Louis Andersen
Public Works Director

Scott Bender
County Engineer

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Greg Stanley
County Manager

PINAL COUNTY PUBLIC WORKS

To: **PROSPECTIVE BIDDERS**

Date: **September 2, 2015**

Project Name: **Tangerine Basin Flood Control Project**

Addendum #1

The following items shall be incorporated in the work. All items shown on the plans or specified in the project manual shall remain unchanged except as specified herein.

Questions:

1. Are CADD files available? Yes, requests can be made by emailing Gloria Bean, the files will be sent electronically.

2. Is a Soils Report available? The report will be available on the Public Works website and will be distributed in a future planned addendum.

3. There is a County pit located at Val Vista and I-10, is this pit available to relocate dirt? The County pit in the area of Val Vista and I-10 is a possible place to take excess, clean fill dirt. The County is interested in receiving the clean, fill dirt. The County pit cannot receive brush or other vegetative waste. The contractor would be responsible for coordinating with the Public Works Maintenance Branch Chief, Jim Higginbotham at (520) 866-6421 prior to bid to learn more about this option and any other potential limitations.

4. Are permits the contractors responsibility? Yes, however there are no charges for Public Works issued permits. All other permits may be subject to fees.

5. What are acceptable work hours? For residential consideration work hours are from 6AM to 4PM, deviation of these hours may be requested to the Project Engineer.

6. Is there a water source? It is the contractor's responsibility to obtain water for this project.

Louis Andersen, Director
Pinal County Public Works

GEOTECHNICAL EVALUATION
HOPI DRIVE DRAINAGE IMPROVEMENTS
CASA GRANDE, ARIZONA

PREPARED FOR:

EPS Group
2045 South Vineyard Avenue, Suite 101
Mesa, Arizona 85210

PREPARED BY:

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
3202 East Harbour Drive
Phoenix, Arizona 85034

March 18, 2013
Project No. 604037001

March 18, 2013
Project No. 604037001

Mr. Elijah Williams, PE
EPS Group
2045 South Vineyard Avenue, Suite 101
Mesa, Arizona 85210

Subject: Geotechnical Evaluation
Hopi Drive Drainage Improvements
Casa Grande, Arizona

Dear Mr. Williams:

In accordance with our proposal dated October 9, 2012, and your authorization, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced site. The attached report describes our evaluation methodology and presents our findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service to you during this phase of the project.

Sincerely,
NINYO & MOORE

REGISTERED GEOLOGIST
CERTIFICATE NO.
49080
JEFFREY RODGERS
Date Signed 3/18/13
ARIZONA, U.S.A.

Jeffrey S. Rodgers, RG
Project Geologist

EXPIRES: 03/31/15

Registered Professional Engineer (Civil)
CERTIFICATE NO.
34866
STEVEN D. NOWACZYK
Date Signed 3.18.13
ARIZONA U.S.A.

Steven D. Nowaczyk, PE
Principal Engineer

EXPIRES 06/30/2015

JSR/SDN/KLP/clj

Distribution: (1) Addressee – Electronic Copy

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1. INTRODUCTION

In accordance with our proposal dated October 9, 2012, and your authorization, we have performed a geotechnical evaluation for the Hopi Drive Drainage Improvements project in Casa Grande, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site in order to formulate geotechnical recommendations for design and construction. This report presents the results of our evaluation along with our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Conducting a visual geologic reconnaissance of the area and reviewing background information including geologic maps and aerial photographs.
- Conducting a site visit to select and mark out the boring locations and notifying Arizona Blue Stake of the locations prior to drilling.
- Drilling, logging, and sampling three small-diameter exploratory borings to depths ranging from approximately 9 to 10 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Performing laboratory tests on selected samples obtained from our borings to evaluate the in-situ moisture content and dry density, gradation analysis, Atterberg limits, consolidation (response-to-wetting), and corrosivity characteristics (including pH, minimum electrical resistivity, and soluble sulfate and chloride contents). The results of the laboratory testing are presented on the boring logs and/or in Appendix B.
- Performing a single-ring infiltration test within the footprint of a proposed basin. The results of the infiltration test are presented in Appendix C.
- Preparing this report presenting our findings, conclusions, and recommendations regarding the design and construction of the project.

3. SITE DESCRIPTION

The project site is located in Section 20 of Township 5 South, Range 6 East in Casa Grande Arizona. The approximate location of the site is depicted on Figure 1. At the time of our evaluation the project site generally consisted of undeveloped desert land with a north-south traversing natural drainage that crosses under Hopi Drive and Val Vista Road. The drainage was channelized between Hopi Drive and Val Vista Road and had residential and commercial structures flanking the east and west sides of the channel. The channel crossed under Hopi Drive, Havasupai Drive, and Val Vista Road via box culverts. Hopi Drive, Havasupai Drive, and Val Vista Road were asphalt-paved, east-west traversing Roadways.

According to the *Casa Grande West, Arizona, 7.5-Minute United States Geological Survey (USGS) Topographic Quadrangle Map (2011),* the elevation at the project site is approximately 1,515 feet relative to mean sea level (MSL) near the northern limits and approximately 1,490 feet MSL at the southern limits. Based on information presented on the topographic quadrangle map, the regional topography at the site generally slopes from the northwest down to the southeast.

Several aerial photographs of the project site were reviewed for this project. A 1993 Historic Aerials photograph and a 1996 USGS photograph depicted the site as having residential development to the west of the alignment. A natural drainage was observed in this photograph that crossed Hopi Drive, Havasupai Drive, and Val Vista Road. A 2002 photograph from Digital Globe depicted the construction of residential development along the east and west sides of the previously described natural drainage, between Havasupai Drive and Val Vista Road. A 2007 photograph depicts the residential structures previously described as being under construction on both sides of the natural drainage. A 2012 photograph depicts a commercial structure constructed between Hopi Drive and Havasupai Drive east of the natural drainage. This photograph depicted the site as being similar to its current condition.

4. PROPOSED CONSTRUCTION

The project consists of the design and construction of improvements to the existing Hopi FCP system. The improvements generally include the construction of two new basins, an overflow drainage channel, a concrete-lined drainage channel, and new box culverts where the channel crosses under Hopi Drive and Val Vista Road.

The alignment generally traverses parallel and adjacent to the east side of Turzigoot Road south from the intersection of Turzigoot Drive and Tangerine Road. The alignment then traverses east-west just north of Hopi Drive and connects into a natural drainage to the east. The alignment traverses south along the alignment of the natural drainage and ends at Val Vista Road.

We understand that the drainage channel that extends from approximately Tangerine Road to Hopi Drive will be on the order of 3 to 5 feet deep. The new concrete-lined channel between Hopi Drive and Havasupai Drive is planned to be on the order of 3 to 4 feet deep. The new box culverts will consist of three, 10-feet wide by 4-feet deep box culverts at Hopi Drive, and two, 10-feet wide by 3-feet deep boxes at Val Vista Road. We understand that no culverts will be added where the drainage crosses Havasupai Drive.

Two basins will be constructed near the northern limits of the site as well as at the location where the drainage and Hopi Drive intersect. The basins will be on the order of 5 feet deep.

5. FIELD EXPLORATION AND LABORATORY TESTING

On February 12, 2013, Ninyo & Moore conducted a subsurface evaluation at the project site in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Our evaluation consisted of drilling, logging, and sampling three small-diameter borings denoted as B-1 through B-3. The borings were advanced using a CME-75 truck-mounted drill rig equipped with hollow-stem augers, and extended to approximately 9 to 10 feet bgs. Bulk and relatively undisturbed soil samples were collected at selected intervals. Detailed descriptions of the soils encountered are presented on the boring logs in Appendix A. The general locations of the borings are depicted 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 (SPT) and bulk samples were sealed in plastic bags to retain their approximate in-place moisture.

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona, for geotechnical laboratory testing. The testing included in-situ moisture content and dry density, gradation analyses, Atterberg limits, consolidation (response-to-wetting) and corrosivity characteristics (including pH, minimum electrical resistivity, and soluble sulfate and chloride contents). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. A description of each laboratory test method and the remainder of the test results are presented in Appendix B.

We also performed an infiltration test within the footprint of the proposed northern retention basin. The infiltration test procedure included drilling a 12-inch diameter borehole to a depth of about 5 feet bgs. The hole was cleaned of loose soil and a 10-inch outer diameter polyvinyl-chloride (PVC) casing was embedded into the soil at the bottom of the hole to a depth of approximately 1-inch. The PVC casing was filled with water in order to pre-wet the soil. The test continued after the pre-wetting period by refilling the casing and monitoring the drop in water level as a function of time until steady-state conditions were achieved. The field measurements and results of this test are provided in Appendix C. These results are based on a sidewall correction factor of 0.333, and a de-rating factor of 0.5 applied to the measured field results. As such, the factored percolation rate was approximately 1.0-inch per hour. This result should be viewed as approximate and is based on the site conditions at the test location on the date tested. This percolation rate may not be representative of the project site. The approximate location of the test is depicted on Figure 2.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections describe the geologic and subsurface conditions at the site.

6.1. Geologic Setting

The project site is situated along the boundary of the Sonoran Desert Section of the Basin and Range Physiographic Province and the Transition Zone (also referred to as the Central Highlands), which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 18 million years ago during the mid- to late-Tertiary 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 river deposition. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains.

The surficial geology of the site is described as being Holocene-age (less than 10,000 years) alluvial fan deposits generally consisting of clay, silt, and sand (Pearthree et al. 1988). The United States Department of Agriculture (USDA) Web Soil Survey described the site as the Mohall Clay Loam, which consists of deposits of clay, clayey sand, and silty sand.

6.2. Subsurface Conditions

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

Alluvium was encountered at the surface of our borings and extended to the total explored depths. The alluvium generally consisted of sandy clay, clayey sand, and silty, clayey sand in our borings. Varying amounts of gravel were observed in this material in our borings. Scattered to numerous caliche nodules were observed in the alluvial material in our borings.

6.3. Groundwater

Groundwater was not encountered in our borings. Based on well data provided by the Arizona Department of Water Resources (ADWR, 2012), the depth to the regional groundwater table, as measured in wells situated near the site, has been estimated to be on the order of 270 feet bgs. Groundwater levels may fluctuate due to the close proximity to the natural drainage, any adjacent ditches and canals, seasonal variations, irrigation, groundwater withdrawal or injection, and other factors. In general, groundwater is not anticipated to be a constraint to the construction of this project.

7. GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site such as land subsidence and earth fissures and faulting.

7.1. Land Subsidence and Earth Fissures

Groundwater depletion, due to groundwater pumping, has caused land subsidence and earth fissures in numerous alluvial basins in 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 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 no known earth-fissures underlying the project site. Review of published ADWR Interferometric Synthetic Aperture Radar (InSAR) maps showed that the project site is situated between two documented subsidence bowls; however, these maps did not depict ongoing subsidence at the project site. The closest documented earth fissure to the site is located approximately 5 miles to the east of the site (AZGS, 2009). Continued groundwater withdrawal in the area may result in subsidence and the formation of new fissures or the extension of existing fissures. While the future occurrence of land subsidence and earth fissures cannot accurately be predicted, these phenomena are not expected to be a constraint to the construction of this project.

7.2. Faulting

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 property. The closest fault to the site is the Sand Tank Fault, situated approximately 51 miles to the west of the site (Pearthree, 1998). The Sand Tank Fault is situated along the western piedmont of the Sand Tank Mountains, to the southeast of Gila Bend. The fault is a northeast striking normal fault that dips to the northwest. The most recent movement along this fault was as recent as approximately 70,000 years ago during the Late Pleistocene epoch. The slip-rate category of this fault is less than 0.2 millimeters per year (Pearthree, 1998). Seismic design considerations are presented in Section 9.3.

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:

- The on-site surface soils should generally be excavatable to the anticipated earthwork depths with heavy-duty earth moving construction equipment in good working condition.
- We estimate an earthwork (shrinkage) factor of 10 to 20 percent if the on-site soils are re-used as engineered fill.
- New box culverts and wing walls should be founded on 12 or more inches of engineered fill as described in Section 9.1.6.
- Imported soils and soils generated from on-site excavation activities that exhibit a relatively low plasticity and very low to low expansion potential can generally be used as engineered fill.
- Groundwater was not observed in our borings. The regional groundwater table has been encountered on the order of 270 feet bgs near the site, based on the nearby well data.
- No known geologic hazards are situated immediately adjacent to or below the surface at the site.
- Corrosivity test results indicate that subgrade soils at the site are generally considered to be corrosive to ferrous materials, and the sulfate content of the soils present a negligible sulfate exposure to concrete.

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction. An additional geotechnical evaluation should be conducted when the details for the proposed construction are available.

9.1 Earthwork

In general, the specifications contained in the latest revisions to Maricopa Association of Governments (MAG) and any City of Casa Grande amendments are expected to apply, except as noted in the following sections.

9.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, site observations, and our experience with similar materials. In our opinion, the excavation of near-surface on-site materials can generally be accomplished with heavy-duty earthmoving or excavation equipment in good operating condition. As previously described above, gravel and numerous caliche nodules were encountered in our borings. This may slow the rate and/or call for more aggressive excavation techniques depending on the actual degree of cementation encountered during construction. It should be noted that due to the wide spacing of our soil borings, excavation conditions different from what was encountered in our borings may be encountered during construction.

Temporary excavations that are 20 feet deep or less could be constructed using a sloped excavation in accordance with Occupational Safety and Health Administration (OSHA) Standards, based on the soil types encountered. We recommend that the OSHA soil "*Type C*" be used for the soils along the alignment. Based on OSHA standards, this corresponds to a temporary side slope of 1.5:1 Horizontal:Vertical (H:V), or flatter, in sloped excavations that are less than 20 feet. Slope stability for trenches deeper than 20 feet, though not anticipated, should be designed by the contractor's engineer based on alignment-specific soil properties and settlement-sensitive features.

9.1.2. Permanent Cut Slopes

Permanent cut slopes that are protected from erosion (by soil cement, riprap, shotcrete, gabions, etc.) for this project can be sloped at an angle of 2:1 (H:V) for excavations less than 5 feet below adjacent grade. Sloughing of the side soils should be anticipated if the slopes are not protected from erosion. Regular maintenance should be anticipated for the slopes.

9.1.3. Temporary Shoring

Due to the adjacent roadway and underground utilities, temporary earth retention systems may be needed for this project. Temporary earth retention systems may include braced systems, such as trench boxes or shields with internal supports or cantilever systems (e.g., soldier piles and lagging); however, the risk of excessive lateral deflection may render the cantilever shoring system inappropriate for the project.

The contractor should retain a qualified and experienced engineer to design the shoring system. We recommend that the contractor take appropriate measures to protect the workers. OSHA requirements pertaining to workers' safety should be observed. Ninyo & Moore should evaluate the soil parameters used by the shoring engineer for appropriateness.

9.1.4. Bottom Stability

The proposed excavations are not anticipated to encounter significant groundwater (with the possible exception of surface run-off or perched zones) during construction. Therefore, trench bottom stability problems during construction are generally not anticipated at this site. However, if excavations are located near drainage ditches, or near washes, arroyos, or drainage areas that are open during a heavy rain event, or near any leaking utilities, the trench material(s) might become saturated and unstable and a dewatering system may be needed for these conditions. Should this occur, remedial measures will be needed.

9.1.5. Construction Dewatering

Stream flow, surface run-off, and perched groundwater will vary seasonally depending on rainfall in the site vicinity. Excavations that do encounter surface run-off (if any) could be dewatered by pumping the water out from the bottom and away from the excavation. However, heavily saturated units or perched groundwater zones, if encountered, may call for more aggressive means of dewatering and consultation with a qualified expert. Discharge of water from the excavations to natural drainage channels may entail securing a special permit.

9.1.6. Grading, Fill Placement, and Compaction

Vegetation and debris from the clearing operation and demolition debris should be removed from the site and disposed of at a legal dumpsite. Obstructions that extend below finish grade, if present, should be removed and the resulting holes filled with compacted soil.

The geotechnical consultant should carefully evaluate any areas of soft or wet soils prior to placement of grade-raise fill or other construction. Drying or overexcavation of some materials may be appropriate.

On-site soils and imported soils that are suitable for re-use as engineered fill should not consist of potentially expansive material as evaluated by the ASTM D 4318 of having a Plasticity Index (PI) more than 20, and/or Expansion Index (EI) more than 50, as evaluated by ASTM D 4829. Our Atterberg limits tests on selected samples indicated that the plasticity indices ranged from 7 to 15. As such, it is our opinion that many of the on-site soils are suitable for re-use as engineered fill during construction. Additional evaluation should be conducted prior to and/or during construction by the Contractor to better delineate areas of unacceptable soils, if encountered.

In addition, suitable fill material should not include organic material (more than 4 percent organic content), construction debris, or other non-soil fill materials. Clay lumps or rock particles should not be larger than 4 inches in dimension.

We recommend that new box culverts and wing wall foundations be supported on 12 inches, or more, of moisture-conditioned and compacted engineered fill. This zone can either be improved by overexcavation or scarification. The fill thickness should be measured from the bottom of the box culvert base and should be compacted by appropriate mechanical methods to 95 percent or more relative compaction, in accordance with ASTM D 698 at a moisture content slightly above its optimum. The improvement below box culverts should extend laterally 2 or more feet horizontally beyond the culvert footprint. An earthwork (shrinkage) factor ranging from 10 to 20 percent for the on-site soils is estimated for this project.

Following the improvement as described above, and prior to the placement of new fill, the resulting exposed surface should be proof-rolled and carefully evaluated by Ninyo & Moore. Based on this evaluation, additional remediation may be needed. This could include scarification of the exposed surface. This additional remediation, if needed, should be addressed by Ninyo & Moore during the earthwork operations.

9.1.7. Imported Fill Material

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

9.2. Soil-Cement Bank Protection

We understand that a soil-cement treated surface may be considered for this project. Soil-cement bank protection is normally constructed in horizontal stair-step lifts, with relatively steep side slopes. Lift thicknesses generally range from 6 to 9 inches and each stair-step is typically on the order of 8 feet wide. The soil-cement treatment should be toed-down to an elevation that is deeper that the estimate total scour depth.

The percentage of cement needed for this type of application is typically based on a desired compressive strength and the composition of the soils used. We recommend utilizing a compressive strength of 750 or more pounds per square inch (psi). However, the percentage of cement content needed may differ along the alignment because of the variety of soil types encountered. The following table represents a typical range of cement content percentages needed to achieve a maximum dry density of about 120 pounds per cubic foot for various soil gradations.

Table 1 – Typical Variation in Cement Content

Material Retained on No 4 Sieve (%)	Typical Cement Content (%)
0 to 14	7 to 8
15 to 29	6 to 8
30 to 45	6 to 9
45 and more	Soil-cement not recommended

Based on the above-mentioned table and our laboratory testing, we estimate that the cement content needed for this project ranges between about 7 to 8 percent. This estimate is based on limited soil sampling and testing and should be used for planning purposes. The contractor should perform independent testing during the earthwork operations to better define the actual percentage of cement needed for this project. It should also be noted that soil-cement treated surfaces may be difficult to manufacture from soil types with a plasticity index in excess of 20 as evaluated by ASTM 4318.

9.3. Seismic Design Considerations

Based on a Probabilistic Seismic Hazard Assessment for the conterminous United States, issued by the USGS (2002 data), the site is located in a zone where the peak ground accelerations having 10, 5, and 2 percent probability of being exceeded in 50 years are 0.04g, 0.05g, and 0.08g, respectively. These ground motion values are calculated for "firm rock" sites, which correspond to a shear-wave velocity of approximately 2,500 feet per second in approximately the top 100 feet bgs. Different soil or rock types may amplify or de-amplify these values. The proposed improvements should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the seismic design parameters for the site in accordance with International Building Code (ICC, 2009) guidelines and mapped spectral acceleration parameters (USGS, 2011).

Table 2 – 2009 International Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.4
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.190 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.063 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.303 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.152 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.202 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.101 g

9.4. Box Culverts/Wing Walls

Box culverts and wing wall foundations should be supported on 12 or more inches of engineered fill, as described in Section 9.1.6. Box culvert foundations may be designed using an allowable bearing pressure of up to 2,000 pounds per square foot (psf) for static conditions.

Total and differential movements of up to about 1 inch and 1/2-inch, respectively, may occur. Distortions of about 1/2-inch (vertical) over 20 feet (horizontal) are possible.

The "at-rest" earth pressure against box culvert and wing walls that are restrained at the top or braced so that they cannot yield, and with level backfill with no water present, may be taken as equivalent to the pressure exerted by a fluid weighing 55 pounds per cubic foot (pcf). For undrained conditions, an equivalent fluid pressure of 91 pcf may be used. Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.5q. The value for "q" represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads.

Box culvert or wing walls that are not restrained from movement at the top (such as a "U"-shaped box with no roof) and have a level backfill behind the wall may be designed using an "active" equivalent fluid unit weight of 35 pcf for drained conditions, and 80 pcf for undrained conditions. This value assumes compaction within about 5 feet of the wall will be accomplished with relatively light compaction equipment, and that very low to low expansive backfill will be placed behind the wall. For any wing walls with sloping backfill behind them, Ninyo & Moore should be contacted for the recommended "active" equivalent fluid pressure based on the actual slope configuration. Retaining walls should also be designed to resist a horizontal earth pressure of 0.30q. The value for "q" represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads.

9.5. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the concrete. Corrosion potential was evaluated using the results of laboratory testing of one 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 samples tested ranged from 8.0 to 8.2, which is considered to be alkaline. The minimum electrical resistivity measured in the laboratory ranged from 1,040 ohm-cm to 1,505 ohm-cm, which is considered to be corrosive to ferrous materials. The chloride content of the samples tested ranged from 43 ppm to 77 ppm, which is also considered to be corrosive to ferrous materials. The soluble sulfate content of the soil samples was 0.003 percent by weight, representing a negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are considered to be corrosive to ferrous materials. Therefore, special consideration may be given to the use of heavy-gauge, corrosion-protected, underground steel pipe or culverts. As an alternative, wrapped/plastic pipe or reinforced concrete pipe may be considered. A corrosion specialist should be consulted for further recommendations.

9.6. Concrete

Laboratory chemical tests performed on an on-site soil sample indicated a sulfate content up to 0.003 percent by weight. Based on the following American Concrete Institute (ACI) table, the on-site soils are generally considered to have a moderate sulfate exposure to concrete.

Table 3 – ACI Requirements for Concrete Exposed to Sulfate-Containing Soil

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

[1] A lower water-cementitious materials ratio or higher strength may be call for low permeability or for protection against corrosion of embedded items or freezing and thawing (ACI Table 4.2.2).
[2] Seawater.
[3] Pozzolan that has been evaluated by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Notwithstanding, the sulfate test results and 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.

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

9.7. Site Drainage

Surface drainage should be provided to divert water away from the paved surfaces and structures. Surface water should not be permitted to pond on or adjacent to pavement areas. To deter accumulation of water below the new pavement sections, the subgrade soils below the new pavement sections should be sloped away from the center toward the edges of the roadway.

9.8. Pre-Construction Conference

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

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

American Concrete Institute, 2005, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

American Society for Testing and Materials (ASTM), Annual Book of ASTM Standards.

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Euge, K.M., et al., 1992, Development of Seismic Acceleration Contour Maps for Arizona: Arizona Department of Transportation Report No. AZ 92-344: dated September.

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Pearthree, P.A., 1998, Quaternary Fault Data and Map for Arizona: Arizona Geological Survey, Open-File Report 98-24.

United States Geological Survey, 2011, Casa Grande West, Arizona, 7.5-Minute Series (Topographic): Scale 1 = 24,000.

United States Geological Survey, 2009, National Seismic Hazard Mapping Project, World Wide Web, http://geohazards.cr.usgs.gov/eq.

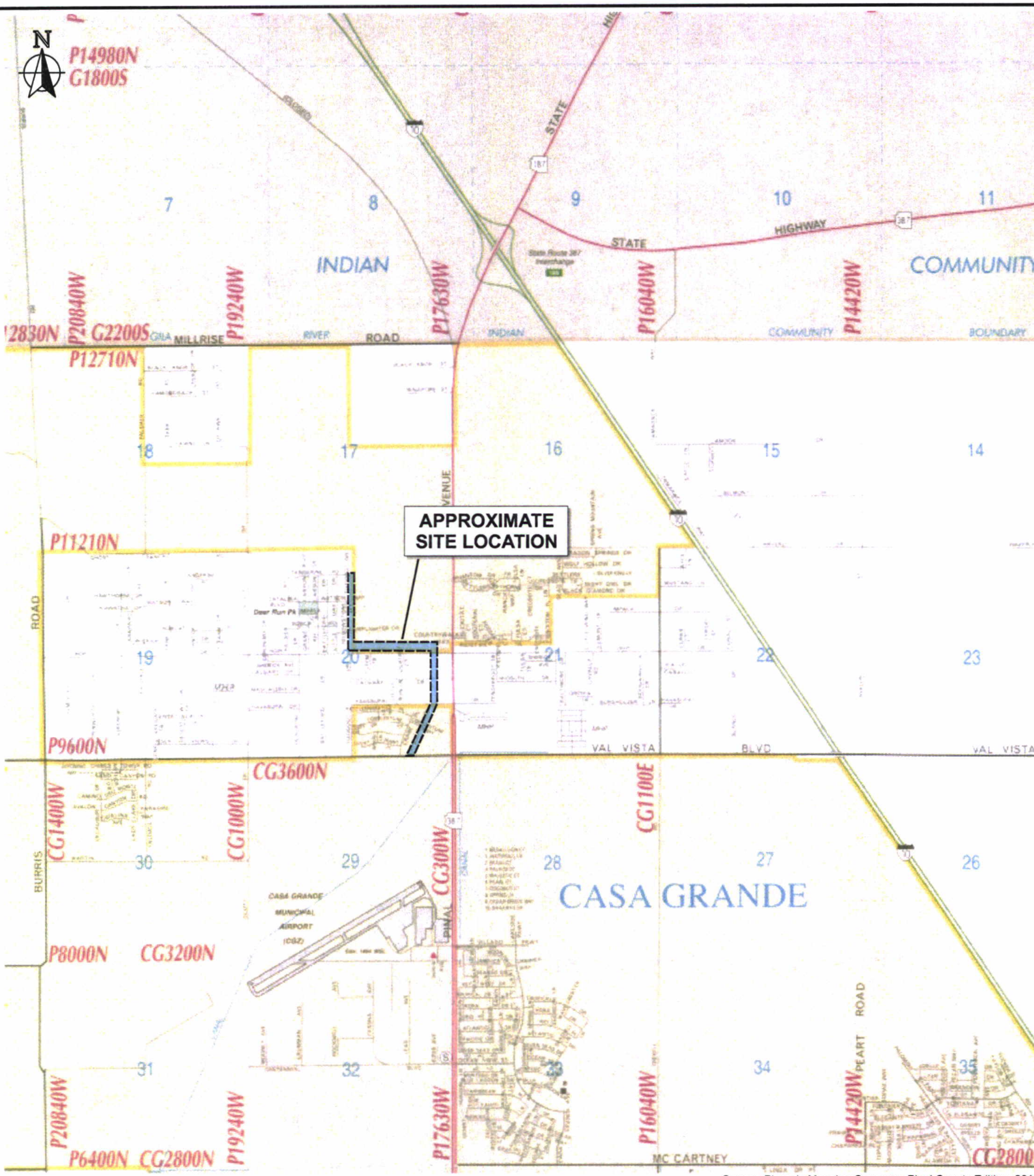
United States Department of Agriculture Web Soil Survey, http://websoilsurvey.nrcs.usda.gov.

AERIAL PHOTOGRAPHS

Source	Date
Historicaerials.com	1993
United States Geological Survey	1996
Digital Globe	2002
USDA Farm Service	2007

AERIAL PHOTOGRAPHS

Source	Date
Google Earth	2013



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

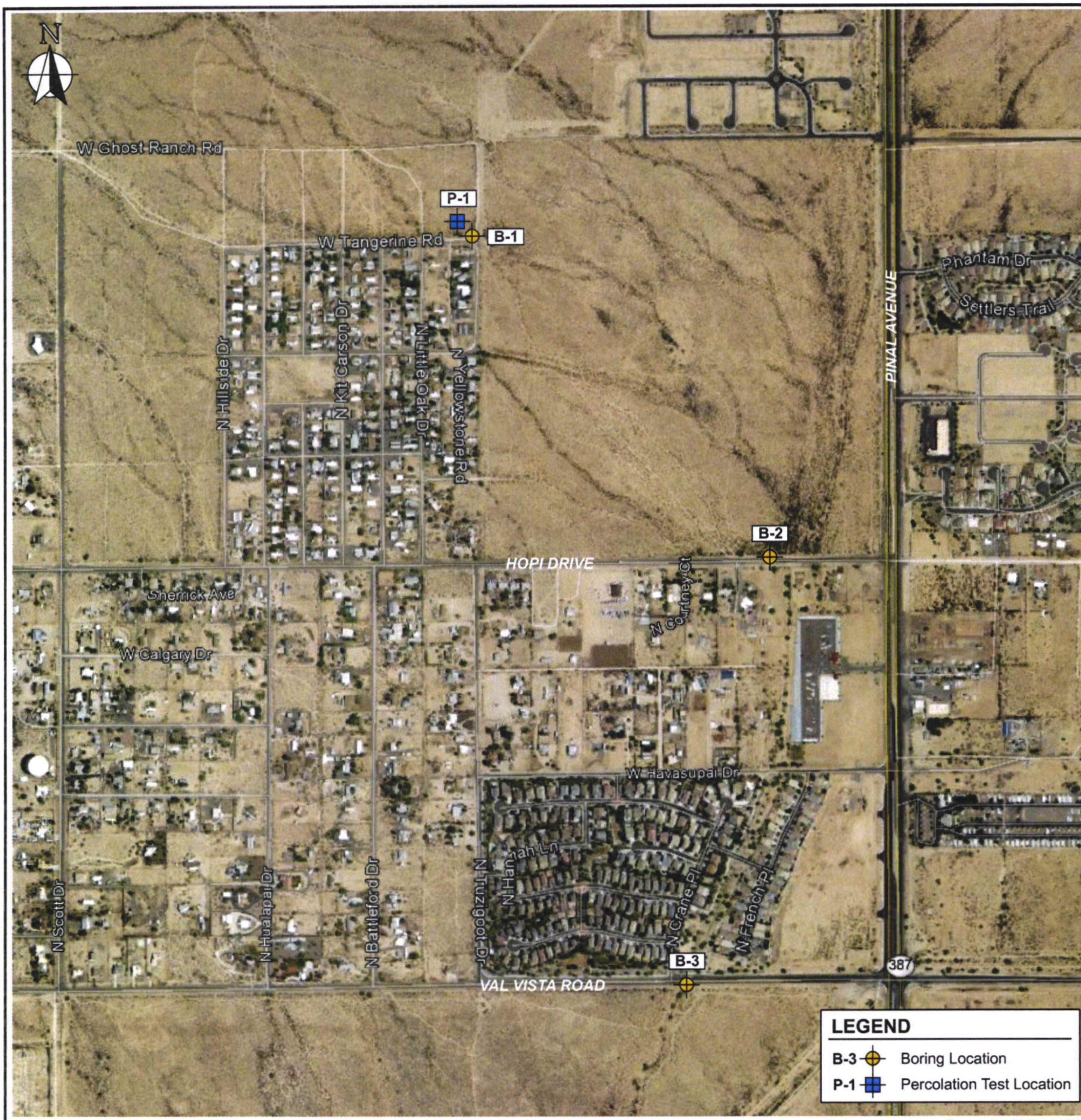
0 3300

Approximate Scale: 1 inch = 3300 feet

Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore		SITE LOCATION	FIGURE 1
PROJECT NO: 604037001	DATE: 3/13	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	

file no: 4037vmap0313



Source: NAVTEQ, 05/26/12.

0 1200

Approximate Scale:
1 inch = 1200 feet

Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore		EXPLORATION LOCATIONS	FIGURE
PROJECT NO: 604037001	DATE: 3/13	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	2

file no: 4037sp0313

APPENDIX A

BORING LOGS

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

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

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

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer or the kelly bar of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

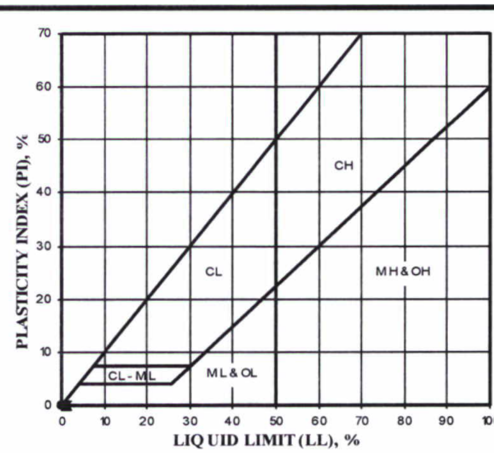
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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 slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & 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	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	306 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.075
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No. 10 to No. 40	2.00 to 0.420
Fine	No. 40 to No. 200	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

Updated Nov. 2011

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	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	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: Shear 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

Updated Nov. 2011

DATE DRILLED	2/12/13	BORING NO.	B-1
GROUND ELEVATION	--	SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (D&S Drilling)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY DM	REVIEWED BY JSR

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Brown, damp, medium dense, clayey SAND with gravel.
			18	3.4	103.9			
			33					Very dense; scattered caliche nodules.
5								
							CL	Brown, damp, hard, sandy CLAY; trace gravel; numerous caliche nodules.
			59	4.4	104.4			
			26					
10								Total Depth = 10 feet. Groundwater not encountered during drilling. Backfilled on 2/12/13 promptly after completion of drilling. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								
20								

Ninyo & Moore

BORING LOG

HOPI DRIVE DRAINAGE IMPROVEMENTS
CASA GRANDE, ARIZONA

PROJECT NO.	DATE	FIGURE
604037001	3/13	A-1

DATE DRILLED	2/12/13	BORING NO.	B-2
GROUND ELEVATION	--	SHEET 1 OF 1	
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (D&S Drilling)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY DM	LOGGED BY DM	REVIEWED BY	JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	ALLUVIUM: Brown, damp, loose to medium dense, clayey SAND.
		7					
						SC-SM	Brown, damp, medium dense, silty, clayey SAND with gravel.
		22	2.6	108.9			
5							
						SC	Brown, damp, medium dense, clayey SAND; scattered caliche nodules.
		18					
		50/5"					Very dense.
							Total Depth = 8.9 feet. Groundwater not encountered during drilling. Backfilled on 2/12/13 promptly after completion of drilling.
10							Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15							
20							

Ninyo & Moore

BORING LOG

HOPI DRIVE DRAINAGE IMPROVEMENTS
CASA GRANDE, ARIZONA

PROJECT NO.	DATE	FIGURE
604037001	3/13	A-2

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC-SM	ALLUVIUM: Brown, damp, dense, silty, clayey SAND with gravel.
			52	2.8	111.9			
			29					
5								
			27	4.9	104.9			Medium dense.
							SC	Brown, damp, dense, clayey SAND; scattered caliche nodules.
			28					
10								Total Depth = 10 feet. Groundwater not encountered during drilling. Backfilled on 2/12/13 promptly after completion of drilling. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								
20								

DATE DRILLED 2/12/13 BORING NO. B-3

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY JSR

Ninyo & Moore

BORING LOG

HOPI DRIVE DRAINAGE IMPROVEMENTS
CASA GRANDE, ARIZONA

PROJECT NO.	DATE	FIGURE
604037001	3/13	A-3

APPENDIX B

LABORATORY TESTING

Classification

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

In-situ 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. 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. The grain-size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figure B-4.

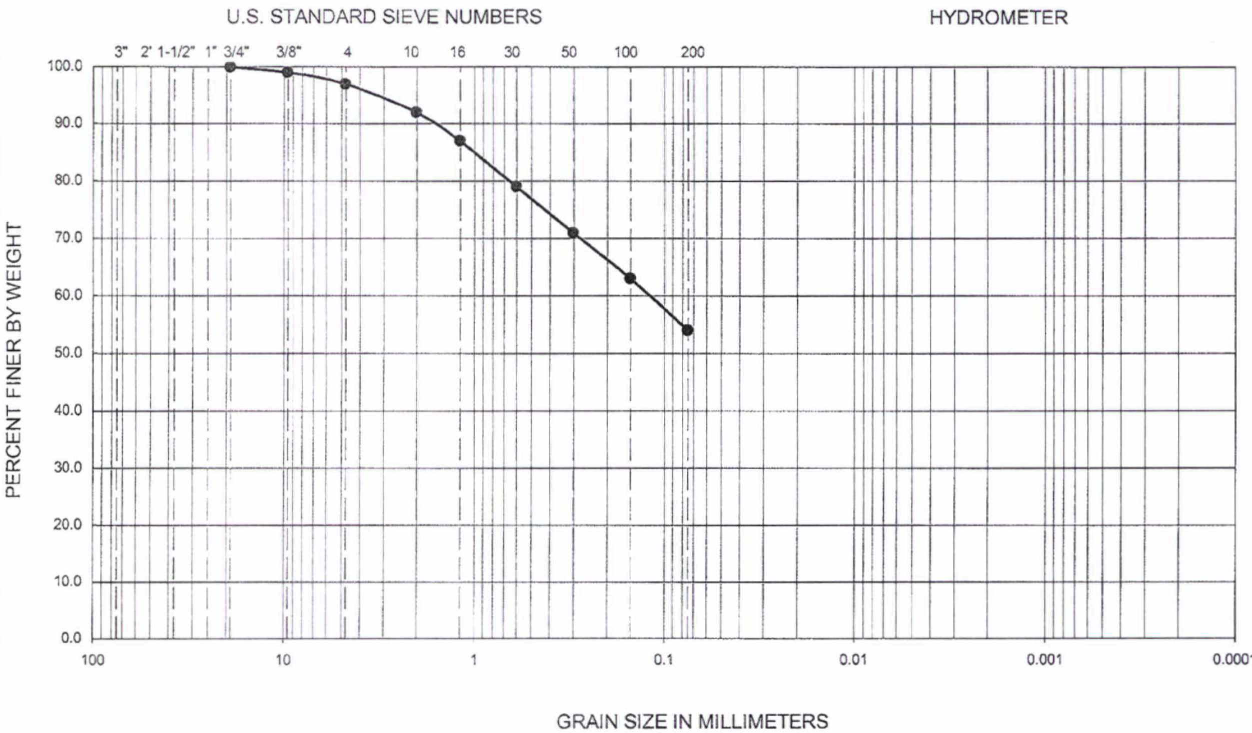
Consolidation Tests

Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. 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-5 through B-6.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with Arizona Test ARIZ 236b. The chloride content of these selected samples was evaluated in general accordance with ARIZ 736. The sulfate content of these selected samples was evaluated in general accordance with ARIZ 733. The test results are presented on Figure B-7.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

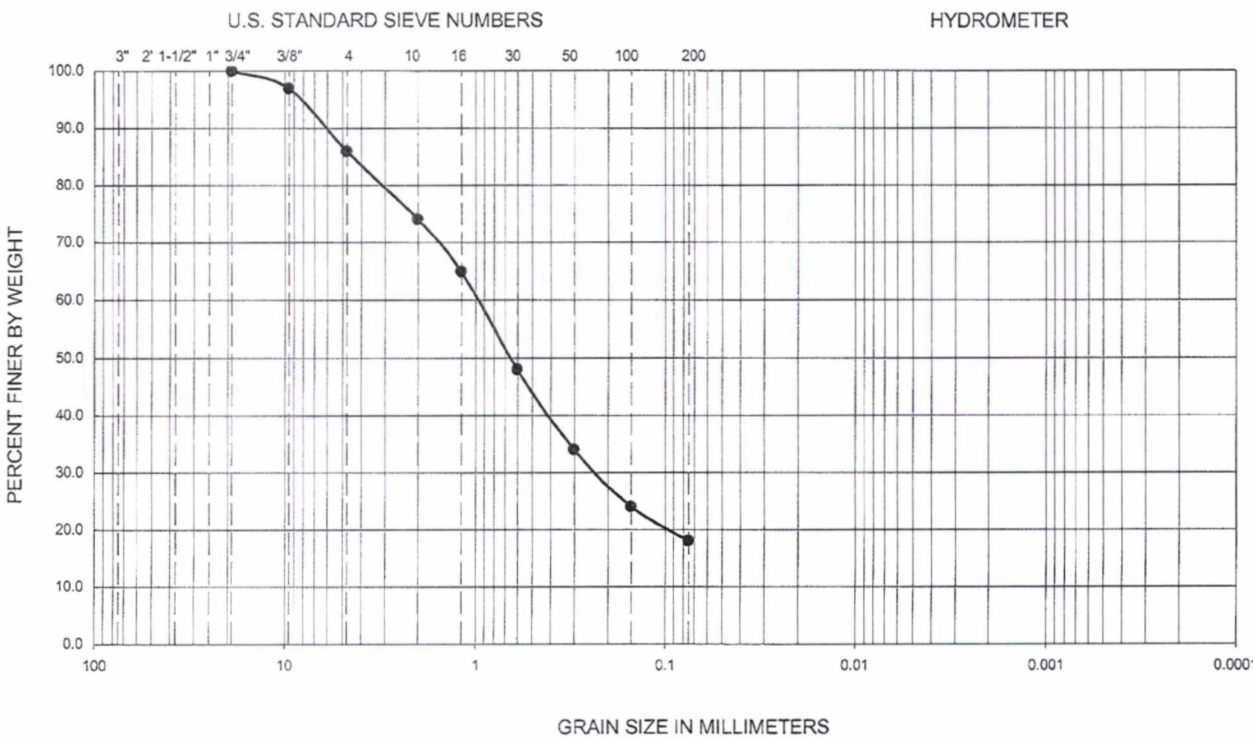


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	6-7.5	33	18	15	--	--	--	--	--	54	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS		FIGURE
PROJECT NO.	DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	B-1
604037001	3/13		

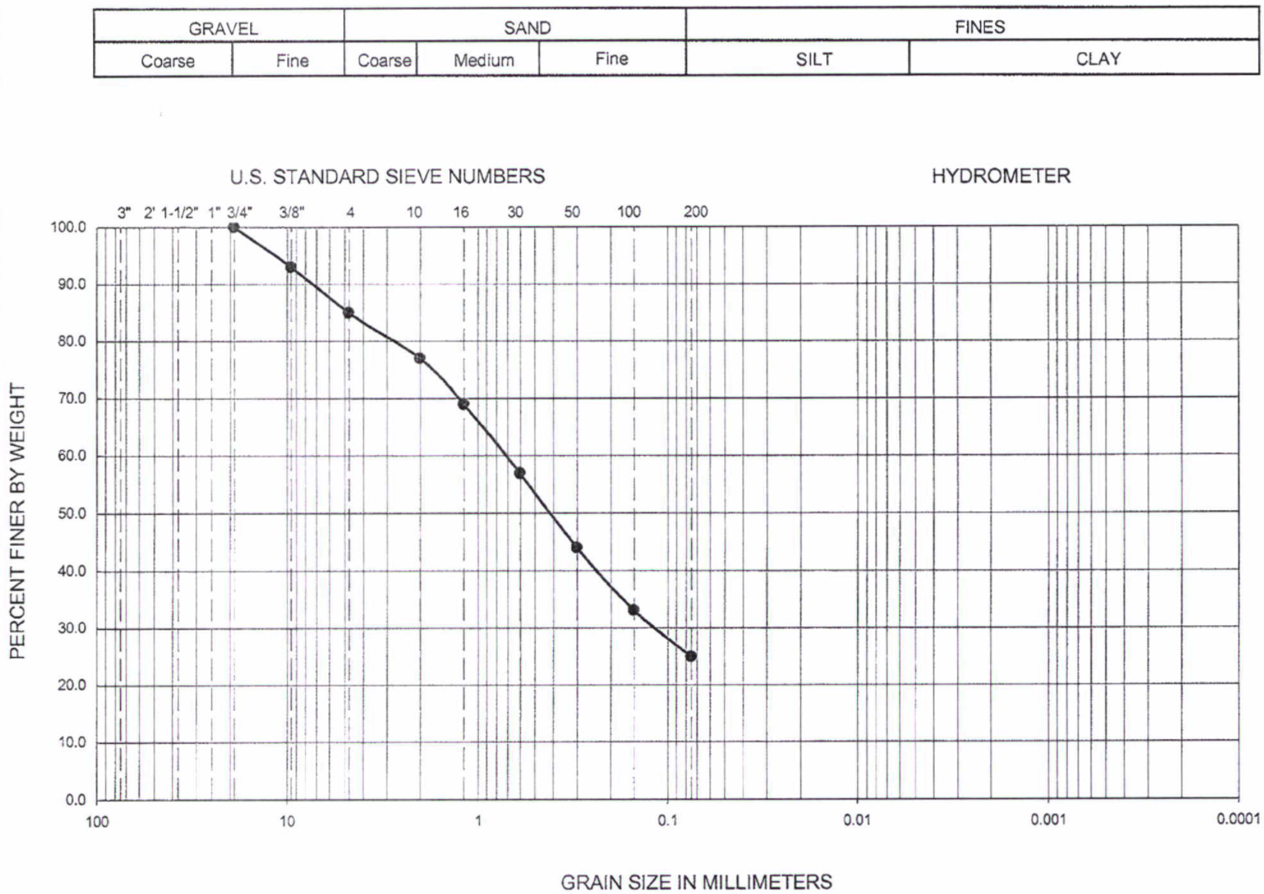
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	3.5-5	24	17	7	--	--	--	--	--	18	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	**GRADATION TEST RESULTS**	FIGURE
PROJECT NO. / DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	**B-2**
604037001 / 3/13		

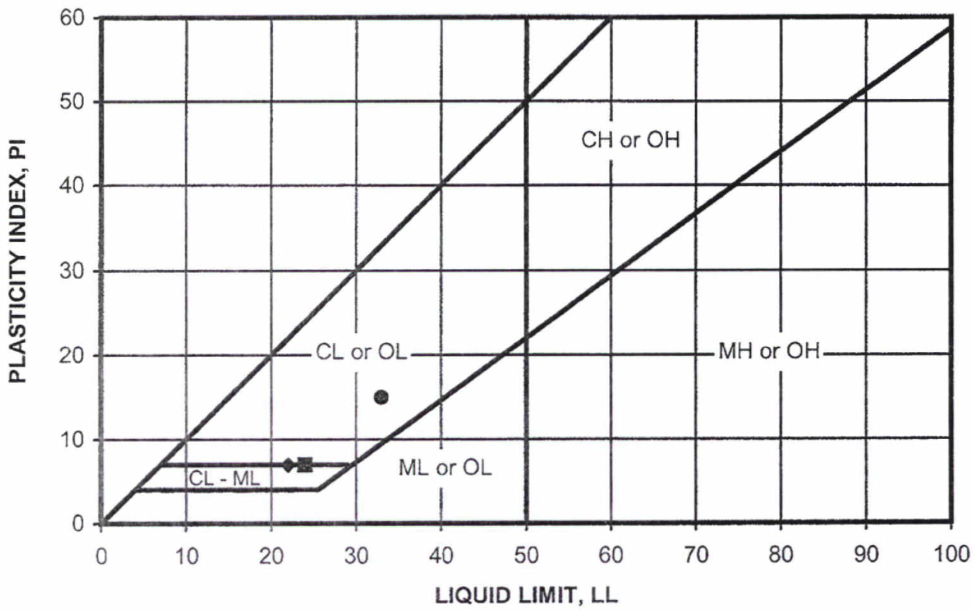


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-3	1-2.5	22	15	7	--	--	--	--	--	25	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

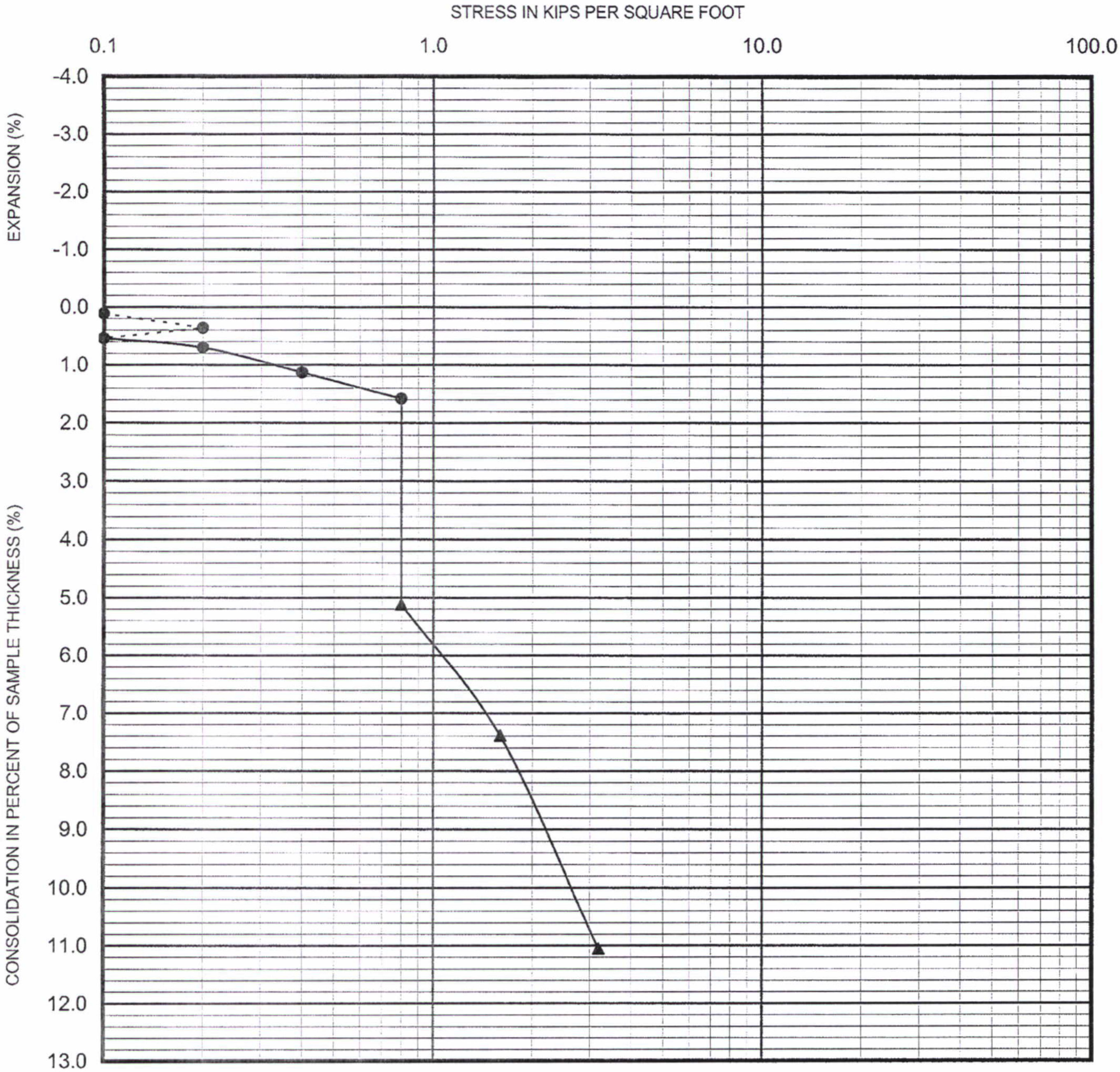
Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	B-3
604037001 / 3/13		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	6-7.5	33	18	15	CL	CL
■	B-2	3.5-5	24	17	7	CL-ML	SC-SM
◆	B-3	1-2.5	22	15	7	CL-ML	SC-SM



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	B-4
604037001	3/13		

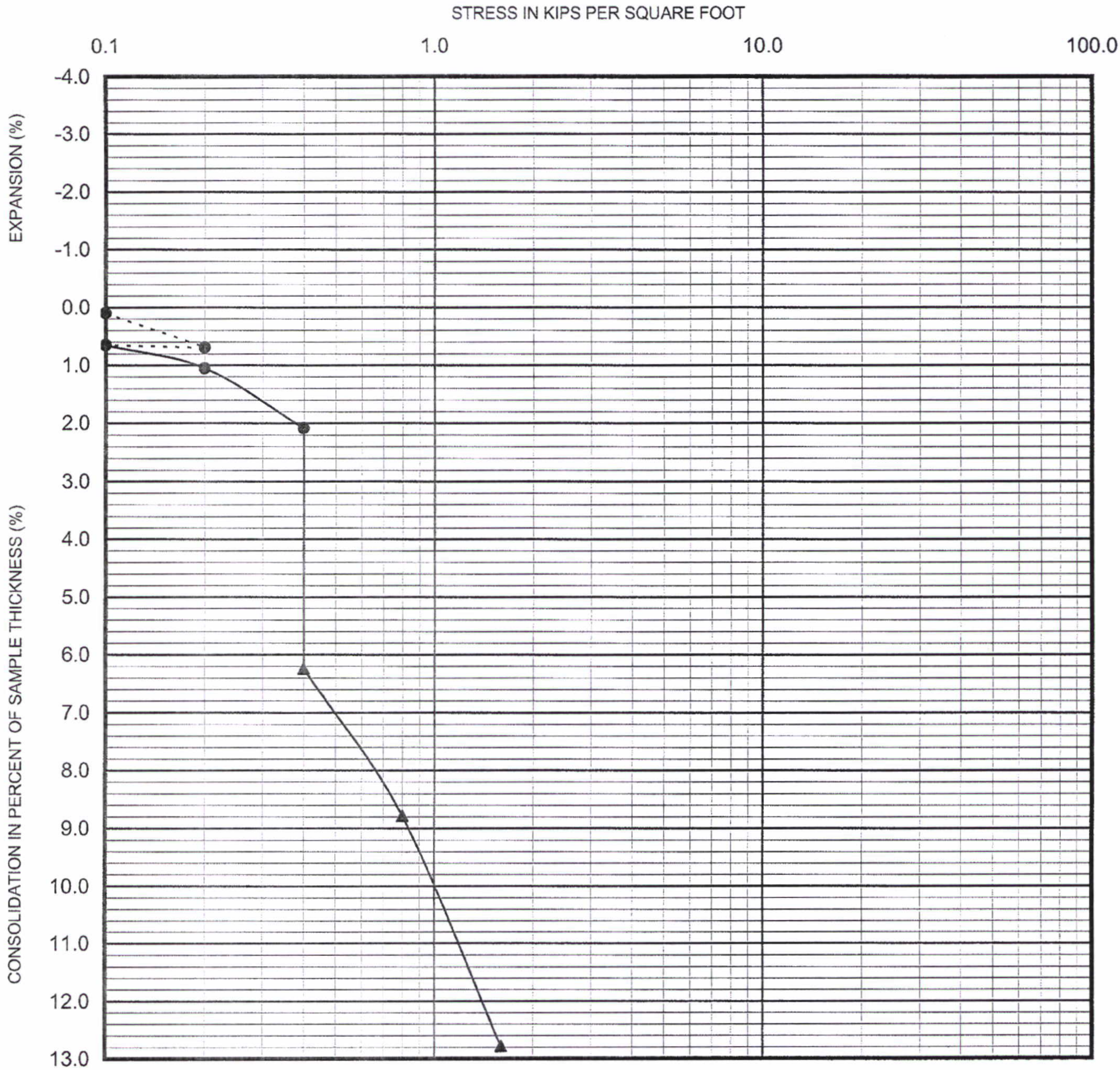


Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-1
Depth (ft.)	6-7.5
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	B-5
604037001	3/13		



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-2
Depth (ft.)	3.5-5
Soil Type	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	HOPI DRIVE DRAINAGE IMPROVEMENTS CASA GRANDE, ARIZONA	B-6
604037001	3/13		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-1	0-5	8.0	1,040	33	0.003	43
B-3	0-5	8.2	1,505	31	0.003	77

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

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
604037001	3/13

HOPI DRIVE DRAINAGE IMPROVEMENTS
CASA GRANDE, ARIZONA

FIGURE
B-7

APPENDIX C

SUMMARY OF INFILTRATION TEST RESULTS

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Hopi Drive Drainage Improvements PROJECT NO.: 604037001

TECHNICIAN: DM LOCATION: P-1

PRE-SOAK

DATE(S)	START TIME (Hr:Min)	END TIME (Hr:Min)
2/12/13	6:45	14:40

PERCOLATION TEST

7/29/10

START TIME (Hr:Min)	END TIME (Hr:Min)	CHANGE IN WATER LEVEL (INCHES)	PERCOLATION RATE (INCHES/HOUR)
14:40	14:50	1.1	6.60
14:50	15:00	1	6.00
15:00	15:10	1	6.00

FACTORED PERCOLATION RATE = (0.333 * 0.50 * 6.00) = 1.00 INCHES/HOUR

Sidewall correction factor = 0.333

Derating factor = 0.50

Performed in general accordance with the EPA Falling Head Percolation test procedure outlined in the Drainage Policies and Standards for Maricopa County (2nd Rev. September 2009)