PRELIMINARY GEOTECHNICAL EXPLORATION

NKU FOUNDATION PROPERTY

DIXIE HIGHWAY AND MOUNT ALLEN DRIVE

COVINGTON, KENTUCKY

CIVIL ENGINEERS


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NKU Foundation
Northern Kentucky University
Highland Heights, Kentucky 41099-8005

Attention: Dr. James L. Alford

Re: Preliminary Geotechnical Exploration
    NKU Foundation Property
    Dixie Highway and Mount Allen Drive
    Covington, Kentucky

Gentlemen & Ladies:

Presented herein is our report of the preliminary geotechnical exploration made at the NKU Foundation Property located on the south side of the Dixie Highway, just east of its intersection with Mount Allen Drive, Covington, Kentucky. This work was accomplished in accordance with our proposal-agreement dated November 15, 1994 and accepted by Dr. James L. Alford on November 30, 1994, Agent for the NKU Foundation.

SCOPE
The purpose of this preliminary exploration was to determine the general constituents of the existing waste fill embankment that was constructed on the NKU Foundation property and to offer our opinions as to the geotechnical feasibility of developing this tract.

PROJECT CHARACTERISTICS
We understand that the tract under consideration for future development consists of that parcel described as a 16.9417 acre tract of land immediately east of the Northern Kentucky University
Covington Campus, located off of Mount Allen Drive in Covington, Kenton County, Kentucky. The property is bordered to the north by the Dixie Highway, to the southwest by Mount Allen Drive, and to the south by parcels owned by C.A. Bramlage and the Sanitation District No. 1 of Campbell and Kenton Counties. The eastern boundary of the tract is the right-of-way of Interstate 75.

Prior to the recent realignment of Interstate 75 from Kyles Lane to Twelfth Street, the subject 16.9417 acre tract of land included a V-shaped valley that drained surface water eastwardly from near the intersection of the Dixie Highway and Mount Allen Drive. The original topography was approximately as illustrated by the topographic contours shown on the Waste Area Study Plan by David E. Estes Engineering, Inc. (Estes), drawing dated June 18, 1990. With the realignment of Interstate 75, we understand that it was proposed to waste approximately 315,140 cubic yards of fill in the western portion of the 16.9417 acre tract within the confines of the V-shaped valley. The Estes Waste Area Study Plan illustrated a proposed toe of slope at the west edge of a State of Kentucky impounding easement, the slope extending upwardly to the west at 3 horizontal to 1 vertical (3:1), 110 feet in height, with one intermediate bench. The proposed waste fill embankment is graphically illustrated by the proposed contours shown on the Estes Waste Area Study Plan. Based upon field density tests performed during the embankment construction by Cartec Technical Consultants, Inc. (Cartec), we understand that the embankment was built during the period of July through November, 1990. As a result of this construction, approximately 4.5 acres of gently sloping terrain fronting approximately 1000 feet along the Dixie Highway exists. We understand that consideration is being given to developing this 4.5 acres. It is the feasibility of developing this acreage by supporting structures upon the fill embankment that is the focus of this report.
FIELD EXPLORATION
This preliminary geotechnical exploration of the waste fill embankment was accomplished with the drilling of 4 test borings, numbered 101 through 104. The test borings were staked in the field by our survey crew relative to the intersection of the Dixie Highway and Mount Allen Drive as illustrated on the Estes Waste Area Study Plan. The test boring locations are illustrated on the Boring Plan, Drawing 94821E-1 included in the Appendix to this report. The Boring Plan was prepared from a copy of the Estes Waste Area Study Plan. Benchmark information was not available on this plan. A copy of the 1963 edition of the Northern Kentucky Area Planning Commission (NKAPC) Map was obtained for the area and 4 locations selected where the topographic contours cross the centerline of the Dixie Highway. The elevation 680 contour was used as a benchmark and the other three selected contour locations field checked for correlation. Relatively close agreement was obtained. Therefore the El. 680 contour intersection with the centerline of the Dixie Highway was utilized as our survey control. The ground surface elevations at the 4 test boring locations were determined from this benchmark. The appropriate elevation is indicated at the bottom of each of the test boring logs.

Access to the project site was restricted due to excessive erosion of the ditch line at the main entrance location. A secondary access location was also blocked with a large diameter pipe. Auxiliary equipment was provided by G. J. Thelen & Associates, Inc. (GJTA) to remove the pipe away from the secondary entrance drive in order to facilitate access to the site with our truck-mounted drill rig.

The 4 test borings were completed with our truck-mounted drill rig by advancing either continuous flight solid stem augers or hollow stem augers. Sampling was accomplished ahead of the augers with either 2-inch or 3-inch O.D. split spoon samplers, an Elasky continuous sampler or 3-inch O. D. Shelby tubes. Representative
portions of the 2-inch split spoon samples were sealed in glass jars. Portions of the 3-inch split spoon samples were sealed in plastic quart jars. The Elasky samples were sealed in Elasky sampler plastic liners. The Shelby tube samples were sealed in the thin walled Shelby tubes. Sealing of the samples preserves the soils' moisture and density. All recovered samples were labeled with the boring number, sample number and sample depth for identification purposes.

As the test borings proceeded, the Drilling Technician prepared field logs of the subsurface profile noting the soil and bedrock descriptions, stratifications, groundwater, standard penetration resistance and other appropriate data.

LABORATORY REVIEW

After the test borings were completed, the recovered samples were transported to our Soil Mechanics Laboratory where they were reviewed by our Project Geotechnical Engineer. As the Engineer examined and visually described each of the samples, representative samples were selected for general soil classification and strength testing including natural moisture content, natural density, specific gravity, unconfined compression tests and triaxial compression tests. A summary of all of the laboratory tests except for the triaxial compression tests is included in the Appendix to this report. The triaxial compression test results are summarized on the stress path graphs on the Triaxial Test Forms included in the Appendix. Also included in the Appendix are the Moisture-Density Test and the Unconfined Compression Test forms.

Based upon the visual examination of the samples, the results of the laboratory tests and the Drilling Technician's field logs, our Project Geotechnical Engineer prepared the finalized test boring logs. Copies of these finalized logs are included in the Appendix to this report along with a Soil Classification Sheet which
summarizes the terms and symbols used in the preparation of the logs.

The lines identifying the changes between soil and bedrock types on the test boring logs were determined by interpolation between samples and should be considered approximate. Only a change which occurs within a sample could be precisely determined. The transitions in soil and bedrock types may be abrupt or gradual.

GENERAL SITE CONDITIONS
The project site consists of the northwestern portion of a 16.9417 acre tract of land bordered to the north by the Dixie Highway and to the southwest by Mount Allen Drive in Covington, Kenton County, Kentucky. Topographic drawings obtained from aerial photographs prepared in 1963 indicate that the project site, prior to having been filled, consisted of a deeply entrenched drainage valley that flowed from west to east. The topographic contours of the prefilling conditions are depicted by the existing contours shown on the Estes Waste Area Study Plan. Those contours indicate that the terrain below the recent fill embankment varied from approximately El. 660 to the west to El. 535 to the east.

The drainage valley in question represents the lower portion of a dendritic drainage channel in the transition between the Covington drainage basin area of the Ohio River Valley and the highlands of the ridgetops to the west of the Covington drainage basin area. This drainage channel originates in the proximity of Interstate 75, crosses westwardly across the project site, and then follows the alignment of the Dixie Highway southwestwardly and eventually across the Covington Catholic High School property, St. James Avenue terminating in the proximity of Sleepy Hollow Road. The highlands generally consist of relatively shallow depths of overburden clayey soils over the bedrock formation, shale and thinly bedded limestone. This is in contrast to the Covington basin area which consist of deep deposits of alluvium associated
with the ancient Teays-aged drainage pattern. The project site, being located within the confines of the lower portion of a major drainage channel, probably included a subsurface profile of low-density near-surface soils over stiff colluvial and residual clays and then bedrock, shale and thinly bedded limestone. Several of the test borings encountered 10 feet or less of clayey soils between the waste shale fill and the underlying bedrock formation.

The bedrock of the Northern Kentucky Area consists of Ordovician-aged interbedded shale and limestone. Shale and limestone is a sedimentary bedrock. The project site is located near the crest of a ridge of bedrock formation through the mid-western portion of the United States known as the Cincinnati Arch. Being near the crest of this ridge, the bedrock, for all practical purposes, is horizontally bedded. A Geologic Quadrangle map of the area suggests the bedrock may have a local dip downward to the north by northeast of approximately 1/2 percent.

The Ordovician-Age bedrock has been subdivided into various formations and the formation have been roughly correlated to USGS elevations. Below approximately El. 690, the bedrock formation consists of the Kope or Eden Formation. The upper two members of the Kope formation, being the McMicken and Southgate Members, would be expected to directly underlie the project site. A copy of the local Geologic Column is included in the Appendix to this report illustrating the various local formations. The Kope or Eden Formation is known for its relatively soft, easily deformed shale and a relatively low percentage of limestone beds.

The shale and limestone fill of the waste fill embankment is understood to have been accumulated from the major roadway cut required for realignment of Interstate 75 between Twelfth Street in Covington and Kyles Lane in Ft. Wright. The excavations would have accumulated bedrock from primarily the Kope Formation and the overlying Fairview Formation, and possibly the Bellevue Tongue of
the Grant Lake Limestone Formation located just above the Fairview Formation. The basic constituents of the fill would be primarily the gray shale with limestone floaters, the floaters being remnants of the interbedded limestone, with some inclusions of brown weathered shales and overburden clayey soils that naturally exist above the unweathered bedrock.

The findings of the 4 test borings confirm the presence of the deep waste shale and limestone floater fill with some remnants of the overburden clayey soils at the original ground surface and then the underlying bedrock, the Kope Formation. Test borings 101A, 103 and 104 were extended entirely through the existing fill and terminated in the parent unweathered bedrock, the gray shale and thinly bedded limestone. Test boring 102 was terminated when the quantity of the drilling footage estimated for our proposal was exceeded. It appears that there was more fill on the project site than initially anticipated for two reasons. First, the top of the fill embankment is believed to be 5 to 12 feet higher than the Estes Waste Area Study Plan as judged by correlation of the elevations at the test boring locations with that suggested by the proposed contours of the Waste Area Study Plan. Secondly, a significantly greater depth of fill was encountered in test borings 103 and 104 than anticipated by comparison of the existing and proposed contours on the Estes Waste Area Study Plan. There may have been some fill in place before the waste fill embankment was constructed, or the topographic plan did not accurately define the original terrain.

Test borings 101A, 103 and 104 were terminated in the parent bedrock, the gray shale and thinly bedded limestone. In test boring 103, there was a zone of olive brown soft weathered bedrock, shale and thinly bedded limestone, above the parent material. Our experience in the Northern Kentucky Area suggests that commonly there exists three zones of bedrock, the zones being distinctive by coloring and the degree of weathering of the shale. More specifically, the parent bedrock from which the two weathered zones
are derived consists of gray shale and thinly bedded limestone. The upper portion of the gray shale is generally soft and increases somewhat in density and strength with depth to a moderately tough to tough consistency. The zone of bedrock immediately above the unweathered bedrock, the gray shale and thinly bedded limestone, generally consists of a soft weathered bedrock, olive brown shale and thinly bedded limestone. The change in color is indicative of the weathering process and the shale is generally slightly less dense and weaker. The uppermost zone is a very soft highly weathered bedrock, brown and gray shale and thinly bedded limestone. With this stratum the shale has almost weathered to a clay-like consistency, although the bedding planes are still very distinct. Either or both of the weathered zones may be absent at a particular location as a result of variations in weathering, erosion and man-made excavation.

The limestone layers of the bedrock formation are hard in comparison to the shale. The limestone tends to be fossiliferous, locally crystalline, randomly jointed and thinly bedded. Experience has found that the limestone layers of the Northern Kentucky Area generally range in thickness from less than 1 inch to 8 inches. Thicker layers or concentrations of layers are occasionally encountered. The percentage of limestone in the Kope Formation is commonly in the 10 to 20 percent range whereas the Fairview Formation can include limestone on the order of 40 to 55 percent of the formation. The Bellevue Formation can include 55 to 70 percent limestone with exceptional thick beds.

Test boring 101A encountered a 10-foot thick stratum described as possible fill on top of the gray shale and thinly bedded limestone. This soil was a relatively stiff silty clay with inclusions of shale fragments and limestone floaters, similar to colluvium. Colluvium is a talus deposit that accumulates along the lower portions of steep bedrock slopes as a result of prior sliding. Test boring 103 encountered no native overburden above the top of
bedrock. Test boring 104 encountered a thin stratum of very stiff silty clay with shale fragments and limestone floaters directly above the unweathered bedrock, the gray shale and thinly bedded limestone. It is possible that borings 101A and 104 were near the centerline of the original drainage valley such that weathered zones of bedrock would likely to have been eroded away.

The test borings confirm that the waste area embankment consists primarily fill, shale and limestone floaters. The majority of the fill recovered from the 4 test borings consists of gray shale and limestone floaters with a slightly lesser degree of olive brown shale and limestone floaters and a minor percentage of brown and gray shale and clayey soils. A battery of tests were performed on the recovered samples of the waste fill. Bag samples were collected upon which standard Proctor moisture-density tests were performed per ASTM D698. A sample of gray shale with limestone fragments resulted in a maximum dry density of 123.6 pounds per cubic foot and an optimum moisture content of 11.2 percent. An olive brown shale and limestone sample resulted in a maximum dry density of 114.3 pounds per cubic foot and an optimum moisture content of 16.2 percent. A sample of primarily silty clay yielded a maximum dry density of 111.1 pounds per cubic foot and an optimum moisture content of 15.2 percent. Another sample of primarily intermixed gray shale and brown shale with limestone floaters reflected a maximum dry density of 126.5 pounds per cubic foot and an optimum moisture content of 10.5 percent. It is anticipated that there can be some variation in the maximum dry densities depending upon the formation from which the shale was recovered. At this point in time, there was no way to distinguish between shale recovered from the Kope Formation versus shale recovered from the Fairview Formation.

The void ratio range for the 4 proctor samples ranged from approximately 0.35 to 0.52 at their maximum dry density. The degree of saturation ranged from approximately 79 to 89 percent.
Representative portions of samples recovered via the 3-inch O. D. split spoon sampler, the 3-inch O. D. Shelby tubes and the continuous Elasky sampler were tested for in situ dry density. The results of these tests suggested that the gray shale samples reflected natural dry densities on the order of 115 to 137 pounds per cubic foot, yielding percent compactions on the order of 94 to 108 percent. Natural moisture contents ranged from approximately 8 to 18 percent, suggesting degrees of saturation on the order of 82 to 98 percent. This specific gravity of the gray shale samples were on the order of 2.73 to 2.77. The void ratio of these samples ranged from approximately 0.25 to 0.49.

The brown shale samples reflected natural dry densities on the order of 110 to 116 pounds per cubic foot, yielding percent compactions typically over 100 percent. Natural moisture contents ranged from approximately 14 to 20 percent, yielding percent saturations on the order of 95 to 100 percent and void ratios approximately 0.42 to 0.56. The specific gravities of these samples were on the order of 2.73 to 2.74.

The samples of silty clay which were recovered and tested yielded natural dry densities on the order of 101 of 114 pounds per cubic foot, suggesting percent compactions on the order of 98 to 103 percent. Natural moisture contents ranged from approximately 15 to 25 percent with degrees of saturation on the order of 82 to 100 percent. The void ratios ranged from approximately 0.46 to 0.57 with specific gravities on the order of 2.70 to 2.78.

The degree of compaction over 100 percent probably indicates that the samples tested for density did not well correlate with the laboratory standard Proctor moisture-density tests.

Visually the samples of fill recovered varied from stiff to hard with only a few exceptions. Near the existing ground surface there was some minor softening due to weathering. In test boring 104,
the sample recovered at a depth of 40 feet was primarily a clay with limestone floaters and described as medium stiff in consistency. Five (5) intact samples recovered were tested for unconfined compressive strength. Compressive strengths ranged from 6010 to 10150 pounds per square foot with natural dry densities ranging from 111.7 to 118.8 pounds per cubic foot and natural moisture contents of 14.2 to 18.2 percent. Comparison of the natural dry densities to the Proctor test results suggest percent compactions on the order of 95 to 102 percent.

Four (4) samples recovered were tested by the triaxial compression testing method. These samples were subjected to confining pressures. Pore pressure measurements obtained during testing yielded a stress path graph from which strength parameters of the soils were interpreted. Upon an assumption of zero cohesion, an angle of internal friction of each of the 4 tests yielded friction angles varying from 23.8 to 38.7 degrees. With an assumption of a minor amount of cohesion, on the order of 100 pounds per square foot, the modified friction angles range from approximately 23.1 to 36.6 degrees. The samples tested had natural dry densities on the order of 112.2 to 123.3 pounds per cubic foot, percents compaction ranging from approximately 98 to 101 percent. Natural moisture contents of the samples varied from approximately 10 to 18 percent.

It should be noted that during the drilling process, significant difficulty was encountered at depths on the order of 17 to 22 feet below the existing ground surface in each of the test borings. Test boring 101 was terminated at a depth of 17 feet due to auger refusal on hard fractions included within the fill, presumably limestone or concrete. In test boring 104, the augers became bound in the profile at a depth of 18 feet and a backhoe was required to excavate around the augers in order to recover them.
Groundwater was not observed during the drilling of either of the 4 test borings. Notations to that effect are indicated at the bottoms of the test boring logs.

DATA EVALUATION

The NKU Foundation provided to us a booklet of field density reports for waste area No. 1 prepared by Cartec during the period of July through November, 1990. A total of 732 field density tests, numbered 43 through 822 with some numbers omitted, were included in this booklet. It is understood that these field density test reports reflect in-place density tests during the construction of the waste fill embankment. The project specification was listed at 95 percent compaction, assumed to be per the standard Proctor moisture-density test of ASTM D698. The test results generally indicate that the project specifications were obtained.

We made a review of this field-density test data and determined that the average reported percent compaction was 98.4 percent and the average moisture content was 12.6 percent. This average moisture content compares to the GJTCA optimum moisture contents from the 4 standard Proctor tests at 13.1 percent. A review of the Cartec compaction test results indicated that 359, or approximately 1/2 of the total number of tests, had field moisture contents of 10 percent or less, that is, 3 percent below the average optimum moisture content determined by the GJTCA moisture-density tests. This is in contrast to the average moisture content of 14.9 percent of all natural density tests performed by GJTCA. This suggests that the field density tests may have been skewed towards the dry side of the range due to the influence of limestone during the field density testing procedure. Only 8 percent of the GJTCA natural density tests resulted in moisture contents of 10 percent or lower.

We also reviewed the locations of the field density tests during construction. The density test reports correlate the test numbers
to stations, offsets and elevations. We were not provided with the horizontal or vertical controls used during construction. A plot of the field density tests by station and offset provided us with a plan that fairly well superimposed upon the fill area as defined by the Waste Area Study plan developed by Estes. A copy of that plan is included in the Appendix to this report, Drawing 94821E-2. We also plotted all of the field density tests by station versus elevation, giving an east-west cross section through the site. A copy of that drawing, Drawing 94821E-3, is also included in the Appendix to this report. Review of the data suggests that there may be a bust in either plan or elevation because many of the tests plot either above the proposed ground surface or below the original ground surface as defined by the Estes Waste Area Study plan. Additionally, the plotting suggests that there were no field density tests performed near the toe of the original fill embankment nor were there field density tests performed in the central mid-elevation range of the embankment. At the east end of the site, no field density tests were performed above El. 600 and to the west end of the site, no field density tests were performed above El. 630. There were also no tests below El. 540.

We have attempted to look at the number of field density tests performed within a 10-foot range in elevation across the project site and compared that to its corresponding yardage that could be anticipated from the Estes Waste Area Study plan. In the El. 550 to El. 560 range, it is estimated that there was one test for every 29 cubic yards of fill placed. Between El. 580 and El. 590, the number of density tests was reduced to one per 126 cubic yards. At El. 600 to El. 610, the number of field density tests reduced to one per 2060 cubic yards. In general, there were a fewer number of tests per cubic yard of fill placed with increasing elevation.

Of particular interest to any fill embankment is the stability of the outslope. The Estes Waste Area Study plan illustrated an outslope at a gradient of 3 horizontal to 1 vertical (3:1) over a
height of 110 feet with one intermediate 18-foot-wide bench. Using the strength information obtained from our laboratory testing program, we performed a slope stability analyses of the fill slope at the eastern end of the embankment with the top of embankment revised to a level 12 feet above that shown on the Estes Waste Area Study Plan to better correlate with our survey information and test boring results. A groundwater table at approximately 15 feet above the original ground surface, depicted on the Estes Waste Area Study Plan was assumed in the analyses.

For an angle of internal friction weighted towards the lower end of the range of the 4 triaxial compression test results, a factor of safety of 1.68 was computed for a friction angle of 27 degrees. With an adjustment to include a minor amount of cohesion of 100 pounds per square foot and a friction angle of 26 degrees, a factor of safety of 1.69 was obtained. These methods of analyses are considered effective stress analyses.

Considering a total stress analysis using a cohesion value equal to half of the weighted unconfined compression test strength results, a factor of safety of 3.04 for a cohesion of 3700 pounds per square foot was obtained. Reducing the cohesive strength to 2000 pounds per square foot resulted in a factor of safety of 1.64.

Only deep-seated modes of failure were considered in these analyses.

A significant problem with deep fill embankments is the potential for consolidation of the fill soils and corresponding settlement of the ground surface. Generally compacted fill embankments build up stress in the ground equal to the weight of the overlying soil. For instance, a unit weight of soil of 135 pounds per cubic foot produces vertical stress in the ground which increases linearly with increasing depth below the ground surface. At depths of 20 feet below the ground surface, the vertical stress is equal to 2700
pounds per square foot (psf). At 40 feet, the vertical stress is 5400 psf. At 80 feet, the stress is 10,800 psf and at the top of bedrock at 94 feet, the pressure is 12,690 psf. Compacted shales and clays with limestone floaters are typically rated at allowable bearing capacities of 3000 to 4000 psf with limited settlement for soils typical of the Northern Kentucky Area. At higher bearing pressures, expected settlements increase. Therefore, with pressures generated in the ground deeper than 25 feet below the existing ground surface, consolidation of the fill embankment under classical consolidation theory should be expected.

Available to us was some historical information concerning the potential consolidation of gray shale fill. We attempted to utilize the void ratio information computed from our natural density tests and the data from the historical consolidation tests on gray shale samples to estimate the maximum degree of settlement that could be expected from the embankment. Over 50 inches of consolidation was estimated. It is common for settlements to be over-estimated as a result of consolidation tests.

Our firm also has direct experience with monitoring the ground surface settlement of another compacted fill embankment in the Northern Kentucky Area constructed primarily of waste shale and limestone floaters. The embankment was constructed to a depth of 48 feet, in 6 to 8 inch lifts, moisture-conditioned and compacted to at least 98 percent of maximum density as determined by the standard Proctor moisture-density test, ASTM D698. Over a two year period, 3.2 inches of surface settlement was observed by monitoring settlement points via conventional surveying techniques. It is estimated that a total of 4 inches of settlement will ultimately occur over an elapsed time period of approximately 5.5 years. Using classical consolidation theory, the amount of surface settlement is directly proportional to the thickness of the consolidating stratum, its original degree of compaction, and the common logarithm of the ratio of the induced stress to its stress.
of compaction. Attempting to make a comparison of a 94 foot depth of soil above the bedrock at the NKU Foundation property, in comparison to the 48 feet of fill of the embankment on the historical project suggests total settlements at the project site could exceed 25 inches. Where the embankment is along the consolidation curve at this point in time is also difficult to predict. The time rate of consolidation is proportional to the square of the depth of fill. Using the historical data as a basis, it is estimated that the eastern portion of the project site could settle over a 16-year period using the classical consolidation theory.

CONCLUSIONS AND RECOMMENDATIONS
Based upon our engineering reconnaissance of the site, the results of the 4 test borings, a visual examination of the samples, the laboratory tests, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the following conclusions and offer the following opinions and recommendations.

The conclusions, opinions and recommendations of this report have been derived by relating the general principles of the discipline of Civil Engineering (Soil Mechanics) to the existing site conditions represented by the 4 preliminary test borings. Because changes in surface, subsurface and climatic conditions as well as economic fluctuations can occur with time, we recommend for our mutual interest that the use of this report be restricted to this specific project.

If conditions are encountered in the field during future explorations which vary from the facts of this report, we recommend that that information be evaluated and appropriate adjustments made in light of the new information.

- 16 -
The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater or air, on or below or around this site. Prior to development of this site, we recommend that an environmental assessment be conducted to address any environmental concerns.

We have performed the test borings and laboratory testing for our evaluation of the site conditions and for the formulation of the conclusions, opinions and recommendations of this report. We assume no responsibility for the interpretation or extrapolation of the data by others.

The project site has been presently brought to a grade estimated to vary from approximately El. 659 to the east to El. 669 to the west, that level being 10 to 25 feet higher than the Dixie Highway. Of the 16.9417 acre tract of land, there remains approximately 4.5 acres of gently sloping terrain available for potential development. The resulting gently sloping plateau has a new slope downward to the north towards the Dixie Highway, a new slope downward to the east towards Interstate 75, and an old slope upward to the southwest towards Mount Allen Drive. Apparently there were some old sewers that crossed the property. We are not aware of whether these sewers remain in place or have been rerouted as part of the waste fill embankment construction.

The findings of the test borings confirm that the subsurface profile at the 4.5 acre gently sloping tract consists of deep varying depths of compacted shales and clayey soils with limestone floaters. Relative to the feasibility of developing this gently sloping tract, it is our opinion that there are 3 items of significance from a geotechnical prospective. These items are: 1) stability of the fill outslopes to the north and east; 2) the bearing capacity of foundations for structures; and 3) the potential for settlement of the embankment transmitted to the
surface. It is our opinion that the feasibility of development must carefully consider each of the above parameters as it pertains to future land usage. The following is a discussion of each of these elements.

**Slope Stability**

The findings of the 4 test borings suggest that the existing fill, in general, appears moist and well compacted. Preliminarily, given the existing conditions, it is expected that the 3:1 outslopes along the northern and eastern borders on the site are stable. In the field of soil mechanics, a factor of safety of 1.5 or greater is typically the target value relative to long-term slope stability issues. The factor of safety represents the ratio of the resistance along a given potential slide plane to the driving forces tending to slide the material above the slide plane. A factor of safety of 3 based upon a short-term total stress analysis was obtained using strength values obtained from unconfined compression tests where the average strength was weighted towards the lower end of the range. A factor of safety of 1.68 was obtained for long-term effective stress analyses using the results of the triaxial compression tests where there average strength was weighted towards the lower end of the range of the angle of internal friction determined from the test results. In each case, stability analyses were performed using the "Rotational Equilibrium Analysis of Multi-layered Embankments" (REAME) computer model program developed by Dr. Yang H. Huang, P.E. at the University of Kentucky. The program searches for the lowest factor of safety for a given set of geometry, strength parameters and groundwater conditions. Provided that there are no significant changes in the topography (i.e., cut or erosion at the toe of slope or surcharging at the crest of slope), or significant changes in groundwater conditions, preliminarily one would expect satisfactory performance of the existing fill slopes.
Groundwater is a significant element relative to local slope stability. It will be important that good surface drainage be maintained across the top of the embankment so that surface water infiltration is minimal. We do not know whether there is an underdrain system beneath the existing embankment. If there is, one would not expect significant changes in the deep groundwater conditions as a properly designed and constructed drainage blanket would remove groundwater out from the beneath the lower portion of the embankment. In the long term there may be some migration of moisture from the side slopes out into the central portion of the embankment. Seepage out of the Kope Formation of the bedrock is considered to be relatively minor because of the low percentage of interbedded limestone. The compact condition of the existing fill embankment reduces the permeability rate relative to groundwater flow.

**Bearing Capacity**

Because the embankment appears to be constructed of shales, clays and limestone floaters that are relatively tight and well-compacted, the soil strengths are considered generally satisfactory for bearing capacity. Bearing pressures are on the order of 3000 psf to 4000 psf with an adequate factor of safety against a bearing capacity failure. The main limitation, however, is overall embankment consolidation as discussed in the following section of this report.

If overall embankment consolidation (and corresponding surface settlement) were not an issue, the findings of this preliminary exploration suggests that nominal size shallow foundations could readily be supported in the compacted fill at 3000 to 4000 psf. Pressures induced upon the soil within the zone of influence of foundations (i.e., depths below footing level equal to 3 to 4 footing widths) could readily be absorbed by soils whose unconfined compressive strengths are in the very stiff to hard range. However, we have some minor concern about uniformity of the fill embankment
in plan and with depth since field density tests may not have been uniformly dispersed through the embankment during construction. In particular, it appears that local regions of the fill were not tested, in particular the upper 20 feet of the embankment. We also experienced some very coarse deposits in the fill during drilling at depths of 17 to 22 feet below the existing ground surface which could be some form of waste granular material, limestone floater deposit or rubble concrete zone. Beyond that particular stratum, 17 to 22 feet, our limited amount of testing did not note any significant differences in the quality of compacted fill either with depth or in plan.

**Settlement**

It is our opinion that the main limitation with development of this site is the impact that continuing consolidation of the fill may have upon structures and infrastructures. Although the fill appears dense and well-compacted, classical consolidation theory suggests that embankments of depths representative across the project site are substantial and will undergo settlement. As discussed in the Data Evaluation section of this report, the magnitude of settlement is directly proportional to the depth of existing fill. By comparison of the existing topography with the original topography, the greatest depth of fill will be at the eastern end of the 4.5 acre tract near the centerline of the original drainage valley. The depth of fill decreases as you move westwardly across the site up through the throat of the original valley. The depth of fill decreases to the north and south out of the center of the old swale. Obviously, the magnitude of the expected settlement will be less towards the western end of the site as well as towards the northern and southern limits of the embankment where the depths of fill are shallowest.

Classical consolidation theory suggests that tens of inches of settlement could occur over a 1 to 2 decade period after construction of the embankment. Since the present elapsed time is
only on the order of 4.0 to 4.5 years since the embankment was constructed, it is our opinion that some significant additional settlement should be expected over the next decade. Because of the variability of material types both in plan and elevation, the variations in density of each material type, the potential differences in consolidation characteristics, the differences in overall depth of fill in plan across the 4.5 acre tract, and the in situ pressure differentials which change with depth, it is extremely difficult to refine the magnitude of settlement that the embankment could eventually undergo. It is also difficult to define at what point the present conditions along the time-settlement curve. We recommend that an immediate long-term monitoring program be established to collect sufficient data to make an educated decision about the consolidation of the fill and the corresponding settlement of the ground surface at the project site. The purpose of the monitoring system would be to develop a time/history relationship that would quantify both the magnitude of settlements on-going and its relationship to time. We suggest that a minimum of 8 settlement monuments be established in the existing ground surface and that the settlement monuments be tied into 2 independent benchmarks. It is our opinion that the development of such a history will support the theories of potential consolidation as discussed in this report.

Recognizing the high probability of settlement with time, the feasibility of developing this site must take that factor into consideration. Multi-story development of the site would generate two potential problems, in our opinion. The first is that high-rise structures will typically generate loads much greater than the allowable bearing capacity of compacted shale, clay and limestone fills. Bearing capacities of 3000 to 4000 psf are typically not adequate to support structures 4 to 5 stories or more in height. Therefore, it would be necessary to support such structures on bedrock in order to get an increased bearing capacity. This requirement has two limitations. The first limitation is that
bedrock is very deep at the project site because of the magnitude of fill which has been placed. Secondly, the bedrock formation below the project site consists of the Kope Formation shale and limestone. The Kope Formation of bedrock is one of the weaker formations of bedrock in the Northern Kentucky Area. Bearing capacities for drilled shafts in the Kope Formation may be limited to 40,000 to 60,000 psf, depending upon the depth of socket into the bedrock. It is generally not practical to install auger cast piles or driven pipe or H piles in this profile because of the limestone in the fill as well as in the weathered and unweathered zones of bedrock.

The second key problem with multi-story buildings is that they are settlement sensitive. With the potential for multiple inches of future settlement, it is not practical to support such a building at shallow depths in the existing fill. The settlement would tend to warp the superstructure which would cause significant internal problems with the structural framing and architectural features. It would therefore be necessary to consider a deep foundation system to penetrate the existing fill to minimize settlement of the superstructure. For deep drilled shafts to bedrock, not only are there two limitations as discussed above, but there is also the influence of drag-down on the drilled shafts due to future consolidation of the fill. Even though the fill is relatively tight and well-compacted, as future consolidation of the mass of the embankment occurs, the frictional drag upon the sides of the drilled shafts will increase the load on the drilled shafts. The drilled shafts will have to be proportioned accordingly for this load increase. An alternative is to design the drilled shafts with a bond breaker between the perimeter of the shafts and the shale and limestone fill so that future consolidation of the fill does not transfer load onto the drilled shafts.

Single-story structures could possibly be supported in the existing fill provided they are flexible enough to absorb settlement. The
capacities of the existing compacted fills are satisfactory to support shallow foundations as discussed above with an adequate factor of safety from a bearing capacity point of view. It is the global settlement that is the key problem. The central portion of the site has the greatest depth of fill and therefore the highest potential for settlement. The side slopes of the original valley were steep such that the depth of fill would vary by 30 to 40 feet in depth over a 100 foot dimension in a north-south direction. Such significant variations in depth of fill could result in substantial differential settlements across the width of the building. For single-story or even possibly two-story buildings distortions in the structures may result in sloping floor slabs, masonry wall crackings, plaster cracking, etc. which could be significant, dependent upon the proposed building placement.

Another significant issue relative to settlement is the construction of utilities. Utilities servicing structures supported on deep foundations would have to be designed to allow for differential movement between the embankment within which the utilities are constructed and the superstructure of the building which may be deep foundation supported. Differential settlements across the site would also have to be taken into consideration relative to utilities. Sufficient gradient would have to be built into the conduits to allow for adequate flow as the long-term settlements begin to accrue. Again, it would have to be recognized that the maximum amount of settlements would occur through the throat of the original drainage valley and the utilities planned accordingly.

Features which are not sensitive to settlement, such as parking lots, recreation facilities, etc., could readily be supported upon the fill embankment. Settlements would not have a significant impact upon those features other than the minor changes which could develop in changes in surface drainage as a result of differential settlement.
In summary, it is our opinion that it will be important to immediately establish a settlement monitoring program in order to develop a settlement history for this fill embankment. The magnitude and rate of settlement determined from the settlement monitoring program will likely dictate the types and heights of structures that can be built at this site.

The available documentation on the compaction of the fill embankment has some apparent voids in the data. We recommend that the baseline and survey control for the original density testing be researched to ascertain whether there are zones of fill where compaction data is limited or absent. We recommend that more detailed subsurface information be obtained for areas of such zones as part of the final geotechnical exploration of the site once a potential plan of development has been selected in light of information provided in this preliminary report.

CLOSURE
We have included in the Appendix to this report a reprint of "Important Information About Your Geotechnical Engineering Report" published by ASFE, The Association of Engineering Firms Practicing in the Geosciences, which our firm would like to introduce to you at this time.

We appreciate the opportunity to provide you with this report of preliminary geotechnical exploration for the fill embankment in question. Should you have any questions concerning the contents of this report, or if we may be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
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Copies submitted: 3