ANDERSON ENGINEERING CONSULTANTS, INC.
3217 NEIL CIRCLE - JONESBORO, ARKANSAS 72401
PHONE (870) 932-3700 FAX (870) 932-3769

March 15, 2004
AECI Job No. 197804

Mr. Mike Smith, EI
Castor Engineering and Surveying
P.O. Box 477
West Memphis, Arkansas 72303

Re: Geotechnical Investigation
Proposed Eakas Industrial Facility
Wynne, Arkansas

Dear Mr. Smith:

We have completed all of the borings for the proposed project, the laboratory testing is being finalized, and the final report will be delivered in accordance with the established schedule. Per your request, pertinent design information is being offered in advance of the final report.

The site of the proposed facility is in an open area on the east side of Arkansas State Highway 1 approximately 0.2 miles south of the city of Wynne, Arkansas. The site has been used historically for agricultural purposes. The near surface soils are basically alluvial deposits of silt (ML) or clayey silt (ML) that had a stiff to hard consistency. Groundwater was not encountered at the time of the investigation in the near surface soils. However, perched or latent groundwater should be expected in the top soil stratum in close proximity of the existing creek especially during prolonged rainy periods or in the wetter part of the year.

Based upon elevation it is most probable that the site is to be raised an estimated 1.0 to 3.0 feet in the building area to assure drainage. Prior to fill placement, approximately eight (8) inches of topsoil should be undercut and removed from structural or pavement areas. After undercutting, the near surface soils should be proof-rolled to locate any localized soft areas and if encountered they should be undercut to sound material and backfilled in 8 inch compacted lifts. The on-site soils may be used as fill and they will provide an adequate bearing capacity when compacted to 95% modified (ASTM D1557) compaction at a water content that is within 2± percent of optimum. However, these soils have a high silt content and this soil type tends to be moisture sensitive. These soils may be difficult to work with during prolonged wet periods. These soils will pump at water contents that are significantly above optimum and will become soft if allowed to saturate. The contractor should practice aggressive moisture control measures and maintain proper drainage at the site at all times during construction. Any off site soils used as fill should consist of a clayey sand (SC), sandy clay (CL), silty sand (SM), or clayey gravel (GC).

It is feasible that the footings will bear in natural ground or compacted fill. At the present the column loading condition for the structure is unknown. Therefore it is feasible that lightly loaded structures will bear on conventional footings and larger loads may bear on straight shaft drilled piers. Considering the results of the field investigation, the tests completed to date, and the potential fill that is to be placed at the site, the following preliminary design criteria is offered:

Foundation Type	
<u>Conventional Footings</u>	
Minimum Depth	2.5 feet
Bearing Strata	Natural ground or Compacted Fill
End Bearing Capacity	2,300 psf
<u>Drilled Piers</u>	
Minimum Diameter	3.0 feet
Minimum Depth	15.0 feet
End Bearing Capacity	3,500 psf
Skin Friction	1,000 psf
Total Settlement	0.50 inch
Differential Settlement	0.30 inch

The predominant soil encountered to the depths investigated is a dense sand (SP). The water table was encountered during the investigation at a depth of approximately 32.5 feet. The data for seismic design provided in the following table is considered applicable to this project site based upon the subsurface soil conditions and the values for Arkansas published by the Arkansas State Building Services, the 1999 Standard Building Code and the 2000 International Building Code. The class C site classification, groundwater, and sandy soils will require an evaluation of the site's liquefaction potential during a seismic event and this will be addressed in the final report. However, the risk of liquefaction at this site is considered very low because of the high silt content of the near surface soils and the dense sands at the greater depths.

SEISMIC DESIGN CRITERIA

Site Class (IBC)	C*
Seismic Zone (ASBS)	3
Soil Profile Type (SBC)	S_2
Site Coefficient (SBC)	1.2
Peak Acceleration Coefficient (A_a) (ASBS)	0.21
Effective Peak Velocity-Related Acceleration Coefficient (A_v) (ASBS)	0.22

Mr. Mike Smith, EI
Castor Engineering and Surveying
Page 2; 03/15/04

The information provided in this preliminary document is complete to date and we do not foresee any significant changes. However, AECI reserves the right to revise the information as additional testing is completed and to reflect any additional information as the project develops. If a variance in the preliminary recommendations provided in this document change then the appropriate parties will be notified as determined appropriate. We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this advance information, or if we may be of further service to you in any way, please do not hesitate to contact us.

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

Sincerely,
ANDERSON ENGINEERING CONSULTANTS INC.

Bill Alumbaugh

Bill Alumbaugh, P.E.
Vice President

Attachment: Bearing Capacity Versus Depth (2 Pages)

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 6536
BILLY R. ALUMBAUGH
3/15/04

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

Design Calculations for Conventional Footings

PROJECT: EAKAS INDUSTRIAL FACILITY
PROJECT NO.: 197804
BORING NO.: SELECT
TESTED BY: AECI
DATE: 03/12/04
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	11	2.9	1.5	3.6	0.188	1.9
4.0	1.5	4.0	2.5	14	3.7	1.8	4.6	0.500	2.6
6.5	4.0	6.5	2.5	16	4.2	2.1	5.3	0.813	3.0
9.0	6.5	9.0	2.5	11	2.9	1.5	3.6	1.125	2.4
11.5	9.0	11.5	2.5	13	3.4	1.7	4.3	1.438	2.9
16.5	11.5	16.5	5.0	16	4.2	2.1	5.3	2.063	3.7
21.5	16.5	21.5	5.0	15	4.0	2.0	4.9	2.688	3.8
26.5	21.5	26.5	5.0	19	5.0	2.5	6.3	3.313	4.8
31.5	26.5	31.5	5.0	32	8.8	4.4	11.0	3.938	7.5
36.5	31.5	36.5	5.0	33	9.1	4.6	11.4	4.563	7.9
41.5	36.5	41.5	5.0	35	9.8	4.9	12.2	5.188	8.4
46.5	41.5	46.5	5.0	49	15.4	7.7	19.2	5.813	12.1
51.5	46.5	51.5	5.0	46	14.0	7.0	17.5	6.438	11.4
56.5	51.5	56.5	5.0	50	15.9	7.9	19.8	7.063	12.7
61.5	56.5	61.5	5.0	49	15.4	7.7	19.2	7.688	12.6
66.5	61.5	66.5	5.0	50	15.9	7.9	19.8	8.313	13.0
71.5	66.5	71.5	5.0	50	15.9	7.9	19.8	8.938	13.2
76.5	71.5	76.5	5.0	50	15.9	7.9	19.8	9.563	13.3

WATER TABLE LEVEL: 32.5 ft.

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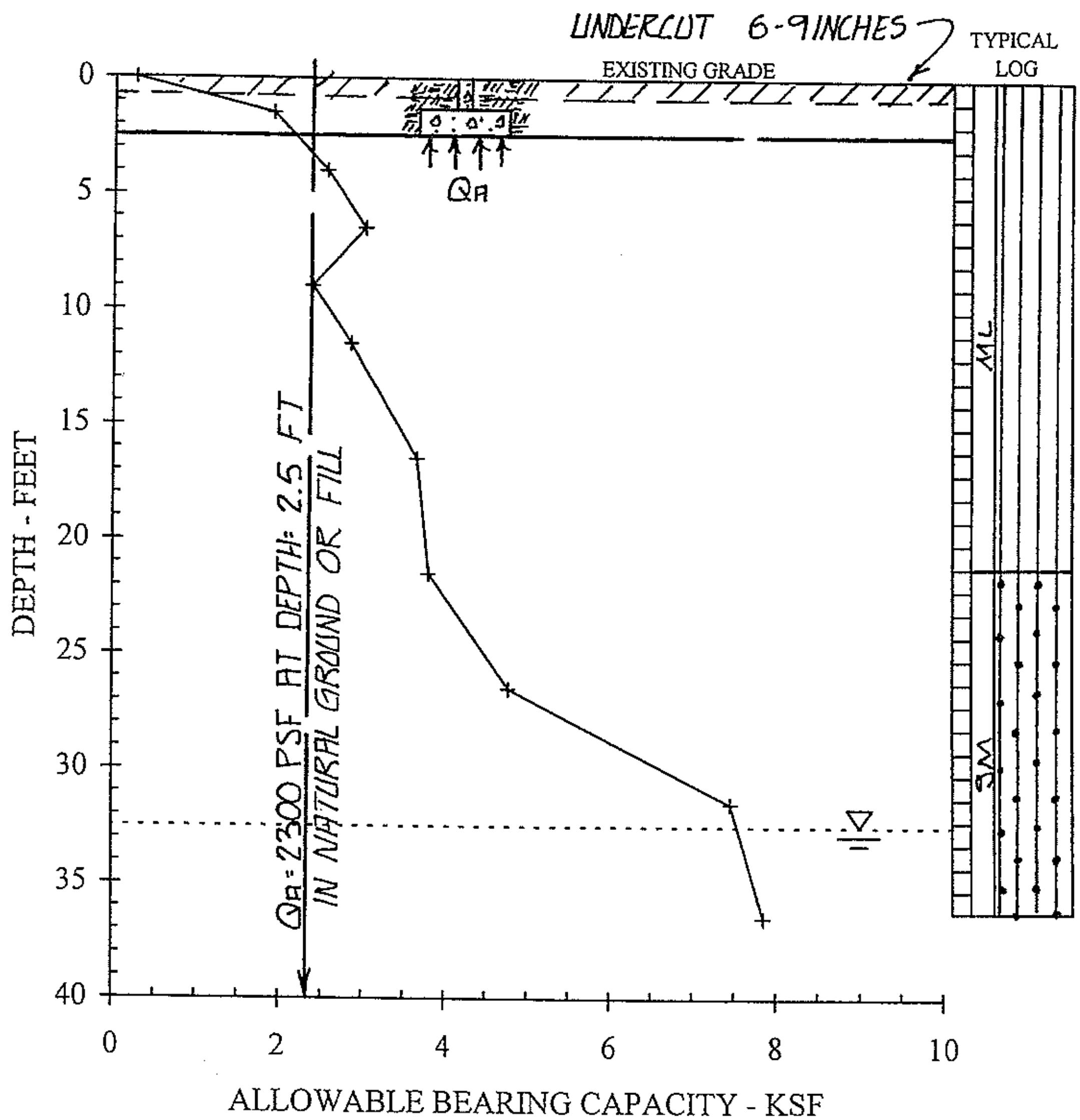
CONVENTIONAL FOOTINGS

PROJECT: **EAKAS INDUSTRIAL FACILITY**
WYNNE, ARKANSAS

BORING NO.: **SELECT**

PROJECT NO.: **197804** WATER TABLE: **32.5 ft.** SAFETY FACTOR: **2.00**



DEPTH - BEARING CAPACITY CURVE

ANDERSON ENGINEERING CONSULTANTS, INC.

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

March 24, 2004

Job No. 197804

Mr. Mike Smith, E.I.
Castor Engineering and Surveying
P.O. Box 477
Wynne, Arkansas 72396

Re: Geotechnical Investigation
Eakas Industrial Facility
Wynne, Arkansas

Dear Mr. Smith:

It is our pleasure to submit this report on the soil and foundation investigation for the referenced project in Wynne, Arkansas. The investigation consisted of field borings, soils laboratory analyses, and foundation and pavement design analyses.

We recommend that our geotechnical services be continued in the foundation construction phase of the project 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 adverse geotechnical conditions are encountered during excavation, they can be identified and evaluated so adequate remedial measures can be implemented during construction.

We wish to express our appreciation for the opportunity of serving you and members of the design team. We are available for further assistance at any time during final design and construction, should you desire additional consultation.

Very truly yours,

ANDERSON ENGINEERING CONSULTANTS, INC.

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

Scott W. Anderson, P.E.
Senior Geotechnical Engineer

BA/SWA/plf
197804.GEO

GEOTECHNICAL INVESTIGATION

FOR

EAKAS INDUSTRIAL FACILITY

WYNNE, ARKANSAS

* * * * *

CASTOR ENGINEERING AND SURVEYING

DESIGN ENGINEER

P.O. BOX 477

WYNNE, ARKANSAS 72396

* * * * *

BY

ANDERSON ENGINEERING CONSULTANTS, INC.

GEOTECHNICAL CONSULTANTS

3217 NEIL CIRCLE

JONESBORO, ARKANSAS 72401

MARCH 24, 2004 JOB NO. 197804

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

ABSTRACT

Eakas Industrial Facility	Job No. 197804
Date of Borings	2/19-20/04
Number of Borings	27
Maximum Depth Investigated	101.5 feet
Type of Samples	Standard Penetration

General Stratigraphy:

The near surface soils consists of a stiff, slightly sandy silt (ML) overlaying a silty clay (CL). The site's base stratum consists of a silty sand (SM) that becomes less silty and more dense with depth. The soil was alluvially deposited and isolated horizontal lenses and layers or silt, sand, and clay were periodically encountered within the depths investigated.

Water Table	9.0 feet
Frost Depth	10.0 inches
Earthwork (Specify)	95% ASTM D 1557 within 2% of optimum moisture
Swell Potential	Low
Potential Vertical Rise	<0.10 inch
Borrow Area Soils	
On-Site	PI < 15
Off-Site (Specify)	Select PI<15; (GC), (SC), (CL)
Conventional Footings	
Bearing Capacity - Natural Ground	2300 psf
Bearing Depth	2.5 feet
Total Settlement	0.50 inch
Differential Settlement	0.25 inch
Straight Shaft Drilled Piers	
Minimum Embedment	15.0 feet
Minimum Diameter	3.0 feet
End Bearing	3,000 psf
Skin Friction	1,000 psf

Note: <u>Undercutting of isolated soft or wet soils may be required in the building and parking areas during wet or winter months.</u>

Abstract - Continued

Pavements	Light Duty	Heavy Duty
Flexible:		
HMAC (AHTD Type II)	2.0″	3.5″
Clay Gravel Base (AHTD Class 5)	8.0″	10.0″
Compacted Subgrade	9.0″	16.0″
Rigid:		
Concrete	5.0″	8.0″
Clay Gravel Base (AHTD Class 5)	4.0″	6.0″
Compacted Subgrade	8.0″	8.0″

PURPOSE

The primary purposes of this geotechnical investigation were:

a. To determine the physical and engineering properties of the soils within the area of the proposed construction with respect to their suitability for the support of the proposed facility.

b. To make recommendations for the earthwork, pavements, and type of foundation(s) suited for the prevailing soil conditions within the proposed construction area.

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

SCOPE

The scope of this geotechnical investigation includes the following:

a. The geological features of the job site area consist essentially of alluvial silty clay soils overlaying silty sands. Thus, the site stratigraphy was defined by 27 wash rotary borings terminated at 6.5 to 101.5 feet.

b. Field testing consisted of Standard Penetration test samples taken in all of the borings. Soils were visually classified in the field by a soils engineering technician.

c. The soils analyses were based on N-values obtained from the Standard Penetration tests, moisture content, Atterberg limits, unconfined compression tests, swell tests, visual observations, and other routine inspection and classification methods. The soils were classified basically in accordance with the Unified Soils Classification System; however, visual classifications are given on the logs.

d. The foundation bearing capacity and settlement analyses were based on our current foundation design procedures, using the Standard Penetration N-values obtained during drilling and the results of the laboratory testing program.

e. The flexible and rigid pavement designs shown in this report are based on the CBR design method estimated from field and laboratory tests in the top 5.0 feet of soil in the pavement areas of the site.

AUTHORITY

This geotechnical investigation was authorized by signed Proposal, dated February 16, 2004, by Mr. Bill Thomas of the Cross County Economic Development Commision, the developer for the proposed project.

GEOTECHNICAL INVESTIGATION

On February 19-20, 2004, 27 geotechnical test borings were made at the proposed site in Wynne, 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 24. The Field Classification System for Soil Exploration and Key to the Soil Classifications and Symbols are given on Plates 25 and 26, respectively. The Unified Soils Classification System is given on Plate 27.

GEOLOGY AND STRATIGRAPHY

The site of the proposed project is located approximately 0.3 miles south of of the city of Wynne, Arkansas on the east side of Arkansas State Highway No. 1. The proposed site has been most recently used for agricultural purposes. The site is relatively flat and it generally slopes from the northeast to the southwest with an elevation variance of approximately 2.0 feet. There is a creek/drainage relief bordering the south side of the property and the water level in the creek was approximately 6 feet below the property at the time of the investigation. The surface soils were relatively stiff; however, there were some isolated areas that were "ponding water" at the time of the investigation and the soils were soft at and in close proximity of these areas. Therefore, the contractor should anticipate some difficulty in excavating the near surface materials in the wetter periods of the year. Some undercutting and replacement may be necessary to facilitate construction.

The Wynne, Arkansas, area lies within the Mississippi Embayment Physiographic region of eastern Arkansas. This area consists of alluvial and terrace deposits of silts, clays, and sands with lenses of clay and gravel. The soils range, in general, from silts to sands. The site stratigraphy essentially consists of a near surface layer of stiff sandy silt (ML) that has been highly weathered and modified as a result of farming operations. The subsequent strata consists of very stiff, silty clay that transitions to a dense silty sand (SM). The sand layer becomes more dense and less silty with depth. The site soils were found to be consistent with the area geology.

GROUNDWATER CONDITIONS

Groundwater was encountered at the proposed site during this investigation at a depth of approximately 9.0 feet. This water level is considered to be a perched or latent water condition and will rise and fall with fluctuations in rainfall and the water level in the creek bordering on the south of the property. The hydrostatic water table was encountered in the silty sand layer at an approximate depth of 25.0 feet.

Perched water should be expected in the near surface. This latent water condition is typically due to storage of recent rainfall or by a barrier to capillary evaporation. Perched water if encountered will most likely be brief in duration and typically in low quantities. Areas likely to contain perched water include old drainage swales, existing utility trenches, in soil that has been modified for agricultural purposes, and within the dripline of historic trees.

Some groundwater can be expected in the near surface soils and should be considered in the design and construction of deep excavations and utility installation. Where perched water is encountered the contractor should expect to excavate gravity drainage ditches to divert it away from the construction area. Additionally, soft, wet and pumpable soils should be expected. In structural areas these soils should be removed and be replaced with a select granular fill soil compacted to project specifications. Since the quantity of undercut is unknown it would be prudent to establish a unit rate for this item of work to minimize construction delays.

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 foundations and structures should be designed using guidelines as set forth in the 1999 Standard Building Code as required by **Arkansas Act 1100-1991** (and subsequent amendments) or the International Building Code, as determined appropriate.

The predominant soil type at the site is a dense sand (SP) that is overlaid by a cohesive overburden. The data provided in the following table is considered applicable to this project site 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. Consultation with the Wynne Department of Public Works in regard to their specific regulations for construction is recommended.

SEISMIC DESIGN CRITERIA

Site Class (IBC)	C
Seismic Zone (ASBS)	3
Soil Profile Type (SBC)	S_2
Site Coefficient (SBC)	1.2
Peak Acceleration Coefficient (A_a) (ASBS)	0.21
Effective Peak Velocity-Related Acceleration Coefficient (A_v) (ASBS)	0.22

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 from an earthquake. When soils experience liquefaction they loose all strength to resist load and temporarily exist in a near liquid state. But, liquefaction is primarily associated with saturated, loose to medium dense cohesionless soils. At this site, very stiff cohesive soils were found to overlay dense cohesionless soils at the proposed site. This near surface layer will help serve as a confining layer for the underlying soils. Additionally the cohesionless soils encountered at the site are dense to very dense silty sands overlaying the poorly graded sands encountered at a depth below 70.0 feet. Seed's stress procedure was used to estimate the liquefaction potential of the site and based upon the blowcounts of the standard penetration tests encountered at the site and the results indicate that these soils are typically not susceptible to a liquefaction type failure mechanism. This was confirmed by the Liquefaction analysis furnished on Plates 28 through 30. Since the probability for liquefaction at this site is considered low then the probability of lateral displacement (spreading), bearing capacity reduction, and differential settlement associated with the liquefaction is also relatively low.

LABORATORY TESTING

Tests were performed on select samples to determine their classification and/or strength characteristics. Laboratory testing included Atterberg limits, mechanical analyses, unconfined compressive strengths, PVC swell index tests, and the establishment of the moisture-density relationship for the on-site soils. 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 cohesive soils encountered in these borings was found to consists primarily of a borderline silty clay (CL) or clayey silt (ML). The liquid limit (LL) for the cohesive material ranged from 18 to 46, with plasticity index (PI) ranging from 1 to 23, resulting in a generalized classification of a silty clay (CL).

Mechanical Analyses

Mechanical grain size analyses were performed to develop a profile of the Minus No. 200 sieve percentage for use in the liquefaction analysis. The results indicate that the foundation soils become coarser with depth. The near surface soils are clays and silts that overlay sands that become coarser with depth. The minimum minus No. 200 sieve percentage in the granular material was found to be less than 11.7% clay-silt percentage which decreases the susceptibility of these soils to liquefaction.

Unconfined Compression Tests

Unconfined compression tests were performed on selected samples in their natural moisture content to correlate with field N-values to predict the in-situ bearing capacity and settlement. The samples investigated resulted in a variable strengths ranging from 0.3 to 2.0 ksf. The moisture content for the samples were found to be 28.1% and 24.4% with dry unit weights of 85.5 and 95.3 pcf, respectively. The test confirm the susceptibility of the near surface soils to water and a significant decrease in the soil's shear strength should be expected if the soils are allowed to become saturated during construction.

Swell Tests

A PVC swell test was performed on a selected sample that was dried to a moisture content that was less than the soil's plastic limit to determine the swelling effects of the sample when saturated. The test was performed on a sample in the 5.0 to 6.5 feet depth which typically will be the most susceptible to moisture change at this site. The plasticity index (PI) of the soil tested was determined to be 23. The swell test indicated a negligible swell pressure; however, a potential vertical rise of 0.10 inch is considered prudent in the foundation design to account for isolated areas of higher PI.

Moisture-Density Relationship

The strength properties of the borrow area soils were analyzed for their moisture-density relationship using a Modified Proctor (ASTM D 1557) compaction effort. The test results indicate that modified compaction efforts should be used to increase the soil's strength and that the on-site soils compacted to this degree will be suitable as fill. The near surface cohesive soil was found to have a maximum dry density of 112.0 pcf at an optimum moisture content of 14.5%. However, the on-site soils in the proposed borrow area have a high silt (ML) content and these soils will pump at water contents that are significantly above optimum. Therefore, it is recommended that the water content at placement should be maintained within the range of ±2 percent of the optimum moisture content.

EARTHWORK

Prior to cut and placement of fill on the site, approximately 8.0 inches of topsoil and vegetation should be removed where present. After stripping and undercutting, proof rolling with a loaded, tandem-axle truck is recommended to locate potential soft spots 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 spots in the natural ground detected by proof rolling should be removed and replaced with compacted stable soil. After stripping, the top 8.0 inches of natural ground should be scarified and compacted to the maximum achievable ASTM D 1557 density as a bridging lift and inspected by the soils engineer or his representative prior to fill placement. Fill should be placed in 8.0-inch, loose lifts, compacted to 95% ASTM D 1557 within two percentage points of optimum moisture content.

Additionally, the test data indicates that the upper surface soils in the proposed on-site borrow area are moderately plastic. On-site material with a PI of less than 15 may be used for fill when dried and it will have good compaction and strength properties when placed at plus or minus two percentage points of optimum moisture content. The overburden soils will, however, pump when the moisture content greatly surpasses the optimum moisture content. The contractor should be prepared to provide temporary construction drainage to facilitate drying of wet soils. Undercutting and replacement may also be required of the soils during wet weather. This may include some additional undercutting to gain access to the site.

At the time of this investigation, the average moisture content of the near surface site soils was well over the optimum moisture content to achieve the required density. Therefore, the contractor should anticipate significant drying and aerating of the on-site soils to achieve 95% Modified compaction. Any off-site fill soils required should be granular, non-expansive type soils and have a PI of less than 15. Acceptable off-site fill materials should be limited to clayey gravel (GC), clayey sand (SC), or a sandy clay (CL). Each of these materials is locally available within a reasonable haul distance. Silty soils may be used; but, stability problems may be periodically encountered if these materials are allowed to become wet or saturated.

Site grading and earthwork operations using on-site soils will be more difficult in wet or winter weather. The on-site near surface silty soils will absorb significant quantities of water which will require significant aeration and working to dry during the wetter seasons. The amount of drying can be reduced 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.

FOUNDATIONS

The foundation loads were not known at the time of the investigation but moderate to heavily loaded industrial facilities typically have a wide range of structural loading conditions. Because of this large range conventional footings, straight shaft drilled piers, or combinations thereof may be used at the site. The design information for the different foundation systems are furnished as follows:

Conventional Foundations

Conventional strip or spot footings with a slab-on-grade may be used at the site as allowed by the structures loading condition. The foundations may be designed using an allowable bearing capacity of 2300 psf at a depth of 2.5 feet into the natural ground or compacted fill. The footings should be designed for a total and differential settlement of 0.50 and 0.25 inch, respectively. Bearing capacities at other depths into the natural soil may be determined from Plates 31 and 32 which show calculations and curves for the bearing capacity with depth for the site. Plate 33 shows an explanation of calculations for conventional spread footings. The bottom of the footing excavations should be either cleaned by hand to remove loose soil or be compacted to consolidate the near surface soils that may have been loosened during the excavation process in an effort to minimize potential differential settlement. Inspection by the soils engineer or his representative is recommended to verify that the allowable bearing value has been achieved in all of the footing excavations prior to concrete placement so that corrective action could be taken should any conditions be encountered which differ materially from that assumed to prevail in the design stage.

Straight Shaft Drilled Piers

Straight shaft drilled piers may be used under the larger concentrated loads that may be encountered in a warehouse or industrial type structure. The straight shaft drilled piers should have a minimum depth of 15.0 feet and a minimum diameter of 3.0 feet. The piers will have an allowable end bearing of 3,000 psf and a circumferential skin friction of 1,000 psf. The drilled piers should be designed for a total and differential settlement of 0.5 and 0.25 inch, respectively; which will make this foundation type compatible with the convetional footings if a combination pier and footing foundation system is chosen for the proposed facility. Belled or under reamed drilled piers should not be used at the site due to the relatively low plasticity of the foundation depth soils.

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 at least 18.0 inches of compacted soil or firm subgrade. Some undercut and replacement may be necessary to provide a suitable subgrade for support of the floor slabs. The use of an impermeable vapor barrier (visqueen) underlain with a 4.0 inch granular moisture barrier is recommended for the proposed structure. Acceptable moisture barriers include: a well graded sand (ASTM C33), a well graded gravel (ASTM C 33, No. 57), or an approved field sand providing the percent passing the No. 200 sieve is less than 5.0 percent. A modulus of subgrade reaction (k) equal to 150 pci can be used for design of conventional slab-on-grade floor slabs if all earthwork criteria are met.

PAVEMENTS

The following pavement designs and recommendations are based on numerous reasonable assumptions concerning the pavement use, site conditions, and maintenance. The pavement designs presented herein are based on the earthwork recommendations presented earlier and an assumed CBR value of 3 based on correlation with the soil physical properties, including plasticity, mechanical grain size analyses, and strength.

Flexible Pavement:

Based upon a CBR of 3, the required parking lot pavement structure for light duty pavement would consist of 9.0 inches of compacted subgrade, 8.0 inches of stone base course (AHTD Class 7), and 2.0 inches of hot mix (AHTD, 1996 ed. Type II, AHTD 2000 ed. 12.5 mm). For heavy duty pavements, 16.0 inches of compacted subgrade, 10.0 inches of stone base course, and 3.5 inches of hot mix would be required. The recommended flexible pavement structures are shown on Plate 34. The base course should be compacted to minimum of 95% Modified compaction to properly support the flexible pavement.

Rigid Pavement:

As an option to the proposed flexible pavement, a non-reinforced concrete pavement may be utilized. The light duty pavement areas should consist of 5.0 inches of concrete, 4.0 inches of stone base, and 8.0 inches of compacted subgrade. The heavy duty pavement areas (including access to dumpsters or truck docks) should consist of 8.0 inches of concrete, 6.0 inches of clay gravel base, and 8.0 inches of compacted subgrade. Plate 34 shows the recommended rigid pavement structures. The base course should be compacted to a minimum of 95% Modified compaction to properly support the concrete pavement. The paving concrete should have a minimum 28-day compressive strength of 4000 psi and be entrained with 5% air as recommended by the ACI code. The jointing pattern and load transfer devices should be as recommended by the ACI and the PCA criteria.

The long term pavement performance will be directly related to several factors such as adequate edge drainage and surface drainage which does not allow water to accumulate on the pavement surface or behind the curbs and pavement edges. All pavement joints must be sealed and should be placed parallel to the overall site drainage direction. All irrigation, water, and other utility lines should be carefully monitored to insure they do not contribute to premature pavement failure by allowing water to migrate onto or under the pavements. Adequate quality control testing including proof rolling, compaction testing, thickness testing of base and HMAC as well as compaction of the HMAC is critical to successful long term pavement performance. In addition, pavements will require regular maintenance such as periodic surface sealing and crack sealing to prolong the desired performance and life.

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 required fill should be compacted to a minimum of 95% Modified compaction, in accordance with ASTM D 1557. The footing and or drilled pier 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. A geotechnical engineering representative should be present to evaluate the bottom of the footing excavations by means of a static cone penetration device. The compaction of the pavement sections should be verified by tests after the earthwork is completed, so as not to invalidate the design criteria. Our recommendations are based upon adequate quality control testing being utilized and further evaluations and reviews during the construction phase of the project.

CONCLUSIONS AND RECOMMENDATIONS

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

1. As previously discussed, conventional footings and/or straight shaft drilled piers would serve satisfactorily for the proposed structure. It is concluded that these will be economical foundations and should be designed in accordance with the necessary structural and/or architectural requirements determined by the designers with the owner's ultimate approval.
2. The conventional foundations should be designed utilizing a maximum allowable bearing of 2,300 psf at a depth of 2.5 feet below final grade when founded on firm stable natural soils or compacted fill. Bearing capacities at other elevations may be determined from Plates 31 and 32 as determined appropriate and with the Geotechnical Engineer's concurrence.

3. Straight shaft drilled piers should be designed for an allowable end bearing capacity of 3,000 psf and a circumferential skin friction of 1,000 psf at a minimum depth of 15.0 feet. The minimum diameter of the drilled piers should be 3.0 feet.
4. Soil at the site or other low PI, non-expansive granular fill shall be placed in 8.0-inch thick lifts and be compacted within two percentage points of optimum moisture content to 95% Modified Proctor density as per ASTM D 1557. The select off-site fill shall not have a PI in excess of 15 and should consist of a select clay gravel (GC), clayey sand (SC), or lean clay (CL).
5. Draining of any perched water encountered during construction and undercutting of soft, wet or pumping soils will be required as indicated previously. The contractor should provide adequate drainage and pumping equipment to facilitate construction.
6. Quality control testing should be utilized in the construction of the foundation, undercutting, fill placement, and floor slab construction with adequate testing to verify that the design requirements have been achieved.
7. Geotechnical engineering services should be utilized in the foundation construction phase, and our recommendations are based upon this so that adequate compensation can be made for conditions that may occur which differ significantly from those assumed as a result of this investigation.
8. Other recommendations are given throughout the text of this report.

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