LIMITED GEOTECHNICAL INVESTIGATION

FOR

PROPOSED COMPASS INDUSTRIAL PROJECT

OSCEOLA, ARKANSAS

* * * * *

GREAT RIVER ECONOMIC DEVELOPMENT FOUNDATION

OWNERS

4701 MEMORIAL DRIVE

BLYTHEVILLE, ARKANSAS 72315

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AUGUST 1, 2006

JOB NO. 256006

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August 1, 2006

Job No. 256006

Mr. Clif Chitwood
Great River Economic Development Foundation
4701 Memorial Drive
Blytheville, Arkansas 72315

Re: Limited Geotechnical Investigation
Proposed Compass Industrial Project
Osceola, Arkansas

Dear Mr. Chitwood:

It is our pleasure to submit the results of this limited geotechnical investigation for the Compass Industrial Project in the City of Osceola, Arkansas. The investigation consisted of widely spaced field test borings, laboratory testing, and general foundation design analyses and recommendations.

The proposed 2000-acre site primarily on the protected side of the Mississippi River flood control levee, south of East State Highway 198, with frontage on the Mississippi River. The borings placed on the site are widely spaced and are basically used to indicate general site conditions within the investigated area. The soils are all alluvially deposited and consists of a complex layering of sands, silts, clays and combinations thereof. The surface soils have a consistency that generally ranges from a medium stiff to stiff sandy clay which typically overlays a soft to medium stiff silt (ML) or clay (CL and CH) layer to a depth of approximately 25.0 feet. This stratum is underlain by a basal, dense to very dense sand (SP) to the terminal depths investigated.

Based upon this preliminary investigation, the site is suitable to sustain applied loads typical of a heavy industrial site. Conventional and deep foundations are typically economical and they may be utilized as appropriate to support the expected structures. We recommend that our geotechnical services be continued when specific locations and loads are determined for the various structures for this is the most feasible means of assuring the owners, designers, and builders that the geotechnical design intent is being achieved. In the event other adverse geotechnical conditions are encountered in the specific structure locations, they can be identified and evaluated so that safe and economical structures may be designed.

We wish to express our appreciation for the opportunity of serving you and other members of your design team. Please contact us at any time during the design and construction should you have a need for further assistance.

Very truly yours,

ANDERSON ENGINEERING CONSULTANTS, INC.

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

Scott W. Anderson, P.E.
Principal Engineer

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

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 6536
BILLY R. ALUMBAUGH
07-01-06

BA/SWA/plf
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BY

ANDERSON ENGINEERING CONSULTANTS, INC.

GEOTECHNICAL CONSULTANTS

10205 ROCKWOOD ROAD

LITTLE ROCK, ARKANSAS 72204

AUGUST 1, 2006 JOB NO. 256006

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Important Information About Your Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

ASFE

The Best People on Earth

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PURPOSE

The primary purposes of this limited geotechnical investigation were:

a. To determine the feasibility of construction at the proposed site with respect to physical and engineering properties of the soil for the support of the proposed facility, equipment, and appurtenances.

b. To evaluate and recommend the general design criteria for the various soil, foundation, and other earthwork items in accordance with current geotechnical engineering practices.

c. To make general recommendations for the earthwork and to determine preliminary foundation design criteria which will be most compatible and economically prudent for the prevailing soil conditions within the proposed construction area.

SCOPE

The scope of this geotechnical investigation includes the following:

a. The general geological features of the area are typical of the Mississippi Embayment Physiographic Region which consist of sands, silts, and clays and combinations thereof. The client requested and authorized a total of fifteen borings for the preliminary subsurface investigation to obtain generalized geotechnical engineering data site and to assess the site to determine its capability of supporting the loads of a typical heavy industrial site. The Vicinity Map on Plate 1 shows the physiographic location of the site in relation to its surroundings in Osceola, Arkansas.

b. Accordingly, the site investigation and analyses reported herein involved the following:

   1) The site stratigraphy was defined by utilizing widely spaced geotechnical test borings placed strategically over the site at locations as shown on the Plan of Borings, Plate 2.

   2) Field test borings for the geotechnical analyses consisted of wash rotary borings with Shelby tube and Standard Penetration samples taken at selected depths. The logs of the borings are shown on Plates 3 through 17. The field classifications and

symbols are given on Plates 18 and 19, respectively. These systems are provided to aid the reader in interpreting the various symbols used on the log of boring. The soils were classified from visual observations and laboratory tests with respect to the Unified Soil Classification System shown on Plate 20. The resulting assumed stratigraphy is provided on Plate 21.

3) The soils laboratory testing included Atterberg limits, mechanical grain size analysis, unconfined compression tests, direct shear tests, and other routine classification tests. Individual laboratory test data is provided in Appendix B.

4) The proposed site is located in a seismic zone 3 area and a limited liquefaction analysis was performed on the near surface soils. The cohesive soils and natural silty sands in the area are relatively stable. The results of the limited liquefaction analysis is provided on Plates 22 through 24.

4) The foundation bearing capacity analysis were based on AECI's current foundation design procedures using the Standard Penetration N-values and the results of the laboratory testing. The calculations and curves for the allowable bearing capacity for conventional shallow foundations are provided on Plates 25 and 26. An explanation of the calculations for conventional footings is provided on Plate 27. Conventional footings will be suitable for relatively lightly loaded areas; but, auger cast pile foundations will be more economical for larger more concentrated loading conditions, i.e. columns, heavily loaded mats. The calculations and curves for a 16-inch diameter pile are provided on Plates 28 through 30. An explanation of the calculations is provided on Plate 31.

AUTHORITY

The geotechnical investigation was authorized by signed acceptance of AECI Proposal No. 24106 dated June 27, 2006 by Mr. Clif Chitwood, representing the Great River Economic Development Foundation.

GENERAL SITE CONDITIONS

The proposed site is approximately 2000 acres in size and is located south of East Arkansas State Highway 198, east of U.S. Hwy 61, with frontage on Mississippi River and south of the city of Osceola, Arkansas. The site is predominately located on the land side (protected side) of the Mississippi River flood control levee with general access. The property is currently being used for agricultural purposes with no visual evidence of past structural usage. The site is generally level within the proposed construction area with the site being primarily drained with constructed ditches.

REGIONAL GEOLOGY AND STRATIGRAPHY

The proposed site is in the Mississippi Embayment Physiographic region of eastern Arkansas. Soils associated with the Mississippi Embayment are usually alluvial sediments deposited by the Mississippi River and its ancient tributaries. These soils usually consist of Quaternary Age silts, sands, and clays.

The stratigraphy of the site is highly variable as expected due to multiple geologic forces since the glacial age. Typically the site consists of overburden sediments of local rivers and streams occurring in varying sequences as a result of glacial outwash. Fine grained soils such as silts and clays indicate low flow periods, whereas sand/gravel deposits indicate more rapid flow. The project site shows many thin and variable layers of silt, sand, and clays within its top 30.0 to 40.0 feet. A stratigraphy of the top 20.0 feet is shown on Plate 21. The surface soils are generally more clayey but grade coarser with depth. Below 40.0 feet site soils become sandy and remain so to the terminal depths explored.

GROUNDWATER CONDITIONS

Groundwater is closely tied to the nearby Mississippi River and at the time of the investigation was approximately at a depth of 12.0 feet. Thus, when the river is at flood stage and the water is retained by the levee, the groundwater at the site will rise to the surface and may even be placed under hydrostatic pressure. Therefore, high water levels should be accounted for in the design and construction of all subsurface features.

Where high water levels are encountered during construction the contractor should be prepared to provide temporary construction drainage consisting of either gravity drainage ditches or sump/pump methods. In large excavation areas a deep well(s) or numerous well points may dewater the localized area more economically and efficiently. Additionally, anywhere a high groundwater table is encountered, soft, wet, and pumpable soils should be expected that may require removal and replacement with select compacted fill.

SEISMICITY

The published earthquake information (U.S. Geologic Survey (USGS), Publication FS-131-02) for the New Madrid fault zone indicates that there is a only a 25% to 40% probability of a magnitude 6.0 earthquake by the year 2050. The published time duration between magnitude 8.0+ earthquakes, similar to the 1811-1812 seismic events, is approximately 500 years; therefore, the probability of a seismic event of this magnitude is only 7% to 10% by the year 2050.

This site is located in the Seismic Zone 3 area of the New Madrid Fault Zone and consideration must be given to designing the proposed structures to withstand the ground motion of an earthquake. The seismic analysis will require the selection of appropriate site coefficients and other seismic values that can be established from the subsurface conditions, guidelines set forth by local, state, and federal codes, and historical seismic information. The predominant stratigraphy at the site is the dense to very dense basal sand (SP) formation existing at depths below 35.0 feet. The surface soil stratum is defined by a layer of relatively stiff and stable sandy clay (CL) and the intermediate soft to medium stiff cohesive layer is not a "quick clay" as justified by the Atterburg Limits but the soils are in a saturated state due to the relatively high water table

The 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 structure's foundations should be designed using guidelines as set forth by the Arkansas State Building Services as required by **Arkansas Act 1100-1991** (and subsequent amendments), the 2000 International Building Code, or as referenced by local ordinances.

A heavy industrial site will heavily depend upon the deeper auger cast piles bearing in the dense basal sand formation which will significantly limit the vertical movement at the site and will dampen the horizontal component of p-wave. Based upon the subsurface soil conditions and the seismic values for Arkansas published by the Arkansas State Building Services, the Standard Building Code and the 2000 International Building Code the following data are considered applicable to this project site:

SEISMIC DESIGN CRITERIA

Seismic Zone	3
Site Class	D
Soil Profile Type	S_3
Site Coefficient	1.5
Peak Acceleration Coefficient (A_a) (ASBS)	0.25
Effective Peak Velocity-Related Acceleration Coefficient (A_v) (ASBS)	0.25

LIQUEFACTION ANALYSES

Liquefaction is the sudden loss of shear strength in a soil as a result of excess pore water pressure which has been induced by vibration from an earthquake. When soils experience liquefaction they loose strength and temporarily exist in a near liquid state. Liquefaction is primarily associated with saturated, loose to medium dense cohesionless soils, and "quick clays." At this site, the predominate soils are a dense to very dense sand (SP) that are basically not susceptible to liquefaction with a relatively thin strata of soft to medium dense cohesive soils. A magnitude 6.0 earthquake was used in the liquefaction analyses presented in this report because of the acceptably low probability of a larger earthquake within the functional design life of most industrial sites.

Due to the seismicity of the region and the strong motion potential at the site, a liquefaction analysis was performed using Seed's stress procedure to estimate the settlement potential of the site. The methodology is primarily based upon the blowcounts of the standard penetration tests, the site's stratigraphy, and the depth to the water table. The resulting analyses are furnished on Plates 22 through 24. The analyses indicates that the soils at the site are acceptable and that the site's safety factor against liquefaction is within the generally published acceptability limits. Based upon this limited evaluation, the soil's at this site should not liquefy for the analyzed conditions.

LABORATORY TESTING

Tests were performed on select samples to determine their classification and/or strength characteristics. Laboratory testing included water contents, unit weight, Atterberg limits, mechanical analyses, unconfined compressive tests, and direct shear tests 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 lean clay (CL) with fine grained sand and silt or moderately plastic clay (CH). The liquid limit (LL) for the cohesive material from the landside of the levee ranged from 29 to 88, with plasticity index (PI) ranging from 11 to 50. Soils in this range are moderately plastic with some shrinkage or swell potential but the more potentially expansive soil layer primarily exists at depths between 10.0 and 20.0 feet which at the time of the investigation were saturated, therefore, the expansion potential is minimal. The soils should be further evaluated during the design investigation to determine the limits; but, based upon previous experience the potential heave associated with this soil type at this site should be less than 0.10 inch, which is considered minimal.

Mechanical Grain Size Analyses

Mechanical grain size analyses were performed to develop a profile of the Minus No. 200 sieve percentage for use in the liquifaction analysis. The results indicate that the cohesive strata is basically 85.0% silt/clay and the basal sandy soils typically had a 5.0% clay-silt percentage. Typically, a higher fine percentage decreases the susceptibility of these soils to liquefaction.

Unconfined Compression Tests

Unconfined compression tests were performed on a selected samples at their natural moisture content to predict the in-situ bearing capacity of the cohesive soils. The corresponding water content and unit weight was also determined for these specimen to assess the strength sensitivity to moisture and to aid the determination of the Seismic Site Classification. The samples were determined to have an unconfined compressive strength that ranged from 0.4 to 2.2 ksf. The moisture content for the samples were 37.4% and 36.3% with a dry unit weight of 82.2 and 108.4 pcf, respectively.

Direct Shear Tests

Direct shear tests were performed on select soil samples to determine the angle of internal friction (Ø) and for design purposes to assess the general change relative to depth. The angle of internal friction is normal for the site, progressively increases with depth and ranges from 20° at the 8.0 to 10.0-foot depth to 32° at the 70.0 to 71.0-foot depth. The high angle of internal friction indicates a strong granular interlock and that the particles are more angular. This will result in higher soil strength and decreases the soil's susceptibility of liquefaction.

GENERAL EARTHWORK

The following sections are intended to provide the designer and contractor with guidelines for preliminary design for the project. They are not intended to be used as a specification for construction procedures or methods but are provided to allow a budgetary estimate to be developed.

Site Conditions

The landside portion of the site is currently used for agricultural purposes. Prior to fill placement, the site should be stripped of all topsoil and organics (6.0 to 12.0 inches). Additionally, any trees and stumps should be properly removed and discarded off-site. The site should be proof rolled with a loaded tandem axle truck to identify any potential soft areas that may need to be undercut. The near surface soils should then be scarified and recompacted to the maximum achievable degree of compaction.

Site Drainage

A comprehensive site drainage plan should be formulated and implemented in order to control the high groundwater levels anticipated during and after construction and to adequately drain the site of rapid rainfall. High water levels encountered during construction of excavations, especially deep foundations and utilities, will require additional drainage considerations. Furthermore, large quantities of rainfall will quickly saturate and pond on top of the slightly permeable site soils, thus, pumping may be required.

Temporary drainage ditches and berms may be required to divert surface and shallow subsurface water from the construction area. French drains or drainage blankets may also be required if persistent and troublesome water cannot be alleviated. The contractor should be prepared to provide sump pumps to remove ponded water from any deep excavations.

Fill Soils

The onsite near surface soils may be used for fill and they will have good strength characteristics when compacted using proper moisture control and Modified Compaction (ASTM D1557). The plastic soils encountered at the deeper depth should not be used as structural fill but they generally are excellent soils for use as liners or other water control features. Any off-site fill should consist of non-expansive, granular material such as clayey gravel (GC), clayey sand (SC), clayey fine grained sand (SM), a cohesive sandy clay (CL), or other natural or manufactured soils with prior approval of the geotechnical engineer. The cohesive soils will readily pump if the moisture content is allowed to significantly surpass the soil's optimum moisture content. Therefore, moisture control should be maintained within two percentage points of optimum.

Modified compaction should be given primary consideration as it will yield higher CBR for pavements, higher allowable bearing capacities for shallow foundations, and will achieve the desired degree of densification necessary to limit differential settlement in the event of a strong motion earthquake. All soils used for fill or backfill should be placed in loose lifts not to exceed 10.0 inches and thinner lifts may be required for hand type compaction or small rollers.

Utilities/Pit Construction

Utility and pit construction will encounter soft soils and potentially shallow groundwater dependent upon the Mississippi River's water level at the time of construction. The clays generally have a low permeability, however, they are not uniform but rather are inter-layered with more permeable silts, sands, and clays. The horizontal permeability of these soils are 6.0 to 10.0 orders of magnitude greater than the vertical permeability. Therefore, the groundwater table should be considered to be consistent with the level of the Mississippi River. Excavations below this elevation will require temporary dewatering by pumping or draining to sump pits during construction. All utility construction and trenching should be conducted in accordance with the applicable local codes and OSHA regulations. Trench boxes and/or shoring will be required for below grade excavations greater than 4.0 feet. The soft, wet, cohesive soils and the loose fine grained sands will tend to slough and cave into excavations and the excavation bottom will likely require the use of a subgrade support material such as "B" stone or flowable fill prior to placement of bedding materials to resist ground heave effects.

Adverse Weather Conditions

Site grading will be more difficult in winter and wet weather. The on-site cohesive soils near the surface of the site will absorb significant quantities of water and can wick water up from the groundwater table below. These soils will require significant aeration and working to dry during wet weather. The contractor may elect to dry the soils using lime or fly ash tilled in the wet soil with a motorized tiller in areas where undercut and removal is not required.

FOUNDATIONS

Option 1 - Conventional Footings

Conventional footings or a mat/raft foundation is recommended as a foundation system for lightly loaded structures. The foundations should be made rigid in an effort to minimize potential differential movements resulting from non-uniform settlement due to consolidation of variable thickness of native and/or fill soils and localized earthquake induced densification/spreading. Conventional shallow footings founded at 2.0 feet below finished grade should have a minimum bearing capacity of 1750 psf and larger allowable bearing capacities may be achieved with some modification of the near surface soils. Bearing capacities at other depths into the natural soil may be determined from Plates 25 and 26 which show calculations and curves for the bearing capacity with depth for the site. Plate 27 provides an explanation of calculations for conventional spread footings. The structural loading conditions of the foundations are not known at the time of this limited investigation but based upon past experience with similar industrial sites the total and diffential settlement of less than 1.00 and less than 0.50 inch, respectively should be expected at the referenced bearing capacity.

Option 2 - Pile Foundations

Pile foundations should be considered as the primary foundation type for heavily loaded structures. The bearing capacity of the skin friction piles in the Mississippi River soils have been determined utilizing the Ø-C characteristics of the soils. This data is obtained from the results of an empirical relationship between the blowcounts ("N") of the Standard Penetration test, the results of a pile load test in the vicinity, and the aid of our in-house computer design techniques which utilizes these soil data.

Auger cast pile calculations were made and the resulting allowable bearing capacity curves for compression and tension loads versus length are given on Plates 28 through 30. An explanation of the calculations is provided on Plate 31. Sixteen-inch diameter auger cast piles set at a depth of approximately 35.0 feet should develop 100 kips (50 tons) allowable compressive capacity on either the landside or the riverside of the levee. The settlement of these structures will be a combination of both plastic deformation of the soil, and elastic shortening of the foundation member itself. Although some settlement of the pile foundations are expected, the total and differential settlements should be well within the tolerances of required structures and should be compatible with the conventional footings when designed as previously described.

Pile Load Testing

It is considered prudent that the pile design criteria be adequately verified by field load testing in accordance with ASTM D 1143 (Axial) and ASTM D 3966 (Lateral) or other applicable building codes as modified by the Geotechnical Engineer to match specific site conditions. This is a necessary function of the pile foundation design, and any design recommendations are based upon a load testing program being evaluated by geotechnical engineers representing this firm prior to giving authorization to order the final pile size and lengths.

It is recommended that a pile load test program utilizing test piles of two separate lengths be performed to determine the exact installed length. The pile load testing program should consist of installing one set of two test piles for each of the pile types selected. The two test piles capacity versus length relationship can be developed for each pile type. A lateral load test is also recommended for each pile type on the shorter of the two piles after axial testing is completed. A pile testing program implemented in the early construction stage would be the most economical means of performing this item of work.

CONSTRUCTION QUALITY CONTROL

In order to achieve quality workmanship, to help ensure that the specified end results are achieved, and to make certain that the continued satisfactory performance of the project is assured, full time quality control and monitoring of the work performed should be provided under the following guidelines:

1. A full time quality control technician, serving in a surveillance and documentation capacity as inspector for the designers, owners, and builders, will provide the best assurance for achieving the specified compliance.
2. With a full time quality control technician, a specified number of various field tests would not be necessary since the technician would be at liberty to perform the tests on all items of work daily and/or routinely as may be required.

3. The quality control testing agency should be given responsibility in the testing and evaluation of the work, under the guidance of the Owner's representative, but not to the extent of negating the contractual documents or the obligation of additional construction funds.
4. The ASTM standard testing procedures should be used to the fullest extent possible in the quality control program, supplemented by various other state or local specifications on some items of the work when applicable.

RECOMMENDATIONS

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

1. The site can be made suitable for construction of a heavy industrial facility without unique or unusual design and/or construction techniques; however, additional geotechnical investigation should be required to aquire site specific information for design purposes.
2. For lightly loaded structures a shallow foundation may be used provided it bear 2.0 feet below finished grade with an allowable bearing capacity of 1750 psf. Higher allowable bearing capacities can be achieved with some modification of the near surface soils on both sides of the levee.
3. For heavily loaded structures, auger cast piles bearing in the dense sands below 35.0 feet depth will adequately support the structures when designed in accordance with the necessary structural and/or architectural requirements determined by the designer. Specific pile capacities versus length should be determined for individual structures with additional geotechnical investigations.
4. The proposed site is in a seismic zone 3 area and a detailed liquifaction study should be performed with the design phase geotechnical investigation for the site. The limited analyses performed as a part of this study determined that the soils should not liquefy at a Magnitude 6 earthquake and the resulting settlement should be within an acceptable range for the proposed structure.

5. Engineered fill used to backfill undercuts, raise grades, and support lightly loaded structures should be compacted to a Modified Proctor density with the moisture within two percentage points of the optimum water content.
6. Concrete should not be placed until after excavations have been observed and tested by cone penetrometer to insure that the bearing capacity is as required. Remediation will be required for those footings having a bearing capacity below the minimum.
7. Surface drainage and roof downspouts should empty into drainage collectors and flow into a below grade storm system in lieu of emptying onto the site surface in order to prevent long term saturation of the subgrade adjacent to foundations.
8. A groundwater table at the top of ground should be considered for the design and construction of all subsurface utilities and structures. Underground tanks or vaults may require anchoring to counteract buoyancy forces.
9. The Contractor should be required to submit test results on samples of the proposed fill and base materials for approval prior to placement, so as to insure compatibility with design assumptions.
10. Specifications normally used for the construction of similar projects should be adequate with the exception of a few items unique to this project, as discussed earlier.
11. Except for the potential of a high water table, the investigation did not reveal any unexpected conditions that would have a significant detrimental effect upon the proposed projects; however, this is not to say that others do not exist, for a complete determination in this regard is beyond the scope of this investigation as authorized.
12. Full time quality control testing procedures are recommended in order to assure that construction is in accordance with the project plans and specifications, and to assist the Engineer and/or Owner's representative during the progress of the work.
13. Other conclusions and recommendations are discussed in the text of this report.

LIMITATIONS

The borings as performed with this investigation are widely spaced and contain information related to the types of soil encountered at specific locations and times and show lines delineating the interface between these materials, as well as results of tests performed in the laboratory on representative samples. The logs contain our field geologist's interpretation of conditions that are believed to exist in those depth intervals between the actual samples taken. Therefore, these boring logs contain both factual and interpretative information. It is not warranted that these logs are representative of subsurface conditions at other locations and times.

The analyses, conclusions, and recommendations contained in this report are based onsite conditions as they existed at the time of our field investigation and on the assumption that the exploratory borings are representative of the subsurface conditions throughout the 2000-acre site. The site investigation and analyses associated with this study are limited in nature and a much more detailed investigation is recommended prior to the design phase. Additionally, during construction if different subsurface conditions from those encountered in our borings are observed, or appear to be present beneath excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary. If conditions have changed due either to natural causes or to construction operations at or adjacent to the site, we urge that we be promptly informed, and retained to review our report to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse.

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