PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED INDUSTRIAL PARK

NEWPORT, ARKANSAS

* * * * *

NEWPORT INDUSTRIAL DEVELOPMENT COMMISSION

OWNERS

201 HAZEL STREET

NEWPORT, ARKANSAS 72112

* * * * *

APRIL 7, 2006

JOB NO. 243906

ANDERSON ENGINEERING CONSULTANTS, INC.

3217 NEIL CIRCLE – JONESBORO, ARKANSAS 72401
PHONE (870) 932-3700 FAX (870) 932-3769

April 7, 2006

Job No. 243906

Mr. John Chadwell
Newport Industrial Development Commission
201 Hazel Street
Newport, Arkansas 72112

Re: Preliminary Geotechnical Investigation
Proposed Industrial Park
Newport, Arkansas

Dear Mr. Chadwell:

It is our pleasure to submit this preliminary report for the referenced property in Newport, Arkansas. The investigation consisted of field test borings, laboratory analyses, and general pavement and foundation recommendations.

The investigation indicated variable soil type with extensive areas of surficial expansive soils and poor to moderate near surface soil consistency. The site, however, will be suitable for development when proper design and construction techniques are employed. We recommend that our geotechnical services be continued when specific building and parking locations are determined for this is the most feasible means of assuring the owners, designers, and builders that the geotechnical design intent is being achieved. In the event other adverse geotechnical conditions are encountered in specific building locations, they can be identified and evaluated so that safe and economical structures may be designed.

We wish to express our appreciation for the opportunity of providing this preliminary soils investigation to you and your company. We are available for further assistance at any time during final design and construction and should you desire additional consultation please feel free to contact us.

Very truly yours,

ANDERSON ENGINEERING CONSULTANTS, INC.

Alexandra W. "Alex" Gangluff, P.E.
Geotechnical Engineer

Billy R. Alumbaugh, P.E.
Senior Geotechnical Engineer

CERTIFICATE OF AUTHORIZATION
ANDERSON ENGINEERING CONSULTANTS, INC.
No. 73
ARKANSAS - ENGINEER

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 11241
ALEXANDRA W. GANGLUFF

AWG/BA/msk
243906.GEO

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* * * * *

BY

ANDERSON ENGINEERING CONSULTANTS, INC.

GEOTECHNICAL CONSULTANTS

3217 NEIL CIRCLE

JONESBORO, ARKANSAS 72403-1655

APRIL 7, 2006 JOB NO. 243906

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Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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e-mail: info@asfe.org www.asfe.org

PURPOSE

The primary purposes of this geotechnical investigation were:

a. To determine the feasibility of construction at the proposed site with respect to physical and engineering properties of the soil within the proposed site.

b. To make general recommendations for the earthwork, pavements, and the type of foundation suited for the prevailing soil conditions within the overall site.

c. To evaluate and recommend the general design procedures for various common soil, pavement, and foundation items in accordance with current engineering practices.

SCOPE

The scope of this geotechnical investigation includes the following:

a. The geologic features of the site were found to consist essentially of alluvial deposits of clay, silt, and sand soils. A total of eleven auger borings were advanced to a maximum depth of 51.5 feet at general locations across the site.

b. Field testing consisted of Standard Penetration test samples taken in all of the borings. Soils were visually classified by a senior geotechnical engineer.

c. The soils analyses were based on N-values obtained from the Standard Penetration tests, Atterberg limits, mechanical grain size analyses, unconfined compression tests, swell tests, visual observations, and other routine inspection and classification methods. The soils were classified basically in accordance with the Unified Soil Classification System (ASTM D 2487); however, visual classifications are given on the logs.

d. The foundation recommendations were based on standard foundation design procedures, including the Standard Penetration N-values obtained during drilling and the results of the laboratory testing program.

e. The flexible and rigid pavement recommendations given this report are related to the subgrade material characteristics of the near surface site soils.

AUTHORITY

This geotechnical investigation was authorized on February 14, 2006, by Mr John Chadwell, the owner's representative for the project.

GEOTECHNICAL INVESTIGATION

On February 14 and 15, 2006, eleven geotechnical test borings were made at the proposed site east of the Newport, Arkansas. The site is located as shown on the Vicinity Map, Plate 1. The borings were placed on site as shown on the Plan of Borings, Plate 2. The logs of the borings are given on Plates 3 through 10. The Field Classification System for Soil Exploration and Key to the Soil Classifications and Symbols are given on Plates 11 and 12, respectively. These systems are provided to aid the reader in interpreting the various symbols used on the logs of borings. The Unified Soil Classification System is given on Plate 13. This system is used to determine the soil classification and to develop the terminology used on the logs of borings.

GENERAL SITE CONDITIONS

The proposed property consists of approximately 47.5 acres located south of Highway 18, east of Highway 18 Spur, and directly north of an abandoned runway at the Newport Municipal Airport located east of the City of Newport, Arkansas. At the time of the investigation, recent usage of the site had been limited to agricultural crop production. The site is relatively flat, however, exact grades were not provided. Overall site drainage is likely to ditches on the north and west property boundaries. However, water retention should be anticipated across the site during periods of wet or winter weather. A truck mounted drill rig was used to access the site.

GEOLOGY AND STRATIGRAPHY

The proposed site is located in the Mississippi Embayment Physiographic Region of northeastern Arkansas, and consists of terraced sediments deposited by the ancient Mississippi River and its tributaries during Quaternary times. These deposits generally consist of a complicated sequence of unconsolidated layers of gravels, sands, silts, and clays. The site soils were found to be consistent with the area geology. The site stratigraphy generally consists of 6.5 to 18.0 feet of soft to very stiff, fat clay (CH) and silty clay (CL) and loose to medium dense, clayey sand (SC). This strata is underlain by medium stiff to stiff, sandy silt (ML) and loose to dense, silty sand (SM) to depths of 35.0 to 44.5 feet. The basal stratum consisted of medium dense to very dense, sand (SP). It should be noted that the surficial soils are predominately fat clay (CH). However, in the vicinity of borings B3, B4, P3, P4, and to some extent B2, the near surface soils are a clayey sand (SC). This variation may be a result of natural processes or historic earthwork associated with the adjacent runway.

GROUNDWATER CONDITIONS

The groundwater was encountered at a depth of 27.5 to 29.5 feet during drilling and is consistent with previous experience in the Newport, Arkansas area. This water level, though, is seasonal in nature and will rise and fall with fluctuations in rainfall. Some perched water should also be expected in the near surface cohesive soils, especially during the winter or wet seasons of the year, and should be considered in design and construction of foundations, deep utilities, equipment pits or elevator shafts. This latent water condition is typically due to storage of recent rainfall or by a barrier to capillary evaporation and will be more prevalent in drainage swales, rubble fills, and in existing utility trenches. Perched water, if encountered, will most likely be brief in duration and typically in low quantities. Where perched water is encountered it should be expected to excavate gravity drainage ditches to divert it away from the construction area. Additionally, soft, wet and pumpable soils can be expected that will require removal and replacement in structural areas.

SEISMICITY

Seismic analyses require the selection of appropriate site coefficients and other seismic values that can be established from subsurface conditions, guidelines set forth by local, state and federal codes, and historic seismic information. The structures and foundations should be designed using guidelines as set forth in either the 1999 Standard Building Code as required by **Arkansas Act 1100-1991** (and subsequent amendments) or the 2000 International Building Code.

The predominant soil types are interbedded sands, silts, and clays that vary from soft to very stiff and loose to very dense. Based upon the subsurface soil conditions and the seismic values for Arkansas published by the Arkansas State Building Services, the 1999 Standard Building Code and the 2000 International Building Code the following data are considered applicable to this project site:

Site Class	C*
Seismic Zone	3
Soil Profile Type	S_2
Site Coefficient	1.2
Peak Acceleration Coefficient (A_a)	0.20
Effective Peak Velocity-Related Acceleration Coefficient (A_v)	0.20

*Not verified by 100-foot boring as per IBC Code. Performing a 100-foot boring may improve your IBC site classification, and therefore, may be an economical means of controlling foundation costs.

LIQUEFACTION ANALYSES

Liquefaction is the sudden loss of all shear strength in a soil as a result of excess pore water pressure which is induced by vibration or shock waves resulting from an earthquake, explosion, or machinery. When soils experience liquefaction they lose strength to resist load and temporarily exist in a near liquid state. Liquefaction is primarily associated with saturated, loose to medium dense cohesionless soils, i.e. sands at high moisture contents or below the water table. At this site, the relatively low water table and dense consistency of the basal sand (SP) strata minimize the potential for liquefaction. However, in borings B1 and B2 a medium stiff to stiff, non-plastic sandy silt (ML) strata exists below the water table at a depth of approximately 30.0 to 40.0 feet that has a potential for liquefaction. Therefore, additional investigation and analysis should be preformed on the site soils to delineate and quantify zones of potential liquefaction at the site.

LABORATORY TESTING

Laboratory testing was performed on select samples to determine their physical properties, classification and, strength characteristics. Laboratory testing included Atterberg limits, mechanical grain size analyses, unconfined compression tests and swell tests. The following sections describe the results of these tests. Individual test results are shown in Appendix B.

Atterberg Limits

Atterberg limit tests were performed on selected samples to aid in classification and to determine the potential volume change of the soils. The results indicated that over half the samples tested were non-plastic (NP) with the remainder moderately to highly plastic clay (CL) and fat clay (CH). The liquid limit (LL) of the cohesive soils ranged from 32 to 81 with the plasticity index (PI) ranging from 14 to 55. The cohesive soils were generally located in the top three samples, with corresponding depths of up to 6.5 feet.

Mechanical Grain Size Analysis

Mechanical grain size analyses were performed on variable soil types from the proposed site. The results indicated no more than 0.2% gravel sized material, between 12.8% and 89.8% sand, and between 10.2% and 87.2% passing the No. 200 sieve. Thus, the samples tested may be classified as fat clay (CH), sandy silt (ML), and silty sand (SM).

Unconfined Compression Tests

Unconfined compression tests were performed on selected cohesive samples at the specimens natural moisture content. The samples investigated resulted in low to moderate strengths ranging from 1.3 to 2.2 ksf. The moisture content for these samples may be considered moderate to high and were found to range from 25.2% to 48.6%. The dry unit weights are generally low, ranging from 70.8 pcf to 92.2 pcf, however, they may be considered normal for more plastic clay soils.

Shrinkage/Swell Tests

Visual inspection and laboratory plasticity tests performed on selected samples suggest that the in-situ clays may be of a critical nature with respect to shrinkage and swell potential, and thus, they could cause some detrimental effects upon any proposed structures. Representative samples were tested to determine the potential swell if the materials become saturated. Table I, shown on the following page, summarizes the results of these tests. The results indicate moderate swell pressures may be encountered, especially if the soils are allowed to dry to a moisture content below their plastic limit. Additional testing should verify that the potential vertical rise (PVR) of these soils should not have a significant detrimental effect upon future improvements at the proposed site.

TABLE I

SUMMARY OF PVC SWELL/LINEAR SHRINKAGE TESTS

Sample Number	B1;P2	B2;P3
Depth (feet)	2.5 – 4.0	5.0 – 6.5
Classification	CH	CH
Liquid Limit, Plastic Limit	74, 26	77, 26
Plasticity Index	48	51
Water at Beginning of Swell (%)	19.0	36.1
Water at End of Swell (%)	28.1	40.2
Swell Pressure (psf)	2,495	1,040
Linear Shrinkage (%)	14.0	12.0

GENERAL EARTHWORK

The following sections are intended to provide the designer and contractor with guidelines for design and construction for future projects. They are not intended to be used as a specification for construction procedures or methods.

Site Preparation

Because the site has been in agricultural use the near surface soils have been tilled and processed. The organic layer of this material may be as deep as 12.0 inches across the site. Prior to cut and placement of any fill on the site, a minimum of 6.0 inches of topsoil and vegetation should be removed. After stripping, proof rolling with a loaded truck or scraper is recommended across the entire site to locate potential soft areas in the subgrade and/or natural ground before any fill is placed and in the cut areas after excavation to the planned elevation. Any soft areas in the natural ground detected by proof rolling should be removed and replaced with compacted stable soil. After stripping and any required undercut, the top 6.0 inches of exposed subgrade should be scarified and recompacted prior to fill placement.

Fill Soils

It is assumed that the on-site soils will be utilized to their fullest extent, however, the test data indicates that the upper surface cohesionless soils are not suitable for use as fill as they are predominately high plastic fat clay (CH). Thus, offsite fill will be required and consideration should also be given to the use of locally available select fills. Generally, select fill should be composed of granular, non-expansive soils such as clay gravel or clay sand. Modified compaction has been given primary consideration as optimum is typically 3.0% to 7.0% less than Standard. Modified is thus recommended as it will also yield higher CBR and allowable bearing capacities for conventional footing foundations.

Utilities

New utilities are anticipated for any proposed facilities. Utility excavations should be easily made with standard excavating equipment. All utility excavations can be backfilled with on-site materials and should be placed and compacted to ASTM D 1557. The on-site soils are considered as clayey and thus, some sloughing or caving can be expected. The contractor should strictly adhere to OSHA excavation standards in utility construction.

Landscaping

Due to the shallow fat clay (CH), care must be exercised to not dry out the subgrade soils after construction which will result in excessive settlement due to drying shrinkage of the more plastic soils. Large moisture demanding trees or vegetation should not be planted near or adjacent to buildings, as drying of the subgrade and foundation supporting soils could result in excessive settlements from soil shrinkage. When this occurs, severe distress can be noted in masonry walls and floor slabs.

The preferred landscaping method is to utilize planters having drainage systems that control and route water away for the building so that saturation of the foundation soils will not occur with swelling or loss of the allowable bearing capacity. As a general rule, the drip line of any existing or future full grown tree should not fall within the building area. Moisture control will also be aided by having sidewalks, paving, or sloping ground surfaces for at least 5.0 feet outside the structure. The sidewalks or paving must have a positive slope away from the building and all joints must be sealed to prevent water infiltration. Implementation of these points will reduce the changes in moisture content of any more plastic soils and movements of the foundation and slabs.

Adverse Conditions

Site grading and earthwork operations will be more difficult in wet or winter weather. The on-site clayey soils will absorb significant quantities of water which will require significant aeration and working to dry during the winter or wet weather. As an alternate, the contractor may elect to dry the soils using lime or fly ash worked into the wet soils. The amount of drying can be required by maintaining the site in a well drained condition during construction including not allowing water to stand or pond on areas of the exposed earthwork. In addition, during wet weather the upper limit on the moisture content should be raised to five percentage points over optimum moisture content, provided the fill meets the specified compaction and is firm and stable.

FOUNDATIONS

Conventional shallow footings would be feasible for use with lightly loaded single and two story structures. The foundations should be made rigid in an effort to minimize potential differential movements resulting from non-uniform settlement due to consolidation of variable thickness of native and/or fill soils. Column and wall footings should be designed in accordance with the various applicable codes. Due to the relatively soft and potentially expansive nature of the near surface soil, conventional shallow footings should bear on 3.0 feet of select compacted fill. An allowable bearing capacity of 2000 psf may be used for footing bearing at a depth of 2.0 feet below the finished floor elevation on 3.0 feet of select fill. The finished floor elevation may be raised above the existing grade to minimize the undercut required. A corresponding settlement value should be with in normal settlement tolerances. The calculations and curves showing the bearing capacity analyses are provided on Plates 14 and 15. An explanation of the bearing capacity calculations is provided on Plates 16. For heavy loading conditions, auger cast piling bearing in the lower sand (SP) soils should be given primary consideration. However, geo-piers bearing at an intermediate depth may also prove economical.

FLOOR SLABS

Differential movement of the floor slab may be caused by a difference in the allowable gross bearing capacity, differing heave conditions, and/or variable thicknesses of compressible soils below the floors. The stiffness effect of a well compacted select fill subgrade and/or engineered fill should greatly diminish the differential floor slab movements to tolerable limits. A conventional slab-on-grade may be utilized provided the slab bears on select compacted fill. The use of an impermeable vapor barrier underlain with free draining material is generally recommended beneath all floor slabs to provide an all-weather pad.

DRIVING AND PARKING AREAS

Either flexible or rigid pavement structures should serve adequately on the proposed property with the design based on numerous reasonable assumptions concerning the pavement use, site conditions, and maintenance. The site soils in their natural condition will likely require undercut and backfill replacement to properly support the required pavement sections. However, flexible pavements will probably require higher maintenance than a comparable rigid pavement structure.

Flexible Pavement

Flexible pavement typically consists of asphalt cement hot mix (ACHM) as specified by Section 407 of the Standard Specifications for Highway Construction (Edition of 2003) as published by the Arkansas State Highway and Transportation Department. The design requirements for ACHM surface course; 12.5 mm (Type II) and 9.5 mm (Type III) are provided in Tables 407-1 and 407-2, respectively. ACHM is most commonly used for light to moderate traffic areas including straight drives and parking areas for light vehicles. It should not be used in traffic lanes where trucks turn, backup, or pick up trash dumpsters.

Rigid Pavement Non-Reinforced

Rigid pavements or Portland Cement Concrete (PCC) pavements consists of concrete materials and construction procedures as specified by Section 501 of the Standard Specifications for Highway Construction (Edition of 2003) as published by the Arkansas State Highway and Transportation Department. The material type and design requirements including admixtures, reinforcing, dowels, jointing, curing, and finish are provided therein. Rigid (PCC) pavements are commonly used for both light and heavy duty traffic applications. Minimally, approach slabs, truck turning areas, docks, and dumpster pads should be PCC.

CONSTRUCTION QUALITY CONTROL

Quality control testing should be utilized in all phases of the construction. To verify that the proper performance of the proposed structure, all fill required should be compacted as required and verified by ASTM D 2922. The foundation excavations should be evaluated to verify that the recommended bearing capacity has not been reduced by disturbance to excavation or massive imperfections in the bearing strata. Our recommendations are based upon adequate quality control being utilized and further evaluations and reviews during the construction phase of the project.

CONCLUSIONS AND RECOMMENDATIONS

As a result of this preliminary geotechnical investigation, the following recommendations are offered for consideration:

1. Additional geotechnical investigation and recommendations should be sought upon determination of a specific building site on the property.
2. The site can be made suitable for construction with proper design and/or construction techniques.

3. Soils and other geologic materials from both on and off the site can be satisfactorily used in the construction of the earthwork with proper handling, design, and construction techniques as previously discussed.

4. The investigation revealed the existence of soft and potentially expansive near surface soils. However, this condition should not have a significant detrimental effect upon future improvements at the proposed site. This is not to say that others do not exist, a complete determination in this regard is beyond the scope of this investigation as authorized by the owner's representative.

5. As previously discussed conventional footings founded on compacted fill should serve satisfactorily for future lightly loaded structures. It is concluded that this will be an economical type of foundation and should be designed in accordance with the necessary structural and/or architectural requirements determined by the designers with the developer's ultimate approval.

6. Modified Proctor density as per ASTM D 1557 should be used in all earthwork including backfill of undercut areas and for building and pavement areas.

7. The use of flexible or rigid pavements should be a function of the anticipated traffic use as determined by the designer. As a minimum PCC pavements should be used for truck and bus lanes as well as dumpster pads.

8. As an additional measure, perimeter surface and subsurface drainage should be directed away from the exterior of the buildings. Other measures should be undertaken to intercept and drain surface runoff, roof drainage, condensate drip water, or seepage water from the near surface and foundation support soils. It would also be a prudent measure to slope backfill soils away from foundation walls.

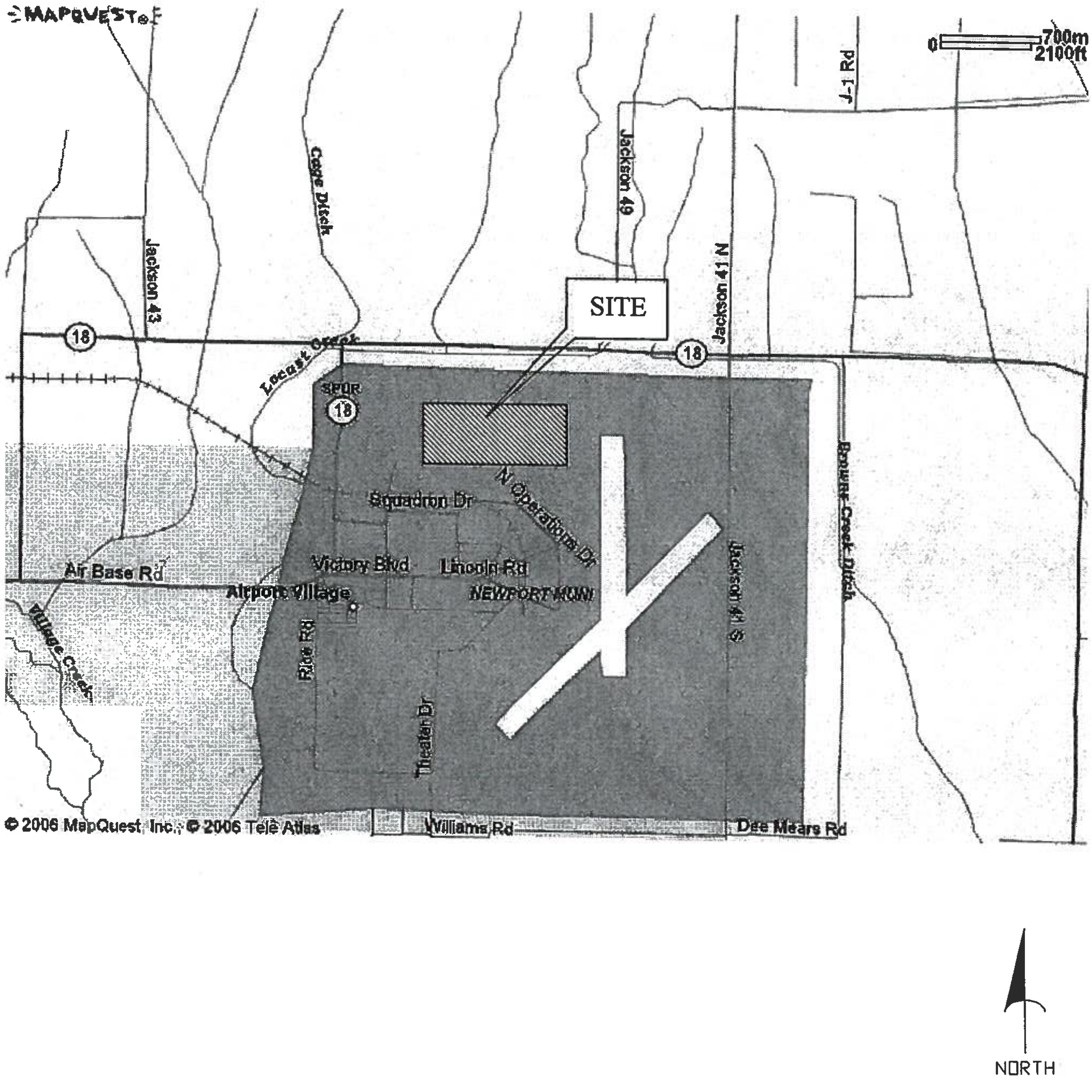
9. Quality control testing should be utilized in the construction of the foundation, fill placement, and floor slab construction with adequate testing to verify that the design requirements have been achieved. Additionally, observation during initial earthwork is recommended to further evaluate the fill existing at the site.
10. Geotechnical engineering services by this firm are recommended during the foundation construction phase so that adequate compensation can be made for conditions that may occur which differ significantly from those assumed as a result of this investigation.
11. Other recommendations are given throughout the text of this report.

* * * * *

APPENDIX A

PLATES

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

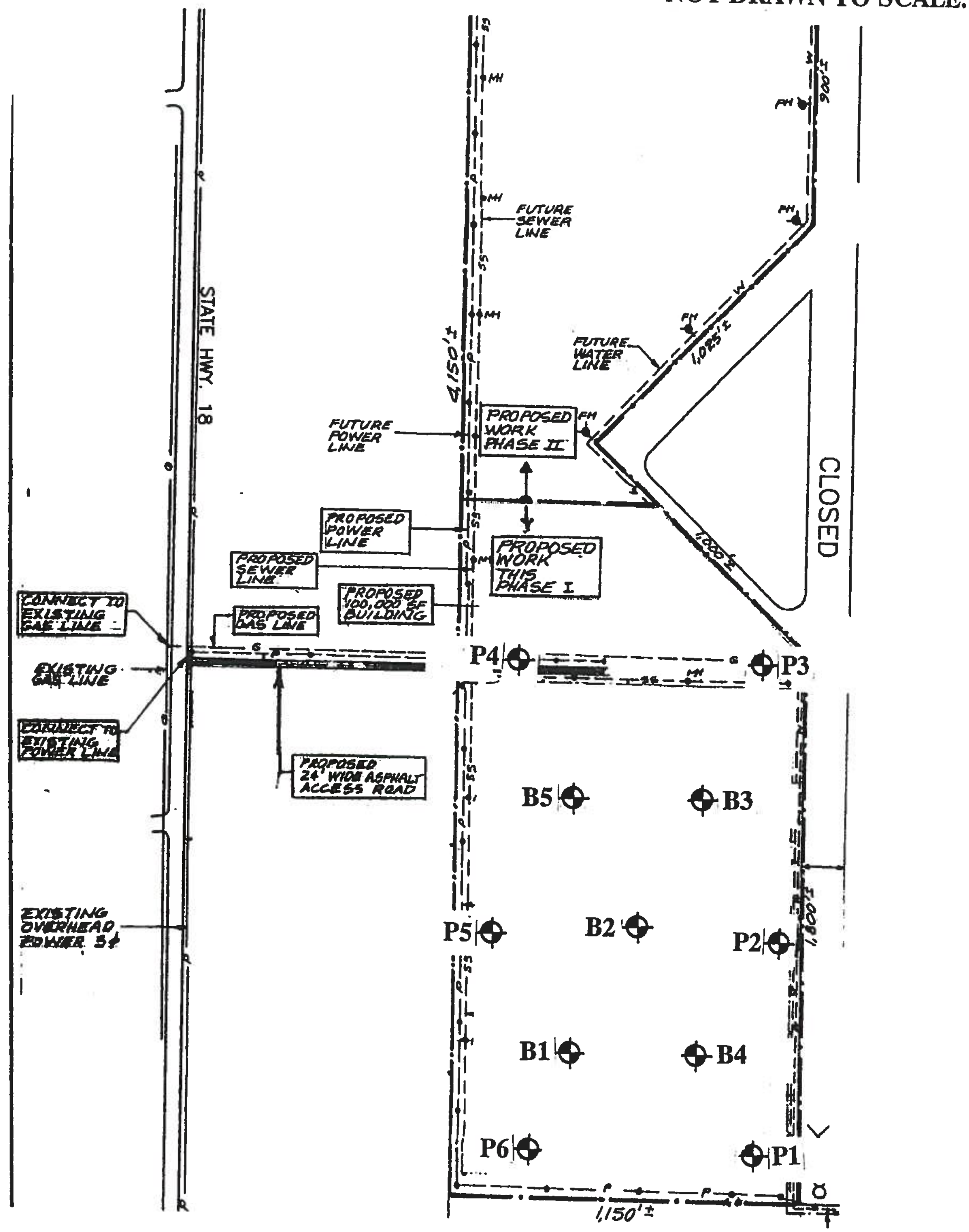


VICINITY MAP

NEWPORT, ARKANSAS

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

NOT DRAWN TO SCALE.



PLAN OF BORINGS

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
DATE: 02/14/06 **JOB NO:** 243906
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
BORING NO: B1
LOCATION: SEE PLAN OF BORINGS
BORING TYPE: AUGER W/SPT
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube; Core; Static Water Table; NX Diamond Core; Standard Penetration; Hydrostatic Water Table; P Penetration Test; J-Jar; No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	5		MEDIUM STIFF MOIST GRAY FAT CLAY (CH) PP = 0.50 TSF
	P2	9		STIFF MOIST GRAY FAT CLAY (CH) PP = 1.00 TSF
5	P3	11		STIFF MOIST LIGHT GRAY AND LIGHT BROWN FAT CLAY (CH) PP = 1.25 TSF
	P4	15		MEDIUM DENSE MOIST LIGHT GRAY AND LIGHT BROWN SILTY SAND (SM)
10	P5	11		MEDIUM DENSE MOIST LIGHT GRAY AND LIGHT BROWN SILTY SAND (SM)
15	P6	14		MEDIUM DENSE MOIST LIGHT GRAY AND LIGHT BROWN SILTY SAND (SM)
20	P7	9		STIFF MOIST DARK GRAY SANDY SILT (ML) PP = 1.00 TSF
25	P8	7		MEDIUM STIFF MOIST DARK GRAY SANDY SILT (ML) PP = 0.75 TSF
30	P9	5		MEDIUM STIFF WET DARK GRAY SANDY SILT (ML) PP = 0.50 TSF
35	P10	7		MEDIUM STIFF WET DARK GRAY SANDY SILT (ML) PP = 0.75 TSF
40	P11	9		STIFF WET DARK GRAY SANDY SILT (ML) PP = 1.00 TSF
45	P12	24		MEDIUM DENSE WET GRAY FINE SAND (SP)
50	P13	22		MEDIUM DENSE WET GRAY FINE SAND (SP)
55				BOTTOM OF HOLE AT 51.5 FEET. BORING CAVED AT 29.5 FEET. WATER WAS ENCOUNTERED AT 29.5 FEET DURING DRILLING. WATER LEVEL AT 28.0 FEET UPON COMPLETION OF DRILLING.

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3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
DATE: 02/15/06 **JOB NO:** 243906
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY

BORING NO: B2
LOCATION: SEE PLAN OF BORINGS
BORING TYPE: AUGER W/SPT
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube; ▮ Core; Static Water Table; NX Diamond Core; Standard Penetration; Hydrostatic Water Table; P Penetration Test; J-Jar; No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	VISUAL DESCRIPTION OF STRATUM
0	P1	8	LOOSE MOIST DARK BROWN CLAYEY SAND (SC)
	P2	9	STIFF MOIST LIGHT GRAY FAT CLAY (CH) PP = 1.00 TSF
5	P3	9	STIFF MOIST LIGHT GRAY FAT CLAY (CH) PP = 1.00 TSF
	P4	25	VERY STIFF MOIST LIGHT GRAY AND BROWN FAT CLAY (CH) PP = 2.50 TSF
10	P5	17	VERY STIFF MOIST LIGHT GRAY AND BROWN FAT CLAY (CH) PP = 1.75 TSF
15	P6	7	MEDIUM STIFF MOIST LIGHT GRAY AND BROWN SILTY CLAY (CL) PP = 0.75 TSF
20	P7	7	MEDIUM STIFF MOIST DARK GRAY CLAYEY SILT (ML) PP = 0.75 TSF
25	P8	7	MEDIUM STIFF MOIST DARK GRAY CLAYEY SILT (ML) PP = 0.75 TSF
30	P9	11	STIFF WET GRAY CLAYEY SANDY SILT (ML) PP = 1.25 TSF
35	P10	14	STIFF WET GRAY SANDY SILT (ML) PP = 1.50 TSF
40	P11	39	DENSE WET GRAY FINE SAND (SP)
45	P12	44	DENSE WET GRAY FINE SAND (SP)
50	P13	29	MEDIUM DENSE WET GRAY FINE SAND (SP)
55			BOTTOM OF HOLE AT 51.5 FEET. BORING CAVED AT 29.5 FEET. WATER WAS ENCOUNTERED AT 29.5 FEET DURING DRILLING. WATER LEVEL AT 27.5 FEET UPON COMPLETION OF DRILLING.

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3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
DATE: 02/15/06 **JOB NO:** 243906
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
BORING NO: B3
LOCATION: SEE PLAN OF BORINGS
BORING TYPE: AUGER W/SPT
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube; ▮ Core; ⊻ Static Water Table; NX Diamond Core; ⊠ Standard Penetration; ⊻ Hydrostatic Water Table; P Penetration Test; J-Jar; No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	7		LOOSE MOIST BROWN CLAYEY SAND (SC)
	P2	7		LOOSE MOIST BROWN CLAYEY SAND (SC)
5	P3	9		LOOSE MOIST BROWN CLAYEY SAND (SC)
	P4	10		LOOSE MOIST BROWN CLAYEY SAND (SC)
10	P5	17		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
15	P6	18		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
20	P7	21		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
25	P8	28		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
30	P9	39		DENSE WET LIGHT BROWN SILTY SAND (SM)
35	P10	45		DENSE WET GRAY FINE SAND (SP)
40	P11	50/11"		VERY DENSE WET GRAY FINE SAND (SP)
45	P12	50/8"		VERY DENSE WET GRAY FINE SAND (SP)
50	P13	50/5"		VERY DENSE WET GRAY FINE SAND (SP)
55				

BOTTOM OF HOLE AT 51.0 FEET.
BORING CAVED AT 29.5 FEET.
WATER WAS ENCOUNTERED AT 29.5 FEET DURING DRILLING.
WATER LEVEL AT 28.0 FEET UPON COMPLETION OF DRILLING.

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
DATE: 02/14/06 **JOB NO:** 243906
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY

BORING NO: B4
LOCATION: SEE PLAN OF BORINGS
BORING TYPE: AUGER W/SPT
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube	NX Diamond Core	P Penetration Test
Core	Standard Penetration	J-Jar
Static Water Table	Hydrostatic Water Table	No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	16		MEDIUM DENSE MOIST BROWN CLAYEY SAND (SC)
	P2	11		MEDIUM DENSE MOIST BROWN CLAYEY SAND (SC)
5	P3	9		LOOSE MOIST LIGHT BROWN CLAYEY SAND (SC)
	P4	11		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
10	P5	17		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
15	P6	7		LOOSE MOIST LIGHT BROWN SILTY SAND (SM)
20	P7	11		MEDIUM DENSE MOIST LIGHT BROWN SILTY SAND (SM)
				BOTTOM OF HOLE AT 21.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
25				

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LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
DATE: 02/14/06 **JOB NO:** 243906
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY

BORING NO: B5
LOCATION: SEE PLAN OF BORINGS
BORING TYPE: AUGER W/SPT
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube; Core; Static Water Table; NX Diamond Core; Standard Penetration; Hydrostatic Water Table; P Penetration Test; J-Jar; No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	3		SOFT MOIST GRAY FAT CLAY (CH) PP = 0.50 TSF
	P2	8		MEDIUM STIFF MOIST GRAY FAT CLAY (CH) PP = 1.00 TSF
5	P3	18		VERY STIFF MOIST LIGHT GRAY AND BROWN FAT CLAY (CH) PP = 2.00 TSF
	P4	25		MEDIUM DENSE MOIST BROWN SILTY SAND (SM)
10	P5	29		MEDIUM DENSE MOIST BROWN SILTY SAND (SM)
15	P6	9		LOOSE MOIST BROWN SILTY SAND (SM)
20	P7	10		LOOSE MOIST DARK GRAY SILTY SAND (SM)
25				BOTTOM OF HOLE AT 21.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.

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LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
BORING NO: P1
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
LOCATION: SEE PLAN OF BORINGS
DATE: 02/14/06
JOB NO.: 243906
BORING TYPE: AUGER W/SPT
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube	NX Diamond Core	P Penetration Test
Core	Standard Penetration	J-Jar
Static Water Table	Hydrostatic Water Table	No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	5		MEDIUM STIFF MOIST GRAY AND BROWN FAT CLAY (CH) PP = 0.50 TSF
	P2	9		STIFF MOIST GRAY AND BROWN FAT CLAY (CH) PP = 1.00 TSF
5	P3	14		STIFF MOIST LIGHT GRAY AND BROWN FAT CLAY (CH) PP = 1.50 TSF
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
BORING NO: P2
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
LOCATION: SEE PLAN OF BORINGS
DATE: 02/14/06
JOB NO.: 243906
BORING TYPE: AUGER W/SPT
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
GROUND ELEVATION: NOT FURNISHED

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	4		SOFT MOIST GRAY AND BROWN FAT CLAY (CH) PP = 0.50 TSF
	P2	7		MEDIUM STIFF MOIST GRAY AND BROWN FAT CLAY (CH) PP = 0.75 TSF
5	P3	30		VERY STIFF MOIST LIGHT GRAY AND BROWN FAT CLAY (CH) PP = 3.00 TSF
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

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LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
BORING NO: P3
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
LOCATION: SEE PLAN OF BORINGS
DATE: 02/14/06
JOB NO.: 243906
BORING TYPE: AUGER W/SPT
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
GROUND ELEVATION: NOT FURNISHED

LEGEND

S Shelby Tube
NX Diamond Core
P Penetration Test
Core
Standard Penetration
J-Jar
Static Water Table
Hydrostatic Water Table
No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	6		LOOSE MOIST BROWN CLAYEY SAND (SC)
	P2	10		LOOSE MOIST BROWN CLAYEY SAND (SC)
5	P3	9		LOOSE MOIST BROWN CLAYEY SAND (SC)
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS
BORING NO: P4
FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION
LOCATION: SEE PLAN OF BORINGS
DATE: 02/14/06
JOB NO.: 243906
BORING TYPE: AUGER W/SPT
DRILLER: BRADBURY
SIMCO 2400
GEOTECHNICIAN: BRADBURY
GROUND ELEVATION: NOT FURNISHED

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	10		LOOSE MOIST BROWN CLAYEY SAND (SC)
	P2	16		MEDIUM DENSE MOIST BROWN CLAYEY SAND (SC)
5	P3	12		MEDIUM DENSE MOIST BROWN CLAYEY SAND (SC)
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

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LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS

FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION

BORING NO: P5

LOCATION: SEE PLAN OF BORINGS

DATE: 02/14/06 **JOB NO.:** 243906 **BORING TYPE:** AUGER W/SPT

DRILLER: BRADBURY
SIMCO 2400

GEOTECHNICIAN: BRADBURY

GROUND ELEVATION: NOT FURNISHED

LEGEND

- S Shelby Tube
- Core
- Static Water Table
- NX Diamond Core
- Standard Penetration
- Hydrostatic Water Table
- P Penetration Test
- J-Jar
- No Recovery

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	9		STIFF MOIST BROWN FAT CLAY (CH) PP = 1.00 TSF
	P2	7		MEDIUM STIFF MOIST BROWN FAT CLAY (CH) PP = 0.75 TSF
5	P3	11		STIFF MOIST BROWN FAT CLAY (CH) PP = 1.25 TSF
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

LOG OF BORING

PROJECT: PROPOSED NEWPORT INDUSTRIAL PARK
NEWPORT, ARKANSAS

FOR: NEWPORT ECONOMIC DEVELOPMENT COMMISSION

BORING NO: P6

LOCATION: SEE PLAN OF BORINGS

DATE: 02/14/06 **JOB NO.:** 243906 **BORING TYPE:** AUGER W/SPT

DRILLER: BRADBURY
SIMCO 2400

GEOTECHNICIAN: BRADBURY

GROUND ELEVATION: NOT FURNISHED

Depth In Feet	Sample Type & No.	N-Blows Per Foot	Graphic Symbol	VISUAL DESCRIPTION OF STRATUM
0	P1	3		SOFT MOIST BROWN FAT CLAY (CH) PP = 0.50 TSF
	P2	7		MEDIUM STIFF MOIST BROWN FAT CLAY (CH) PP = 0.75 TSF
5	P3	9		STIFF MOIST BROWN FAT CLAY (CH) PP = 1.00 TSF
				BOTTOM OF HOLE AT 6.5 FEET. BORING REMAINED OPEN. NO WATER WAS ENCOUNTERED IN THIS BORING.
10				

FIELD CLASSIFICATION SYSTEM
FOR SOIL EXPLORATION

NON COHESIVE SOILS

(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 0 - 4 blows/ft.
Loose	- 4 to 10 blows/ft.
Medium Dense	- 10 to 30 blows/ft.
Dense	- 30 to 50 blows/ft.
Very Dense	- over 50

Relative Proportions

Descriptive Term	Percent
Trace	1 - 10
Little	11 - 20
Some	21 - 35
And	36 - 50

Particle Size Identification

Boulders	- 8-inch diameter or more	
Cobbles	- 3 to 8-inch diameter	
Gravel	- Coarse	- 1 to 3-inch
	Medium	- ½ to 1-inch
	Fine	- ¼ to ½-inch
Sand	- Coarse	- 0.6 mm to ¼-inch (dia. of pencil lead)
	Medium	- 0.2 mm to 0.6 mm (dia. of broom straw)
	Fine	- 0.05 mm to 0.2 mm (dia. of human hair)
Silt		- 0.06 mm to 0.002 mm (Cannot see particles)

COHESIVE SOILS

(Clay, Silt and Combinations)

Consistency

Very Soft	- <2 blows/ft.
Soft	- 2 to 4 blows/ft.
Medium Stiff	- 4 to 8 blows/ft.
Stiff	- 8 to 15 blows/ft.
Very Stiff	- 15 to 30 blows/ft.
Hard	- over 30

Plasticity

Degree of Plasticity	Plasticity Index
None to slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	over 22

NOTES

Classification on logs are made by visual inspection.

Standard Penetration Test - Driving a 2.0-inch O.D., 1⅜-inch I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140-pound hammer free falling a distance of 30.0 inches. It is customary for AECI to drive the spoon 6.0 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6.0 inches of penetration on the drill log (Example: 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e., 8 + 9 = 17 blows/ft.).

Strata Changes - In the column "Soil Descriptions" on the drill log the horizontal lines represent strata changes. A solid line (------) represents an actually observed change, a dashed line (- - - -) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

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KEY TO SOIL CLASSIFICATIONS AND SYMBOLS

UNIFIED SOIL CLASSIFICATION SYSTEM(1)

Major Divisions		Letter	Symbol Hatching	Symbol Color	Name
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		RED	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP		RED	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM		YELLOW	Silty gravels, gravel-sand-silt mixtures
		GC		YELLOW	Clayey gravels, gravel-sand-clay mixtures
	SAND AND SANDY SOILS	SW		RED	Well-graded sands or gravelly sands, little or no fines
		SP		RED	Poorly-graded sands or gravelly sands, little or no fines
		SM		YELLOW	Silty sands, sand-silt mixtures
		SC		YELLOW	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS LL<50	ML		GREEN	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		GREEN	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		GREEN	Organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS LL>50	MH		BLUE	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		BLUE	Inorganic clays of high plasticity, fat clays
		OH		BLUE	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		Pt		ORANGE	Peat and other highly organic soils

TERMS CHARACTERIZING SOIL STRUCTURE(2)

SLICKENSIDED - having inclined planes of weakness that are slick and glossy in appearance.

FISSURED - containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.

LAMINATED (VARVED) - composed of thin layers of varying color and texture, usually grading from sand or silt at the bottom to clay at the top.

CRUMBLY - cohesive soils which break into small blocks or crumbs on drying.

CALCAREOUS - containing appreciable quantities of calcium carbonate, generally nodular.

WELL GRADED - having wide range in grain sizes and substantial amounts of all intermediate particle sizes.

POORLY GRADED - predominantly of one grain size (uniformly graded) or having a range of sizes with some intermediate size missing (gap or skip graded).

SYMBOLS FOR TEST DATA

M/C = 15 - Natural moisture content in percent.
γ = 95 - Dry unit weight in pounds/cubic foot.
Qu = 1.23 - Unconfined compression strength in tons/square foot.
Qc = 1.68 (21 psi) - Confined compression strength at indicated lateral pressure.
51-21-30 - Liquid limit, Plastic limit, and Plasticity index.
30% FINER - Percent finer than No. 200 mesh sieve.
30 B/F - Blows per foot, Standard Penetration test.
▼ - Hydrostatic water table.
▽ - Static water table.

TERMS DESCRIBING CONSISTENCY OF SOILS(2)

COARSE GRAINED SOILS		FINE GRAINED SOILS		
DESCRIPTIVE TERM	NO. BLOWS/FOOT STANDARD PEN. TEST	DESCRIPTIVE TERM	NO. BLOWS/FOOT STANDARD PEN. TEST	UNCONFINED COMPRESSION TONS PER SQ. FT.
Very Loose	0 - 4	Very Soft	<2	<0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Firm (medium dense)	10 - 30	Plastic (medium stiff)	4 - 8	0.50 - 1.00
Dense	30 - 50	Stiff	8 - 15	1.00 - 2.00
Very Dense	over 50	Very Stiff	15 - 30	2.00 - 4.00
		Hard	over 30	over 4.00

Field classification for "Consistency" is determined with a 0.25-inch diameter penetrometer.

(1) - From Waterways Experiment Station Technical Memorandum No. 3-357

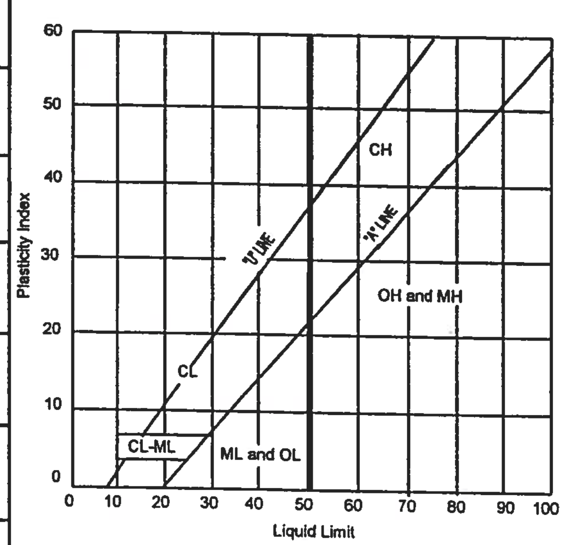
(2) - From "Soil Mechanics in Engineering Practice" by Terzaghi and Peck

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 2487)

Major divisions			Group Symbols	Typical Names	Laboratory Classifications Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (more than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent ....GW, GP, SW, SP More than 12 percent ....GM, GC, SM, SC 5 to 12 percent ....Borderline cases requiring dual symbols **	$C_u = \frac{D_{60}}{D_{10}}$ *greater than* 4; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ *between* 1 *and* 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM* d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. Between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ *greater than* 6; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ *between* 1 *and* 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM* d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 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 and clays (Liquid limit greater than 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 silts			
	Highly Organic soils		Pt	Peat and other highly organic soils			



Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u are for roads and airfield only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; u used when L.L. is greater than 24.
**Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

Design Calculations for Conventional Footings

PROJECT: Proposed Industrual Park
PROJECT NO.: 243906 DATE: 04/06/06
BORING NO.: AVG N TESTED BY: AECI SAFETY FACTOR: 2.00

Df ft	Depth - from	ft. to	STRATA H - ft	N B/F	Qu KSF	Qu/2 KSF	1.25Qu KSF	.125Df KSF	Qa KSF
1.5	0.0	1.5	1.5	7	1.9	0.9	2.3	0.188	1.3
4.0	1.5	4.0	2.5	9	2.4	1.2	3.0	0.500	1.7
6.5	4.0	6.5	2.5	13	3.4	1.7	4.3	0.813	2.6
9.0	6.5	9.0	2.5	17	4.5	2.2	5.6	1.125	3.4
11.5	9.0	11.5	2.5	18	4.7	2.4	5.9	1.438	3.7
16.5	11.5	16.5	5.0	11	2.9	1.5	3.6	2.063	2.9
21.5	16.5	21.5	5.0	12	3.2	1.6	4.0	2.688	3.3
26.5	21.5	26.5	5.0	14	3.7	1.8	4.6	3.313	4.0
31.5	26.5	31.5	5.0	18	4.7	2.4	5.9	3.938	4.8
36.5	31.5	36.5	5.0	22	5.8	2.9	7.3	4.563	5.6
41.5	36.5	41.5	5.0	32	8.8	4.4	11.0	5.188	7.7
46.5	41.5	46.5	5.0	39	11.2	5.6	14.0	5.813	9.3
51.5	46.5	51.5	5.0	33	9.1	4.6	11.4	6.438	8.2

WATER TABLE LEVEL: 27.5 ft.

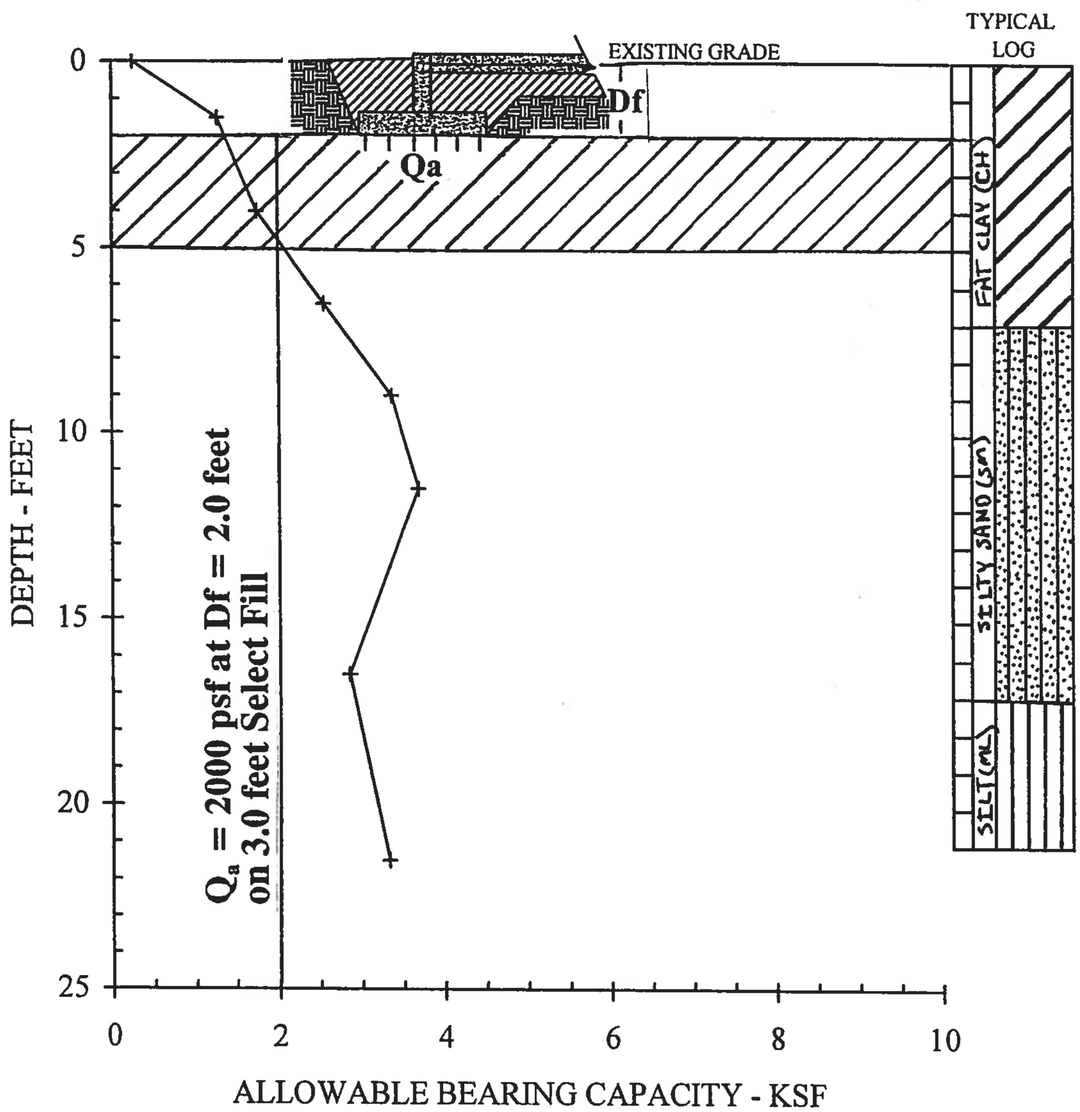
ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

CONVENTIONAL FOOTINGS

PROJECT: **Proposed Industrual Park**
Newport, Arkansas

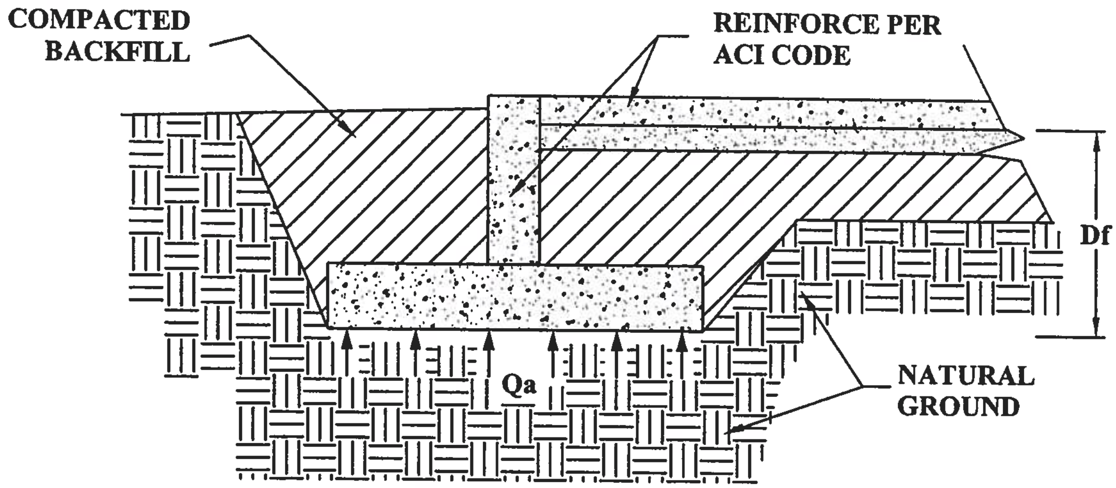
BORING NO.: **AVG N**

PROJECT NO.: **243906** WATER TABLE: **27.5 ft.** SAFETY FACTOR: **2.00**



DEPTH - BEARING CAPACITY CURVE

CONVENTIONAL FOOTINGS



Explanation of Calculations Shown in Tables

D_f = Depth from ground surface to bottom of footing (feet)

Depth = Depth from top to bottom of soil strata (feet)

Strata H = Thickness of soil strata (feet)

N = Standard penetration N-value (blows per foot)

Q_u = Ultimate soil strength (ksf)

$1.25\ Q_u$ = Soil Strength parameter (ksf)

$0.125\ D_f$ = Depth factor (ksf)

Q_a = Allowable bearing capacity = $(1.25\ Q_u + 0.125\ D_f) \div$ Safety Factor (ksf)

EXPLANATION OF BEARING CAPACITY CALCULATIONS

APPENDIX B

SUPPORTING LABORATORY DATA

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

ATTERBERG LIMIT DETERMINATION
ASTM D 4318

Project: PROPOSED INDUSTRIAL PARK **Date:** 02/22/06
Location: NEWPORT, ARKANSAS **Job No.:** 243906

LIQUID LIMIT

Sample Number	B1;P2	B1;P3	B1;P7	B2;P3	B3;P2	B3;P5
Tare Number	16	46	NON - PLASTIC	81	NON - PLASTIC	NON - PLASTIC
Number of Blows	25	24		24		
Tare + Wet Soil (g)	24.70	41.73		25.94		
Tare + Dry Soil (g)	20.08	37.70		20.94		
Tare (g)	13.86	31.02		14.45		
Water (g)	4.62	4.03		5.00		
Dry Soil (g)	6.22	6.68		6.49		
Water Content (%)	74.28	60.33		77.04		
Liquid Limit	74	60	NP	77	NP	NP

PLASTIC LIMIT

Sample Number	B1;P2	B1;P3	B1;P7	B2;P3	B3;P2	B3;P5
Tare Number	9	28	NON - PLASTIC	4	NON - PLASTIC	NON - PLASTIC
Tare + Wet Soil (g)	14.82	32.66		21.50		
Tare + Dry Soil (g)	14.55	32.45		21.26		
Tare (g)	13.53	31.62		20.34		
Water (g)	0.27	0.21		0.24		
Dry Soil (g)	1.02	0.83		0.92		
Water Content (%)	26.47	25.30		26.09		
Plastic Limit	26	25		26		
Plasticity Index	**48**	**35**		**51**		
Classification (#40)	CH	CH	NP	CH	NP	NP

LIQUID LIMIT

Sample Number	B3;P10	B4;P3	B4;P6	B5;P2	B5;P3
Tare Number	NON - PLASTIC	NON - PLASTIC	NON - PLASTIC	11	37
Number of Blows				25	25
Tare + Wet Soil (g)				41.39	45.51
Tare + Dry Soil (g)				36.60	41.94
Tare (g)				30.69	30.90
Water (g)				4.79	3.57
Dry Soil (g)				5.91	11.04
Water Content (%)				81.05	32.34
Liquid Limit	NP	NP	NP	81	32

PLASTIC LIMIT

Sample Number	B3;P10	B4;P3	B4;P6	B5;P2	B5;P3
Tare Number	NON - PLASTIC	NON - PLASTIC	NON - PLASTIC	45	31
Tare + Wet Soil (g)				32.56	33.03
Tare + Dry Soil (g)				32.20	32.74
Tare (g)				30.81	31.14
Water (g)				0.36	0.29
Dry Soil (g)				1.39	1.60
Water Content (%)				25.90	18.12
Plastic Limit				26	18
Plasticity Index				**55**	**14**
Classification (#40)	NP	NP	NP	CH	CL

MECHANICAL GRAIN SIZE ANALYSES
ASTM D 422

Project: PROPOSED INDUSTRIAL PARK
Location: NEWPORT, ARKANSAS
Sample No.: B1;P7
Soil Description: SANDY SILT

Project No.: 243906
Date: 02/23/06
Sample Depth: 20.0-21.5

Sieve or Screen	Weight Retained (grams)	Cumulative Weight Retained (grams)	Percent Retained	Percent Passing
3"	0.0	0.0	0.0	100.0
3/4"	0.0	0.0	0.0	100.0
#4	0.0	0.0	0.0	100.0
#10	0.0	0.0	0.0	100.0
#40	0.1	0.1	0.0	100.0
#200	23.8	23.8	12.8	87.2
PAN	162.2	186.0	100.0	0.0

Percent Sample Gravel: 0.0
Percent Sample Sand: 12.8
Percent Sample Silt/Clay: 87.2

Sample Weight: 186.0g
Washing Loss: 162.2g

Project: PROPOSED INDUSTRIAL PARK
Location: NEWPORT, ARKANSAS
Sample No.: B3;P2
Soil Description: SILTY SAND

Project No.: 243906
Date: 02/23/06
Sample Depth: 5.0-6.5

Sieve or Screen	Weight Retained (grams)	Cumulative Weight Retained (grams)	Percent Retained	Percent Passing
3"	0.0	0.0	0.0	100.0
3/4"	0.0	0.0	0.0	100.0
#4	0.0	0.0	0.0	100.0
#10	0.0	0.0	0.0	100.0
#40	0.1	0.1	0.0	100.0
#200	206.9	207.0	71.9	28.1
PAN	81.0	288.0	100.0	0.0

Percent Sample Gravel: 0.0
Percent Sample Sand: 71.9
Percent Sample Silt/Clay: 28.1

Sample Weight: 288
Washing Loss: 81.0g

ANDERSON ENGINEERING CONSULTANTS, INC.
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MECHANICAL GRAIN SIZE ANALYSES
ASTM D 422

Project: Proposed Industrial Park — **Project No.:** 243906
Location: Newport, Arkansas — **Date:** 02/23/06
Sample No.: B3;P5 — **Sample Depth:** 10-11.5 ft.
Soil Description: Silty Sand

Sieve or Screen	Weight Retained (grams)	Cumulative Weight Retained (grams)	Percent Retained	Percent Passing
3"	0.0	0.0	0.0	100.0
3/4"	0.0	0.0	0.0	100.0
#4	0.4	0.4	0.2	99.8
#10	0.4	0.8	0.4	99.6
#40	- 0.7	0.1	0.0	100.0
#200	185.0	185.1	86.5	13.5
PAN	28.9	214.0	100.0	0.0

Percent Sample Gravel: 0.2
Percent Sample Sand: 86.3
Percent Sample Silt/Clay: 13.5

Sample Weight: 214.0g
Washing Loss: 28.9g

Project: Proposed Industrial Park — **Project No.:** 243906
Location: Newport, Arkansas — **Date:** 02/23/06
Sample No.: B4;P6 — **Sample Depth:** 15-16.5 ft.
Soil Description: Silty Sand

Sieve or Screen	Weight Retained (grams)	Cumulative Weight Retained (grams)	Percent Retained	Percent Passing
3"	0.0	0.0	0.0	100.0
3/4"	0.0	0.0	0.0	100.0
#4	0.0	0.0	0.0	100.0
#10	0.0	0.0	0.0	100.0
#40	0.2	0.2	0.1	99.9
#200	188.4	188.6	89.8	10.2
PAN	21.4	210.0	100.0	0.0

Percent Sample Gravel: 0.0
Percent Sample Sand: 89.8
Percent Sample Silt/Clay: 10.2

Sample Weight: 210.0g
Washing Loss: 21.4g

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MECHANICAL GRAIN SIZE ANALYSES
ASTM D 422

Project: PROPOSED INDUSTRIAL PARK
Location: NEWPORT, ARKANSAS
Sample No.: B5;P2
Soil Description: LIGHT BROWN FAT CLAY

Project No.: 243906
Date: 02/23/06
Sample Depth: 2.5-4.0

Sieve or Screen	Weight Retained (grams)	Cumulative Weight Retained (grams)	Percent Retained	Percent Passing
3"	0.0	0.0	0.0	100.0
3/4"	0.0	0.0	0.0	100.0
#4	0.0	0.0	0.0	100.0
#10	0.0	0.0	0.0	100.0
#40	0.4	0.4	0.2	99.8
#200	52.7	53.1	22.4	77.6
PAN	183.9	237.0	100.0	0.0

Percent Sample Gravel: 0.0
Percent Sample Sand: 22.4
Percent Sample Silt/Clay: 77.6

Sample Weight: 237.0g
Washing Loss: 183.9g

ANDERSON ENGINEERING CONSULTANTS, INC.
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UNCONFINED COMPRESSION TEST
ASTM D 2166

Project:	PROPOSED INDUSTRIAL PARK	**Project No.:**	243906
Location:	NEWPORT, ARKANSAS	**Date:**	02/14/06
Sample No.:	B1;P3	**Sample Depth:**	5.0-6.5 FT
Soil Description:	GRAY FAT CLAY	**K PRC:**	2.0

Tare Number............	19	Height....................................	2.800 in
Tare + Wet Soil.........	156.1 g	Avg. Diameter........................	1.400 in
Tare + Dry Soil	123.2 g	Gs (Estimated)........................	2.67
Tare........................	31.2 g	Trimmed Sample Total Weight....	124.91 g
Water........................	32.9 g	Initial Area.............................	9.931 sq.cm
Wet Soil.....................	124.9 g	Void Ratio = (Vo-Vs)/Vs...........	1.051
Dry Soil	91.9 g	Saturation = GsWo/Eo................	91.1 %
Water Content...........	**35.8 %**	**Dry Density = 62.4(Ws/Vo)........**	**81.2 pcf**
		Wet Density = 62.4(W/Vo).........	**110.4 pcf**

Deflection Dial Reading (10^{-3}in)	Proving Ring Dial Reading (10^{-4}in)	Axial Load (lbs)	Axial Strain $\in = \Delta H/Ho$	$A_{corr} = Ao/(1-\in)$ (sq.cm)	Compressive Strength $0.93(P/A_{corr})$ (ksf)
10	5.0	10.0	0.0034	9.965	0.9
20	1.0	2.0	0.0071	10.003	0.2
30	1.0	2.0	0.0107	10.039	0.2
40	1.5	3.0	0.0142	10.075	0.3
50	2.0	4.0	0.0178	10.111	0.4
60	2.5	5.0	0.0213	10.148	0.5
70	3.0	6.0	0.0249	10.185	0.5
80	4.0	8.0	0.0284	10.222	0.7
90	5.0	10.0	0.0320	10.259	0.9
100	6.0	12.0	0.0355	10.297	1.1
110	7.0	14.0	0.0390	10.335	1.3
120	8.0	16.0	0.0426	10.373	1.4
130	8.0	16.0	0.0461	10.412	1.4
140	8.0	16.0	0.0497	10.451	1.4
150	7.5	15.0	0.0533	10.491	1.3
160	7.5	15.0	0.0569	10.530	1.3

QuMax = 1.4 ksf at Strain = 4% ±

P.P.= 1.25 ksf

Type of Failure: **BULGE**

BULGE

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

UNCONFINED COMPRESSION TEST
ASTM D 2166

Project:	PROPOSED INDUSTRIAL PARK	**Project No.:**	243906
Location:	NEWPORT, ARKANSAS	**Date:**	02/14/06
Sample No.:	B2;P2	**Sample Depth:**	2.5-4.0 FT
Soil Description:	GRAY FAT CLAY	**K PRC:**	2.0

Tare Number............	45	Height................................	2.800 in
Tare + Wet Soil.........	149.7 g	Avg. Diameter......................	1.400 in
Tare + Dry Soil	110.8 g	Gs (Estimated)........................	2.72
Tare........................	30.8 g	Trimmed Sample Total Weight....	119.08 g
Water........................	38.9 g	Initial Area............................	9.931 sq.cm
Wet Soil....................	118.9 g	Void Ratio = (Vo-Vs)/Vs..........	1.398
Dry Soil	80.0 g	Saturation = GsWo/Eo...............	94.6 %
Water Content..........	**48.6 %**	**Dry Density = 62.4(Ws/Vo).......**	**70.8 pcf**
		Wet Density = 62.4(W/Vo).........	**105.2 pcf**

Deflection Dial Reading (10^{-3}in)	Proving Ring Dial Reading (10^{-4}in)	Axial Load (lbs)	Axial Strain $\in = \Delta H/Ho$	$A_{corr} = Ao/(1-\in)$ (sq.cm)	Compressive Strength $0.93(P/A_{corr})$ (ksf)
10	1.0	2.0	0.0035	9.967	0.2
20	2.0	4.0	0.0071	10.002	0.4
30	3.0	6.0	0.0106	10.038	0.6
40	3.0	6.0	0.0142	10.074	0.6
50	4.0	8.0	0.0177	10.111	0.7
60	5.0	10.0	0.0213	10.147	0.9
70	5.0	10.0	0.0248	10.184	0.9
80	6.0	12.0	0.0284	10.221	1.1
90	6.0	12.0	0.0319	10.259	1.1
100	6.0	12.0	0.0355	10.297	1.1
110	7.0	14.0	0.0390	10.335	1.3
120	7.0	14.0	0.0426	10.373	1.3
130	7.0	14.0	0.0462	10.412	1.3
140	7.0	14.0	0.0498	10.451	1.2
150	7.0	14.0	0.0533	10.491	1.2
160	6.5	13.0	0.0569	10.531	1.1

QuMax = 1.3 ksf at Strain = 3.9

Type of Failure: **BULGE**

P.P.= 1.00 ksf

BULGE

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

UNCONFINED COMPRESSION TEST
ASTM D 2166

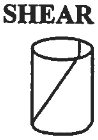
Project:	PROPOSED INDUSTRIAL PARK	**Project No.:**	243906
Location:	NEWPORT, ARKANSAS	**Date:**	02/14/06
Sample No.:	B4;P2	**Sample Depth:**	2.5-4.0 FT
Soil Description:	GRAY SANDY CLAY	**K PRC:**	2.0

Tare Number............	47	Height....................................	2.870 in
Tare + Wet Soil.........	165.2 g	Avg. Diameter..........................	1.400 in
Tare + Dry Soil	138.3 g	Gs (Estimated)............................	2.68
Tare.........................	31.4 g	Trimmed Sample Total Weight....	133.91 g
Water........................	27.0 g	Initial Area................................	9.931 sq.cm
Wet Soil..................	133.8 g	Void Ratio = (Vo-Vs)/Vs...........	0.815
Dry Soil	106.8 g	Saturation = GsWo/Eo................	83.0 %
Water Content...........	**25.2 %**	**Dry Density = 62.4(Ws/Vo)........**	**92.2 pcf**
		Wet Density = 62.4(W/Vo).........	**115.4 pcf**

Deflection Dial Reading (10^{-3}in)	Proving Ring Dial Reading (10^{-4}in)	Axial Load (lbs)	Axial Strain $\in = \Delta H/Ho$	A_{corr} $= Ao/(1-\in)$ (sq.cm)	Compressive Strength $0.93(P/A_{corr})$ (ksf)
10	1.5	3.0	0.0034	9.966	0.3
20	2.5	5.0	0.0069	10.000	0.5
30	4.0	8.0	0.0103	10.035	0.7
40	5.5	11.0	0.0137	10.070	1.0
50	8.0	16.0	0.0171	10.105	1.5
60	8.5	17.0	0.0206	10.140	1.6
70	9.5	19.0	0.0241	10.176	1.7
80	10.0	20.0	0.0275	10.213	1.8
90	10.5	21.0	0.0310	10.249	1.9
100	11.0	22.0	0.0345	10.286	2.0
110	11.5	23.0	0.0379	10.323	2.1
120	12.0	24.0	0.0414	10.360	2.2
130	12.0	24.0	0.0449	10.398	2.1

QuMax = 2.2 ksf at Strain = 4% ±
P.P.= 2.00 ksf

Type of Failure: **70 DEGREE SHEAR**



SHEAR

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

UNCONFINED COMPRESSION TEST
ASTM D 2166

Project: PROPOSED INDUSTRIAL PARK
Location: NEWPORT, ARKANSAS
Sample No.: B5;P3
Soil Description: SANDY CLAY

Project No.: 243906
Date: 02/14/06
Sample Depth: 5.0-6.5 FT
K PRC: 2.0

Tare Number	12
Tare + Wet Soil	159.6 g
Tare + Dry Soil	128.4 g
Tare	30.9 g
Water	31.2 g
Wet Soil	128.7 g
Dry Soil	97.5 g
Water Content	**32.0 %**

Height	2.800 in
Avg. Diameter	1.400 in
Gs (Estimated)	2.67
Trimmed Sample Total Weight	128.96 g
Initial Area	9.931 sq.cm
Void Ratio = (Vo-Vs)/Vs	0.930
Saturation = GsWo/Eo	91.9 %
Dry Density = 62.4(Ws/Vo)	**86.3 pcf**
Wet Density = 62.4(W/Vo)	**113.9 pcf**

Deflection Dial Reading (10^{-3}in)	Proving Ring Dial Reading (10^{-4}in)	Axial Load (lbs)	Axial Strain $\in = \Delta H/Ho$	$A_{corr} = Ao/(1-\in)$ (sq.cm)	Compressive Strength $0.93(P/A_{corr})$ (ksf)
10	2.5	5.0	0.0035	9.966	0.5
20	4.5	9.0	0.0070	10.001	0.8
30	8.0	16.0	0.0104	10.036	1.5
40	8.5	17.0	0.0140	10.072	1.6
50	10.0	20.0	0.0175	10.108	1.8
60	10.5	21.0	0.0211	10.145	1.9
70	11.0	22.0	0.0246	10.182	2.0
80	11.5	23.0	0.0282	10.219	2.1
90	12.0	24.0	0.0317	10.257	2.2
100	12.0	24.0	0.0353	10.295	2.2
110	12.0	24.0	0.0389	10.333	2.2
120	12.0	24.0	0.0424	10.372	2.2
130	11.5	23.0	0.0460	10.411	2.1
140	11.5	23.0	0.0496	10.450	2.0
150	11.0	22.0	0.0532	10.489	2.0
160	11.0	22.0	0.0568	10.529	1.9

QuMax = 2.2 ksf at Strain = 3.2
P.P.= 2.25 ksf

Type of Failure: **BULGE**


BULGE

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

SHRINKAGE / SWELL INDEX TESTS

Project:	Proposed Industrial Park	**Project No.:**	243906
Location:	Newport, Arkansas	**Date:**	02/26/06
Sample No.:	B1;P2	**Sample Depth:**	2.5-4.0 ft
Soil Description:	Dark Gray Fat Clay	**K PRC:**	2.0

Liquid Limit:	74	**Est. Specific Gravity:**	2.67
Plastic Limit:	26	**No. of Layers:**	4
Plasticity Index:	48	**No. Blows/Layer:**	7

WATER CONTENT

	Before Test		After Test
Tare Number	47	Tare Number	38
Tare + Wet Soil	40.7 g	Tare + Wet Soil	174.0 g
Tare + Dry Soil	39.2 g	Tare + Dry Soil	147.3 g
Tare	31.4 g	Tare	52.4 g
Water Content	19.0 %	Water Content	28.1 %
Saturation	73.1 %	Saturation	100.0 %
Dry Density	98.3 pcf	Dry Density	97.8 pcf

VOID RATIO DETERMINATION

Vo	60.801 ccm	Vf	60.976 ccm
Wt of Soil + Ring	356.7 g	Wt of Soil + Ring	365.1 g
Wt of Ring	242.6 g	Wt of Ring	242.6 g
Moist Wt of Soil	114.1 g	Moist Wt of Soil	122.5 g
Vs	35.898 ccm	Vs	35.898 ccm
Eo	0.6937	Ef	0.6986

SWELL DATA

Time	Dial (* 0.0001)	Pressure	Void Ratio
16.00	0.00	0.0	0.6937
31.00	15.00	2079.0	0.6978
41.00	18.00	2494.8	0.6986

Final Dial Reading: 18.00

Swell Pressure: **2,495 PSF**

Heave = 0.288 % = 0.0346 inches/foot

SHRINKAGE DATA

Linear Shrinkage (Bar Method):

Linear Shrinkage: **14.0 %**

Volumetric Shrinkage: **36.4 %**

ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE, JONESBORO, ARKANSAS 72401

SHRINKAGE / SWELL INDEX TESTS

Project:	Proposed Industrial Park	**Project No.:**	243906
Location:	Newport, Arkansas	**Date:**	02/26/06
Sample No.:	B2;P3	**Sample Depth:**	5.0-6.5 ft
Soil Description:	Light Gray Fat Clay	**K PRC:**	2.0

Liquid Limit:	77	**Est. Specific Gravity:**	2.71
Plastic Limit:	26	**No. of Layers:**	4
Plasticity Index:	51	**No. Blows/Layer:**	7

WATER CONTENT

	Before Test		After Test
Tare Number	8	Tare Number	1
Tare + Wet Soil	57.3 g	Tare + Wet Soil	323.3 g
Tare + Dry Soil	50.4 g	Tare + Dry Soil	292.1 g
Tare	31.4 g	Tare	214.6 g
Water Content	36.1 %	Water Content	40.2 %
Saturation	93.0 %	Saturation	100.0 %
Dry Density	82.4 pcf	Dry Density	81.3 pcf

VOID RATIO DETERMINATION

Vo	60.801 ccm	Vf	60.874 ccm
Wt of Soil + Ring	351.8 g	Wt of Soil + Ring	353.7 g
Wt of Ring	242.5 g	Wt of Ring	242.5 g
Moist Wt of Soil	109.3 g	Moist Wt of Soil	111.2 g
Vs	29.647 ccm	Vs	29.647 ccm
Eo	1.0509	Ef	1.0533

SWELL DATA

Time	Dial (* 0.0001)	Pressure	Void Ratio
16.30	0.00	0.0	1.0509
19.00	4.00	554.4	1.0522
31.00	7.00	970.2	1.0532
33.30	7.50	1039.5	1.0533

Final Dial Reading: 7.50

Heave = 0.120 % = 0.0144 inches/foot

Swell Pressure: **1,040 PSF**

SHRINKAGE DATA

Linear Shrinkage (Bar Method):

Linear Shrinkage: **12.0 %**

Volumetric Shrinkage: **31.9 %**