Ninyo & Moore
Geotechnical and Environmental Sciences Consultants

GEOTECHNICAL EVALUATION
83RD AVENUE IMPROVEMENTS
HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PREPARED FOR:
City of Peoria
9875 North 85th Avenue
Peoria, Arizona 85345

PREPARED BY:
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September 8, 2010
Project No. 602727001

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Mr. Richard Costa
City of Peoria
Engineering Department
9875 North 85th Avenue
Peoria, Arizona 85345

Subject: Geotechnical Evaluation
83rd Avenue Improvements
Hatcher Road to Butler Drive
Peoria, Arizona

Dear Mr. Costa:

In accordance with our revised proposal dated May 13, 2010, and your authorization, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced project. The attached report presents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

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

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with our revised proposal dated May 13, 2010, and your authorization, we have performed a geotechnical evaluation for the proposed 83rd Avenue Improvements Project in Peoria, Arizona. The project consists of the widening of 83rd Avenue from Hatcher Road to Butler Drive, and paving Butler Drive from 83rd Avenue to 79th Avenue (Figure 1). Additional improvements include the installation of underground utilities along the alignments.

The purpose of our evaluation was to assess the subsurface conditions along the alignments in order to formulate geotechnical recommendations for the design and construction. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed improvements.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Obtaining right-of-way permits through the City of Peoria and Maricopa County Department of Transportation (MCDOT) to perform the field work within the respected right-of-ways.
- Reviewing available geologic literature, geologic maps, and aerial photographs pertinent to the project site.
- Conducting a visual geologic reconnaissance of the project area.
- Conducting a site visit to mark out the boring locations and notifying Arizona Blue Stake of the locations prior to drilling.
- Drilling, logging, and sampling six small-diameter exploratory borings to depths ranging from approximately 5 to 15 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Performing laboratory tests of selected samples obtained from the borings to evaluate in-situ moisture content and dry density, gradation analysis, Atterberg limits, consolidation (response-to-wetting) and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the boring logs and/or in Appendix B.
- Preparing this report presenting our findings, conclusions, and recommendations regarding the design and construction of the proposed improvements.

3. PROJECT DESCRIPTION

The project consists of widening of 83rd Avenue from Hatcher Road to Butler Drive and paving Butler Drive from 83rd Avenue to 79th Avenue, in Peoria, Arizona. We understand that the improvements along 83rd Avenue will include widening the roadway in each direction, and additions, including a raised, landscaped median, bicycle lanes, street lighting, landscaping, and a new regional storm drain, overhead 12 kilovolt power lines, and new communication lines will be placed under ground.

The improvements along Butler Drive will include constructing a new lane in each direction, bicycle lanes, sidewalks, and street lighting. In addition, a new storm drain will be installed along the 83rd Avenue alignment. We understand that the existing Butler Drive alignment will be shifted to the north to match the current alignment west of 83rd Avenue.

New storm drains will be constructed as part of the improvements, as discussed above. The new storm drains, and other utilities along the alignment, will be installed using conventional cut-and-cover techniques. The invert elevation of the utilities will be on the order of 12 feet or less.

At the time of our evaluation, grading and drainage plans were not available; however, we assume that positive drainage will be maintained along the alignment and that drainage will be directed off and away from the new pavement.

4. SITE DESCRIPTION

The project alignment along 83rd Avenue between Hatcher Road and Butler drive is situated along the section boundary of Sections 26 and 27, and 34 and 35. The project alignment along Butler Drive is situated in the western portion of Section 35. Both of the alignments are in Township 2 North, Range 1 East.

At the time of our evaluation, 83rd Avenue was a north-south traversing, paved roadway. From Hatcher Road to approximately 1,000 feet south of Olive Avenue, 83rd Avenue was a two-lane roadway (one lane in each direction) with a right turn lane for access to Olive Avenue. From approximately 1,000 feet south of Olive Avenue to approximately 1,000 feet south of Butler

Drive, 83rd Avenue was a three-lane roadway (two lanes southbound, one lane northbound) with a middle turning lane. Residential structures and overhead power lines were situated along the west side of the roadway, and agricultural land was situated to the east. A gas station was observed at the northwest corner of 83rd Avenue and Olive Avenue. It is our understanding that there is a former concrete surface beneath the roadway; however, the location of the concrete is not known.

Butler Drive was an unpaved, graded, east-west traversing roadway at the time of our evaluation. Agricultural land was situated to the north side of the roadway, and industrial businesses and agricultural land were situated along the south side. Overhead power lines were observed traversing parallel and adjacent to the south side of the roadway

Based on the *Glendale, Arizona 7.5-Minute United States Geological Survey (USGS) Topographic Quadrangle Map (1982)*, the elevation of the project site generally ranges from approximately 1,130 feet relative to mean sea level (MSL) at the northern limits of the project site to approximately 1,110 feet MSL at the southern limits. Based on the information presented on this topographic map, the regional topography generally slopes from the northeast down to the southwest.

Several aerial photographs from the Flood Control District of Maricopa County were reviewed for this project. A 1949 aerial photograph depicted 83rd Avenue as being surrounded by agricultural land on the east and west sides of the roadway. Two residential structures were situated at the northwest and southwest corners of the intersection of 83rd Avenue and Olive Avenue. Approximately 1,000 feet south of Olive Avenue on the east side of 83rd Avenue, a residential structure was depicted. A lined ditch was depicted traversing parallel to 83rd Avenue along the west side of the roadway. Butler drive was surrounded by agricultural land to the north of the roadway, and undeveloped desert land to the south. A 1959 aerial photograph depicted the site as being similar to the 1949 photograph; however, a small structure was observed at the southeast corner of 83rd Avenue and Butler Drive. In photographs from 1964 and 1969, the previously mentioned lined ditch adjacent to the west side of 83rd Avenue was not depicted, and the area became agricultural land. A 1979 photograph depicted residential development at the

southwest corner of 83rd Avenue and Butler Drive. Photographs from 1993 and 1996 depicted residential development along the west side of 83rd Avenue. Industrial development was depicted along the south side of Butler Drive. The previously mentioned residential structure at the northwest corner of 83rd Avenue and Olive Avenue was not depicted in this photograph. The parcel of land the residential structure was situated at was depicted as agricultural land. A 1999 photograph depicted the site as being similar to its current condition with a gas station situated at the parcel of land previously occupied by a residential structure and agricultural land at the northwest corner of 83rd Avenue and Olive Avenue.

5. FIELD EXPLORATION AND LABORATORY TESTING

On May 24, 2010, Ninyo & Moore conducted a subsurface exploration at the project site in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. The field exploration consisted of drilling, logging, and sampling six small-diameter exploratory borings, denoted as B-1 through B-6, extended to depths ranging from approximately 5 to 15 feet bgs. Borings B-1 through B-4 were situated in the roadway of 83rd Avenue. Borings B-5 and B-6 were situated along the north side of Butler Drive. The borings were drilled using a Diedrich D-50 truck-mounted drill rig equipped with hollow-stem augers. Bulk samples and relatively undisturbed soil samples were collected at selected depth intervals from the exploratory borings. The approximate locations of the borings are presented on Figure 2. Detailed descriptions of the soils encountered in the borings are presented on the boring logs in Appendix A.

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 analysis, 200 wash, Atterberg limits, consolidation (response-to-wetting) and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides).The results of the in-situ moisture content and dry density testing 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.

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

The basins and surrounding mountains were formed approximately 10 to 18 million years ago during the Mid- to Late-Tertiary. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains as well as from 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 Late to Middle Pleistocene (10,000 to 790,000 years) alluvium. The alluvium is described as silt, sand, and gravel deposits with scattered cobbles. The alluvium is also described as having strong argillic (clay) horizons, and calcic cementation ranging from Stage II to Stage IV [partial grain coating to complete coating (Demsey, 1988)].

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 generalized descriptions of the materials encountered in the borings. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1. Asphalt Concrete and Aggregate Base

Asphalt concrete (AC) was encountered at the surface of borings B-1 through B-4. The AC thickness measured in these borings generally ranged from approximately 4 to 7.5 inches. Aggregate base (AB) was not encountered underlying the AC at borings B-1 and B-2; however, AB was encountered in borings B-3 and B-4. The AB thickness encountered in our borings was approximately 10 inches.

6.2.2. Fill

Fill material was encountered in our borings underlying the pavement section described above in borings B-1 and B-2, and at the surface of B-5 and B-6. The fill was approximately 1 foot thick, and generally consisted of consisted of clayey sand in our borings.

6.2.3. Alluvium

Alluvium was encountered underlying the pavement section and fill in borings B-1 and B-2, and under the fill in borings B-3 through B-5. The alluvium was generally heterogeneous, stratified, and generally consisted of sandy clay and clayey sand in our borings. Caliche filaments and nodules were observed in the alluvium, as well as varying quantities of gravel 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), the regional table groundwater has historically been measured to be approximately 200 feet below the ground surface in nearby wells. In general, the historic measurements of the groundwater table generally show that regional groundwater levels that are higher in the south, and deeper to the north. Groundwater levels can fluctuate due to seasonal variations in precipitation, irrigation, groundwater withdrawal or injection, and various other factors. In general groundwater is not expected to be a constraint to the design and construction of this project.

7. GEOLOGIC HAZARDS

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

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 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 no known earth-fissures underlying or adjacent to the site. The closest documented earth fissure to the site is approximately 6 miles to the east. In general, earth fissures are not expected to be a constraint to the 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 Quaternary fault to the project site is the Carefree Fault Zone, situated approximately 27 miles to the northeast of the site. The Carefree Fault Zone is a series of northwest striking normal faults that dip to the southwest. Recent movement along this fault was approximately 750,000 years ago during the Middle Pleistocene epoch. The slip-rate category of this fault is less than 0.2 millimeters per year (Pearthree, 1998).

7.3. Liquefaction

Based on the lack of near surface water, and the low ground motion hazard (relatively low ground accelerations), the potential for liquefaction at the project site is not a design consideration for this project.

8. CONCLUSIONS

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

- The on-site soils should generally be excavatable to expected depths, with heavy-duty earthmoving construction equipment in good working condition. However, scattered caliche filaments and nodules were encountered in the borings, which could be more difficult to excavate depending on the degree of cementation encountered during construction.
- Because of the close proximity of the adjacent roadway and other structures, shoring the trench sidewalls may be needed.
- Some of the on-site soils are not recommended for re-use as engineered fill.
- Imported soils and soils generated from onsite excavation activities that exhibit relatively low plasticity indices and very low to low expansion potential can generally be used as engineered fill soil.

- Corrosivity test results indicate that the subgrade materials may be corrosive to ferrous metals and the sulfate content of the soils present a negligible sulfate exposure for concrete.
- Groundwater levels based on historic nearby well data indicate that the regional groundwater table has been encountered at approximately 200 feet bgs.
- No known or documented geologic hazards are present underlying or immediately adjacent to the site.

9. RECOMMENDATIONS

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

9.1. Site Preparation

Construction areas for new utility lines and pavement should be cleared of deleterious materials, including grass, weeds, construction debris, and any other material that might interfere with the performance or progress of the work. These materials, if encountered along the alignment of the proposed improvements, should be disposed at a legal dumpsite.

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

9.2. Excavation Characteristics

Based on our borings, site observations, and knowledge of the regional geology around the site, it is our opinion that excavations can be excavated to targeted depths using heavy-duty excavation equipment in good operating condition. The on-site materials encountered generally consist of sandy clay, and clayey sand in our borings. Scattered caliche filaments

and nodules were encountered in our borings, and may be more difficult to excavate depending on the actual degree of cementation encountered during construction.

For the new utility and water lines, excavation equipment and procedures should be used that do not cause significant disturbance to the trench bottoms. Excavators and backhoes with buckets having large claws to loosen the soil should be avoided when excavating the last 6 to 12 inches of the trench. Such equipment could disturb the trench bottom subgrade. Wet or saturated soils found along the proposed utility line alignments may soften under the action of light equipment and foot traffic. If the subgrade becomes disturbed, it should be compacted before placing the pipe bedding material.

9.3. Utility and Water Lines

The following sections provide our recommendations for the design and construction of new utility and water lines. If the proposed construction is changed from that discussed in this report, Ninyo & Moore should be contacted for additional recommendations

9.3.1. Temporary Slope Stability

The sides of the excavation and trenches for this project should be stabilized in order to minimize damage to adjacent structures resulting from vertical or lateral movement of the soil. The sides of the excavation may be stabilized by sloping back the sides and/or by using bracing. The granular, low cohesion, and sometimes low-density soils encountered in our borings may be prone to sloughing. Additionally, vibrations caused by nearby traffic or construction equipment could 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) requirements for employees working in excavations that may expose them to the danger of moving ground. The OSHA requirements 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. For planning purposes and according to OSHA soil classifications, a "Type C" soil should be considered for this project.

In general, excavations in “Type C” soils with no groundwater and shallower than 20 feet should be constructed at a slope ratio of 1.5:1 (horizontal: vertical) or flatter. Upon making the excavations, soil and/or rock classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA specifications.

If construction or earthwork 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 surface seepage may need shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage, if any, should be evaluated on a case-by-case basis. During wet weather, earthen berms or other methods should be used to divert runoff water from entering excavations. Runoff water should be collected and disposed of outside the construction limits.

9.3.2. Shoring

Given the presence of existing structures (e.g., roadways, power lines, etc.), and existing nearby utilities along the project alignment, plus the need to maintain traffic flow, it may be preferable to shore or brace the trench excavations than to utilize open cuts in these areas. Temporary earth retaining systems will be subjected to lateral loads resulting from earth pressures. Shored or braced trench excavations in soils may be designed using the parameters presented on Figure 3. Based on the soil profile encountered in the boring locations, trench boxes may also be a suitable alternative to laying back the side walls. However, some sloughing is possible at the ends of the trench box; therefore, loose material as a result of this should be removed prior to backfilling of the trench.

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

9.3.3. Bottom Stability

Based on our borings and well data from nearby wells, the excavations for new utility and water lines are not anticipated to encounter groundwater, with the exception of possible surface run-off, or soft materials at their base. Therefore, trench bottom stability problems during construction are not anticipated; however, if excavations are located within a known wash, arroyo, or drainage area, (e.g., nearby irrigation ditches or canals), and are open during a heavy rain event, the trench bottom may become saturated and unstable. This scenario should be evaluated on a case-by-case basis.

9.3.4. Construction Dewatering

No significant seepage from the groundwater table is anticipated on site during construction. Stream flow and surface run-off will vary seasonally depending on rainfall periods. Given the relatively low probability of encountering significant seepage on the site, we anticipate that the excavations that do encounter seepage or surface run-off could be dewatered by pumping it from the bottom of the trench. 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, if needed, may entail securing a special permit.

9.3.5. Pipe Bedding and Pipe Zone Backfill

We recommend that the new utility and water lines be supported on 4 or more inches (or 1/12 of the outside diameter, whichever is more) of graded granular bedding material such as sand and gravel, or crushed rock with a particle size of 3/4-inch or less and 3 to 15 percent passing the No. 200 sieve (pea gravel or crushed chips are not acceptable). This bedding/pipe-zone backfill should extend 1 foot above the pipe crown (Figure 4). Care should be taken not to allow voids to form beneath the pipe, (i.e., the pipe haunches should be continuously supported), to avoid damaging the pipelines. This may involve fill placement by hand or small compaction equipment. The bedding/pipe zone should be placed in horizontal lifts approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to a relative compaction of 95 percent, (as evaluated by the American Society for Testing and Materials [ASTM] D 698), and at a moisture content generally above the laboratory optimum. Further, bedding material and compaction requirements should be in accordance with the Maricopa Association of Governments (MAG, 2010) or Standard Specifications for Public Works Construction (American Public Works Association, 1991). The use of water consolidation (i.e. jetting) to gain compaction is not recommended.

When backfilling, care should be taken to fill voids with compacted material so that excessive settlement of the backfill will not occur. Settlement can be mitigated by backfilling with granular material that is easy to compact or by using a Controlled Low Strength Material (CLSM), sometimes referred to as Controlled Density Fill. More detailed recommendations regarding the use of CLSM are provided in Section 9.3.7.

9.3.6. Trench Backfill

Trench backfill as discussed herein refers to the material placed above the pipe zone/bedding backfill material. Trench backfill material should be mechanically compacted to a relative compaction of 95 percent as evaluated by ASTM D 698 at a moisture-conditioned generally above the laboratory optimum moisture content. Lift thickness for backfill will be dependent upon the type of compaction equipment

utilized, but should generally be placed in 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.

The trench backfill in the upper 2-foot zone that is situated below pavement/flatwork sections should also be mechanically compacted to a relative compaction of 100 percent for granular material, or 95 percent for clayey material, as evaluated by ASTM D 698 at a moisture-conditioned generally above the laboratory optimum.

Backfilling should generally be accomplished in a manner consistent with the standards provided by MAG (2010). Some of the soils encountered during our excavations, as well as processed materials generated during construction, might be suitable for re-use as trench backfill provided they are free of organic material, clay lumps, debris, and rocks larger than 4 inches in diameter. Some screening of larger particles may be needed. The content of gravel in the backfill larger than 1-1/2 inches in diameter should not exceed 40 percent of the backfill volume. Generated excavation materials that contain this oversize fraction shall not be used as backfill unless the material meets the criteria given above and/or the oversize fraction has been processed and removed from the material. Imported backfill material, if utilized, should meet the criteria for imported fill as presented in Section 9.4 of this report.

9.3.7. Controlled Low Strength Material

If desired, controlled low strength material (CLSM), also known as Controlled Density Fill or Slurry Cement Backfill, may be used as an alternate pipe or trench zone bedding or backfill material. 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 higher values of modulus of soil reaction (E'=3000 pounds per square inch [psi]) can be used to design the pipe;

3. There is less risk of damaging the pipe, since little compaction is needed to place CLSM;
4. If native soils are used to formulate the CLSM, less imported material will be needed;
5. CLSM can be batched to flow into irregularities in the trench bottom and walls; and
6. The number of workers needed inside the trench excavation is reduced.

The CLSM design mix should be in accordance with the MAG (2010) or Standard Specifications for Public Works Construction (American Public Works Association, 1991). Additional mix design information can be provided upon request. The 28-day strength of the material should be no less than 50 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.

Buoyant or uplift forces on the piping should be considered if a CLSM is used. Prudent construction techniques may need multiple pours to avoid inducing excessive uplift forces. The construction methods should not allow for the pipeline 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.

9.3.8. Soil Parameters for Pipeline Design

The modulus of soil reaction (E´) is used to characterize the stiffness of soil backfill placed at the sides of buried pipelines for the purpose of evaluating deflection caused by the weight of the backfill over the pipe. We understand that the depth of cover along the pipeline will generally be up to 15 feet. For granular backfill soils for the proposed pipeline, we recommend using an E´ value of 1,500 psi.

The coefficient of friction between the soil and the pipe depends upon the type of each material in the interaction. Assuming that a few different pipe materials may be considered, we suggest a coefficient of friction of 0.3 to 0.4 depending on the pipe's smoothness (these values do not have any factor of safety associated with them). The

manufacturer of the pipe should be consulted for this parameter once the pipe material has been chosen.

9.4. Fill Materials

Imported soils and soils generated from on-site excavation activities, (excluding cobbles and large diameter particles, if encountered), that exhibit relatively low plasticity indices and very low expansion potential are generally suitable for use as engineered fill. Relatively low plasticity indices are defined as having a Plasticity Index [(PI) by ASTM 4318] value of 20 or less. Very low to low expansion potential soils are defined as having an Expansion Index [(EI) evaluated in accordance with ASTM D 4829] of 50 or less. Atterberg limits tests performed on soil samples obtained from various borings indicated PI values ranging from 13 to 21, demonstrating low to moderate plasticity. Based on these results, many of the on-site soils are suitable for re-use as engineered fill, under roadways or in the upper 3 feet of trench backfill. Recommendations for EI and PI may be waived for trench backfill deeper than 3 feet below finished grade. Further testing should be conducted prior to or during construction to help delineate areas of unacceptable soil. An earthwork (shrinkage) factor of 10 to 20 percent is estimated for the on-site soils.

In addition, suitable fill should not include deleterious or organic material (more than 4 percent); clay lumps, construction debris, rock particles, and other non-soil fill materials larger than 3 inches in dimension. This material should be disposed of off-site or in non-structural areas.

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

9.5. 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 1 presents the seismic design parameters for the site in accordance with International Building Code (IBC, 2006) guidelines and mapped spectral acceleration parameters (USGS, 2008).

Table 1 – 2006 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	0.179g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.061 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.286 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.146 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.191 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.097 g

9.6. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on any buried pipelines. Corrosion potential was evaluated using the results of our laboratory testing on soil samples obtained from various borings that was considered representative of the subsurface soils at the project 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 Tests 733 and 736 respectively. The results of the corrosivity tests are presented in Appendix B.

The soil pH values of selected samples from various borings along the project alignment generally ranged from 8.6 to 8.8, which is considered to be alkaline. The minimum electrical resistivity values from the samples collected ranged from 1,710 to 1,850 ohm-cm, which represents a corrosive environment to ferrous materials. The chloride content of the samples tested ranged from 55 to 71 ppm, which is also considered corrosive to ferrous materials. The soluble sulfate content of the samples were both approximately 0.009 percent by weight, which is considered to be a negligible amount for sulfate attack on concrete.

The results of the chloride content test on the samples indicated that the on-site materials could be corrosive to ferrous metals. Therefore, we recommend that special consideration be given to the use of heavy gauge, corrosion protected, underground steel pipe. As an alternative, plastic pipe could be considered. A corrosion specialist should be consulted for further recommendations.

9.7. Concrete

Laboratory chemical tests performed on select soil samples from borings indicated a sulfate content of 0.009 percent by weight. Based on the following tabulated American Concrete Institute (ACI) guidelines, the on-site soils should be considered to have a negligible sulfate exposure for concrete.

Table 2 – ACI Requirements for Concrete Exposed to Sulfate-Containing Soil

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

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

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

The concrete should have a water-cementitious materials ratio no more than 0.50 by weight for normal weight aggregate concrete. The structural engineer should 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.

9.8. Pavements

The following sections present our design assumptions and recommendations for new flexible pavement sections along the project limits. In providing these recommendations, we assumed that AC pavement will be used and that the subgrade preparation recommendations outlined in this report will be employed. Based on conversation with the City of Peoria, we

understand that Butler Drive will be designed to the minimum pavement section for a street designated as "collector".

9.8.1. Existing Pavement

Based on our field observations, pavement distresses of low to medium severity alligator cracking, longitudinal and transverse cracking, edge cracking, and rutting were noted at various locations along 83rd Avenue. It is our opinion that the various distress features noted on the existing pavement may be the result of repeated traffic loadings, age of the pavement, and/or environmental factors. It is our opinion that the distresses noticed indicate both structural and functional failure of the pavement.

9.8.2. New Pavement Subgrade Preparation

Due to the presence of near surface loose soils, we recommend that the new pavement sections associated with this project be founded on a zone of adequately moisture-conditioned and compacted engineered fill that extends 6 or more inches below the bottom of the AB layer. This new fill should be placed in horizontal lifts not more than approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to 95 percent relative compaction, in accordance with ASTM D 698, at a moisture content generally above optimum. This overexcavation zone should extend 1 or more feet horizontally beyond the edge of the pavement. Any imported fill for the new roadway should have an R-Value of more than 20 for this project.

Following the overexcavation as described above, and prior to the placement of new fill, the resulting exposed surface should be proof-rolled and carefully evaluated by the geotechnical consultant. 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 the geotechnical consultant during the earthwork operations.

9.8.3. Traffic

The traffic loading information used to conduct the pavement design for 83rd Avenue within the project limits was based on traffic volumes obtained from the City of Peoria's website. Based on this information, the flexible pavement was designed for an average weekday traffic count of 7,884 vehicles per day. The resulting 20-year Equivalent Single Axle Loads (ESALs) was estimated to be approximately 1,901,000. Based on traffic counts from previous years, we assumed a growth factor of 3 percent per year and 5 percent heavy trucks for the design of new flexible pavement.

9.8.4. R-value

The soils encountered in our borings typically consisted of clay, clayey sand and silty graded. Correlated R-values ranging from 19 to 29 were obtained using Table 202.02-3 in the Arizona Department of Transportation (ADOT) Materials Manual (ADOTM). For this segment of 83rd Avenue, a design R-Value of 19 was used.

For purposes of new construction, it is assumed that soils placed within 3 feet of the finished roadway subgrade will exhibit an R-value that corresponding to the design R-values mentioned above. If the project needs fill from an offsite source, we recommend the soils used for subgrade support should also have an R-value that correlates to the number above.

9.8.5. Resilient Modulus

As discussed in the preceding section, a design R-value of 19 was used for the new pavement section of 83rd Avenue. Based on Figure 202.02-2 obtained from ADOTM, correlating the R-value and a seasonal variation factor of 1.0 taken from Table 202.02-4 in the ADOTM, resilient modulus values of 12,000 psi was estimated.

9.8.6. Drainage Coefficient

A drainage coefficient of 1.0 was established from Table 202.02-7 in the ADOTM, based on a seasonal variation factor of 1.0 and good drainage conditions.

9.8.7. Recommended Asphalt Pavement Sections

Based on the traffic count and the resilient modulus of the subgrade soils, the calculated asphalt pavement section is presented in Table 3 below. The AASHTO method was used to evaluate bituminous layer thicknesses given below. Based on the AASHTO method, a pavement section consisting of 5 inches of AC over 8 inches of AB would be acceptable for this project; however, the City of Peoria minimum pavement section consists of 5 inches of AC over 12 inches of AB.

Table 3 – Recommended Pavement Sections

Roadway	Layer	Thickness (inches)
83rd Avenue* (Major Arterial)	Asphalt Concrete	5.0
	Aggregate Base	12.0
Butler Drive** (Collector)	Asphalt Concrete	4.0
	Aggregate Base Course	8.0

* Minimum Per City of Peoria Standards for Major Arterial Streets
**Minimum per City of Peoria Standards for Collector Streets

The AC mentioned above should meet MAG Standard Specifications. The AB mentioned above should meet Section 702 of the MAG specifications requirements, as shown in Table 4.

Table 4 – Recommended Aggregate Base Gradation

Sieve Size (per ASTM D422-63)	Percent Passing by Weight
1 1/8 inch	100
No. 4	38-65
No. 8	25-60
No. 30	10-40
No. 200	3-12
P.I. Max.	5

AB material should be compacted to a relative compaction of 100 percent of the maximum dry density, as evaluated by ASTM D 698, at a moisture content generally near the optimum.

9.9. Site Drainage

Surface drainage should be provided to divert water away from the paved surfaces. Surface water should not be permitted to pond on pavement areas. Positive drainage for this project is defined as a slope of 2 percent or more for a distance of 5 feet or more away from the pavements. 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 of the roadway.

9.10. 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.

9.11. Construction Observation and Testing

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

techniques and construction materials should perform construction of the proposed improvements.

10. LIMITATIONS

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

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

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

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are

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

This report is intended exclusively for use by the client. Any use or 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. REFERENCES

American Concrete Institute, 1991a, Guidelines for Concrete Floor and Slab Construction (ACI 302.1R).

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United States Geological Survey, 1982, Glendale- Arizona, Maricopa County 7.5 minute topographic map. Scale: 1" = 2000'.

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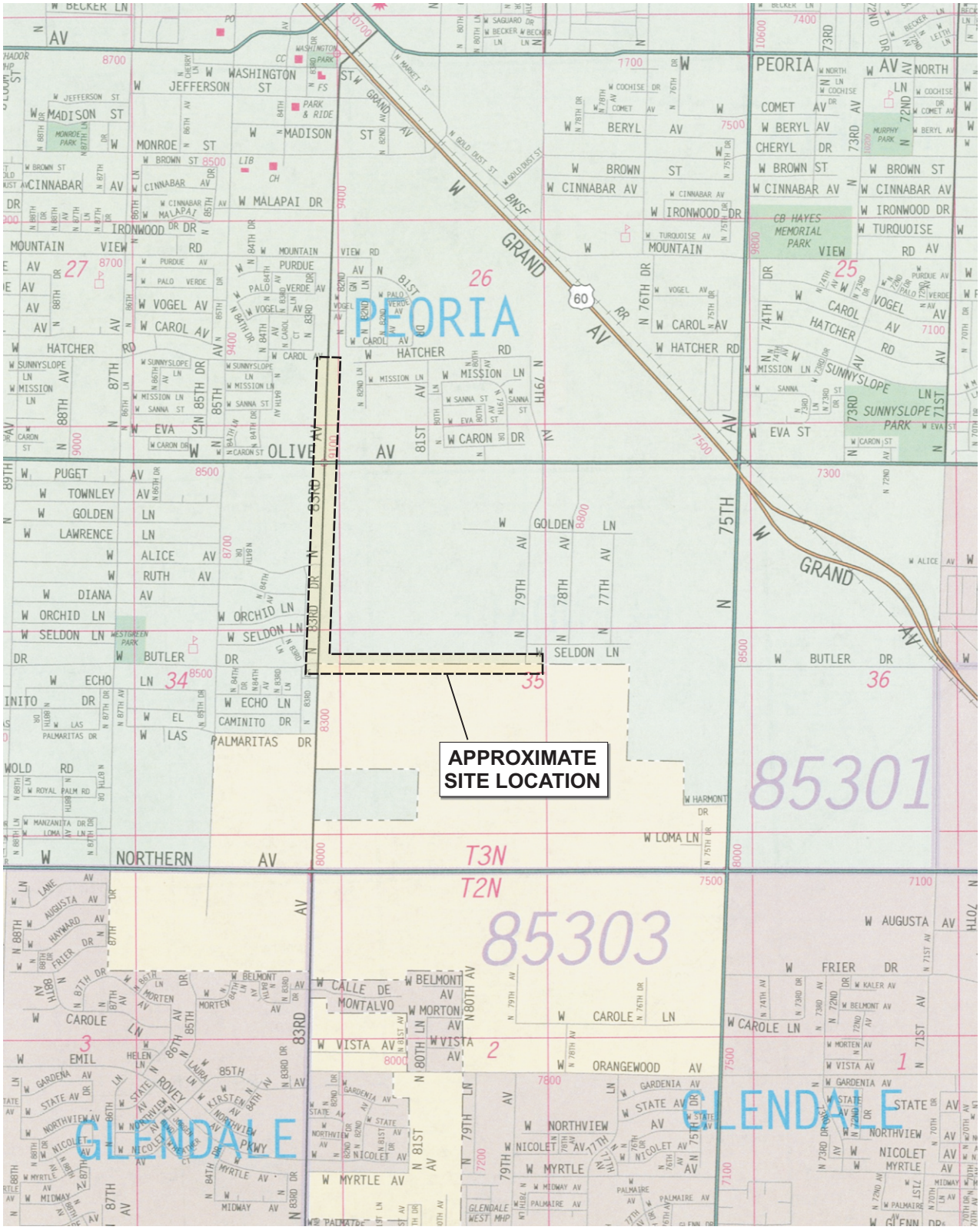
Referenced Aerial Photographs

Source	Photo Dates
Flood Control District of Maricopa County	1949, 1959, 1964, 1969, 1979, 1993, 1996, 1999



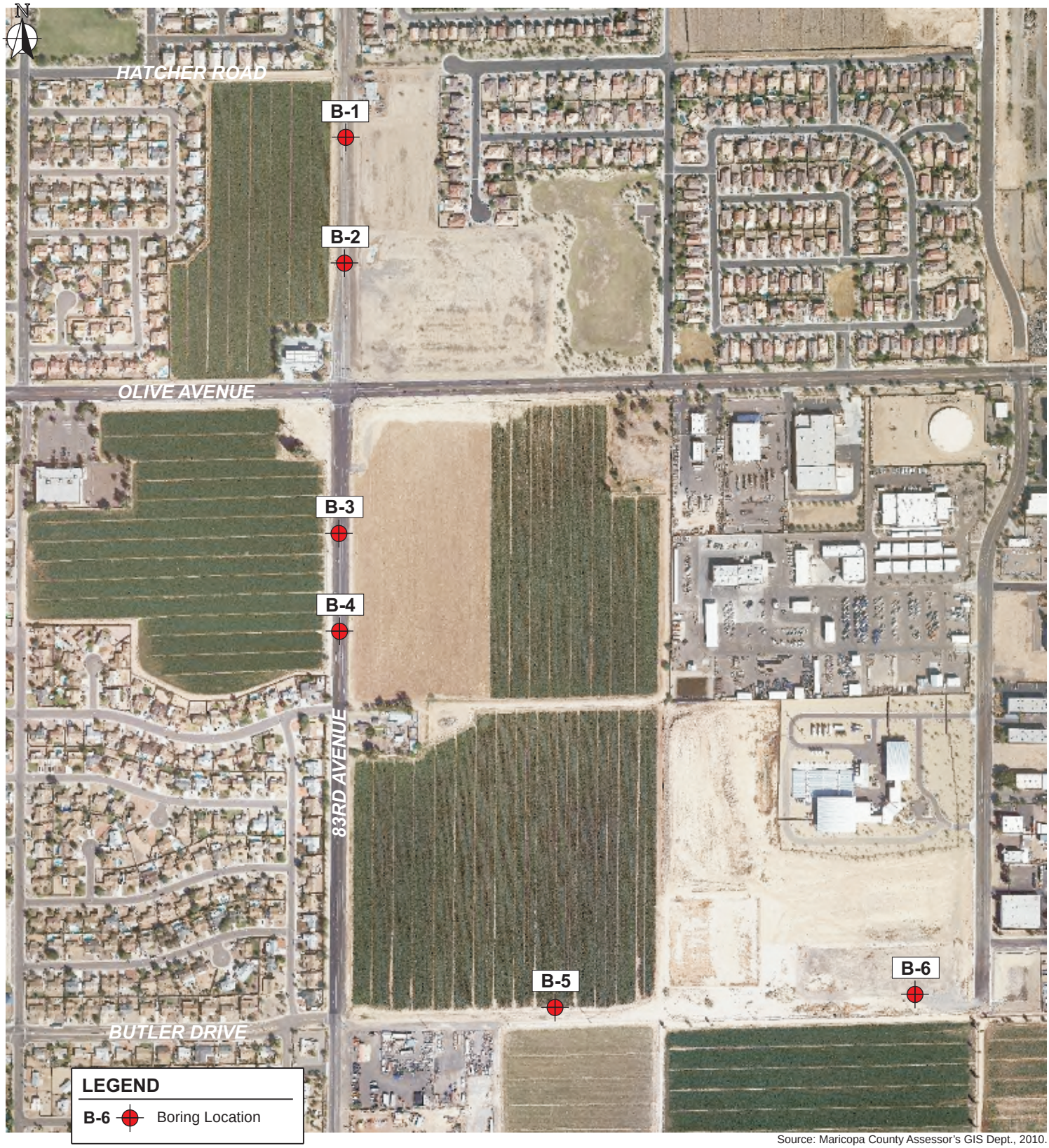
0 1900

Approximate Scale:
1 inch = 1900 feet

Source: The Thomas Guide, Phoenix Metro Edition, 2009.

Ninyo & Moore	SITE LOCATION MAP	FIGURE
PROJECT NO: 602727001 / DATE: 9/10	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	1

file no: 2727vmap0510



Source: Maricopa County Assessor's GIS Dept., 2010.

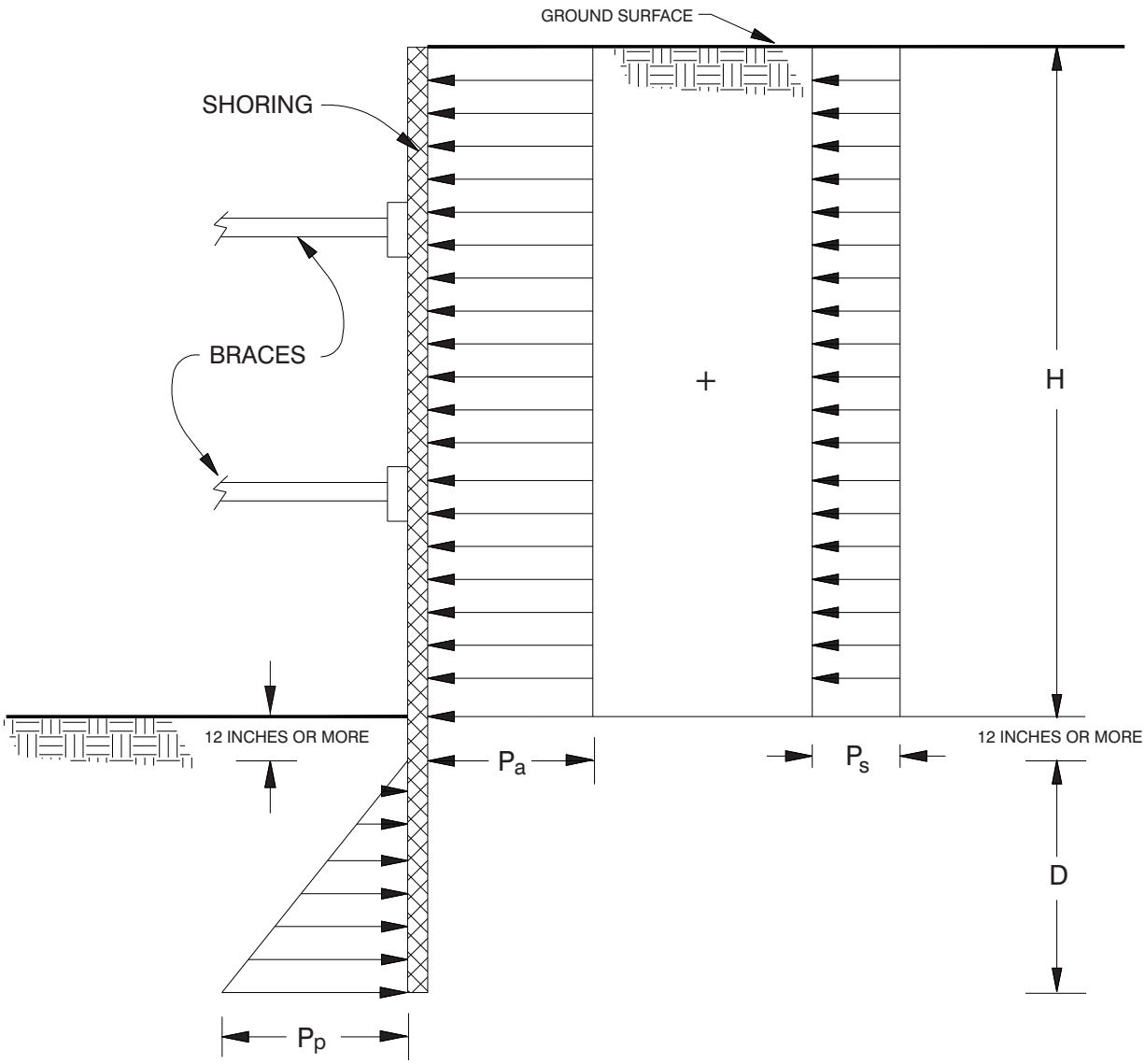
0 800

Approximate Scale:
1 inch = 800 feet

Note: All dimensions, directions and locations are approximate.

Ninyo & Moore	BORING LOCATION MAP	FIGURE
PROJECT NO: 602727001 / DATE: 9/10	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	2

file no: 2727blm0510



NOTES:

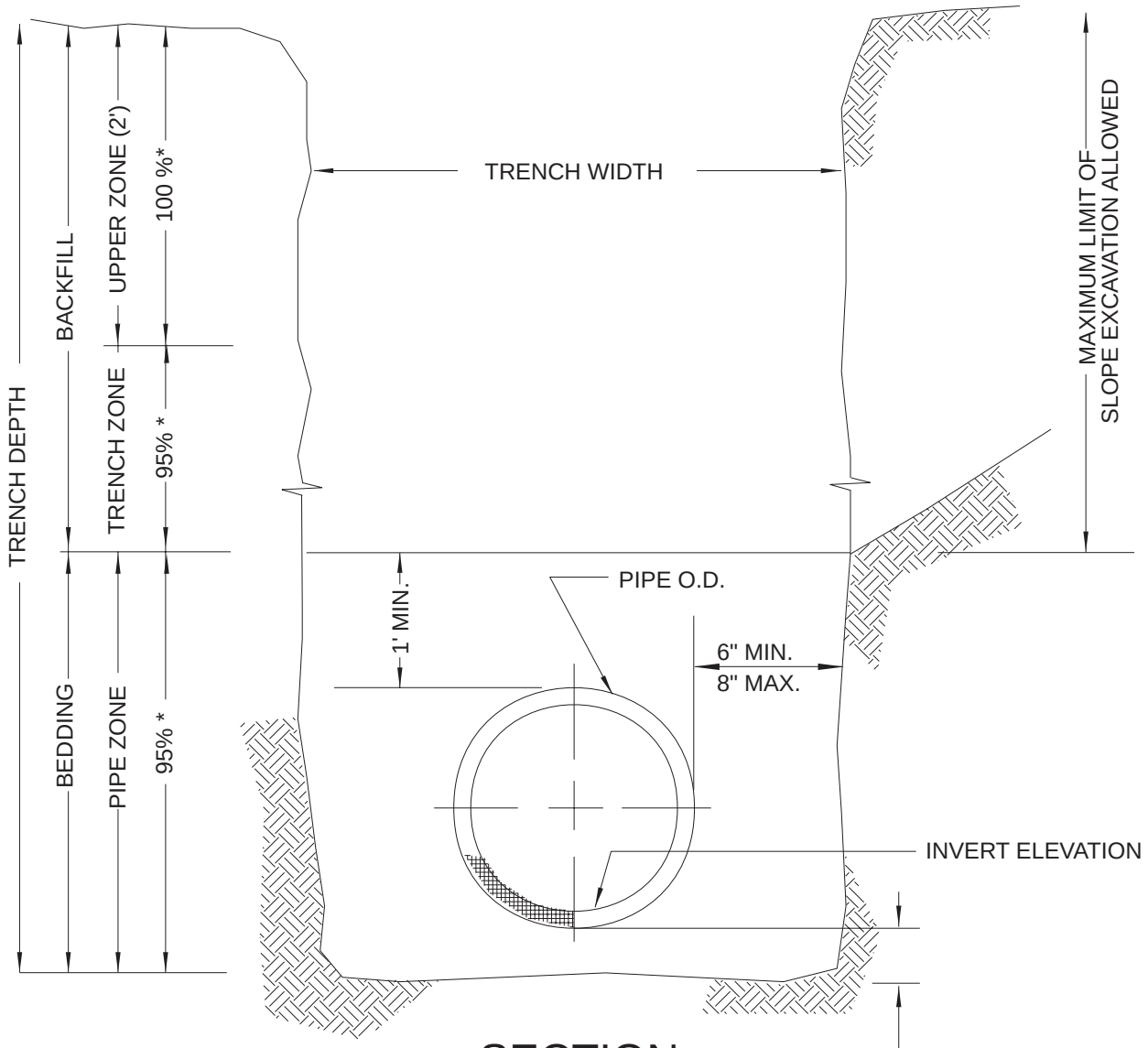
1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 23 H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 300 D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

NOT FOR CONSTRUCTION

Ninyo & Moore	LATERAL EARTH PRESSURE FOR BRACED EXCAVATION	FIGURE
PROJECT NO: 602727001 / DATE: 9/10	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	3

file no: 2727lep0510



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

NOT FOR CONSTRUCTION

Ninyo & Moore	PIPE BEDDING GUIDLINES	FIGURE
PROJECT NO: 602727001 / DATE: 9/10	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	4

file no: 2727pbd0510

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 (SPT) 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 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 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 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 140-pound hammer 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, 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.

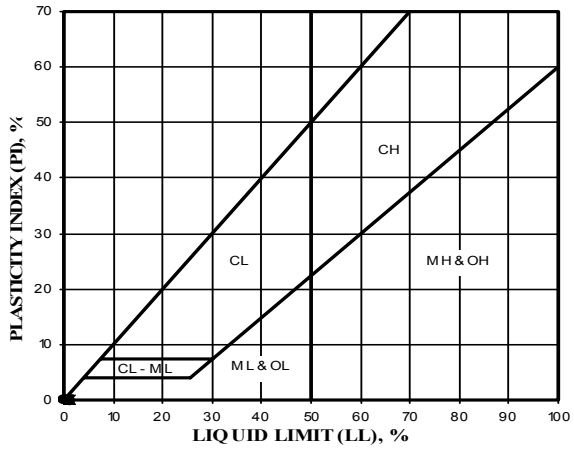
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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
		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	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

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0	■							Bulk sample.
		■						Modified split-barrel drive sampler.
		☒						No recovery with modified split-barrel drive sampler.
		▮						Sample retained by others.
		◤						Standard Penetration Test (SPT).
5		◥						No recovery with a SPT.
		▯	XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
		◥						No recovery with Shelby tube sampler.
		▯						Continuous Push Sample.
				♀				Seepage.
10				▽				Groundwater encountered during drilling.
				▼				Groundwater measured after drilling.
						▦	SM	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	

DATE DRILLED 5/24/10 BORING NO. B-1

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY KLT LOGGED BY KLT REVIEWED BY JSR

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 7 inches thick.
							SC	FILL: Brown, damp, medium dense, clayey fine to coarse SAND.
			14	3.4	109.3		SC	ALLUVIUM: Brown, damp, loose to medium dense, clayey fine to medium SAND; scattered caliche nodules.
							CL	Brown, damp, very stiff, sandy CLAY; numerous caliche nodules; weakly cemented.
			19					
5								
			53	19.5	92.8			Hard.
			24					Scattered caliche filaments and nodules.
10								
			34				SC	Brown, damp, medium dense, clayey fine to coarse SAND with gravel.
15								Total Depth = 15 feet. Groundwater not encountered during drilling. Grouted, backfilled, and asphalt patched on 5/24/10 promptly after completion of drilling. 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.
20								

Ninyo & Moore

BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-1

DATE DRILLED	5/24/10	BORING NO.	B-2
GROUND ELEVATION	--	SHEET 1	OF 1
METHOD OF DRILLING	Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	KLT	LOGGED BY KLT	REVIEWED BY JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 7.5 inches thick.
						SC	FILL: Brown, moist, medium dense, clayey fine to coarse SAND; trace gravel.
		14				CL	ALLUVIUM: Light brown, damp, very stiff, sandy CLAY; trace gravel; numerous caliche nodules; weakly cemented.
		45				SC	Light brown, damp, dense, clayey fine to coarse SAND; few to little gravel; numerous caliche nodules; moderately cemented.
5							Total Depth = 5 feet. Groundwater not encountered during drilling. Grouted, backfilled, and asphalt patched on 5/24/10 promptly after completion of drilling. 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.
10							
15							
20							

Ninyo & Moore

BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-2

DATE DRILLED 5/24/10 BORING NO. B-3

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY KLT LOGGED BY KLT REVIEWED BY JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 4 inches thick.
							AGGREGATE BASE: Approximately 10 inches thick. Brown, moist, medium dense, poorly graded GRAVEL with sand.
		6				SC	ALLUVIUM: Brown, moist, loose, clayey fine to medium SAND; scattered caliche nodules; weakly cemented.
		36	11.0	111.2		CL	Light brown, damp, hard, sandy CLAY; few gravel; numerous caliche nodules; weakly cemented.
5		15					Very stiff; gravel not observed; scattered caliche filaments.
		50/5"					Hard; abundant caliche nodules.
10		22					Very stiff.
15							Total Depth = 15 feet. Groundwater not encountered during drilling. Grouted, backfilled, and asphalt patched on 5/24/10 promptly after completion of drilling. 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.
20							

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BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-3

DATE DRILLED 5/24/10 BORING NO. B-4

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY KLT LOGGED BY KLT REVIEWED BY JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 5.5 inches thick.
							AGGREGATE BASE: Approximately 10.5 inches thick. Brown, damp, medium dense, poorly graded GRAVEL with sand.
		16 14	19.3	99.7		CL	ALLUVIUM: Brown, moist, very stiff, sandy CLAY; few gravel; numerous caliche nodules; weakly cemented.
5							Total Depth = 5 feet. Groundwater not encountered during drilling. Grouted, backfilled, and asphalt patched on 5/24/10 promptly after completion of drilling. 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.
10							
15							
20							

Ninyo & Moore

BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-4

DATE DRILLED 5/24/10 BORING NO. B-5

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY KLT LOGGED BY KLT REVIEWED BY JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Brown, damp, medium dense, clayey fine to coarse SAND; scattered roots.
		23	19.1	96.2		CL	ALLUVIUM: Brown, moist, very stiff, sandy CLAY; trace gravel; scattered caliche filaments and nodules; weakly cemented.
		4				SC	Brown, moist, loose, clayey fine to coarse SAND; few gravel.
5						CL	Brown, moist, stiff, sandy CLAY; few gravel; numerous caliche filaments and nodules; weakly cemented.
		8	16.7	89.0			
		30					Hard.
10							
		34				SC	Brown, moist, medium dense, clayey fine to coarse SAND with gravel.
15							Total Depth = 15 feet. Groundwater not encountered during drilling. Backfilled on 5/24/10 promptly after completion of drilling. 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.
20							

Ninyo & Moore

BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-5

DATE DRILLED 5/24/10 BORING NO. B-6

GROUND ELEVATION -- SHEET 1 OF 1

METHOD OF DRILLING Diedrich D-50, 8" Diameter Hollow Stem Auger (D&S Drilling)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY KLT LOGGED BY KLT REVIEWED BY JSR

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Brown, damp, medium dense, clayey fine to coarse SAND; few gravel; scattered roots.
		14				SC	ALLUVIUM: Brown, damp, medium dense, clayey fine to medium SAND; scattered caliche nodules; moderately cemented.
		35	11.1	99.4		CL	Brown, damp, hard, CLAY with sand; scattered caliche filaments and nodules; weakly cemented.
5							
		26					Numerous caliche filaments and nodules.
		50/5"	14.0	81.0			Abundant caliche nodules; few gravel.
10							
		18					Very stiff.
15							Total Depth = 15 feet. Groundwater not encountered during drilling. Backfilled on 5/24/10 promptly after completion of drilling. 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.
20							

Ninyo & Moore

BORING LOG

83RD AVENUE IMPROVEMENTS - HATCHER ROAD TO BUTLER DRIVE
PEORIA, ARIZONA

PROJECT NO.	DATE	FIGURE
602727001	9/10	A-6

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-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. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (USCS).

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-2.

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-3.

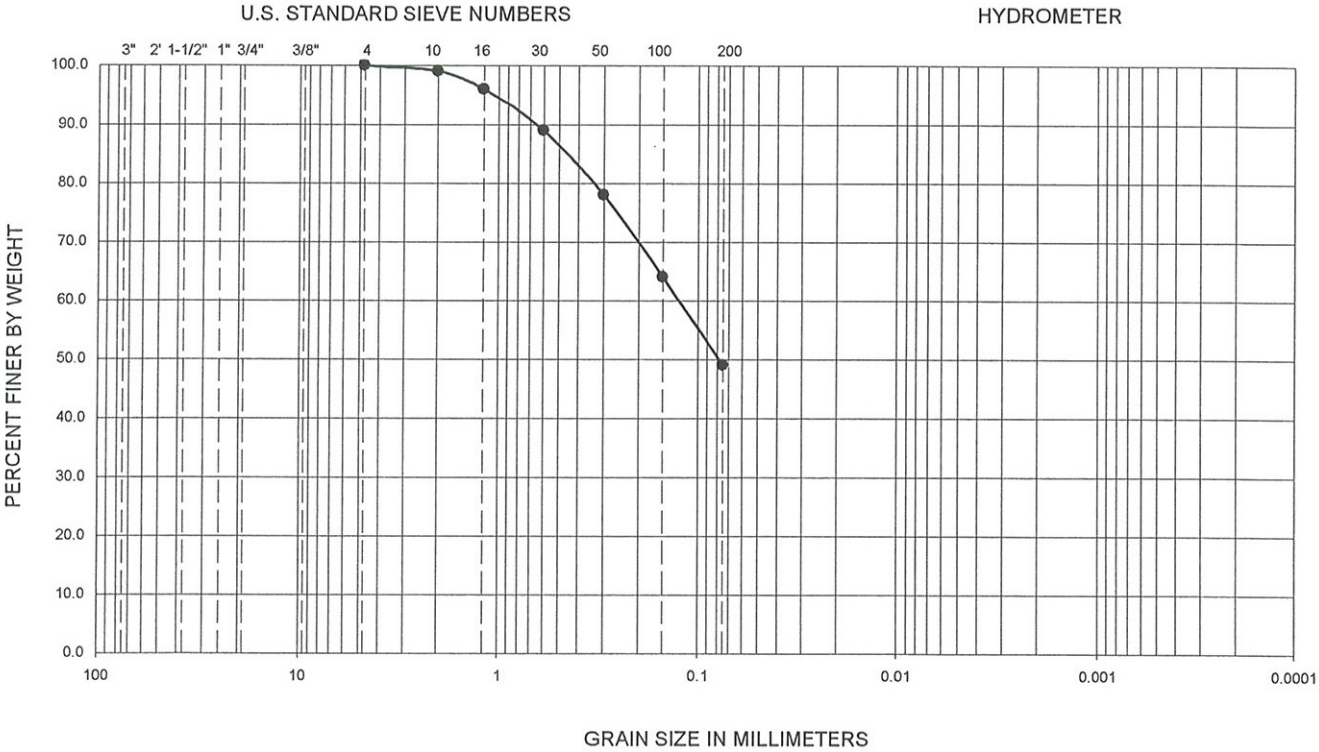
Consolidation Tests

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-4 and B-5.

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-6.

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	1-2.5	32	14	18	--	--	--	--	--	49	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

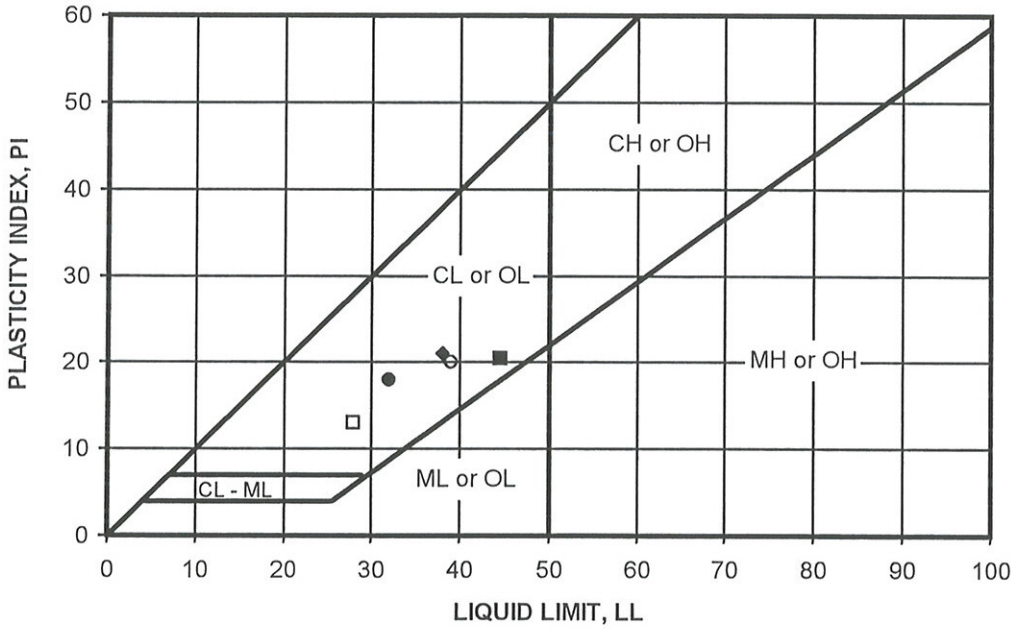
Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-1
602727001 / 9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-2	3.5-5	Clayey SAND	86	29	SC
B-4	1.5-5	Sandy CLAY	95	59	CL
B-5	1-2.5	Sandy CLAY	98	57	CL
B-6	3.5-5	CLAY with Sand	100	79	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

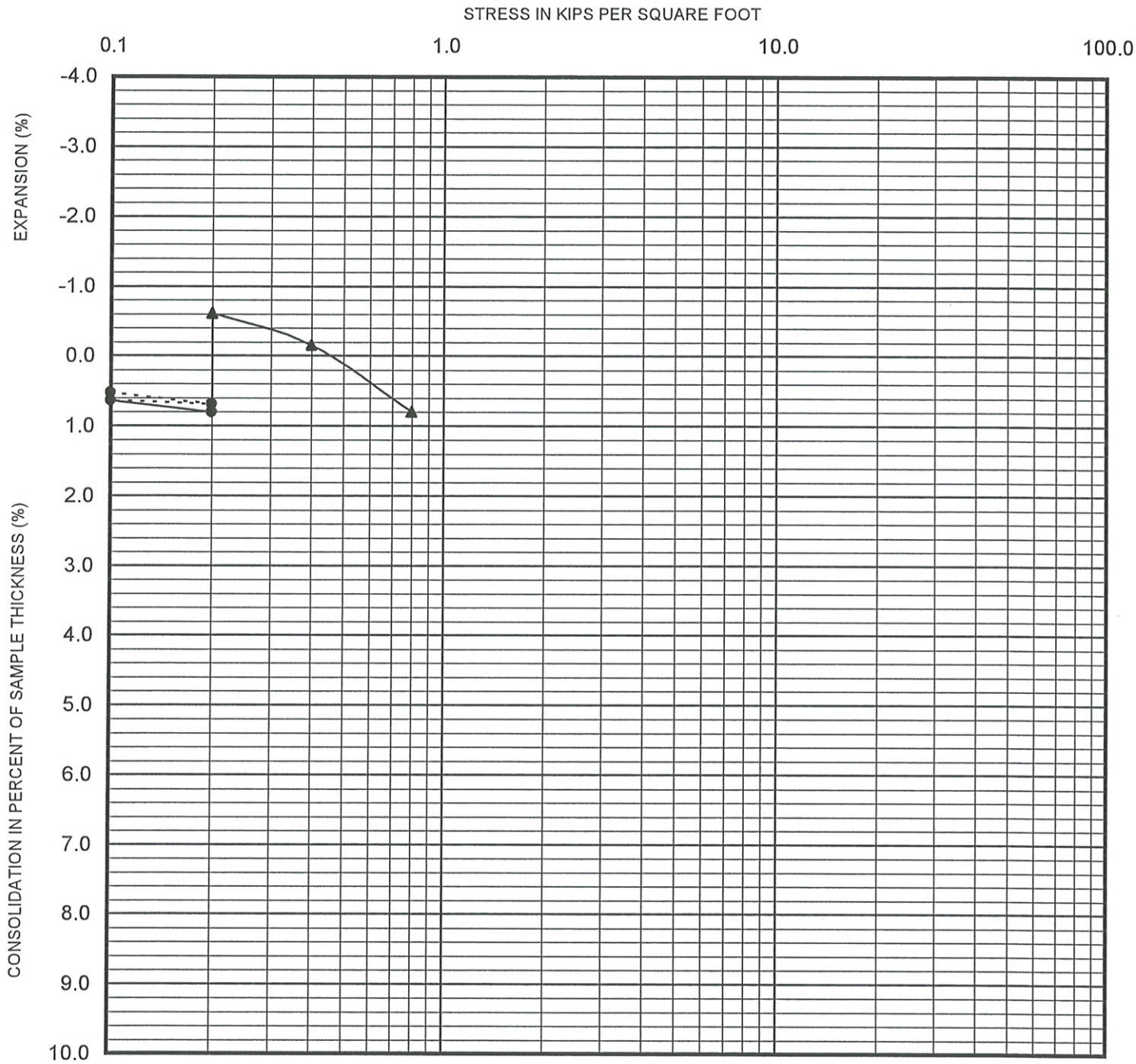
Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-2
602727001	9/10		

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	1-2.5	32	14	18	CL	SC
■	B-2	3.5-5	45	24	21	CL	SC
◆	B-4	1.5-5	38	17	21	CL	CL
○	B-5	1-2.5	39	19	20	CL	CL
□	B-6	3.5-5	28	15	13	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

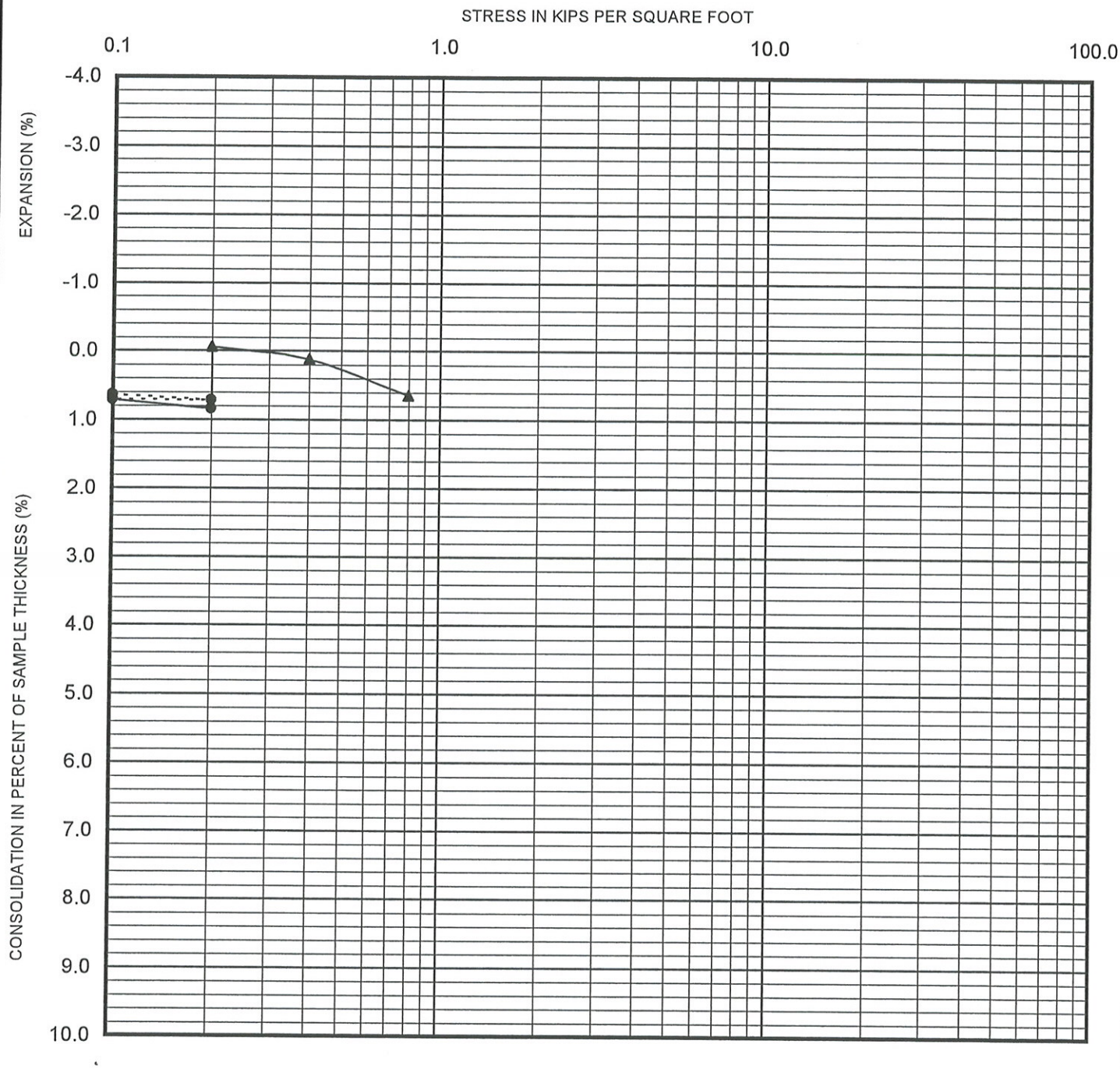
Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. 602727001 / DATE 9/10	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-3



- - -●- - -	Seating Cycle	Sample Location	B-1
—●—	Loading Prior to Inundation	Depth (ft.)	1-2.5
—▲—	Loading After Inundation	Soil Type	SC
- -▲- -	Rebound Cycle		

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-4
602727001	9/10		



Seating Cycle		Sample Location	B-5
Loading Prior to Inundation		Depth (ft.)	1-2.5
Loading After Inundation		Soil Type	CL
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-5
602727001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	5-10	8.8	1,850	90	0.009	71
B-6	10-15	8.6	1,710	92	0.009	55

[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

1

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	83RD AVENUE IMPROVEMENTS HATCHER ROAD TO BUTLER DRIVE PEORIA, ARIZONA	B-6
602727001	9/10		