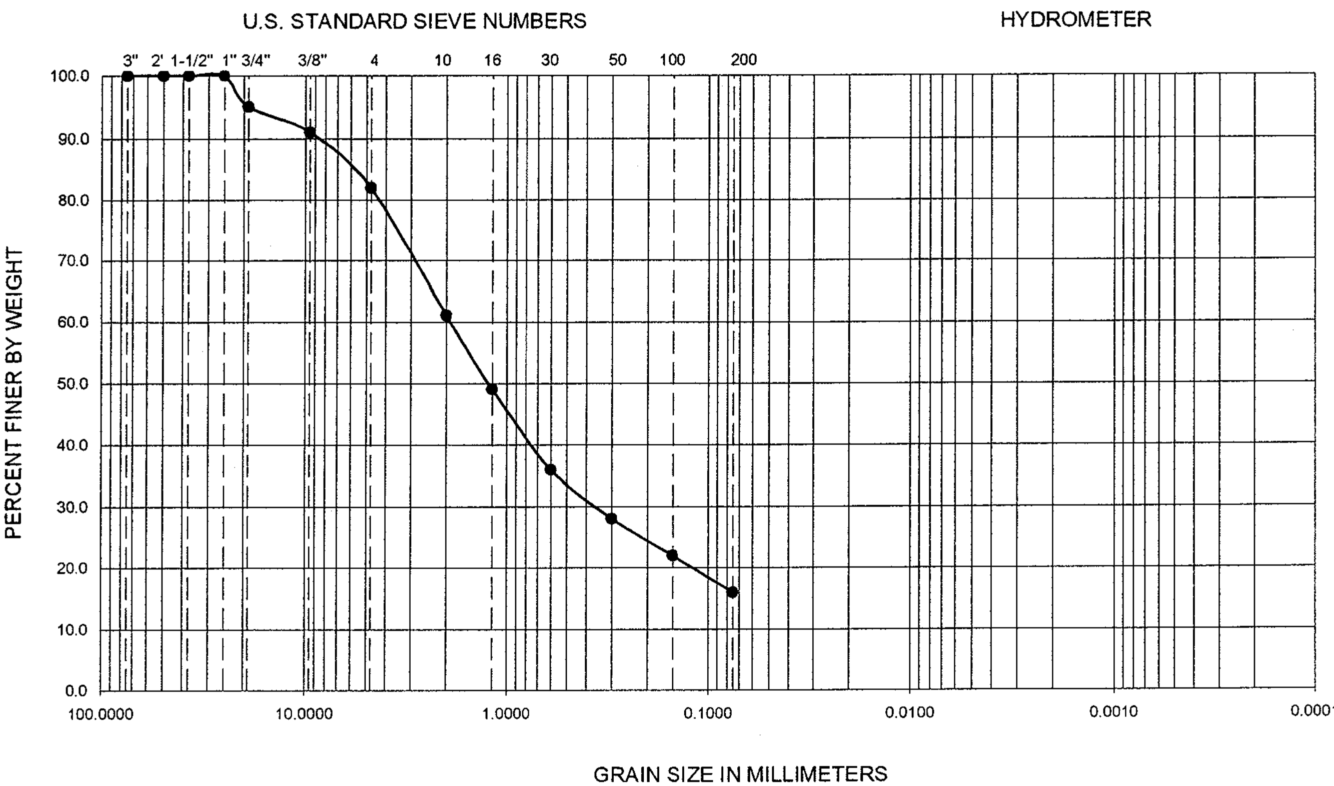
Ninyo & Moore

GRADATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
MCDOWELL ROAD FROM SOSSAMEN TO HAWES
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-13	8.5-9	29	20	9	--	--	--	--	--	16	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-02

Ninyo & Moore

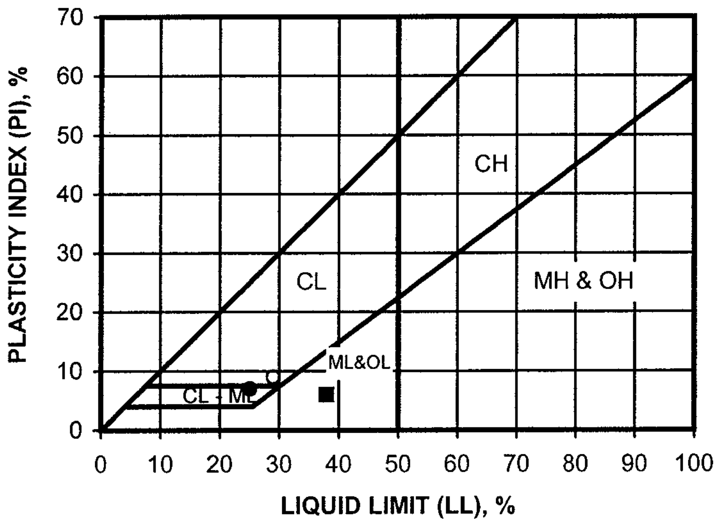
GRADATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-4

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	B-1	13.5-13.7	25	18	7	CL-ML	SC-SM
■	B-5	1-1.5	38	32	6	ML	SM
◆	B-9	13.5-13.6	NP	NP	NP	NP	SM
○	B-13	8.5-9	29	20	9	CL	SC

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-00

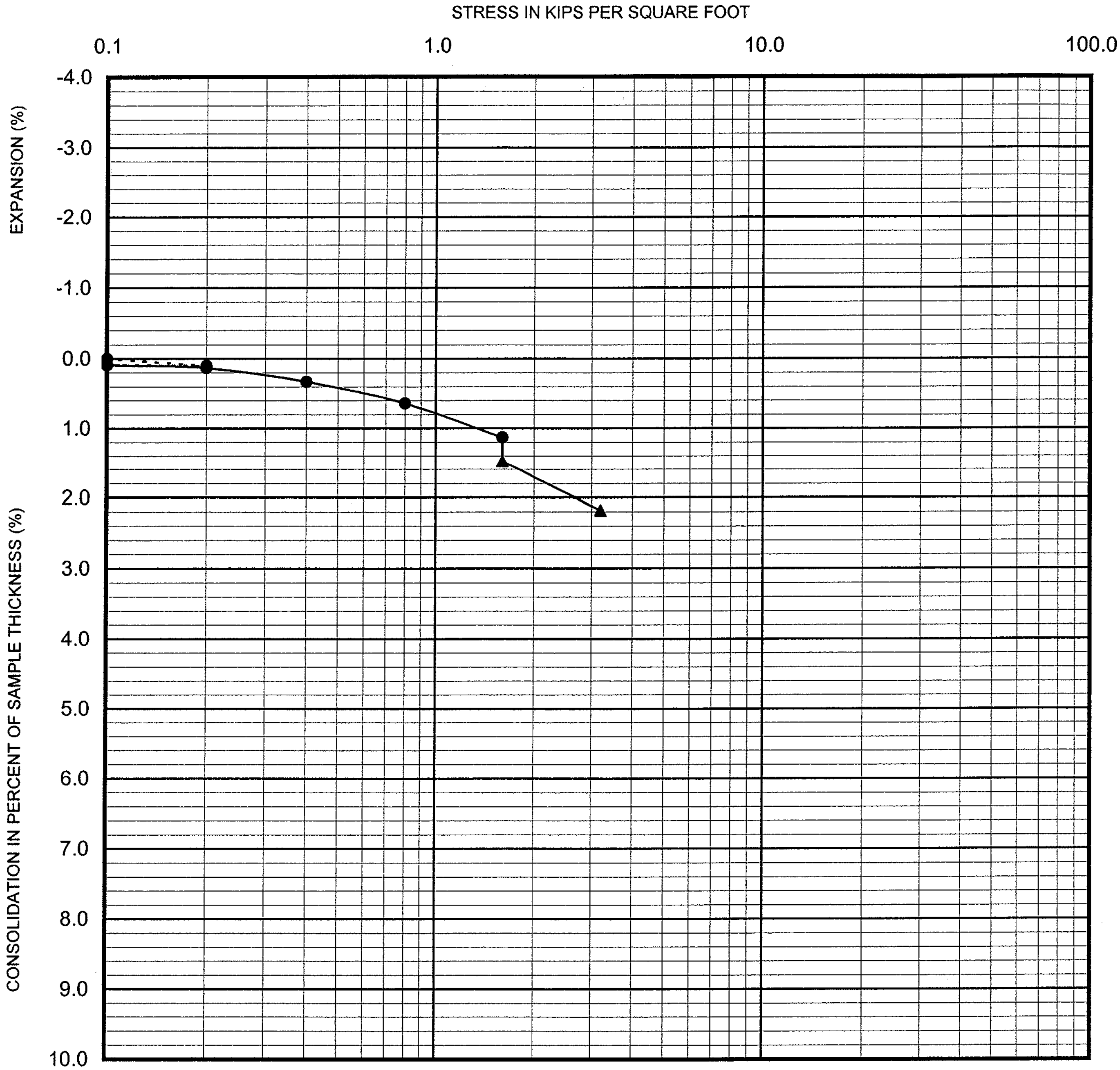
Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE
601052001	1/06

FIGURE
B-5



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-6
Depth (ft.)	13.5-15
Soil Type	SM

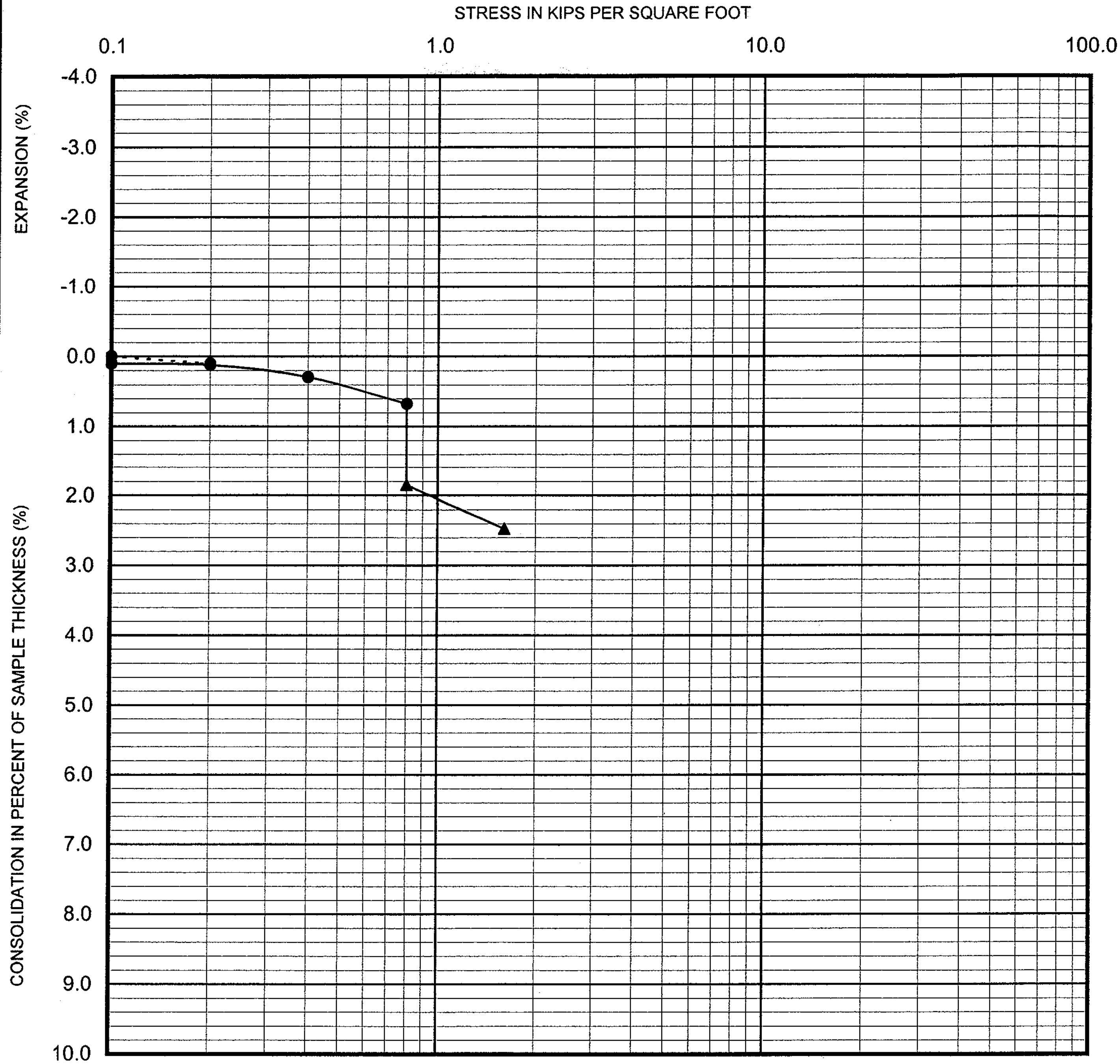
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-03

Ninyo & Moore

CONSOLIDATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-7



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-4
Depth (ft.)	13-15
Soil Type	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-03

Ninyo & Moore

CONSOLIDATION TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-6

CONSOLIDATION B-4 13-15

EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	EXPANSION POTENTIAL
B-1	0-5	7.0	124.5	9.3	-	2	Very Low
B-12	0-5	7.2	117.0	13.2	-	0	Very Low

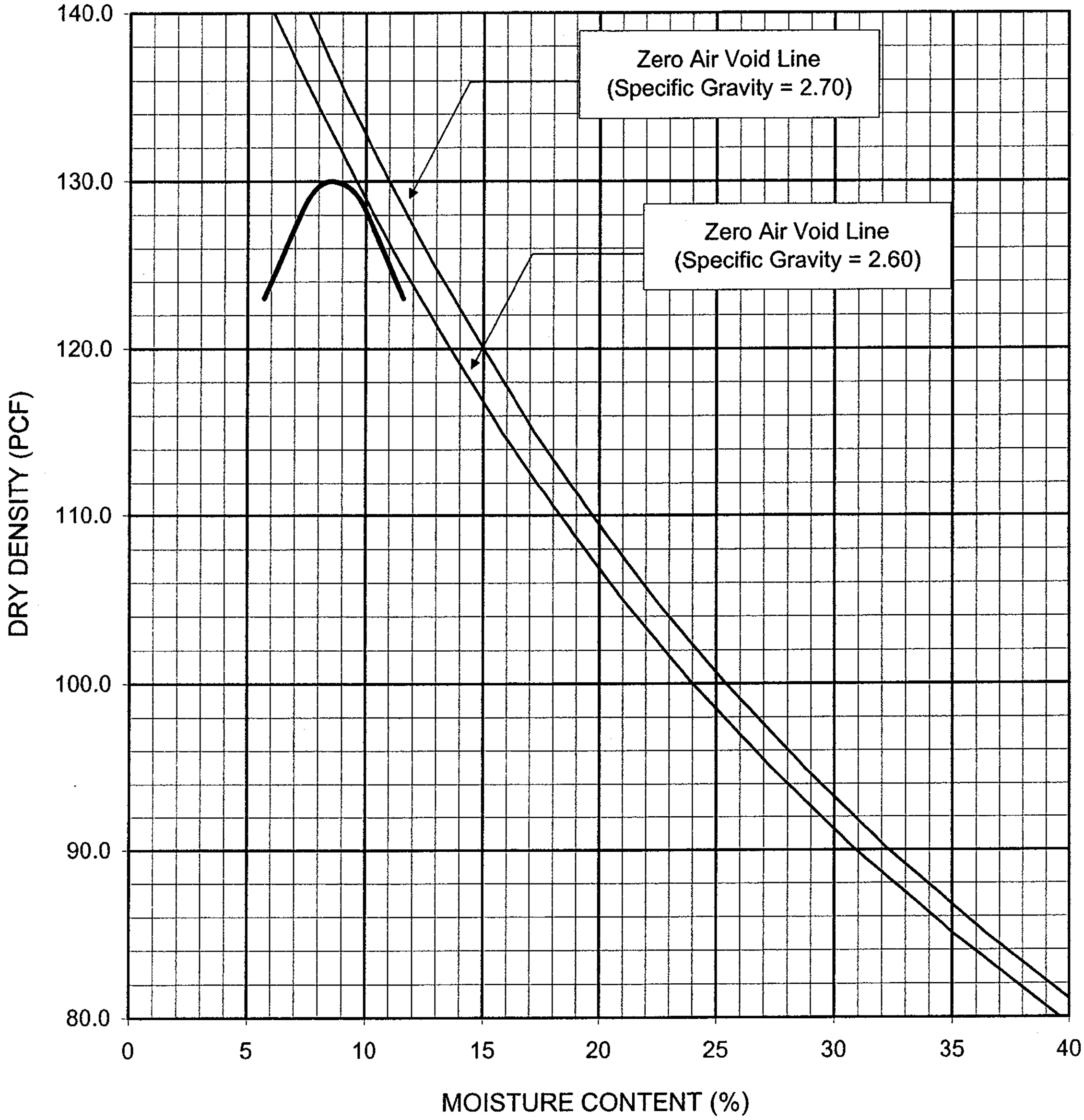
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4829-03

Ninyo & Moore

EXPANSION INDEX TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-8



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	0-5	Silty Fine to Coarse SAND	130.0	8.5

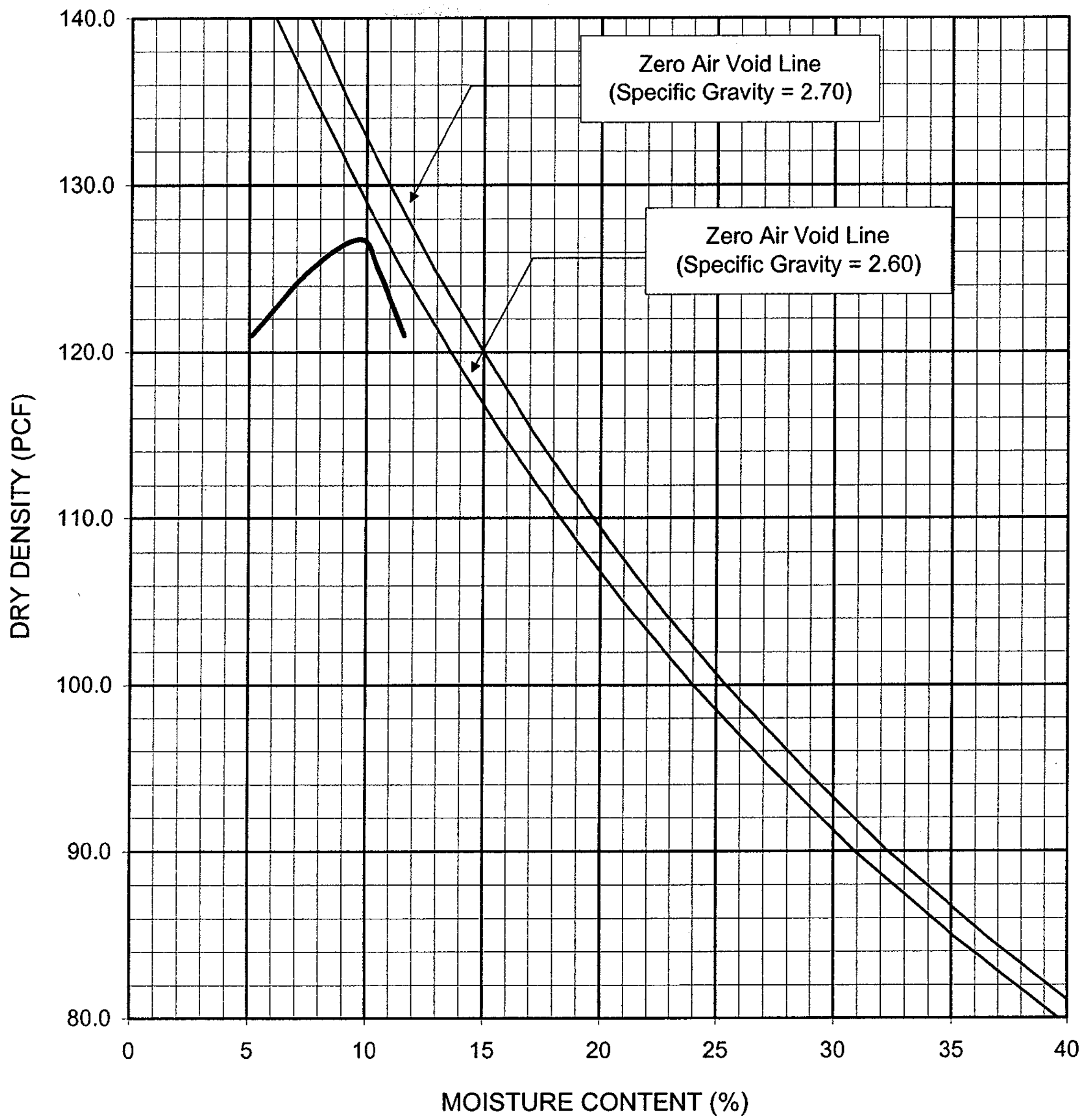
PERFORMED IN GENERAL ACCORDANCE WITH ☐ ASTM D 1557-02 ☑ ASTM D 698-00a METHOD "A"

Ninyo & Moore

PROCTOR DENSITY TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-9



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-12	0-5	Silty Fine to Coarse SAND	126.8	9.7

PERFORMED IN GENERAL ACCORDANCE WITH ☐ ASTM D 1557-02 ☑ ASTM D 698-00a METHOD "A"

Ninyo & Moore

PROCTOR DENSITY TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-10

R-VALUE TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-1	0-5	SM	72
B-5	0-5	SM	69

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844-94

Ninyo & Moore

R-VALUE TEST RESULTS

MCDOWELL ROAD STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-11

CORROSIVITY TEST RESULTS

SAMPLE ID	DEPTH (FT)	pH *	RESISTIVITY * (ohm-cm)	WATER-SOLUBLE SULFATE CONTENT IN SOIL ** (%)	CHLORIDE CONTENT *** (ppm)
B-1	0-5	7.7	4,514	0.010	41
B-5	0-5	8.8	2,736	0.001	10
B-12	0-5	8.6	1,642	0.004	40

* PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 236b
** PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 733
*** PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 736

Ninyo & Moore

CORROSIVITY TEST RESULTS

MCDOWELL ROAD BASIN & STORM DRAIN
SOSSAMEN ROAD TO HAWES ROAD
MESA, ARIZONA

PROJECT NO.	DATE	FIGURE
601052001	1/06	B-12

Ninyo & Moore

APPENDIX C

SEISMIC REFRACTION SURVEYS

APPENDIX C

SEISMIC REFRACTION SURVEYS

Ninyo and Moore personnel conducted seismic refraction surveys at the site on September 28, 2005 to evaluate the rippability characteristics of the subsurface materials. The seismic refraction data were collected with a SmartSeis S12, high performance digital exploration seismograph and 12 vertical component geophones. A 10-pound hammer and metal plate were used as the seismic wave source. A total of 5 seismic refraction traverses were performed along the south edge of McDowell Road between Sossaman Road and Hawes Road. The approximate locations of the surveys are depicted on Figure 2.

The seismic refraction method uses first-arrival times of refracted seismic waves to determine the thicknesses and seismic velocities of subsurface layers. Seismic waves generated at the surface are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface geophones and recorded with a seismograph. The travel times of the seismic waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

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 above will not be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity can also result in the misinterpretation of the subsurface conditions.

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. 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.

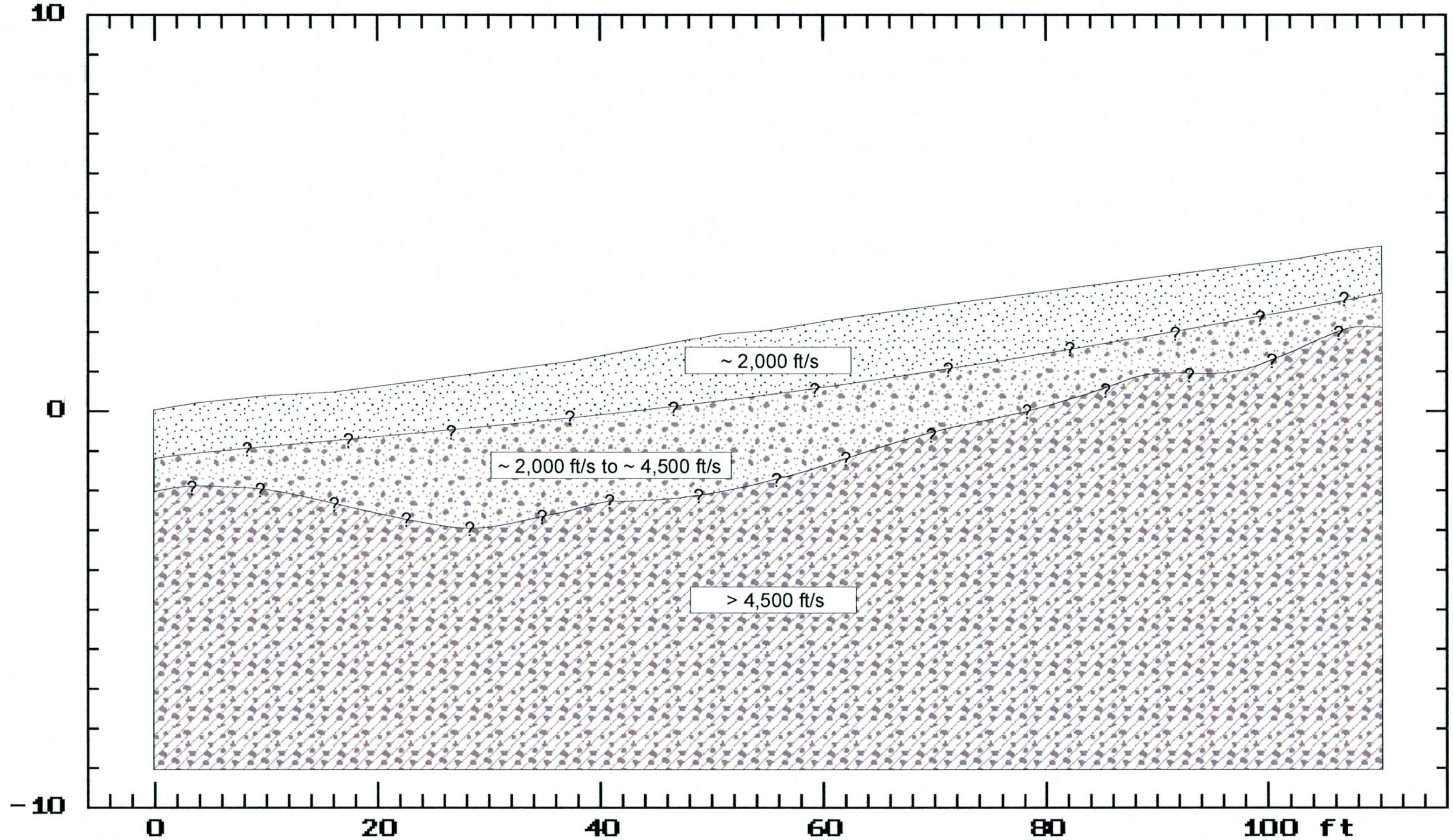
The following rippability chart (Table C-1) is based on our experience with similar materials. It assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that soil characteristics such as degree of cementation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

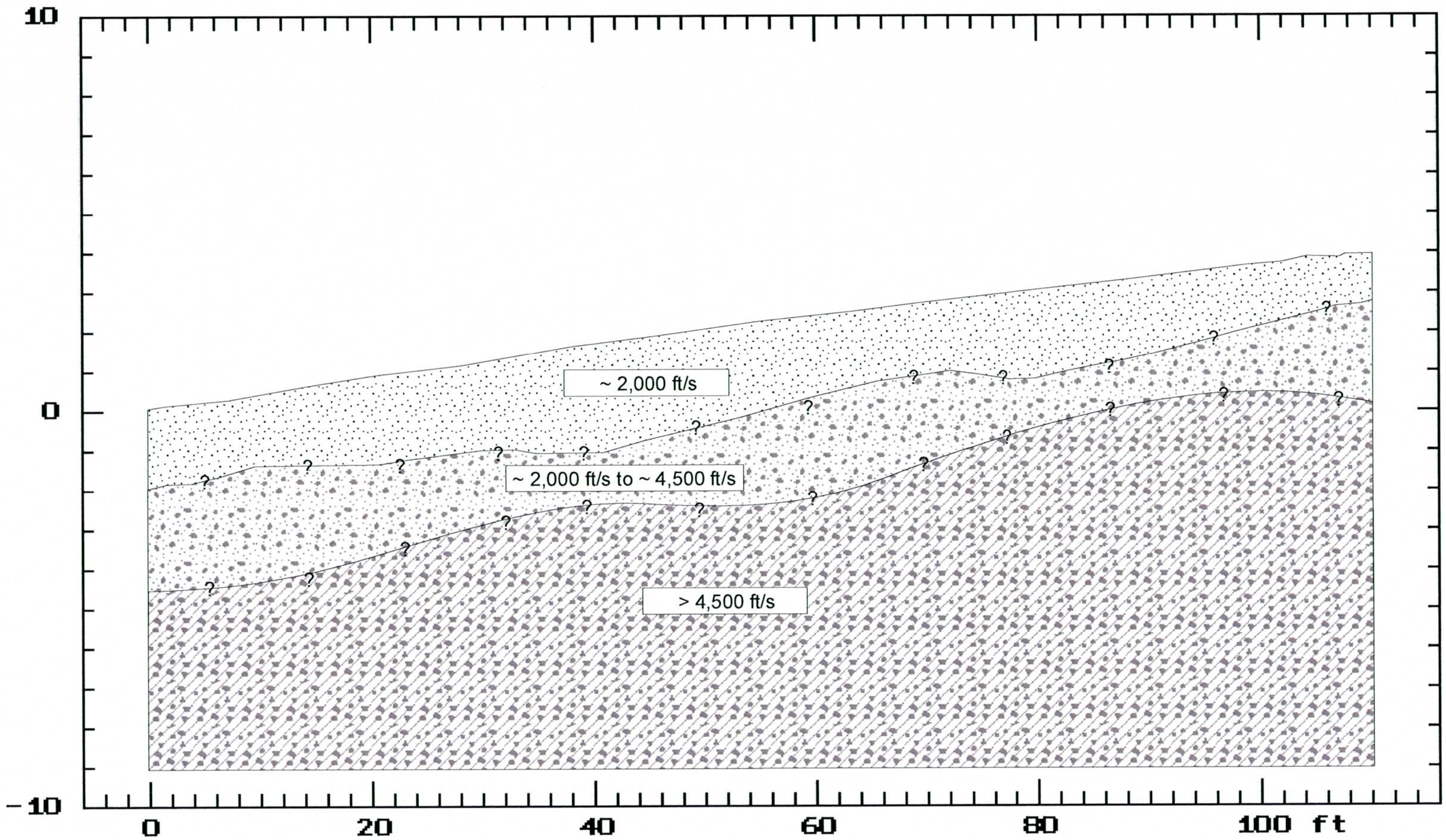
For trenching operations, the rippability figures should be scaled downward. For example, velocities as low as 3,500 feet per second may indicate difficult ripping during trenching operations. In addition, the presence of cobbles and boulders, which can be troublesome in a narrow trench, should be anticipated. 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.

Approximate layer profiles are presented in Figures C-1 through C-5, which are attached to this appendix.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.



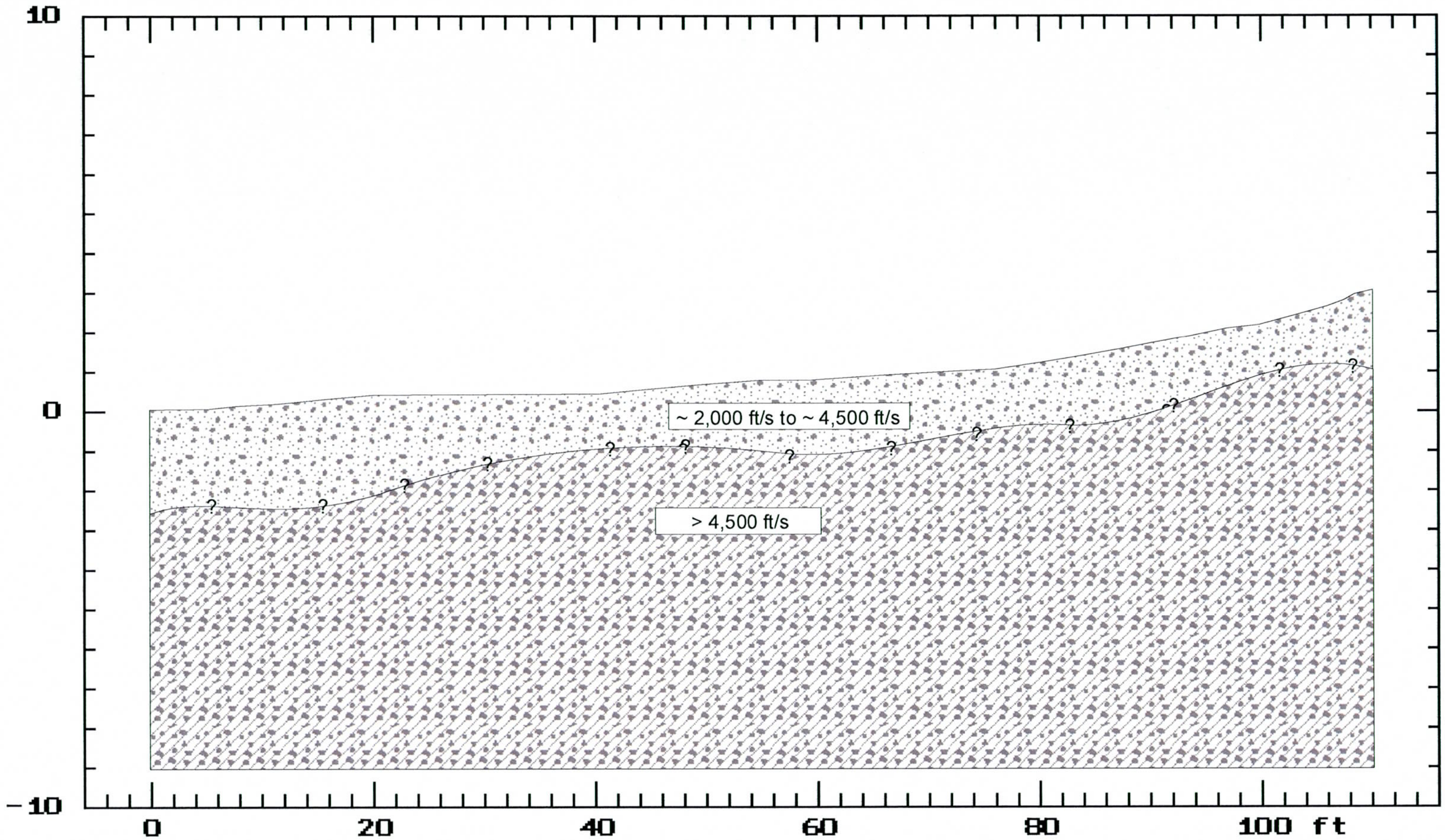
NOTE: This profile is based on seismic refraction surveys, exploratory borings, and test trenches. The layer changes shown are approximate and should not be used for detailed construction planning or estimating.

Ninyo & Moore	APPROXIMATE EXCAVATION PROFILE AT SL-1		
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA			FIGURE C-1
PROJECT No: 601052001	FILE No: 1052S-1	DATE: 01/06	



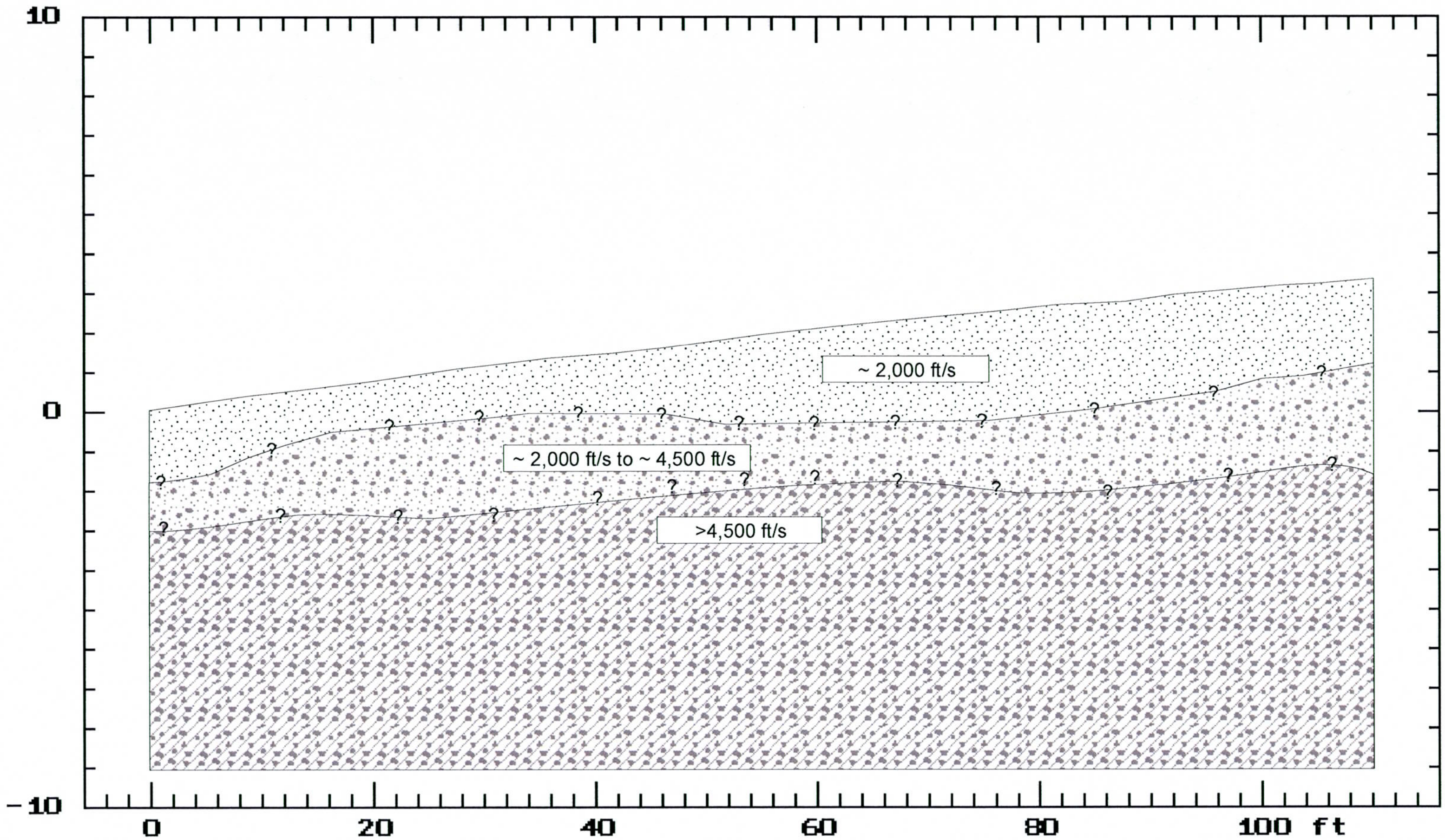
NOTE: This profile is based on seismic refraction surveys, exploratory borings, and test trenches. The layer changes shown are approximate and should not be used for detailed construction planning or estimating.

Ninyo & Moore	APPROXIMATE EXCAVATION PROFILE AT SL-2		
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA			FIGURE C-2
PROJECT No: 601052001	FILE No: 1052S-2	DATE: 01/06	



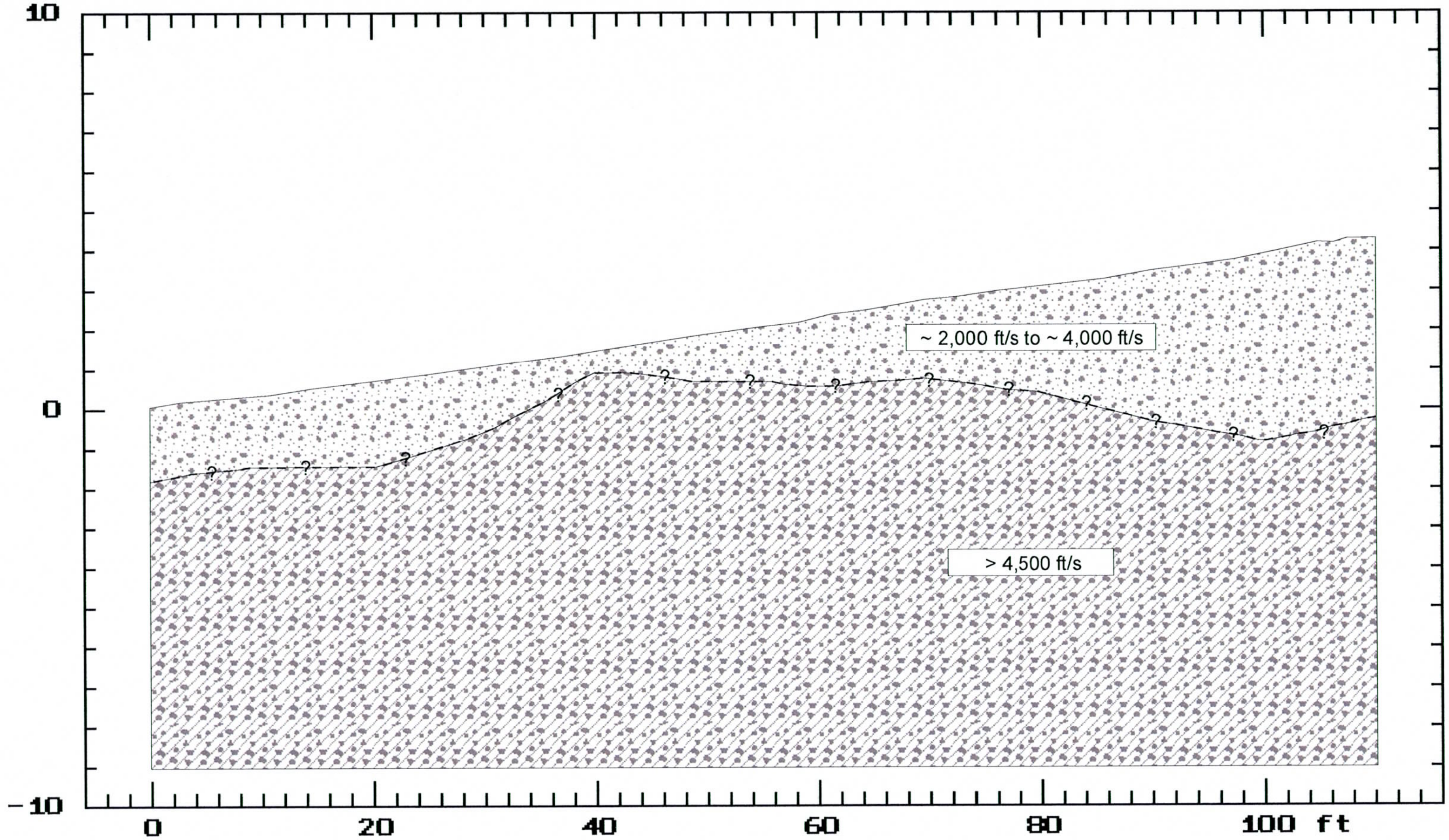
NOTE: This profile is based on seismic refraction surveys, exploratory borings, and test trenches. The layer changes shown are approximate and should not be used for detailed construction planning or estimating.

Ninyo & Moore	APPROXIMATE EXCAVATION PROFILE AT SL-3		
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA			FIGURE C-3
PROJECT No: 601052001	FILE No: 1052S-3	DATE: 01/06	



NOTE: This profile is based on seismic refraction surveys, exploratory borings, and test trenches. The layer changes shown are approximate and should not be used for detailed construction planning or estimating.

Ninyo & Moore	APPROXIMATE EXCAVATION PROFILE AT SL-4		
McDOWELL ROAD BASIN AND STORM DRAIN DESIGN MESA, ARIZONA			FIGURE C-4
PROJECT No: 601052001	FILE No: 1052S-4	DATE: 01/06	



NOTE: This profile is based on seismic refraction surveys, exploratory borings, and test trenches. The layer changes shown are approximate and should not be used for detailed construction planning or estimating.

Ninyo & Moore

APPROXIMATE EXCAVATION PROFILE AT SL-5

McDOWELL ROAD BASIN AND STORM DRAIN DESIGN
MESA, ARIZONA

FIGURE C-5

PROJECT No:	FILE No:	DATE:
601052001	1052S-5	01/06

Ninyo & Moore

APPENDIX D

AGRONOMIC TEST RESULTS

FRUIT GROWERS LABORATORY, INC.

ANALYTICAL CHEMISTS

September 9, 2005

Lab ID : SP 0509021-001
Customer ID : 2-18569

Ninyo & Moore
5710 Ruffin Road
San Diego, CA 92123-1013

Recommendation for McDowell Road Storm Drain

The following report presents the results of analyses conducted on your soil. See page 4 for sample information and analyses results. The following recommendations are based upon the current conditions of the soil. All application recommendations are for each 1,000 square feet of growing area. Please be sure to read the standard application notes presented on page 3.

I. Plant Selection

The Analyses of this soil indicates the following plant selection requirements:

A. Select only non-acidic loving plants for this soil.

B. Select only those plants that have a high or greater tolerance to free limestone for planting at this site.

C. Select only those plants that have a moderate or greater tolerance to Salinity for planting at this site. A review of the plants growing in the immediate area of the site to be landscaped will provide some additional guidelines as to the proper plant selection.

II. Preplant Soil Amendments and Fertilizers

A. Turf and Groundcover

	Apply per 1000 sq. ft.
1. Soil Amendments	
a. Organic (Well-composted)	2.00 cu. yds.
b. Limestone	0.00 lbs.
c. Soil Sulfur	0.00 lbs.

	Apply per 1000 sq. ft.
2. Fertilizers	
a. Nitrogen (N)	0.00 lbs.
b. Phosphorus (P2O5)	3.90 lbs.
c. Potassium (K2O)	2.00 lbs.
d. Magnesium (Mg)	0.00 lbs.
e. Zinc (Zn)	0.00 lbs.
f. Manganese (Mn)	0.00 lbs.
g. Iron (Fe)	0.60 lbs.
h. Copper (Cu)	0.05 lbs.
i. Boron (B)	0.01 lbs.

B. Tree and Shrub Backfill Mix

1.	Native (site) soil	66%
2.	Nitrogen Fertilized Organic Material	33%
3.	Commerical Fertilizer (8-8-4)	1 lb./cu. yd.
4.	Iron	2 oz./cu. yd.
5.	Zinc	1 oz./cu. yd.
6.	Manganese	1 oz./cu. yd.

When planting specifications do not call for a separate backfill mix then backfill the holes that are excavated to install containerized plants using the native (site) soil amended according to the preplant recommendations given on page 1.

III. Leaching Requirement

It is recommended that this soil be thoroughly leached to lower the Sulfate, Chloride prior to planting. This leaching operation should be made after the application of any recommended soil amendments, but prior to applying any of the recommended preplant fertilizers. The leaching operation should consist of three applications of irrigation water with enough water being applied at each irrigation to thoroughly wet this soil to a depth of twenty-four inches with the water being applied at a rate slow enough to prevent any runoff. A two to three day waiting period between applications of water should occur to allow for internal soil drainage.

Sulfate, ChlorideSulfate, Chloride levels should be rechecked after the above leaching operation is completed to determine the degree of improvement. These new levels will allow for the selection of plants having the appropriate salt tolerances.

IV. Post-Plant Fertilization - lbs./1000 sq. ft.

Nitrogen	1 lb.
Phosphorus	1/4 lb.
Potassium	1/4 lb.

The actual post-plant requirements for fertilizers and soil amendments will vary depending upon the specific site conditions. Periodic post-plant analyses can be used to assure proper soil conditions and balanced levels of plant nutrition.

V. Irrigation

Make certain that the irrigation water being applied is penetrating to a depth slightly greater than the root zone of the plants being grown. Water with a frequency needed to maintain moist soil at all times - never wet for long periods and never let the soil dry out.

Application Notes

The application instructions listed below apply only if the material(s) is recommended in this report on page 1. Materials not included in the recommendations are excluded either because the analyses data did not indicate a need or the analysis to determine if a need existed was not requested.

Organic Materials

Nitrolized redwood compost is preferred but other organic mixes may be substituted depending upon the site requirements. Organic materials should be spread uniformly over the surface soils and when possible should be incorporated to a depth of two to three inches.

Limestone, Dolomite & Sulfur

These materials should be broadcast uniformly over the surface soils and then incorporated to a depth of two to three inches.

Gypsum

This material should be broadcast uniformly over surface soils for water penetration. For best results do not incorporate.

Preplant Phosphorous, Zinc, Manganese, Iron & Copper

These materials should be broadcast uniformly over the surface soils and then incorporated to a depth of two to three inches. Post-plant applications can be surface applied for water penetration.

Nitrogen, Potassium & Magnesium

These materials are highly water soluble and can be applied uniformly over the surface soils for water penetration or they can be incorporated with the other materials. Magnesium sources for plant nutrition include Epsom salts (Magnesium Sulfate), and the double salt of Potasium-Magnesium Sulfate (Sulfate of Potash-magnesia).

FRUIT GROWERS LABORATORY, INC.

ANALYTICAL CHEMISTS

September 9, 2005

Ninyo & Moore
5710 Ruffin Road
San Diego, CA 92123-1013

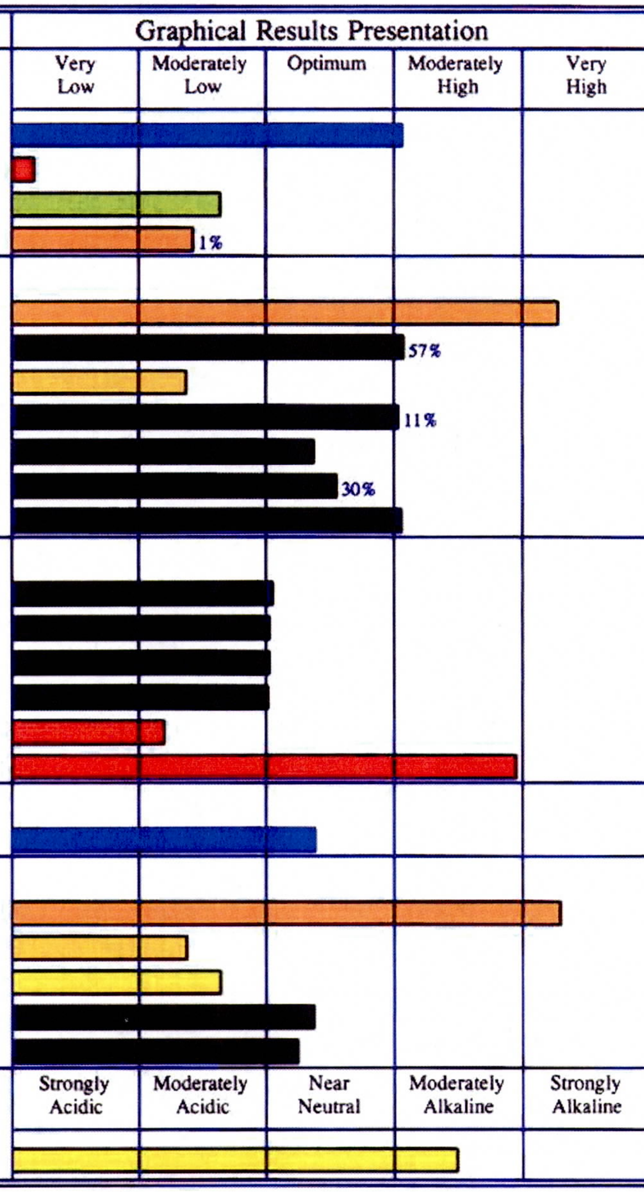
Description : Detention Basin Composite
Project : McDowell Road Storm Drain

Lab ID : SP 0509021-001
Customer ID : 2-18569

Sampled On : August 2, 2005
Sampled By : Ninyo and Moore
Received On : August 30, 2005
Depth : 0-60"
Meth Irrg. : S.S. Sprinklers

LANDSCAPE SOIL ANALYSIS

Test Description	Result	Units	Optimum Range
Primary Nutrients			
Nitrate-Nitrogen	104	Lbs/AF	58 - 98
Phosphorus-P_2O_5	30	Lbs/AF	280 - 370
Potassium-K_2O (Exch)	490	Lbs/AF	1060 - 2650
Potassium-K_2O (Sol)	90	Lbs/AF	130 - 500
Secondary Nutrients			
Calcium (Exch)	20900	Lbs/AF	13500 - 18000
Calcium (Sol)	1890	Lbs/AF	130 - 610
Magnesium (Exch)	680	Lbs/AF	1370 - 2730
Magnesium (Sol)	220	Lbs/AF	0 - 140
Sodium (Exch)	320	Lbs/AF	0 - 1290
Sodium (Sol)	1140	Lbs/AF	0 - 2070
Sulfate	4880	Lbs/AF	820 - 4660
Micro Nutrients			
Zinc	12.0	Lbs/AF	3.6 - 174
Manganese	13.2	Lbs/AF	7.2 - 261
Iron	33.2	Lbs/AF	27.4 - 290
Copper	1.6	Lbs/AF	1.1 - 44.8
Boron	1.1	Lbs/AF	1.9 - 7.9
Chloride	1460	Lbs/AF	39 - 748
CEC	28.1	meq/100g	5 - 65.0
% Base Saturation			
CEC - Calcium	92.9	%	60 - 80.0
CEC - Magnesium	5.0	%	10 - 20.0
CEC - Potassium	0.93	%	2 - 5.00
CEC - Sodium	1.25	%	0 - 5.00
CEC - Hydrogen	0.00	%	0 - 3.00
pH	7.94	---	6.5 - 7.50



Good ▬ Problem ▬ Indicates physical conditions and/or phenological and amendment requirements.

Note: Color coded bar graphs have been used to provide you with 'AT-A-GLANCE' interpretations.

Corporate Offices & Laboratory
PO Box 272 / 853 Corporation Street
Santa Paula, CA 93061-0272
TEL: 805/392-2000
FAX: 805/525-4172

Office & Laboratory
2500 Stagecoach Road
Stockton, CA 95215
TEL: 209/942-0181
FAX: 209/942-0423

Field Office
Visalia, CA
TEL: 559/734-9473
FAX: 559/734-8435
Mobile: 559/737-2399

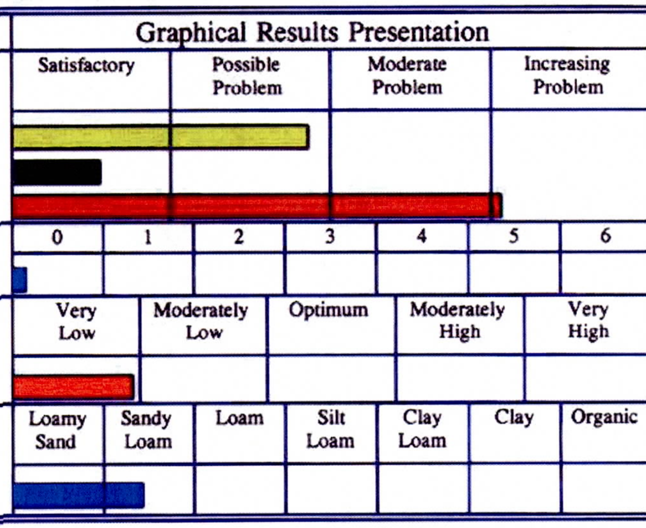
September 9, 2005

Ninyo & Moore

Lab ID : SP 0509021-001
Customer ID : 2-18569
Description : Detention Basin Composite

LANDSCAPE SOIL ANALYSIS

Test Description	Result	Units	Optimum Range
Others			
Soil Salinity	3.72	mmhos/cm	0.5 - 2.00
SAR	3.3		0 - 6.0
Limestone	8.5	%	0 - 0.5
Lime Requirement	0	Tons/AF	2.5 - 3.50
Moisture	3.4	%	6.1 - 18.2
Saturation	24.3	%	40 - 50.0



Good Problem Indicates physical conditions and/or phenological and amendment requirements.

Note: Color coded bar graphs have been used to provide you with 'AT-A-GLANCE' interpretations.

Soil pH & Limestone levels are important to consider when making plant selections. Soil pH levels above 7.0 are not suitable for acid loving plants. Soils containing limestone are not suitable for plants sensitive to Limestone.

FRUIT GROWERS LABORATORY, INC.

William L. Pidduck, Vice President

WLP:JRJ

Ninyo & Moore

APPENDIX E

PROJECT PHOTOGRAPHS


Photo 1. L-1 - View of pit at depth of 4 feet.

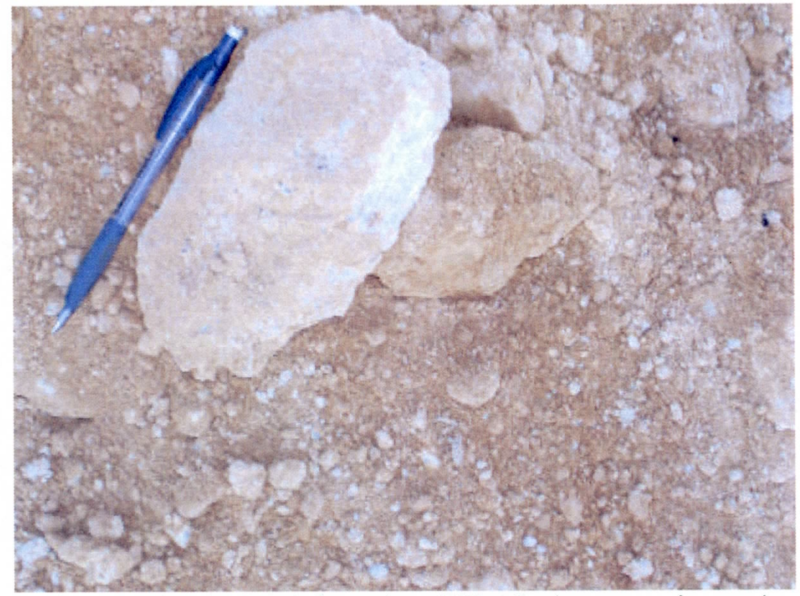


Photo 2. L-1 - Material causing difficulty in excavation at 4 feet.


Photo 3. L-1 - View of test pit at 5 feet.

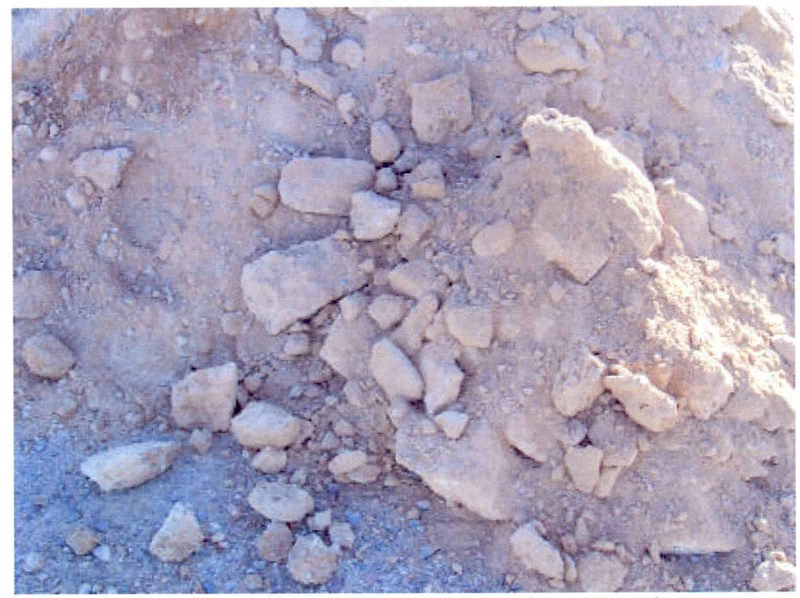


Photo 4. L-1 - View of cuttings from test pit.

Photo 5. L-1 - Pit at depth of 10.9 feet.

Photo 6. L-1 - View of pit at depth of 10.9 feet.

Photo 7. L-2 - View of pit at 6.9 feet.

Photo 8. L-2 - View of pit at 6.9 feet.

Photo 9. L-4 - Well-cemented layer with teeth marks.

Photo 10. L-4 - View of pit at 7.5 feet.

Photo 11. L-4 - Piece of well cemented layer with teeth mark.

Photo 12. L-4 - View of pit at 10.9 feet.


Photo 13. L-4 - View of pit at 10.9 feet.


Photo 14. L-3 - Spoils pile with large well-cemented clasts.


Photo 15. L-3 - Walls of test pit.


Photo 16. L-5 - View of test pit.

Photo 17. L-5 - Walls of test pit.

Photo 18. B-1A - Mini-sonic drill rig.

Photo 19. B-1A - Location of B1-A.

Photo 20. B-1A - Sample bags.

Photo 21. B-1ACollecting sample from 5.5 to 7.5 feet.

Photo 22. B-1A - Collecting sample from 5.5 to 7.5 feet.

Photo 23. B-1A - Collecting sample from 5.5 to 7.5 feet.

Photo 24. B-1A - Sample bags.

Photo 25. B-1A - Collecting sample from 5.5 to 7.5 feet.

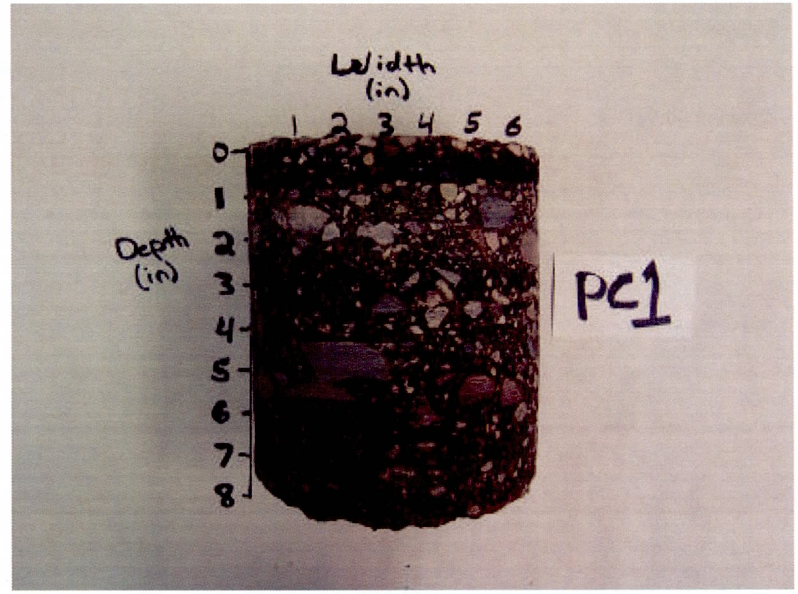


Photo 26. PC-1 - Side view of asphalt core.

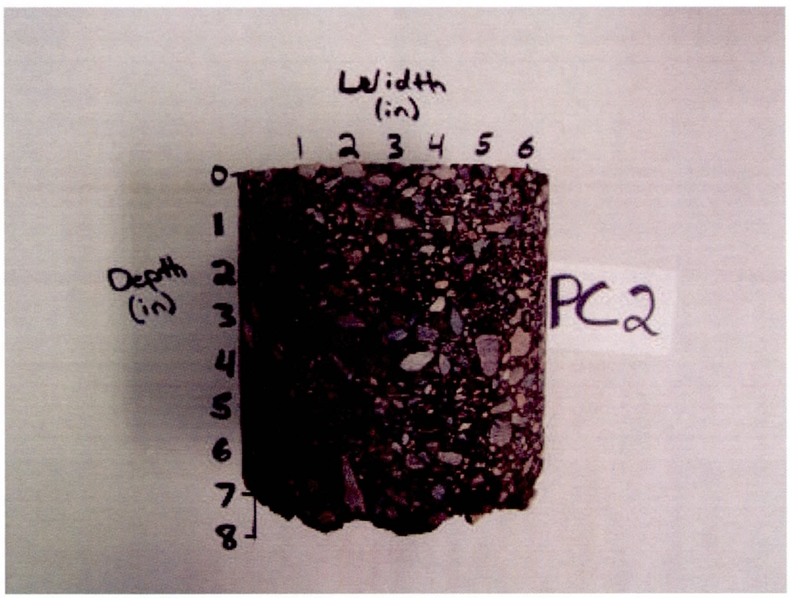


Photo 27. PC-2 - Side view of asphalt core.