# REPORT OF
# PRELIMINARY SUBSURFACE EXPLORATION

## PROPOSED NATIONAL ENRICHMENT FACILITY
## LEA COUNTY, NEW MEXICO

**Prepared for:**

**LOCKWOOD GREENE**

**Spartanburg, South Carolina**

**Prepared By:**

**MACTEC ENGINEERING AND CONSULTING, INC.**

**Knoxville, Tennessee**

**MACTEC Project 3043031049/0001**

**October 17, 2003**

MACTEC

MACTEC

October 17, 2003

Mr. Philip Clarkson
Lockwood Greene
1500 International Drive
Spartanburg, SC 29304

Subject: **Report of Preliminary Subsurface Exploration**
**Proposed National Enrichment Facility**
**Lea County, New Mexico**
**MACTEC Project 3043031049/0001**

Dear Mr. Clarkson:

We at MACTEC Engineering and Consulting, Inc., (MACTEC) are pleased to submit this Report of Preliminary Subsurface Exploration for your project. Our services, as authorized by you, were provided in general accordance with our proposal number Prop03Knox/404, Revision 2, dated September 2, 2002.

The purpose of this preliminary exploration was to develop information about the site and subsurface conditions that could be used for assistance in determining the feasibility of constructing the proposed facilities at the site. This report describes the work performed, and presents the results obtained and our preliminary geotechnical exploration.

The preliminary exploration does not provide the necessary information to complete the design of the facilities. When specific project details concerning building and pavement locations and the foundation loads and site grades are developed, subsequent and more detailed exploration and analysis will be necessary to provide the final geotechnical design parameters.

Thank you for the opportunity to provide our professional geotechnical services during this phase of your project. We will be pleased to discuss our recommendations with you and would welcome the opportunity to provide the geotechnical engineering services needed to successfully complete your project.

Sincerely,

MACTEC ENGINEERING AND CONSULTING, INC.

Matthew B. Haston
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## EXECUTIVE SUMMARY

We were selected by Lockwood Greene to perform a preliminary geotechnical exploration for the proposed National Enrichment Facility. The proposed project site is located north of New Mexico Highway 234 in the western portion of Lea County, New Mexico. The objectives of our preliminary exploration were to develop information about the site and subsurface conditions that could be used for assistance in determining the feasibility of constructing the proposed facilities at the site.

The exploration consisted of drilling five test borings in the proposed building area to depths of 40 to 100 feet. The major findings and recommendations of our subsurface exploration are as follows:

- Underlying an approximately two-foot veneer of loose eolian sands, the soil test borings encountered Quaternary age firm to very dense silty sands. Zones of rocklike calcium carbonate cemented soils (caliche) were encountered at varying intervals within this zone. Underlying the upper sandy soils, Triassic age very hard clays were encountered at depths of 35 to 40 feet. Each of the borings terminated in the clay soils at their predetermined depths.

- Ground water was not encountered in the borings performed for this exploration at the time of drilling. Based upon information from other explorations at this site and adjacent sites, we do not expect ground water to present difficulty during construction considering the preliminary maximum excavation depths.

- Shallow spread foundations bearing in the firm and better sandy soils such as those encountered in the soil test borings may be preliminarily designed using the allowable bearing pressure of 7,000 pounds per square foot.

- The upper loose eolian sands are not likely suitable for direct shallow foundation support or for the subgrade support of new engineered fills, pavements, or grade slabs. Therefore, we recommend that where encountered within building areas, these upper loose sands be stripped from the site during mass grading.

This summary is only an overview and should not be used as a separate document or in place of reading the entire report, including the appendices. Further, this is a preliminary exploration and does not provide the information for the final design of the proposed facilities. Additional field exploration, engineering analysis, laboratory and field testing may be required to develop the final geotechnical design parameters. We recommend the owner retain MACTEC to provide these services based on our familiarity with the project, the subsurface conditions, the intent of the preliminary recommendations, and our experience in this area.

## 1.0 OBJECTIVES OF EXPLORATION

The objective of this preliminary exploration was to develop information about the site and subsurface conditions that could be used for assistance in determining the feasibility of constructing the proposed facilities at the site and to aid in the development of a construction cost estimate and design basis. The preliminary exploration does not provide the necessary information to complete the design of the facilities. When specific project details concerning building, pavement, and railroad spur locations and the foundation loads and site grades are developed, subsequent and more detailed exploration and analysis will be necessary provide the final geotechnical design parameters. An assessment of site environmental conditions or an assessment for the presence of pollutants in the soil, rock, surface water, or ground water of the site was beyond the proposed scope of this exploration.

## 2.0 SCOPE OF EXPLORATION

The scope of services for this preliminary exploration has included a site reconnaissance, drilling the five requested soil test borings at the locations selected by Lockwood Greene within the proposed project area, and visually classifying the soil samples obtained from the standard penetration testing.

We collected two undisturbed and six bulk samples in conjunction with the drilling for laboratory testing. California bearing ratio (CBR), compaction, Atterberg limits, grain size, and moisture content laboratory tests were conducted on selected bulk samples to evaluate their suitability for use as engineered fill and to determine a representative CBR value for preliminary design purposes. Resistivity, pH, Atterberg limits, grain size, and moisture content laboratory tests were conducted on selected samples obtained during the standard penetration testing to evaluate the index properties of the site soils, aid in the classification of the soil type, and to assist in the evaluation of the corrosion potential of the site soils.

## 3.0 PROJECT INFORMATION AND SITE CONDITIONS

Project information was provided in a "Request for Proposal" (RFP) from Mr. Philip Clarkson of Lockwood Greene, dated August 18, 2003. Also provided was a drawing entitled "Conceptual Site Plan" (Drawing SKC-006), by Lockwood Greene, dated September 2, 2003. The drawing shows the preliminary location of the proposed facilities in relation to existing site features and existing

topography. Lockwood Greene also provided a copy of a geotechnical exploration performed by others at a nearby site for informational purposes.

The proposed project is to consist of the construction of a National Enrichment Facility (NEF) north of Highway 234 just west of the New Mexico-Texas state line, near Eunice, New Mexico. The NEF is to be comprised of several building modules which will cover a combined total area in excess of 1,000,000 square feet. The individual building modules are to be of reinforced concrete frame construction with precast concrete exterior wall panels. The walls will act as shear walls providing lateral support for the structure. Based upon the provided drawing, several individual concrete storage pads are to be constructed to the north of the building area. These concrete pads cover an area having plan dimensions of approximately 1,000 by 2,000 feet. Also proposed for construction as part of the NEF are various paved access drives and parking areas, above ground storage tanks, cooling towers, and ancillary security and visitor center structures.

Maximum individual total column foundation loads are reportedly in the range of 400 to 1,300 kips. We understand much of the total column load is dead load. The project is in the preliminary planning phases at this time and information regarding finished grade elevations of the proposed facility has not been provided. Based upon the provided information, the finished floor is to be at about Elev.3414 feet. Site grades within the proposed project grading area range from Elev. 3402 to 3424 feet. We understand the proposed construction will not include basements or subsurface pits of more than five feet in depth.

Based upon a review of the provided topographic drawing, site grades range from about Elev. 3455 feet in the northeast corner of Section 33 down to about Elev. 3380 feet in the southwest corner of Section 32. The site is covered by sparse vegetation consisting of grasses, brush, and cacti. Surficial soils are loose sands. An existing unimproved road crosses the central portion of the site in a north-south orientation. A subsurface pipeline crosses the site in a northwest-southeast orientation.

## 4.0 AREA AND SITE GEOLOGY

### 4.1 PHYSICAL SETTING

The topography in southeastern New Mexico generally consists of broad plains and gently rolling hills with locally some bluffs and shallow river valleys. The geologic structure is relatively simple,

generally consisting of flat-lying to gently warped sedimentary rocks ranging in age from Permian to Pliocene.

The site is located in the Great Plains physiographic province. In the project area, the Great Plains province is divided into two sections, the Pecos Valley section and the Southern High Plains section (Doleman, 1997). Mescalero Ridge, an escarpment (steep cliff) located about ½ mile to the northeast of the project site, is considered the boundary between the two sections. The primary difference between the two sections is the topography: the topographically lower Pecos Valley section is characterized by an irregular erosional surface while the topographically higher Southern High Plains section is a large flat mesa that slopes very gently to the southeast (Doleman, 1997).

The site is located in the Pecos Valley section of the Great Plains. Topography in this area was formed by erosion of the Tertiary age fluvial deposits and localized exposure of the underlying Mesozoic and Paleozoic rocks (Doleman, 1997). In the project area, the topography slopes gently to the southwest (at a gradient of approximately 15 feet per mile) and the surface geology is characterized by Quaternary age eolian (wind blown) deposits that mantle the underlying Quaternary and Tertiary age sediments. Local variations in local topography are reflective of the thickness and distribution of the eolian deposits.

## 4.2 REGIONAL GEOLOGY

The southeastern portion of New Mexico (and adjacent West Texas) is located in a geologically stable area known as the Permian Basin. This large subsurface structural basin, named for the geologic period in which it was formed, is a broad, down-warped area filled in with thick sequences of sedimentary rocks. During the late Cretaceous to early Tertiary time, tectonic uplift (mountain-building processes) to the west of the Permian Basin resulted in a structural high in the area that is now the Rocky Mountains and the southern extension of the Rocky Mountains (including the mountain ranges to the west of the project area). Erosion of this structural high provided the source area for sediments that now make up the younger Tertiary and Quaternary age formations that are locally exposed in the site vicinity.

## 4.3 LOCAL GEOLOGIC CONDITIONS

### 4.3.1 General

In the project area, the bedrock is relatively shallow (within 40 feet of the ground surface) and consists of sedimentary rocks of the Triassic age Chinle Formation. The Chinle Formation bedrock consists of a thick sequence of massive, unsaturated red, reddish purple or green claystone and siltstone with some localized fine-grained sandstone interbeds. The bedrock is overlain by approximately 35 to 40 feet of Quaternary age alluvial sediments. Based on published geologic maps and geotechnical reports for other projects in the area, the Quaternary age materials are part of the Gatuna Formation and consist of moderately cemented sand and gravel. The Gatuna Formation materials are mantled with a thin veneer of eolian (wind blown sand).

### 4.3.2 Geologic Materials

The materials encountered in our geotechnical borings at the site consist of Quaternary age eolian sand (wind-blown dune deposits) that is predominantly dry reddish brown silty sand. The upper loose eolian deposits were generally encountered to depths of up to two feet below existing ground surface in the exploratory borings. However, in areas where sand dunes have formed, the actual depth of the eolian soils is likely more than 2 feet.

Quaternary age alluvial deposits of the Gatuna Formation underlie the eolian deposits. As encountered in our borings, these alluvial deposits consist of dry, light yellow to reddish yellow, dense to very dense sand and silty sand. The sand is fine- to medium-grained, slightly to moderately cemented, and locally contains subangular to rounded gravel and caliche.

Dark red and purple, very hard high plasticity clay (claystone) of the Triassic age Chinle Formation uncomformably underlies the dense Gatuna Formation materials. As encountered in our borings, the Chinle Formation bedrock is at a depth of about 35 feet at the location of Boring B-2 and at a depth of about 40 feet at the location of Borings B-1, B-3, B-4, and B-5. In Boring B-3, drilled to a depth of approximately 100 feet beneath the existing ground surface, the very hard high plasticity clay was encountered from a depth of 40 feet to the maximum depth drilled (100.5 feet).

### 4.3.3 Ground-Water Conditions

Ground water was not encountered in the borings drilled at the site as part of our investigation. Also, ground-water wells being drilled in the eastern portion of the site by others (at the same time of our field investigation at the site) did not encounter water to a depth of about 220 feet (maximum depth drilled at the time of our investigation).

Based on the information from geotechnical investigations for other projects in the immediate area, the depth to ground water is greater than 150 feet in the general site vicinity. Ground water was not encountered in exploratory borings drilled as part of an investigation east of the site (between the site and the Texas border) within the 250-foot total depth explored (Weaver Boos & Gordon, Inc., 1998). Also, piezometers installed for a project in Andrews County Texas (located approximately ½-mile east of the site) indicate that the depth to ground water ranged from approximately 150 feet to 188 feet beneath the existing ground surface in January 1993 (Jack H. Holt, Ph.D. & Associates Inc., 1993).

## 4.4 TECTONIC SETTING

### 4.4.1 General

The tectonic regions in the project area (and in the state of New Mexico) can be defined based on historic seismicity and tectonic (structural) history. The project site is located within the seismically stable Permian Basin. The Permian Basin is defined by a broad subsurface structural feature composed of a series of Paleozoic age (greater than 250 million years before present) sedimentary basins whose last episodes of large-scale subsidence occurred during late Permian time (about 250 million years before present). The structural relief of these basins now exists as subsurface features buried beneath a thick sequence of younger, relatively undeformed sediments. Relative structural stability has been maintained since Permian time within this region as indicated by a lack of deep-seated, active faults within the post-Permian strata.

A prominent subsurface structural feature within the Permian Basin is the Central Basin Platform (CBP). The project site is located in the CBP where the top of the Permian deposits are approximately 1,400 feet below the ground surface, where outside the limits of the CBP, the Permian deposits are much deeper (Weaver Boos Consultants, Inc., 1998). The Permian deposits

are primarily limestone and constitute the main reservoir rocks for the oil and gas fields in the general area.

The Permian Basin is bounded on the west by a seismically active area known as the Rio Grande Rift. The Rio Grande Rift is a major continental rift extending north-south through the state of New Mexico from north of Taos to south of Las Cruces. The overwhelming majority of active faults in New Mexico are located within the boundaries of the Rio Grande Rift.

The seismically active Basin and Range province borders the western margin of the Rio Grande Rift. This province is characterized by fault block mountain ranges commonly bounded by range front normal faults separated by intervening valleys. The valleys are typically formed on structural grabens overlain by valley fill sediments derived from the adjacent mountain blocks. Major development of basin and range structures occurred from late Tertiary (5 million years before present) to Pleistocene time (11,000 years before present) and continue into the present time. A number of fault offsets of late Tertiary age along the western flanks of the Guadalupe, Delaware, Sacramento and San Andres Mountains are observed within the Basin and Range physiographic province in Trans-Pecos, Texas.

Leveling surveys between El Paso, Texas and Carlsbad, New Mexico and the historic seismic record for the New Mexico and West Texas regions support the interpretation that current tectonic activity is occurring in the Rio Grande Rift and the Basin and Range province while the Permian Basin (in which the site is located) remains stable and tectonically or seismically quiet.

### 4.4.2 Faults

The majority of Quaternary age faults within New Mexico are mapped along the north-south trending Rio Grande Rift located approximately 180 miles west of the site.

According to Machette et al. (1998), Quaternary age faults are not identified in New Mexico within 100 miles of the site. Quaternary age faults within 150 miles of the site include the Guadalupe fault, located approximately 115 miles west of the site in New Mexico, and in Texas, the West Delaware Mountains fault zone, East Sierra Diablo fault, East Flat Top Mountain fault, and the East Baylor Mountain-Carrizo Mountain fault located 110 miles southwest, 120 miles southwest, 120 miles west-southwest, and 120 miles southwest of the site, respectively.

### 4.4.3 Seismicity

Current research indicates that the Rio Grande Rift and the adjacent Basin and Range tectonic province are seismically active and the Permian Basin (in which the site is located) is considered to be seismically quiet or inactive. As previously indicated, the overwhelming majority of active faults in New Mexico are located within the boundaries of the Rio Grande Rift. The majority of seismic activity reported in New Mexico for the period 1869 to 1998 is concentrated along the rift (Sanford, Lin, Tsai, and Jaksha, 2002). However, even though the Permian Basin is considered seismically inactive, there is a documented cluster of seismic activity in the Central Basin Platform area since the mid-1960s (U. S. Department of Energy, 2003). In this area, the spatial distribution of epicenters correlate with known locations of oil and natural gas fields and is believed to be induced by production, secondary recovery, or waste injection activities within this natural gas and petroleum province, rather than seismic sources (Sanford, Lin, Tsai, and Jaksha, 2002; U. S. Department of Energy, 2003).

## 5.0 SUBSURFACE CONDITIONS

Subsurface conditions were explored with five widely spaced borings drilled in general accordance with the procedures presented in Appendix A. The boring locations were selected by Lockwood Greene and boring depths were selected by MACTEC. The borings locations and elevations shown on the Boring Location Plan (Figure 2) and the Soil Test Boring Records were surveyed by others prior to the field exploration.

Subsurface conditions encountered at the boring locations are shown on the Soil Test Boring Records in Appendix B. These Soil Test Boring Records represent our interpretation of the subsurface conditions, based on the field logs and visual examination of the field samples by one of our engineers. The lines designating the interfaces between various strata on the Soil Test Boring Records represent the approximate interface locations.

The soil test borings drilled at this site typically encountered Quaternary age eolian and alluvial silty sands underlain by Triassic age clays. A discussion of the origin of these materials is presented in Section 4.0 of this report.

The upper eolian silty sand soils were encountered to a maximum depth of about two feet at the soil test boring locations. These sandy soils were observed to be loose. It should be noted that in areas where these soils have accumulated and formed dunes, the thickness of the eolian sands will be more than two feet. Dunes having estimated heights of up to about eight feet were observed during the field exploration. The upper approximately one foot of the eolian sands were observed to have fine roots and various organic materials.

Silty sand soils of Quartenary age were encountered underlying the surficial eolian deposits. These soils were classified as firm to very dense based upon their Standard Penetration Test (SPT) resistance values which ranged from 20 to in excess of 100 blows per foot. Calcium carbonated cementation was noted within this zone to a varying degree, with zones of caliche encountered at irregular intervals. Fine gravel was observed in some of the split-spoon samples from the soils within this zone.

Triassic age high plasticity clays were encountered underlying the Quaternary age sands at depths of 35 to 40 feet. These materials were classified as very hard based upon SPT resistance values which ranged from 53 to in excess of 100 blows per foot. Each of the soil test borings drilled as part of this project were terminated in the Triassic age clays at the predetermined depths.

## 6.0 GROUND-WATER CONDITIONS

Ground water was not observed in the test borings at the time of drilling. Also, it was reported that ground water was not encountered in borings drilled at the site by others for water well development to depths of 220 feet. For safety reasons, the borings drilled as part of MACTEC's scope of work were backfilled promptly after drilling; consequently, long-term measurements for the presence or absence of ground water were not obtained.

Fluctuations in the ground-water level occur because of variations in rainfall, evaporation, construction activity, surface run-off, and other site-specific factors. Given the proposed preliminary excavation depths, we expect ground water will not present significant construction problems for this project. The selected contractor should, however, be prepared to promptly remove surface waters which could impact construction activities.

## 7.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

As previously noted, this exploration was for assistance in preliminary planning and design for the proposed NEF facility. Five borings, with an average spacing of 1,700 feet, across the proposed site for a project of this size are not sufficient to adequately define subsurface conditions for final design purposes. While these borings do, in our professional opinion, provide a sound basis for assistance in judging the feasibility of developing the site, there is insufficient information for developing specific, final recommendations for site preparation and foundation type or types and design parameters. The following information, therefore, should be considered as preliminary recommendations, subject to refinement when additional project details are available so that a more detailed program of borings and field or laboratory testing can be performed.

Assuming subsurface conditions encountered at the boring locations are representative of subsurface conditions elsewhere on the site, subsurface conditions generally appear to be suitable for the proposed construction supported on a system of shallow foundations. Footings bearing in the firm and dense sandy soils below the upper loose eolian (wind deposited) soils may be preliminarily designed for an allowable bearing pressure of 7,000 pounds per square foot (psf).

The upper loose eolian deposits were generally encountered to depths of up to two feet below existing ground surface in the soil test borings. However, in areas where sand dunes have formed, the actual depth of the eolian soils is likely more than two feet. If the eolian deposits are not removed as a part of mass site grading, it will be necessary to extend the foundation excavations into the underlying firm or better sandy soils to achieve the recommended preliminary allowable bearing pressure and reduce the likelihood of excessive differential settlements; therefore, we recommend that the upper eolian soils be removed during mass grading in structural areas.

The preliminary geotechnical and structural data available at this stage precludes performing rigorous settlement analyses. However, based upon the available data, our preliminary analyses indicate that spread footings bearing in the firm or better sands below the upper eolian soils as described above may be preliminarily assumed to be subject to maximum total settlements of up to about one inch for column loads of up to 1,300 kips. Under these circumstances, differential settlements may range up to ½ of an inch between similarly loaded columns. Settlements between differentially loaded or closely spaced columns would likely be more than the aforementioned values.

The behavior of a shallow foundation with respect to settlement is dependent upon a variety of factors. The information provided herein is intended to demonstrate the feasibility of shallow foundation support. Excessive differential settlement between adjacent columns, while uncommon in the firm or better soils such as those encountered in the borings, can cause structural and architectural damage. A more thorough exploration and analysis will be required to accurately estimate the range of expected total and differential settlements which may be expected when project specifics have been developed.

## 8.0 PRELIMINARY SITE PREPARATION RECOMMENDATIONS

Existing vegetation, surficial organic containing soils and loose eolian sands should be stripped and removed from the construction area. Typically, the upper approximately one foot of the site soils were observed to contain fine roots and limited organic materials. The eolian sands were encountered to depths of about two feet at the soil test boring locations; however, sand dunes were observed across the proposed building locations during the field exploration. These sand dunes are likely composed of loose sands and should therefore be stripped from the site as part of mass grading. Based upon our observations, sand dunes of up to eight feet in height will likely be encountered during grading. We recommend that prior to the preparation of the final bid documents, additional geotechnical exploration be performed to evaluate the depth to which the upper loose eolian sands may be encountered, especially with regard to the dune areas.

Information regarding the use of the upper loose sands for use as new fill is presented in the Engineered Fill section of this report.

After stripping the site and before placing new fill, we recommend the exposed subgrade in the building and pavement areas be proofrolled to detect unsuitable soil conditions. Proofrolling should be done after a suitable period of dry weather to avoid degrading an otherwise acceptable subgrade. Proofrolling should be performed with a heavily-loaded dump truck or with similar approved construction equipment. The proofrolling equipment should make at least four passes over each section, with the last two passes perpendicular to the first two.

We recommend the exposed subgrade and proofrolling operation be observed and documented by our personnel. If unstable conditions are encountered at the subgrade level, our geotechnical engineer will make appropriate recommendations to the owner's representative for dealing with the conditions.

Earth moving, selective borrowing and soil compaction will be required to achieve the final grades proposed for this project. Contractors bidding on this work should be supplied with this preliminary geotechnical information as well as supplemental exploration results because bids based on such data are generally more competitive, time schedules are more accurate and potential cost overruns are smaller as a result. Typical information required for grading operations would require further refinement of the items listed below in relationship to the selected site surface elevations:

- Classification tests to identify soil type
- Existing soil moisture contents to plan moisture content control measures
- Additional compaction tests to determine the maximum dry density and optimum moisture content for verifying the adequacy of compaction operations
- Evaluation of the compaction test results and recommendations of proper compaction procedures such as lift thickness, proper equipment types, moisture content control measures, etc.
- Further delineation of the stratification of materials to be excavated

## 9.0 DIFFICULT EXCAVATION

Construction of the proposed NEF will require excavation of the existing site soils to get within the range of the preliminary finished floor at about Elev. 3,414 feet. Based upon the limited number of borings performed for the preliminary exploration, much of these materials may be removed using conventional earthmoving equipment. However, zones of very dense soils and caliche were encountered above the proposed finished floor elevation. Therefore, such materials will likely be encountered during site grading. Heavy excavating equipment with ripping tools could be required to remove much of these materials. Materials sufficiently hard to cause refusal to the power auger equipment used to drill the borings were not encountered during the preliminary exploration.

Typically, there is no sharp transition between uncemented and cemented soils in geologic settings such as this site. The caliche encountered during this exploration could be penetrated by the mechanical auger used to drill the borings and can likely be excavated without blasting. It is, however, often difficult to excavate these materials without the use of specialized equipment or blasting, especially if harder or more extensive zones of caliche are encountered during site grading. The excavation of very dense soils or caliche in confined excavations, such as for shallow

foundations or utilities, is often extremely difficult. The ease of excavation depends on the quality of the grading equipment, skill of the equipment operators, and geologic structure of the material to be excavated. Materials that cannot be penetrated by the mechanical auger often require blasting prior to removal.

# 10.0 PRELIMINARY SEISMIC CONSIDERATIONS

## 10.1 SITE COEFFICIENT AND SEISMIC ZONATION

The site coefficient, S, for seismic design of the proposed buildings can be determined as established in the Earthquake Regulations under Section 1629 of the Uniform Building Code (UBC) 1997 edition. Based on Figure 16-2 of the 1997 UBC, the site is located within Seismic Zone 1. In addition, based on our review of the site soil conditions as encountered in our borings and local geology, the Soil Profile Type may be assumed to be Type $S_C$ as defined in Table 16-J of the 1997 UBC.

## 10.2 LIQUEFACTION

Liquefaction potential is greatest where the ground-water level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

Ground water was not encountered in our borings drilled to a maximum depth of 100 feet below the ground surface. In addition, it was reported that a boring on the site drilled for water well development did not encounter water at a depth of 220 feet. Also, the soils at the site were dense to very dense. The absence of ground water near the surface would make the potential for liquefaction remote.

## 11.0 COMPACTED FILL RECOMMENDATIONS

We recommend compacted fill be constructed by spreading acceptable soil in loose layers not more than 8 inches thick. The soils used within the proposed construction areas should be compacted in lifts to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). The upper 24 inches of fill beneath pavements and upper 12 inches beneath grade slabs should be compacted to at least 98 percent of the modified Proctor maximum dry density.

As a general rule, the moisture content of the fill soils compacted to 95 percent of the modified Proctor density should be maintained within +3 to -3 percentage points of the optimum moisture content as determined from the compaction test. This provision may require the contractor to dry soils during periods of wet weather or to wet soils during warm or dry periods. The fill soils should have a plasticity index (PI) of less than 15, and a maximum dry density of no less than 90 pounds per cubic foot (pcf).

A sample of potential borrow material was collected from Borings B-3 and B-4 and tested to determine the maximum dry density, optimum moisture content, natural moisture content, and PI. These tests are used to determine if the soil is suitable for use as engineered fill.

The laboratory test data indicate potential on-site borrow soils are typically dryer than the optimum moisture content. Since some of the natural moistures are substantially less than optimum moisture content, the contractor should anticipate wetting of the borrow soils will likely be required to achieve adequate compaction. Our laboratory test data also indicate the potential on-site borrow soils have maximum dry densities and PI values within the recommended ranges. In our opinion, the laboratory data indicate the potential on-site borrow soils are suitable for use as compacted fill. Additional testing should be performed to verify the suitability of the proposed borrow materials prior to final design or the initiation of site grading. The results of the laboratory tests along with a description of the test procedures are provided in Appendix C.

If calcium carbonate cemented soils (caliche) are to be used as engineered fill, it is imperative this material be reduced to particles having a maximum dimension of six inches by the excavation and compaction equipment. Sufficient quantities of soil should be mixed with these materials such that voids do not result between the pieces of caliche and so that the fill meets compaction requirements.

The upper eolian sands removed as part of site stripping are likely suitable for use as engineered fill provided the organic content of these materials is within an acceptable range. It is our preliminary recommendation that soils having an organic content of less than two percent, when subject to organic loss on ignition testing, are suitable for use as engineered fill. The upper approximately one foot of the eolian sands were observed to contain fine roots and organic materials. The upper eolian sands are likely to have in-situ moisture contents well below the materials optimum moisture content and the addition of water will be necessary to achieve the recommended degree of compaction.

## 12.0 CORROSION POTENTIAL

Corrosion is a major factor in reducing the service life of metal and concrete structures within the soil. Therefore, measuring the corrosion potential of soils is an important consideration when designing or selecting protective measures for buried structures or potions of structures.

There are several measurable soil properties which may be used to estimate the potential corrosiveness of a soil. These properties include resistivity, pH, chloride concentration, and sulfide content. Resistivity and pH are the two soil properties which have the greatest influence on underground corrosion and are relatively easily measured.

The electrical resistivity of a soil is measured in the laboratory by immersing the probe of a conductivity meter into a prepared slurry of soil sample and deionized water. Split-spoon samples from the depths of 5 and 10 feet were combined and tested from Borings B-2 and B-4. The measured resistivity of these samples was 7,400 and 2,100 ohm-centimeters for the samples from Borings B-2 and B-4, respectively.

The pH of a soil is a measure of the hydrogen-ion concentration and indicates the intensity of acidity or alkalinity of a soil. A pH value of 7 indicates neutrality; higher values, alkalinity; lower values, acidity. Soil pH values were determined by immersing the probe of a pH meter into a prepared slurry of soil sample and deionized water. The pH values were 7.99 and 7.93 for the samples from Borings B-2 and B-4, respectively.

Based upon published information, the measured pH and resistivity values place the site soils in the non-corrosive to questionable range for corrosion potential.

As the sample from Boring B-4 fell within the questionable range, we recommend further evaluation of the potential for corrosion. This evaluation will likely include additional pH and resistivity tests, as well as testing of the soil's sulfide and chloride concentration. Measures such as special coatings or cathodic protection for buried steel structures or the use of admixtures or chemically resistant cement for concrete protection may be required depending upon the results of such testing.

## 13.0 PRELIMINARY PERCOLATION RATE

Percolation testing was conducted at the proposed NEF site in general accordance with the procedures presented in Title 20, Chapter 7, Part 3 of the New Mexico Administrative Code (NMAC). Percolation testing was performed at locations approximately 25 and 75 feet east of soil test Boring B-4. The test holes were each drilled to depths of about 10 feet below existing ground surface in the silty sand soils such as those encountered in the upper portion of the subsurface profile across the site. Each hole was filled with water to a depth of about 1½ feet below existing ground surface; therefore, the upper loose eolian sands are not represented in the percolation test results.

The measured percolation rates for the two test locations were 6.7 and 10.0 minutes per inch. Averaging the two test results, as is suggested in the NMAC, the percolation rate of 8.4 minutes per inch is recommended for the preliminary design of systems leaching into materials similar to those tested. Additional percolation testing should be performed once information concerning specific drain field locations and elevations has been developed.

## 14.0 PRELIMINARY CALIFORNIA BEARING RATIO INFORMATION

Two remolded CBR tests were performed on bulk soil samples collected from auger cuttings. The samples were obtained from the depths of 5 to 10 feet in Boring B-3 and from ground surface to 15 feet in Boring B-4. Since the as-molded densities and moisture contents differed somewhat from the targeted values, interpolation and extrapolation of the CBR data were required to estimate the CBR values at 95 percent of the modified Proctor maximum dry density at optimum moisture content. The CBR test results are attached in Appendix C.

The CBR values corresponding to 95 percent of the modified Proctor maximum dry density at optimum moisture were estimated to be 34.4 and 10.5 for the samples from B-3 and B-4, respectively. We recommend the lower CBR value of 10.5 for use in preliminary design purposes.

Additional testing should be performed to evaluate the CBR values of proposed fill soils prior to the completion of the final design.

## 15.0 RECOMMENDED ADDITIONAL GEOTECHNICAL EXPLORATION

This exploration is preliminary in nature and should be used for general site planning and feasibility evaluation only. Due to the relatively limited information available at this preliminary stage of the project, preparation of a complete report of geotechnical study with specific recommendations for foundation design and site preparation will require significant supplemental exploration and analysis. Project details and performance criteria should, however, be initially further developed. As project details are developed, additional exploration, field and laboratory testing, and engineering analysis will be required. Additional field testing may include items such as more soil test borings, the collection of relatively undisturbed and disturbed samples, and possibly in-situ testing. Additional laboratory testing may include triaxial shear tests, consolidation tests, direct shear tests, grain size testing, unit weight, Atterberg limits, moisture content and compaction tests. Field resistivity testing may be used in conjunction with laboratory pH, chloride content, and sulfate tests to further evaluate the corrosion potential of the site soils. The geotechnical engineer should be retained to consult with the designer during design development, design and construction phases to be sure that the recommendations are properly interpreted and further developed as necessary.

## 16.0 BASIS OF RECOMMENDATIONS

The preliminary recommendations provided herein are based on the subsurface conditions and on project information provided to us; they apply only to the specific project and site discussed in this report. If the project information section in this report contains incorrect information or if additional information becomes available, you should convey the corrected or additional information to us and retain us to review our recommendations. We will then modify them if the new information has rendered them inappropriate for the proposed project. As mentioned previously, additional exploration and analysis along with interaction of the design team will be required to develop final recommendations for foundation design and site preparation.

Our exploration services include storing the collected samples and making them available for inspection for a period of 30 days. The samples are then discarded unless you request otherwise.

# 17.0 REFERENCES

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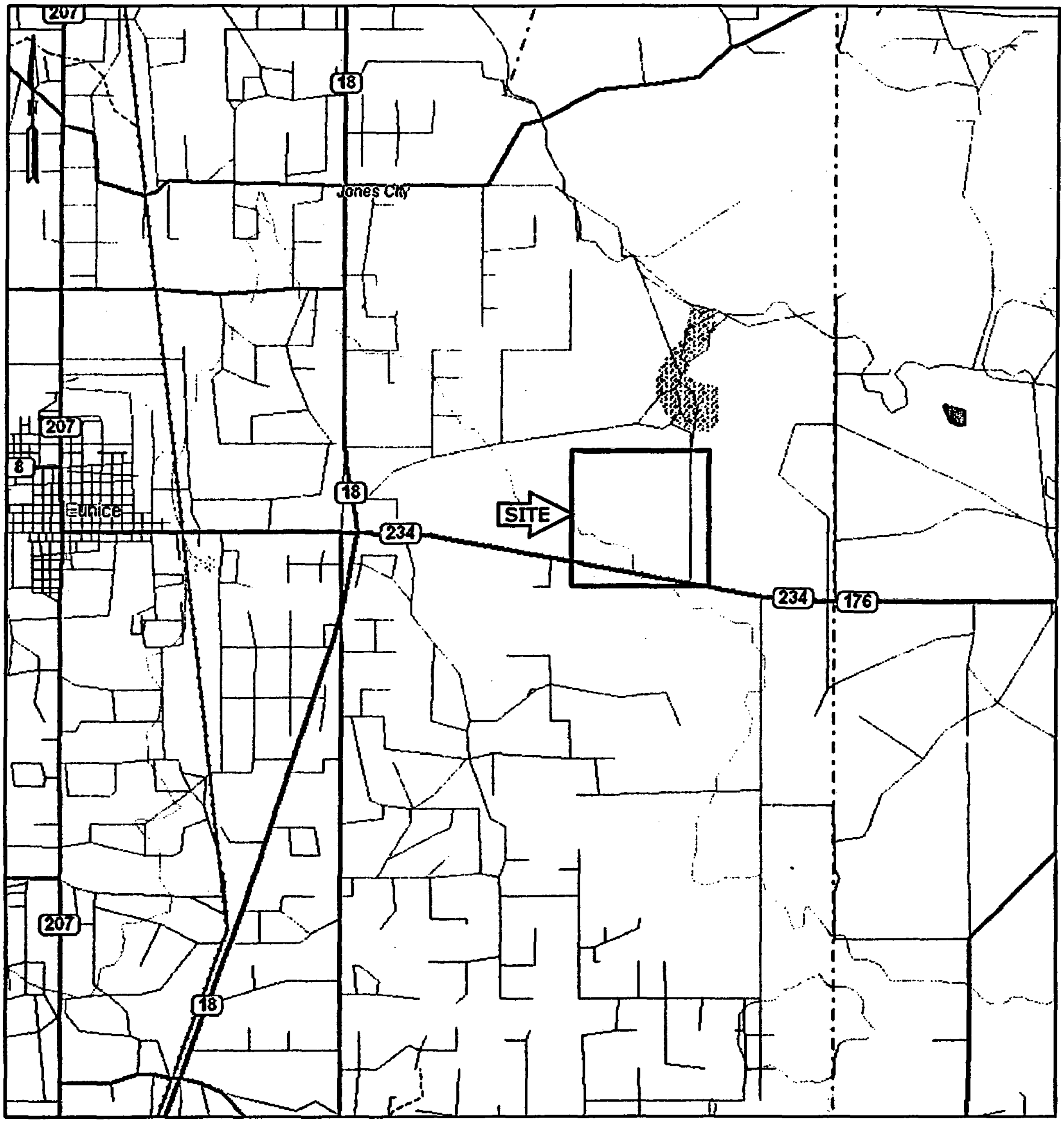
# FIGURES




SOURCE: DELORME MAPPING AND OBSERVATIONS BY MACTEC PERSONNEL


MACTEC

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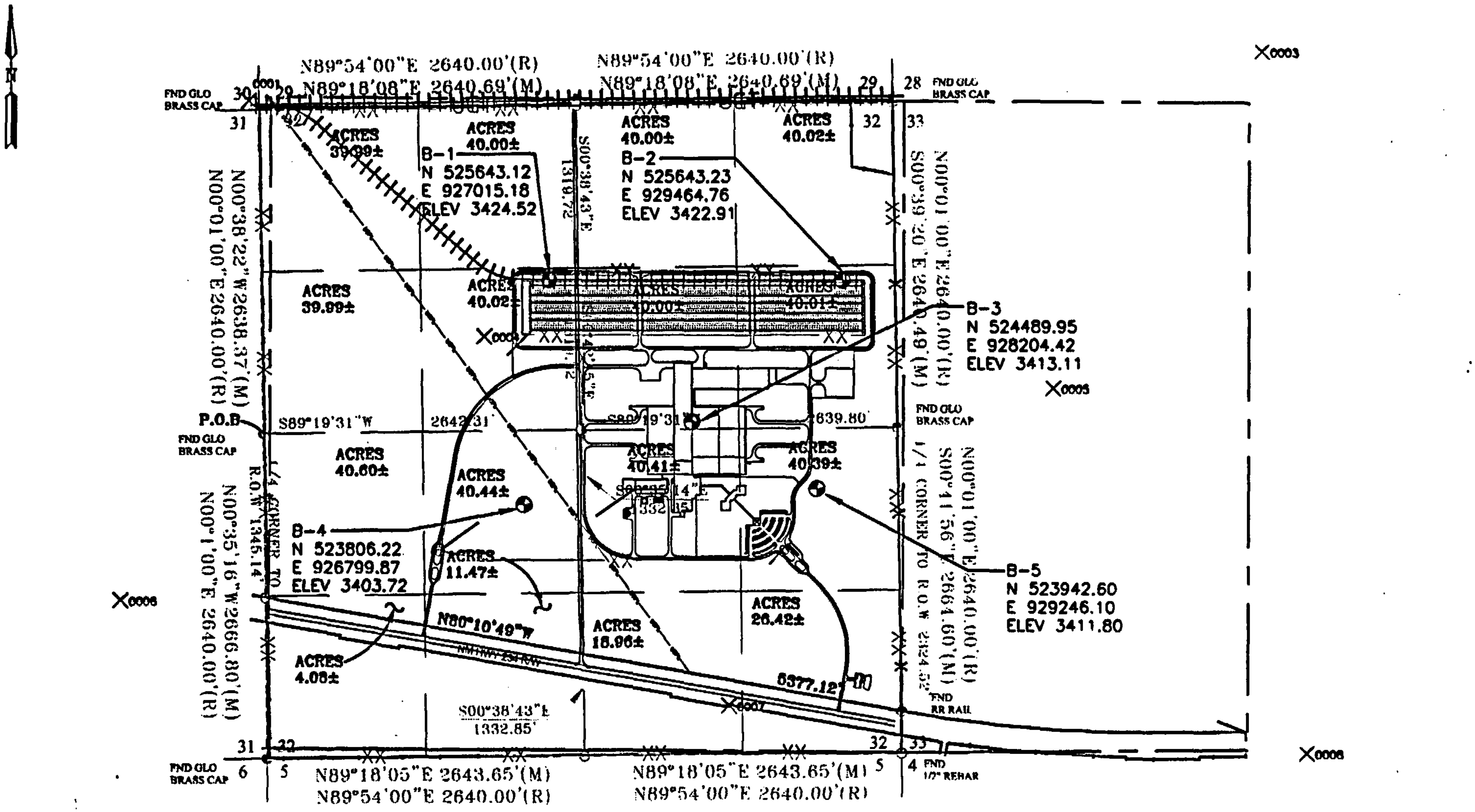
1725 Louisville Drive
Knoxville, Tennessee 37921-5904
865-588-8544 • Fax: 865-588-8026


# FIGURE 1: SITE LOCATION PLAN
# NATIONAL ENRICHMENT FACILITY
# LEA COUNTY, NEW MEXICO

| | | |
|---|---|---|
| DRAFTING BY: | PREPARED BY: mBH | CHECKED BY: JMS |
| JOB NUMBER: 3043031049/0001 | DATE: OCTOBER 2, 2003 | SCALE: 0 6250' |

COORDINATES:




SOURCE: THIS DRAWING WAS ADAPTED FROM A CONCEPTUAL SITE PLAN SOIL BORING LAYOUT DATED SEPTEMBER 2, 2003 AND PROVIDED BY LOCKWOOD GREENE.

**LEGEND**

B-1 BORING LOCATION AND IDENTIFICATION

MACTEC


MACTEC Engineering and Consulting, Inc.
1725 Louisville Drive
Knoxville, Tennessee 37921-5904
865-588-8544 • Fax: 865-588-8026


# FIGURE 2: BORING LOCATION PLAN
# NATIONAL ENRICHMENT FACILITY
# LEA COUNTY, NEW MEXICO

| | | |
|---|---|---|
| DRAFTING BY: | PREPARED BY: MBH | CHECKED BY: JMS |
| JOB NUMBER: 3043031049/0001 | DATE: OCTOBER 2, 2003 | SCALE: 0 1000' |

COORDINATES:

# APPENDIX A

# FIELD EXPLORATORY PROCEDURES

## FIELD EXPLORATORY PROCEDURES

### Soil Test Boring (Hollow Stem)

Soil test borings and sampling operations were conducted in general accordance with ASTM D 1586. The borings were advanced by mechanically turning continuous steel hollow-stem auger flights into the ground. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot of penetration was recorded and is designated the "standard penetration test (SPT) resistance." Proper evaluation of the penetration resistance provides an index to the soil's strength, density, and ability to support foundations.

Representative portions of the soil samples obtained from the split-tube sampler were examined by our engineer to assign manual soil classifications. Representative portions of the split-spoon samples were then placed in containers and shipped to our laboratory. Test Boring Records are attached, graphically showing the soil descriptions and penetration resistances.

### Boring Backfill

The borings were backfilled shortly after drilling for safety purposes. We backfilled the borings with auger cuttings to the ground surface.

You are advised that, even with this backfill technique, there is the possibility of future borehole subsidence depending on actual subsurface conditions, surface drainage, etc. The property owner should monitor the boring locations over time to discover subsidence and make any necessary repairs.

### Bulk Samples

Bulk samples of several soil types obtained at various elevations were collected for testing to determine the suitability of soil for reuse as engineered fill, its maximum dry density and CBR value.

**Undisturbed Sampling**

The relatively undisturbed samples were obtained by pushing a section of 3-inch O.D., 16-gauge steel tubing into the soil at the desired sampling level. The sampling procedure is described by ASTM D-1587. The tube, together with the encased soils, was carefully removed from the ground, made airtight, and transported to our laboratory.

# APPENDIX B

## KEY TO SYMBOLS AND DESCRIPTIONS

## SOIL TEST BORING RECORDS

| GROUP SYMBOLS | TYPICAL NAMES | GROUP SYMBOLS | TYPICAL NAMES |
|---|---|---|---|
| | **TOPSOIL** | | **CONCRETE** |
| | **ASPHALT** | | **DOLOMITE** |
| | **GRAVEL** | | **LIMESTONE** |
| | **FILL** | | **SHALE** |
| | **SUBSOIL** | | **LIMESTONE/SHALE** - Limestone with shale interbeds |
| | **ALLUVIUM** | | **SANDSTONE** |
| | **COLLUVIUM** | | **SILTSTONE** |
| | **RESIDUUM** - Soft to firm | | **AUGER BORING** |
| | **RESIDUUM** - Stiff to very hard | | **UNDISTURBED SAMPLE ATTEMPT** |

| | | | |
|---|---|---|---|
| | Undisturbed Sample<br>1.5-2.0 = Recovered (ft) / Pushed (ft) | | |
| | Split Spoon Sample | | Auger Cuttings |
| | Rock Core<br>60-100 = RQD / Recovery | | Dilatometer |
| | No Sample | | Crandall Sampler |
| | Rotary Drill | | Pressure Meter |
| ▽ | Water Table at time of drilling | ○ | No Recovery |
| | | ▼ | Water Table after 24 hours |

Correlation of Penetration Resistance with Relative Density and Consistency

| SAND & GRAVEL | | SILT & CLAY | |
|---|---|---|---|
| No. of Blows | Relative Density | No. of Blows | Consistency |
| 0 - 4 | Very Loose | 0 - 2 | Very Soft |
| 5 - 10 | Loose | 3 - 4 | Soft |
| 11 - 20 | Firm | 5 - 8 | Firm |
| 21 - 30 | Very Firm | 9 - 15 | Stiff |
| 31 - 50 | Dense | 16 - 30 | Very Stiff |
| Over 50 | Very Dense | 31 - 50 | Hard |
| | | Over 50 | Very Hard |

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

| SILT OR CLAY | SAND | | | GRAVEL | | Cobbles | Boulders |
|---|---|---|---|---|---|---|---|
| | Fine | Medium | Coarse | Fine | Coarse | | |

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

# KEY TO SYMBOLS AND DESCRIPTIONS

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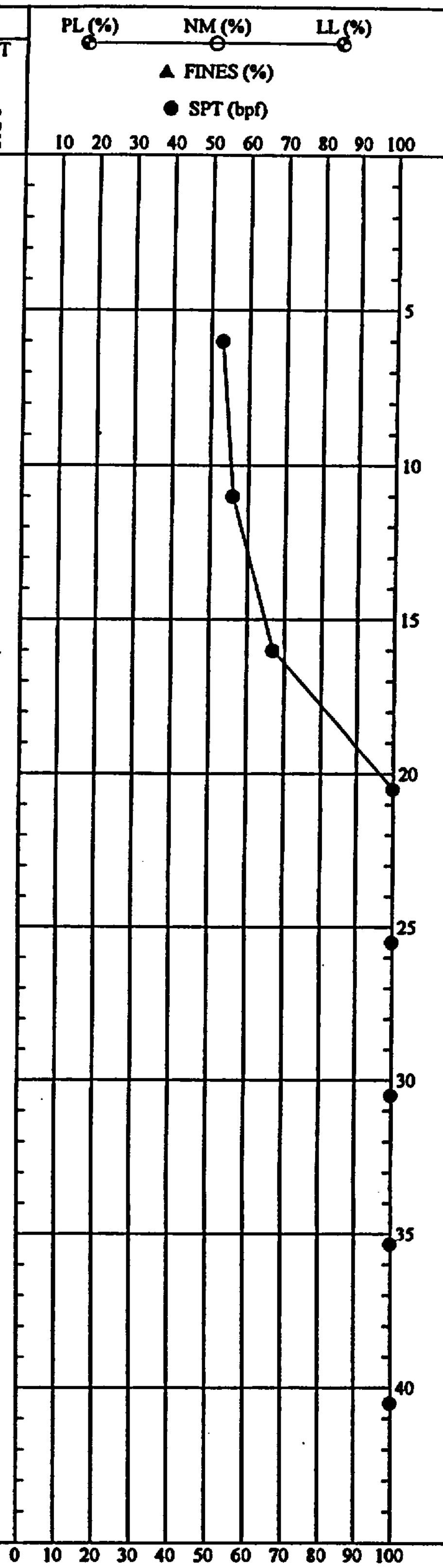
| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS | ELEV (ft) | SAMPLES IDENT | N-COUNT (1st 6" - 2nd 6" - 3rd 6") |
|---|---|---|---|---|
| 0 | REDDISH BROWN, DRY, SILTY FINE SAND WITH SOME FINE ROOTS - EOLIAN | 3423.4 | | |
| | VERY DENSE, REDDISH BROWN, DRY, SILTY FINE TO MEDIUM SAND | | | |
| 5 | | 3418.4 | SPT-1 | 15-25-28 |
| 10 | | 3413.4 | SPT-2 | 21-30-26 |
| 15 | VERY DENSE, DARK RED, DRY, FINE TO MEDIUM SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND | 3408.4 | SPT-3 | 15-30-37 |
| 20 | VERY DENSE, LIGHT REDDISH YELLOW, DRY, SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND | 3403.4 | SPT-4 | 50/6" |
| | | | UD-1 | 1.0-2.0 |
| 25 | | 3398.4 | SPT-5 | 50/6" |
| 30 | | 3393.4 | SPT-6 | 50/6" |
| 35 | VERY DENSE, LIGHT REDDISH YELLOW, SILTY FINE TO COARSE SAND WITH SOME SUBANGULAR TO ROUNDED FINE GRAVEL | 3388.4 | SPT-7 | 50/4" |
| 40 | VERY HARD, DARK RED, DRY, HIGH PLASTICITY CLAY | 3383.4 | SPT-8 | 50/6" |
| | BORING TERMINATED 40.5' | | | |
| 45 | | 3378.4 | | |

SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.




SOIL 3045031049 01.GPJ LAW_GIBB.GDT 10/3/03

REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/9/2003.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

## SOIL TEST BORING RECORD

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 9, 2003 **BORING NO.:** B-1

**PROJ. NO.:** 3043031049/0001 **PAGE 1 OF 1**

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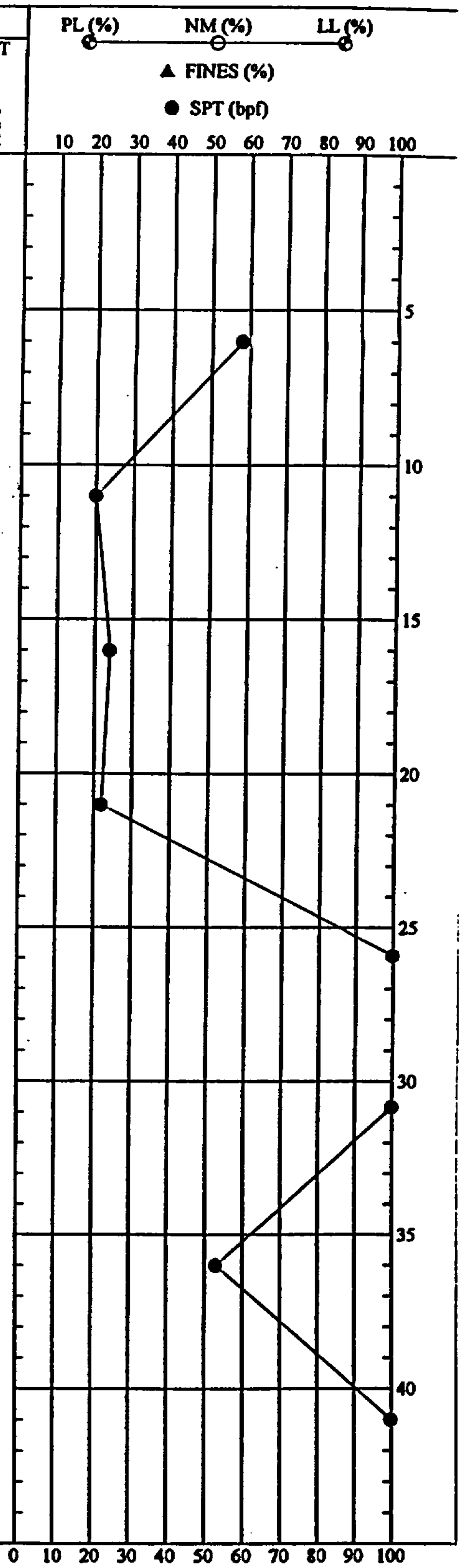
| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS (SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.) | ELEV (ft) |
|---|---|---|
| 0 | REDDISH BROWN, DRY, SILTY, FINE SAND WITH SOME FINE ROOTS - EOLIAN | 3422.2 |
| | VERY DENSE, LIGHT YELLOW, DRY, FINE SILTY SAND WITH CLASTS OF CEMENTED SAND - CALICHE FROM 6.0' - 7.5' | |
| 5 | | 3417.2 |
| 10 | FIRM TO VERY FIRM, LIGHT YELLOW, DRY, SILTY FINE TO MEDIUM SAND | 3412.2 |
| 15 | | 3407.2 |
| 20 | VERY FIRM TO VERY DENSE, LIGHT YELLOW, SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND FROM 25.0' TO 35.0' | 3402.2 |
| 25 | | 3397.2 |
| 30 | | 3392.2 |
| 35 | VERY HARD, DARK RED AND PURPLE, MOIST, HIGH PLASTICITY CLAY | 3387.2 |
| 40 | | 3382.2 |
| | BORING TERMINATED AT 41.4' | |
| 45 | | 3377.2 |

**SAMPLES**

| IDENT | N-COUNT (1st 6" - 2nd 6" - 3rd 6") |
|---|---|
| SPT-1 | 13-29-29 |
| SPT-3 | 11-10-10 |
| SPT-4 | 9-12-12 |
| SPT-5 | 13-10-12 |
| SPT-6 | 15-50/5" |
| SPT-7 | 50/1" |
| SPT-8 | 14-23-30 |
| SPT-9 | 12-33-50/5" |




REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/9/2003.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

## SOIL TEST BORING RECORD

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 9, 2003 **BORING NO.:** B-2

**PROJ. NO.:** 3043031049/0001 **PAGE 1 OF 1**

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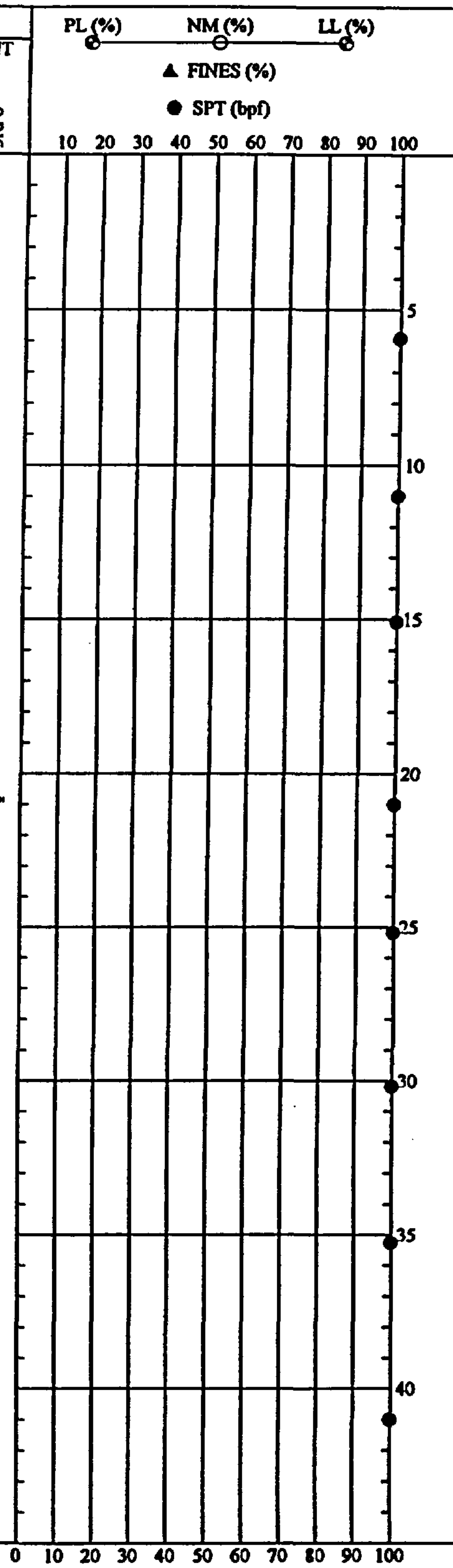
SOIL 3043031049 01.GPJ LAW GIBB.GDT 10/3/03

| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS | ELEV (ft) |
|---|---|---|
| 0 | REDDISH BROWN, DRY, SILTY FINE SANDY WITH SOME ROOTS - EOLIAN | 3412.5 |
| | VERY DENSE, REDDISH BROWN, DRY, SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND | |
| 15 | VERY DENSE, LIGHT YELLOW, DRY SILTY FINE TO MEDIUM SAND - CALICHE | 3397.5 |
| 20 | VERY DENSE, REDDISH YELLOW, DRY SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND AND SOME FINE SUBANGULAR TO ROUNDED GRAVEL | 3392.5 |
| 25 | VERY DENSE, LIGHT YELLOW, DRY SILTY FINE TO MEDIUM CEMENTED SAND - CALICHE | 3387.5 |
| 35 | VERY DENSE, REDDISH BROWN, DRY, SILTY FINE TO MEDIUM SAND | 3377.5 |
| 40 | VERY HARD, DARK RED, DRY, HIGH PLASTICITY CLAY | 3372.5 |
| 45 | | 3367.5 |

SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.

| SAMPLES IDENT | N-COUNT (1st 6" / 2nd 6" / 3rd 6") |
|---|---|
| SPT-1 | 30-50/5" |
| SPT-2 | 33-50/6" |
| SPT-3 | 50/1" |
| SPT-4 | 22-40-50/3" |
| SPT-5 | 50/2" |
| SPT-6 | 50/2" |
| SPT-7 | 50/3" |
| SPT-8 | 23-50/6" |




REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/10/2003.

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**SOIL TEST BORING RECORD**

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 10, 2003 **BORING NO.:** B-3

**PROJ. NO.:** 3043031049/0001 **PAGE 1 OF 3**

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SOIL 3045031049 01.GPJ LAW GIBB.GDT 10/3/03

| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS | ELEV (ft) | SAMPLE IDENT | N-COUNT |
|---|---|---|---|---|
| 45 | VERY HARD, DARK RED, DRY, HIGH PLASTICITY CLAY | 3367.5 | SPT-9 | 27-50/5" |
| 50 | | 3362.5 | SPT-10 | 39-50/2" |
| 55 | VERY HARD, DARK RED AND PURPLE, DRY, HIGH PLASTICITY CLAY | 3357.5 | SPT-11 | 35-50/4" |
| 60 | | 3352.5 | SPT-12 | 31-50/5" |
| 65 | | 3347.5 | SPT-13 | 32-50/4" |
| 70 | | 3342.5 | SPT-14 | 22-50/5" |
| 75 | | 3337.5 | SPT-15 | 39-50/4" |
| 80 | VERY HARD, DARK RED AND PURPLE, DRY, HIGH PLASTICITY CLAY WITH OCCASIONAL THIN CEMENTED LAYERS OF GREENISH GRAY | 3332.5 | SPT-16 | 50/4" |
| 85 | VERY HARD, REDDISH PURPLE WITH GRAY MOTTLING, DRY HIGH PLASTICITY CLAY | 3327.5 | SPT-17 | 50/6" |
| 90 | | 3322.5 | | |

SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.

PL (%) — NM (%) — LL (%); ▲ FINES (%); ● SPT (bpf)

REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/10/2003.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

**SOIL TEST BORING RECORD**

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 10, 2003 **BORING NO.:** B-3

**PROJ. NO.:** 3043031049/0001 **PAGE 2 OF 3**

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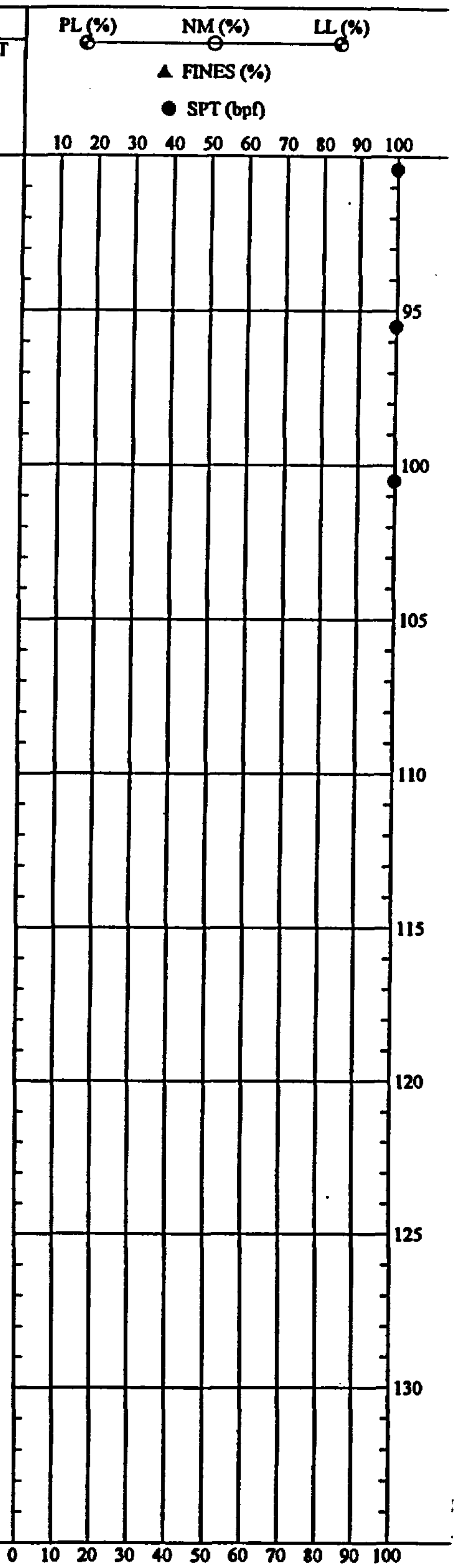
SOIL 3045031049 01.GPJ LAW_GIBB.GDT 10/3/03

| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS<br>SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW. | ELEV (ft) | SAMPLES IDENT | TYPE | N-COUNT (1st 6", 2nd 6", 3rd 6") |
|---|---|---|---|---|---|
| 90 | VERY HARD, RED AND PURPLE, DRY, HIGH PLASTICITY CLAY | 3322.5 | SPT-18 | | 50/5" |
| 95 | | 3317.5 | SPT-19 | | 50/6" |
| 100 | BORING TERMINATED AT 100.5' | 3312.5 | SPT-20 | | 50/6" |
| 105 | | 3307.5 | | | |
| 110 | | 3302.5 | | | |
| 115 | | 3297.5 | | | |
| 120 | | 3292.5 | | | |
| 125 | | 3287.5 | | | |
| 130 | | 3282.5 | | | |
| 135 | | 3277.5 | | | |




SOIL 3045031049 01.GPJ LAW GIBB.GDT 10/3/03

REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/10/2003.

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**SOIL TEST BORING RECORD**

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 10, 2003 **BORING NO.:** B-3

**PROJ. NO.:** 3043031049/0001 **PAGE 3 OF 3**

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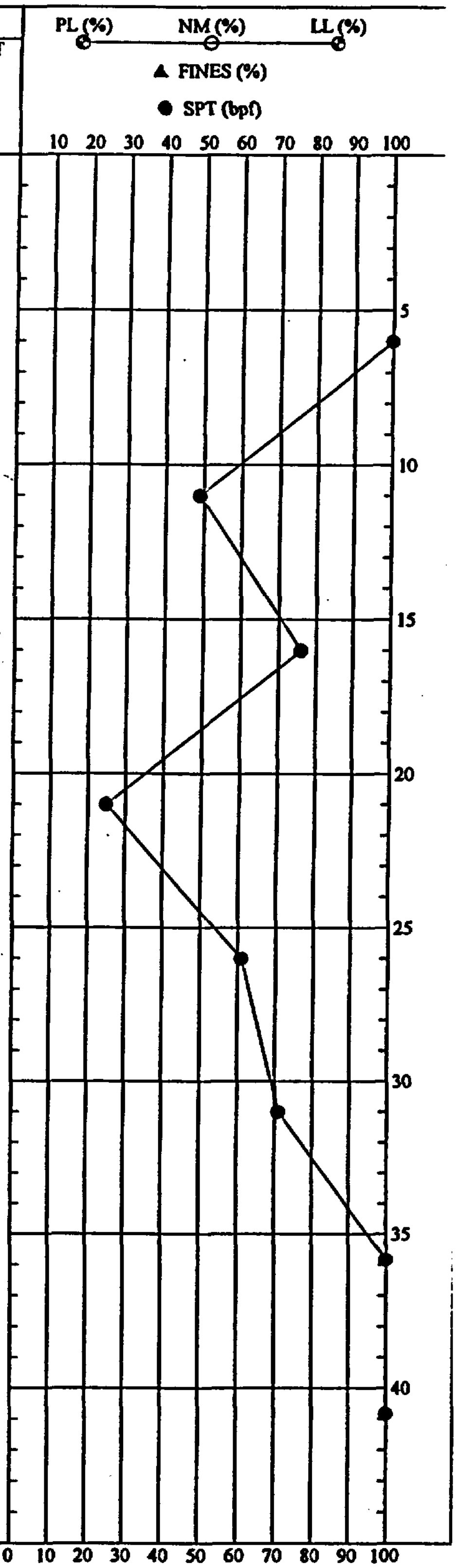
| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS<br>SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW. | ELEV (ft) | SAMPLES IDENT | TYPE | N-COUNT (1st 6" 2nd 6" 3rd 6") |
|---|---|---|---|---|---|
| 0 | REDDISH BROWN, DRY, SILTY FINE SAND WITH SOME ROOTS - EOLIAN | 3403.5 | | | |
| | VERY DENSE, REDDISH BROWN, DRY, SILTY FINE SAND | | | | |
| 5 | | 3398.5 | SPT-1 | | 26-50/6" |
| 10 | DENSE TO VERY DENSE, LIGHT RED, DRY, SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND | 3393.5 | SPT-2 | | 17-26-23 |
| 15 | | 3388.5 | SPT-3 | | 20-39-37 |
| 20 | VERY FIRM TO VERY DENSE, LIGHT RED, DRY, SILTY FINE SAND WITH CEMENTED ZONES FROM 25.0' TO 30.0' | 3383.5 | SPT-4 | | 7-13-12 |
| | | | UD-2 | | 1.0-2.0 |
| 25 | | 3378.5 | SPT-5 | | 16-34-27 |
| 30 | VERY DENSE, LIGHT REDDISH BROWN, DRY FINE SILTY FINE TO MEDIUM SAND | 3373.5 | SPT-6 | | 16-30-41 |
| 35 | | 3368.5 | SPT-7 | | 39-50/4" |
| 40 | VERY HARD, DARK RED, DRY, HIGH PLASTICITY CLAY | 3363.5 | SPT-8 | | 35-50/4" |
| | BORING TERMINATED AT 40.8' | | | | |
| 45 | | 3358.5 | | | |




SOIL 3045031049 01.GPJ LAW_GIBB.GDT 10/3/03

REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/9/2003.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

## SOIL TEST BORING RECORD

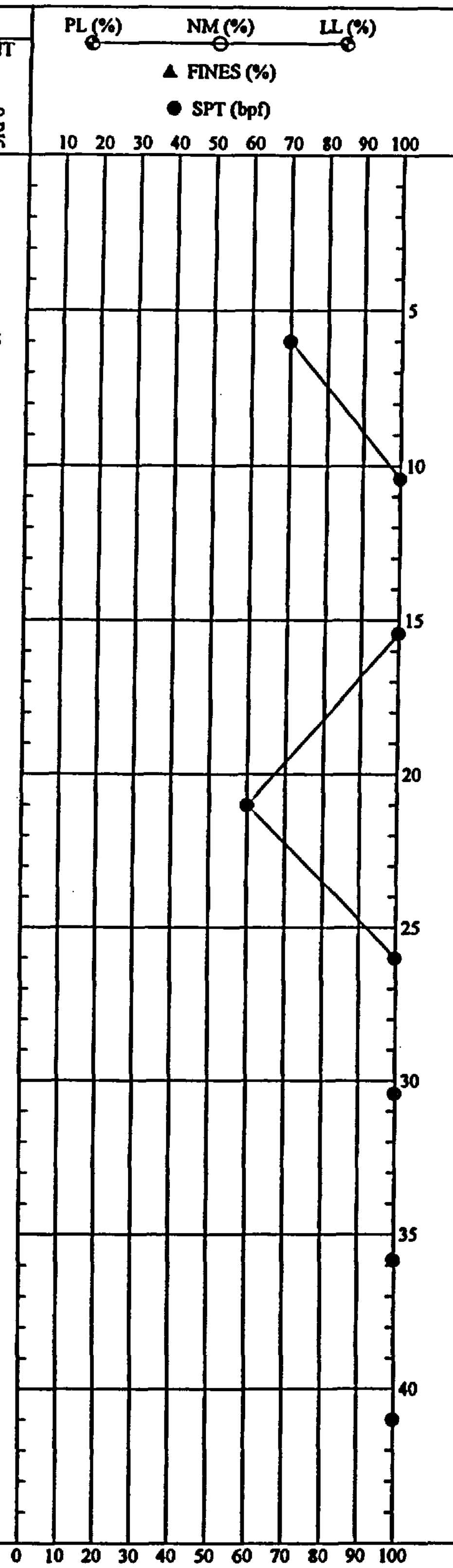
**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 9, 2003 **BORING NO.:** B-4

**PROJ. NO.:** 3043031049/0001 **PAGE 1 OF 1**

MACTEC

| DEPTH (ft) | SOIL CLASSIFICATION AND REMARKS<br>SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW. | ELEV (ft) | SAMPLES IDENT | N-COUNT (1st 6" / 2nd 6" / 3rd 6") |
|---|---|---|---|---|
| 0 | REDDISH BROWN, DRY, SILTY FINE SAND WITH SOME FINE ROOTS - EOLIAN | 3411.2 | | |
| | VERY DENSE, REDDISH YELLOW, DRY SILTY FINE TO MEDIUM SAND | | | |
| 5 | | 3406.2 | SPT-1 | 25-35-35 |
| 10 | VERY DENSE, LIGHT YELLOW, DRY, SILTY FINE TO MEDIUM CEMENTED SAND - CALICHE | 3401.2 | SPT-2 | 50/5" |
| 15 | | 3396.2 | SPT-3 | 50/5" |
| 20 | VERY DENSE, LIGHT YELLOW, DRY SILTY FINE TO MEDIUM SAND WITH CLASTS OF CEMENTED SAND | 3391.2 | SPT-4 | 15-24-36 |
| 25 | | 3386.2 | SPT-5 | 9-38-50/4" |
| 30 | VERY DENSE, REDDISH YELLOW, DRY, SILTY FINE TO MEDIUM SAND WITH SOME FINE SUBANGULAR TO ROUNDED GRAVEL | 3381.2 | SPT-6 | 50/5" |
| 35 | | 3376.2 | SPT-7 | 30-50/4" |
| 40 | VERY HARD, REDDISH PURPLE, DRY, HIGH PLASTICITY CLAY | 3371.2 | SPT-8 | 18-50/6" |
| | BORING TERMINATED AT 41.0' | | | |
| 45 | | 3366.2 | | |




SOIL 3045031049 01.GPJ LAW_GIBB.GDT 10/3/03

REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING A SAFETY HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION. BACK FILLED ON 9/10/2003.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

**SOIL TEST BORING RECORD**

**PROJECT:** NEF - Lea County, New Mexico

**DRILLED:** September 10, 2003 **BORING NO.:** B-5

**PROJ. NO.:** 3043031049/0001 **PAGE 1 OF 1**

MACTEC

# APPENDIX C

## LABORATORY TEST PROCEDURES

## LABORATORY TEST RESULTS

# LABORATORY TEST PROCEDURES

## Atterberg Limits

Originally, the Atterberg Limits consisted of seven "limits of consistency" of fine-grained soils. In current engineering usage, the term usually refers only to the liquid limit (LL) and plastic limit (PL). The LL (between the liquid and plastic states) is the water content at which a trapezoidal groove of specified shape, cut in moist soil held in a special cup, is closed after 25 taps on a hard rubber plate. The PL (between plastic and semi-solid states) is the water content at which the soil crumbles when rolled into threads of 1/8 inch in diameter.

The LL has been found to be proportional to the compressibility of the normally consolidated soil. The PI is the calculated difference in water contents between the LL and the PL. Together the LL and PI are used to classify silts and clays according to the Unified Soil Classification System (ASTM D 2487). The PI is used to predict the potential for volume changes in confined soils beneath foundations or grade slabs. The LL, PL, and PI are determined in accordance with ASTM D 4318.

## Moisture Content

The moisture content in a given mass of soil is the ratio, expressed as a percentage, of the weight of the water to the weight of the solid particles. This test was conducted in accordance with ASTM D 2216.

## Grain Size Distribution

Grain Size Tests are performed to aid in determining the soil classification and the grain size distribution. The soil samples are prepared for testing according to ASTM D 421 (dry preparation) or ASTM D 2217 (wet preparation). If only the grain size distribution of soils coarser than a number 200 sieve (0.074-mm opening) is desired, the grain size distribution is determined by washing the sample over a number 200 sieve and, after drying, passing the samples through a standard set of nested sieves. If the grain size distribution of the soils finer than the number 200 sieve is also desired, the grain size distribution of the soils coarser than the number 10 sieve is determined by passing the sample through a set of nested sieves. Materials passing the number 10 sieve are dispersed with a dispersing agent and

suspended in water, and the grain size distribution calculated from the measured settlement rate of the particles. These tests are conducted in accordance with ASTM D 422.

### Compaction Tests (Moisture-Density Relationship)

Compaction tests are performed on representative soil samples to determine the maximum dry density and optimum moisture content. The results of the tests are used in conjunction with other tests to determine engineering properties relating to settlement, bearing capacity, shear strength, and permeability. The results may also be used as a standard to determine the percent compaction of any soil embankment.

The two most commonly used compaction tests are the standard Proctor test and the modified Proctor test. They are performed in accordance with ASTM D 698 and D 1557, respectively. Generally, the standard Proctor compaction test is run on samples from building areas and areas where moderate loads are anticipated. The modified Proctor compaction test is generally used for analyses of highways and other areas where large building loads are expected. Both tests have three procedures, depending upon soil particle size:

| Test | Procedure | Hammer Weight | Hammer Fall | Mold Diameter | Screen Size (Material Finer Than) | Number of Layers | Number of Blows per Layer |
|---|---|---|---|---|---|---|---|
| Standard (D 698) | A | 5.5 lb. | 12" | 4" | No. 4 sieve | 3 | 25 |
| | B | 5.5 lb. | 12" | 4" | No. 3/8" sieve | 3 | 25 |
| | C | 5.5 lb. | 12" | 6" | 3/4" sieve | 3 | 56 |
| Modified (D 1557) | A | 10 lb. | 18" | 4" | No. 4 sieve | 5 | 25 |
| | B | 10 lb. | 18" | 4" | No. 3/8" sieve | 5 | 25 |
| | C | 10 lb. | 18" | 6" | 3/4" sieve | 5 | 56 |

Test results are presented as a curve depicting dry unit weight versus moisture content. The compaction method used and any deviations from the recommended procedures are noted in the report.

**Laboratory California Bearing Ratio Tests**

The results of the compaction test are utilized in compacting the test sample to the desired density and moisture content for the laboratory California Bearing Ratio test. The California Bearing Ratio, generally abbreviated CBR, is a punching shear test and is a comparative measure of the shearing resistance of a soil. It provides data that is a semi-empirical index of the strength and deflection characteristics of a soil that has been correlated with pavement performance to establish design curves. The CBR is used with empirical curves to design pavement structures.

A laboratory CBR test is conducted according to ASTM D 1883. A representative sample is compacted to a specified density at a specified moisture content. The test is performed on a 6-inch diameter, 4.585-inch-thick disc of compacted soil that is confined in a steel cylindrical mold. The sample is compacted in accordance with Method B or D of ASTM D 698 or D 1557. These compaction procedures are outlined in this report in the section on compaction tests.

CBR tests may be run on the compacted samples in either soaked or unsoaked conditions. During testing, a piston approximately 2 inches in diameter is forced into the soil sample at the rate of 0.05 inches per minute to a depth of 0.5 inches to determine the resistance to penetration. The CBR is the percentage of the load it takes to penetrate the soil to a 0.1-inch depth compared to the load it takes to penetrate a standard crushed stone to the same depth.

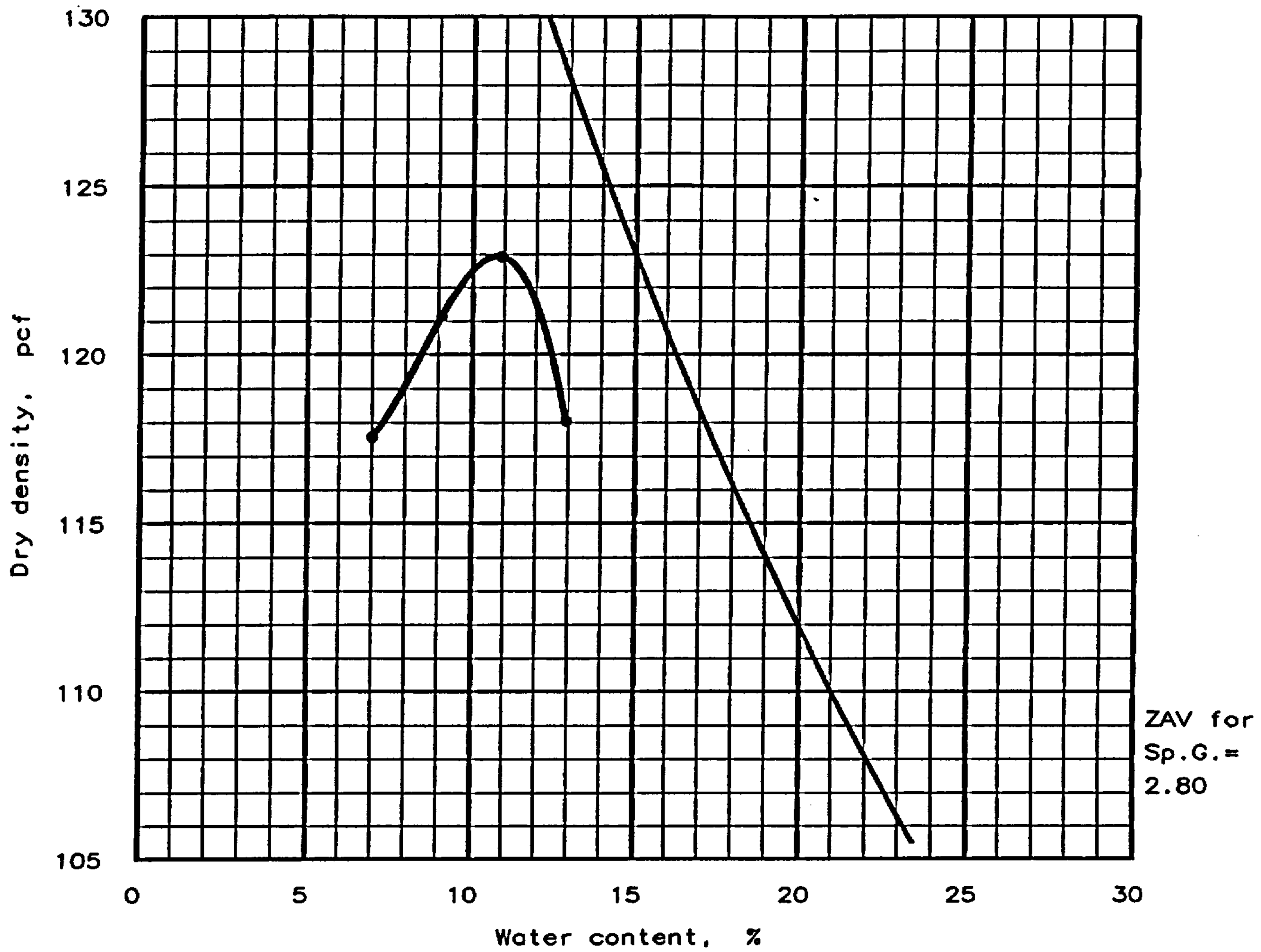
# Table C.1
## Soil Data Summary
## Index and Compaction Properties

<table>
<tr><th>Boring Number</th><th>Sample Type</th><th>Sample Depth (Feet)</th><th>Maximum Dry Density, pcf (ASTM D1557)</th><th>Optimum Moisture Content, % (ASTM D 1557)</th><th>Natural Moisture Content, %</th><th>Liquid Limit, %</th><th>Plastic Limit, %</th><th>Plasticity Index, %</th><th>Percent Finer Than #200 Sieve</th><th>pH</th><th>Resistivity (Ohm-cm)</th></tr>
<tr><td>B-2</td><td>SS</td><td>5</td><td></td><td></td><td>4.1</td><td></td><td></td><td>NP</td><td>16.7</td><td></td><td></td></tr>
<tr><td>B-2</td><td>SS</td><td>25</td><td></td><td></td><td>3.9</td><td></td><td></td><td>NP</td><td>18.9</td><td></td><td></td></tr>
<tr><td>B-2</td><td>SS</td><td>5</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="2">7.99</td><td rowspan="2">7,400</td></tr>
<tr><td>B-2</td><td>SS</td><td>10</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>5</td><td></td><td></td><td rowspan="2">7.5</td><td rowspan="2">26</td><td rowspan="2">17</td><td rowspan="2">9</td><td rowspan="2">31.0</td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>10</td><td></td><td></td><td></td><td></td></tr>
<tr><td>B-3</td><td>BULK</td><td>5 - 10</td><td>122.9</td><td>10.8</td><td>4.3</td><td></td><td></td><td>NP</td><td>24.8</td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>45</td><td></td><td></td><td rowspan="2">11.4</td><td rowspan="2">60</td><td rowspan="2">23</td><td rowspan="2">37</td><td rowspan="2">91.9</td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>50</td><td></td><td></td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>70</td><td></td><td></td><td rowspan="2">13.7</td><td rowspan="2">50</td><td rowspan="2">18</td><td rowspan="2">32</td><td rowspan="2">96.1</td><td></td><td></td></tr>
<tr><td>B-3</td><td>SS</td><td>75</td><td></td><td></td><td></td><td></td></tr>
<tr><td>B-4</td><td>SS</td><td>5</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="2">7.93</td><td rowspan="2">2,100</td></tr>
<tr><td>B-4</td><td>SS</td><td>10</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>B-4</td><td>BULK</td><td>0 - 15</td><td>122.8</td><td>9.6</td><td>3.2</td><td></td><td></td><td>NP</td><td>22.8</td><td></td><td></td></tr>
<tr><td>B-5</td><td>SS</td><td>5</td><td></td><td></td><td rowspan="2">2.8</td><td></td><td></td><td rowspan="2">NP</td><td rowspan="2">21.6</td><td></td><td></td></tr>
<tr><td>B-5</td><td>SS</td><td>10</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

Bulk - Bulk Sample
SS - Standard Penetration Test Sample
NP - Non-Plastic

Prepared By MBH Date 10-15-03 Checked By [signature] Date 10-15-03

# MOISTURE-DENSITY RELATIONSHIP TEST




Test specification: ASTM D 1557-02 Procedure B, Modified

| Elev/ Depth | Classification | | Nat. Moist. | Sp.G. | LL | PI | % > 3/8 in | % < No.200 |
|---|---|---|---|---|---|---|---|---|
| | USCS | AASHTO | | | | | | |
| 5-10' | SM | A-2-4(0) | 4.3 % | NT | NV | NP | 0.0 % | 31.2 % |

| TEST RESULTS | MATERIAL DESCRIPTION |
|---|---|
| Maximum dry density = 122.9 pcf<br>Optimum moisture = 10.8 % | Brown silty sand |

Project No.: 3043031049.0001

Project: NEF Lea County, New Mexico

Location: Boring B-3 Bulk

Date: October 13, 2003

Remarks:

Sample Number 2837

NT- No Test

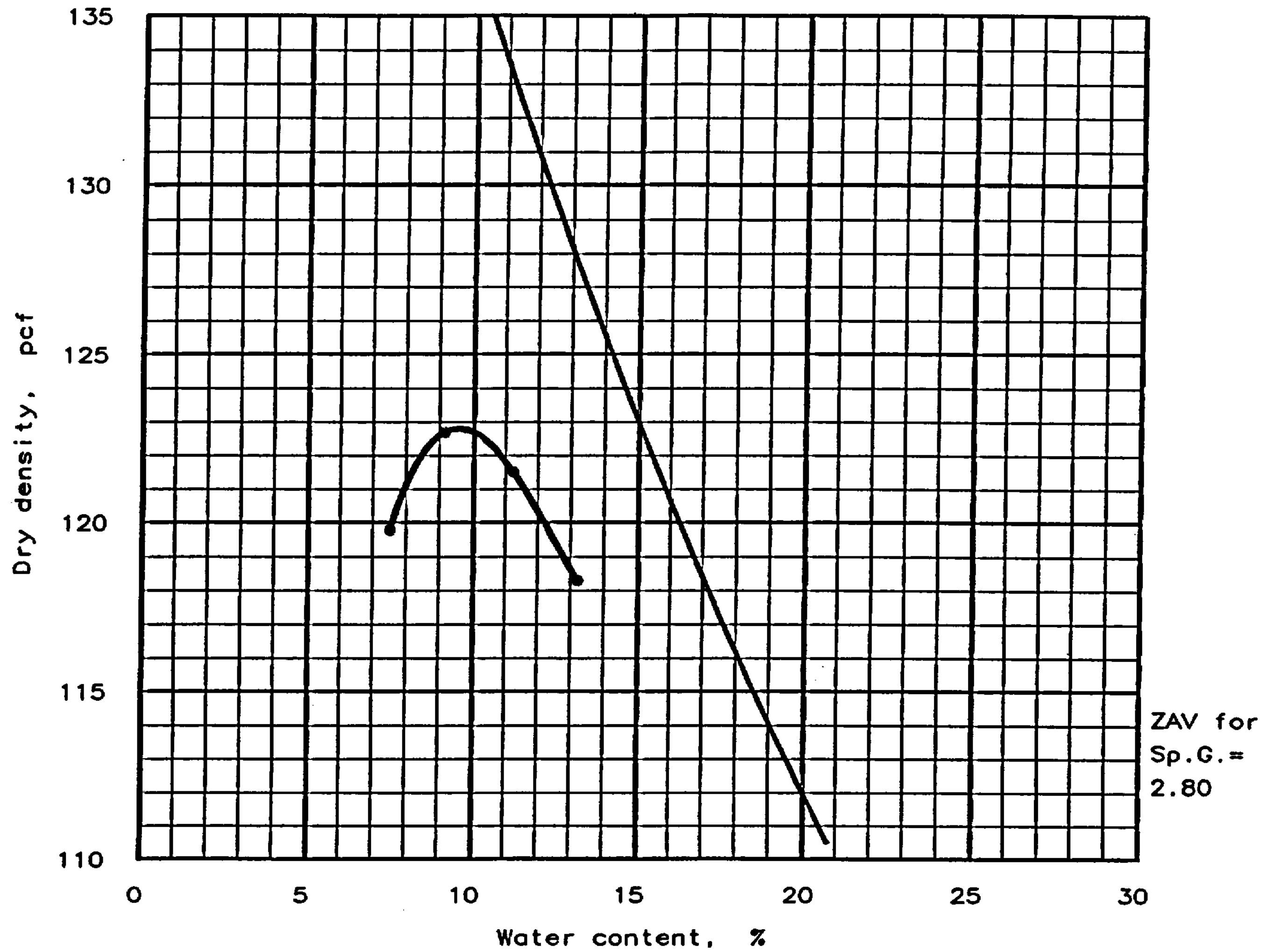
DNS- Data Not Submitted

MOISTURE-DENSITY RELATIONSHIP TEST

LAW ENGINEERING AND ENVIRONMENTAL SERVICES, INC.

Fig. No. 2837

# MOISTURE-DENSITY RELATIONSHIP TEST




Test specification: ASTM D 1557-02 Procedure B, Modified

| Elev/ Depth | Classification | | Nat. Moist. | Sp.G. | LL | PI | % > 3/8 in | % < No.200 |
|---|---|---|---|---|---|---|---|---|
| | USCS | AASHTO | | | | | | |
| 0-15' | SM | A-2-4(0) | 3-8% % | NT | NP | NP | 0.0 % | 22.7 % |

| TEST RESULTS | MATERIAL DESCRIPTION |
|---|---|
| Maximum dry density = 122.8 pcf<br>Optimum moisture = 9.6 % | Tan silty sand |

Project No.: 3043031049.0001

Project: NEF Lea County, New Mexico

Location: Boring B-4 , 0-7' and 7-15' combined

Bulk Sample

Date: October 13, 2003

MOISTURE-DENSITY RELATIONSHIP TEST

LAW ENGINEERING AND ENVIRONMENTAL SERVICES, INC.

Remarks:

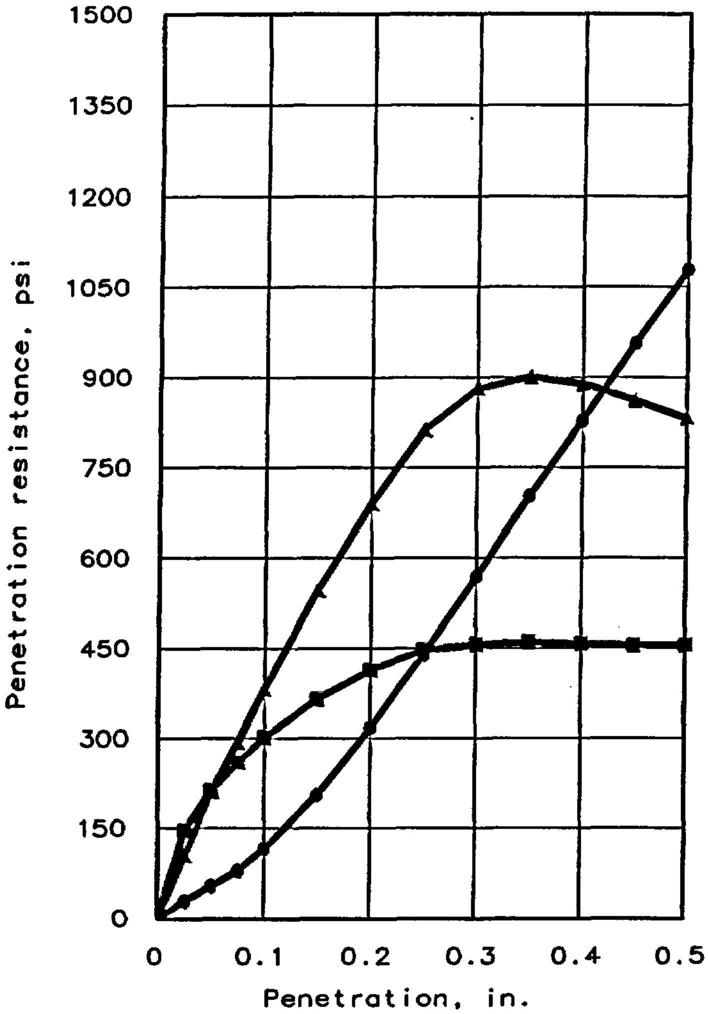
Sample Number 2836

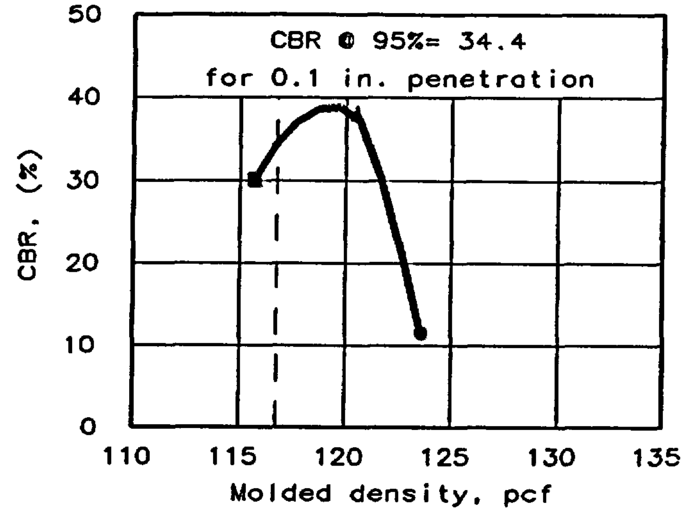
NT- No Test

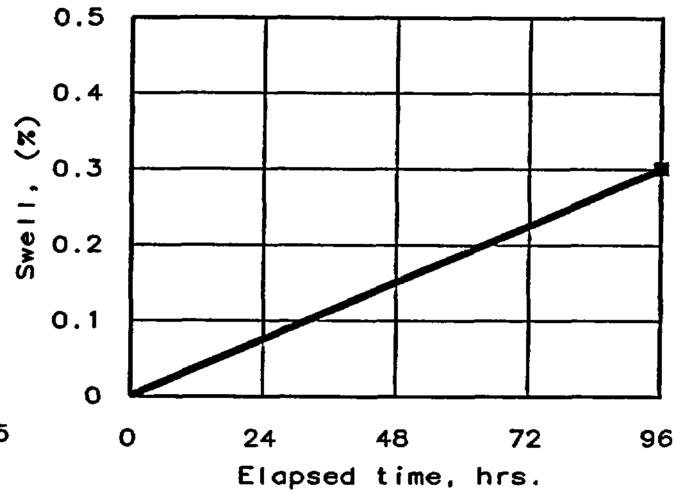
DNS- Data Not Submitted

Fig. No. 2836

# BEARING RATIO TEST REPORT




| | Molded | | | Soaked | | | CBR, (%) | | Lin. | Pen. | Swell |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Dens. | % max | moist | Dens. | % max | moist | 0.1" | 0.2" | Cor. | Sur. | % |
| 1 ● | 123.6 | 100.6 | 11.2% | 123.2 | 100.2 | 13.1% | 11.6 | 21.1 | 0 | 10.0 | 0.3 |
| 2 ▲ | 120.5 | 98.0 | 11.2% | 120.1 | 97.7 | 12.7% | 38.1 | 46.0 | 0 | 10.0 | 0.3 |
| 3 ■ | 115.7 | 94.1 | 11.1% | 115.4 | 93.9 | 13.9% | 30.1 | 27.5 | 0 | 10.0 | 0.3 |

| MATERIAL DESCRIPTION | USCS | Max. dens. | Opt. w.c. | LL | PI |
|---|---|---|---|---|---|
| Brown silty sand | SM | 122.9 | 10.8 | NV | NP |

Project No: 3043031049.0001
Project: NEF - Lea County, New Mexico
Location: Boring B-3, 5-10' Bulk

Date: October 13,2003

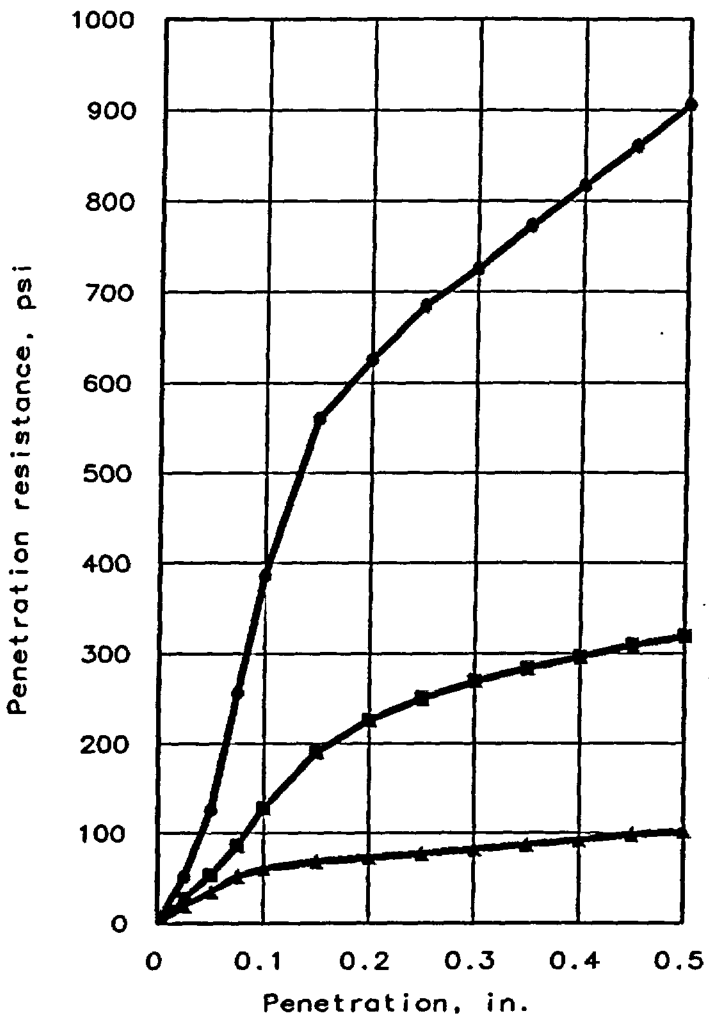
BEARING RATIO TEST REPORT

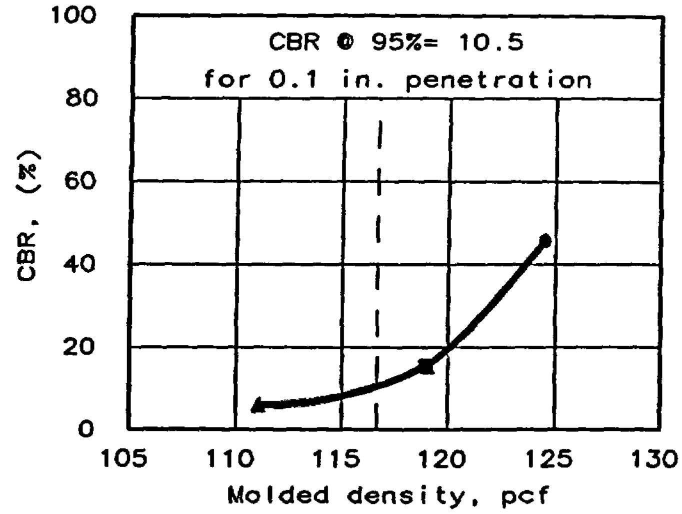
**LAW ENGINEERING AND ENVIRONMENTAL SERVICES, INC.**

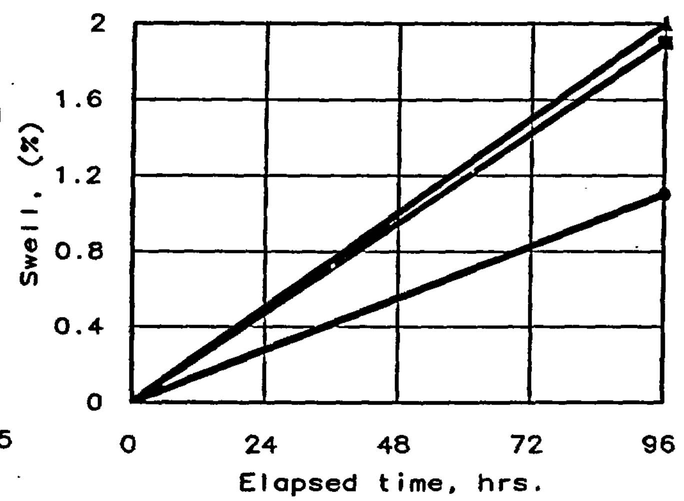
Test Descr./Remarks:
ASTM D1883-99 C.B.R.
ASTM D 1557-02 B /
Sample Number 2837
Penetration on
soaked samples.

Fig. No.: 2837

# BEARING RATIO TEST REPORT




| | Molded | | | Soaked | | | CBR, (%) | | Lin. | Pen. | Swell |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Dens. | % max | moist | Dens. | % max | moist | 0.1" | 0.2" | Cor. | Sur. | % |
| 1 ● | 124.5 | 101.4 | 9.5% | 123.1 | 100.2 | 14.0% | 45.5 | 43.3 | 0.020 | 10.0 | 1.1 |
| 2 ▲ | 111.1 | 90.5 | 9.7% | 108.9 | 88.7 | 18.5% | 6.0 | 4.8 | 0 | 10.0 | 2.0 |
| 3 ■ | 119.0 | 96.9 | 9.7% | 116.8 | 95.1 | 16.3% | 15.3 | 15.7 | 0.020 | 10.0 | 1.9 |

| MATERIAL DESCRIPTION | USCS | Max. dens. | Opt. w.c. | LL | PI |
|---|---|---|---|---|---|
| Tan silty sand | SM | 122.8 | 9.6 | NV | NP |

Project No: 3043031049.0001

Project: NEF - Lea County, New Mexico

Location: Boring B-4, 0-7' & 7-15' combined Bulk

Date: October 13,2003

BEARING RATIO TEST REPORT

LAW ENGINEERING AND ENVIRONMENTAL SERVICES, INC.

Test Descr./Remarks:

ASTM D1883-99 C.B.R.

ASTM D 1557-02 B /

Sample Number 2836

Penetration on

soaked samples.

Fig. No.: 2836