REPORT OF SUBSURFACE EXPLORATION AND PRELIMINARY GEOTECHNICAL SITE ASSESSMENT

Proposed Base Housing
Anderson Golf Course, Ft. Knox Military Reservation
Ft. Knox, Kentucky

AMEC Project No. 03-7590-0000

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June 2003

Amendment 4

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16 June 2003

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Re: Preliminary Geotechnical Site Assessment
Proposed Base Housing, Anderson Golf Course
Ft. Knox Military Reservation, Ft. Knox, Kentucky
AMEC Project No. 03-7590-0000

Dear Mr. Fitzhugh:

We have completed a series of borings, site reconnaissance and a laboratory testing program for the above referenced project. Presented on the following pages are summaries of the field and laboratory data developed for the proposed base housing development along with general information and/or recommendations regarding geotechnical site development issues and identified karst features. Important information regarding studies of the type performed has been prepared by the ASFE organization and this is presented in Appendix 1 for your review. This study has been completed in accordance with our proposal dated 17 April 2003 and your scope of work dated 23 April 2003. This study represents a preliminary geotechnical site assessment for approximately 90 acres. It is not intended for specific foundation design of any particular structure as subsurface conditions may vary significantly over relatively short distances. Site specific geotechnical subsurface exploration and site and structure specific recommendations should be prepared for any proposed buildings. Our professional services and findings obtained have been prepared in accordance with generally accepted engineering principles and practices. No other warranty, expressed or implied is made. Environmental services are beyond the scope of this study; accordingly, this report does not address the chronology of previous land use or any related, environmentally sensitive issues. Additionally, this report does not address regulatory issues associated with storm water runoff, the identification and modification of regulated wetlands or groundwater recharge areas.

EXPLORATION AND TESTING

A total of 49 boring locations were identified for drilling. Boring locations were determined in the field using the site topographic survey map of the property, readily accessible landmarks and a tape measure or pacing. Subsequent to the completion of our borings, Gresham Smith and Partners

surveyed each boring location. The final surveyed locations and elevations were provided to us by Gresham Smith and Partners in the form of an AutoCAD file that was used as the base map for Figure 3. Borings were advanced to a depth of 15 feet or auger refusal, whichever was encountered first. Bedrock was cored at a series of boring locations where bedrock was encountered at depths of approximately 10 feet or less. Table 1 provides a summary of the completed borings, corresponding ground surface elevation, their overall depth and thickness of overburden and bedrock cored. Corresponding boring logs are presented in Appendix 2.

The borings were auger-drilled and drive sampled in general accordance with the requirements of ASTM D 1586 (*Penetration Test and Split-Barrel Sampling of Soils)* commonly referred to as the Standard Penetration Test (SPT). Split-spoon soil samples were obtained at approximately 2.5-foot intervals throughout the overburden depth. In addition to the split-spoon samples, 26 relatively undisturbed samples were obtained by means of thin-wall (Shelby) tubes for the purpose of laboratory determinations of soil insitu unit weight and unconfined compressive strength. The locations of Shelby tubes are indicated on the boring logs. Large-volume, disturbed soil samples were also obtained to provide material used to develop moisture-density (Proctor) curves and California Bearing Ratio (CBR) values. A total of five bulk samples were obtained. Upon completion of auger drilling, rock cores were obtained from nine of the borings where auger refusal was encountered. Up to five-foot long cores were obtained using an NX-size core barrel. The Rock Quality Designation (RQD) and percent recovery values are listed on the corresponding boring logs presented in Appendix 2. Upon completion of drilling operations, each boring was monitored for the presence of water. Finally, the boreholes were backfilled with soils removed from the borings, an 18-inch thick plug of concrete placed in the borehole and the plug topped with about six inches of topsoil.

The soil samples were visually examined and logged in our laboratory by members of our professional staff. Laboratory testing was performed on selected soil samples to provide data from which to estimate engineering properties. Testing consisted of:

- 63 moisture content determinations,
- 5 grain size analyses including hydrometers,
- 15 Atterberg limits determinations,
- 25 unconfined compressive strength determinations,
- 25 unit weight determinations,
- 5 resistivity determinations,
- 5 sulphate content determinations,
- 5 modified Proctor moisture-density relationships, and
- 5 CBR determinations.

Our interpretation(s) of the soil and rock samples along with laboratory test results are included on the graphic logs presented in Appendix 2. A laboratory test summary table, grain size distribution curves, unconfined compression test reports, modified Proctor moisture-density curves and CBR test reports are presented in Appendix 3. The soil and rock samples not consumed by laboratory testing will be stored at our laboratory for a period of at least 60 days. Thereafter, they will be discarded unless otherwise directed by you.

In addition to the above soil borings and sample acquisition, AMEC also performed a site reconnaissance to catalog and describe known or readily evident closed depressions (karst features or sinkholes) located within the limits of the study area. This task was performed in conjunction with golf course superintendent Mr. Kenny Shively who was familiar with many of these features located within the limits of Anderson Golf Course. These features are numbered and identified on Figure 3. Closed depressions shown on the provided topographic site survey map or noted during our site reconnaissance were characterized and those observations are tabulated in Table 2. It should be noted that although we have endeavored to identify most existing depressed areas, our experience with similar sites indicates additional subsidence features not detected by our fieldwork may exist within areas currently forested or heavily vegetated. Mr. Shively pointed out that sinkholes which appear within the golf course are generally repaired and grassed fairly quickly; therefore, it was impossible to locate some repaired sinkholes.

SITE AND SUBSURFACE CONDITIONS

The proposed base housing project will be built over approximately 90 acres, located west of Wilson Road and south of Dixie Street on the Ft. Knox Military Reservation, Ft. Knox, Kentucky. A portion of the proposed study area is located on the existing northern portion of the Anderson Golf Course. Within the overall study limits, the area including the golf course clubhouse and base Guesthouse is excluded from the development area. The general site location is identified on the USGS topographic quadrangle maps for Vine Grove and Fort Knox, Kentucky, a copy of which is presented as Figure 1.

The Anderson Golf Course, generally in the area surrounding the existing clubhouse and Guesthouse, makes up approximately two-thirds of the study area. Remaining existing structures and infrastructure within the study area include Binter Court and eight associated buildings, electric, gas, water, sanitary and irrigation utility lines, and an unpaved road extending southwest from the end of Binter Court. The general terrain is rolling to relatively flat with typical slopes ranging from 3H:1V (Horizontal: Vertical) to about 30H:1V. Some locally steeper slopes (<2:1V) and flat areas

appear to be the result of grading or former development activities. Topographic contours on the site survey map provided to us exhibit topographic relief of approximately 22 meters (72 feet) and elevations are between 200 and 221 meters (656 to 725 feet, MSL). Approximately 25 acres on the western side of the subject study area are predominantly covered with woods or dense vegetation. Small pockets of trees and landscaping are found throughout the remainder of the site; however, with the exception of building or paved areas the remainder of the study area is open grass.

Drainage within the study area is provided by overland runoff as well as some improved drainage facilities including a series of roadside ditches and culverts along Wilson Road and Binter Court. Natural drainage features include one intermittent blueline stream that flows from west to east across the northern third of the site and a series of closed depressional landforms likely associated with underlying karst features (sinkholes) that detain surface water and either convey it underground or allow it to evapotranspirate. The stream directly captures runoff from approximately two-thirds of the site and may capture, through underground connections, essentially all precipitation that falls on the site and percolates into the subsurface. After leaving the northeast corner of the study site, the stream flows north parallel with Wilson Road, then east and discharges to Mill Creek. Mill Creek flows north discharging to the Salt River which subsequently discharges to the Ohio River.

Characterizations of karst features (in this case, closed topographic depressions commonly referred to as sinkholes) were made at 10 locations across the study area as shown on Figure 3. Depression identified as S-101 was determined to be related to former grading work and was not considered the result of karst activity. At several locations, multiple small karst features in close proximity or located within one larger depression were described together (such as S-100, S-102, S-104, S-105 and S-106). The majority of the features were very nearly round in shape and varied in diameter from several yards to tens of yards. Almost all of the features (or “dropouts”) identified were located in the southwestern portion of the study area. Several features were noted along a broad swale (see S-103, S-106 and S-100) that ended in a closed depression (S-100). (We understand from conversations with the golf course maintenance personnel that other drainage features in other parts of the golf course (not within the study area) also have a number of “drop-outs” or sinks that follow the axis of the existing drainage features.) Due to the demands of operating a golf course, “dropouts”, often only several feet in diameter and several feet in depth, are regularly filled and grassed to eliminate hazards to golfers. Depression S-108 was completely obscured by re-grading activities including the placement of a sidewalk over the area and was located for us by golf course maintenance personnel.

Isolated areas generally within existing larger depressions were noted to have experienced relatively recent subsidence and/or repair (such as S-109); however, the overall bottom of the larger features appeared to be relatively stable, were soil lined and often contained some fill material. Fill materials typically consisted of crushed rock and/or soil that had been placed to repair them. Large mature trees were generally not present in the features reviewed. Water was found in S-100 but was not found in any of the other features at the time of our field program. Based on conversations with maintenance personnel, we understand several features do temporarily contain water following heavy periods of precipitation.

General Soils Information

A review of published soil survey literature indicates a majority of the property has a soil description designation of Silt Loam – Crider Silt Loam (CrD, 2% to 6% slopes), Nolin Silt Loam (No, 0% to 2% slopes), Vertrees Silt Loam (VrD, 12% to 20% slopes) and Vertrees Silty Clay Loam (VtD3, 6% to 20% slopes, severely eroded). The Vertrees Silty Clay Loam soil series covers approximately one-half of the study area and dominates the southern half of the site. Located primarily in the northern half of the site the Nolin Silt Loam covers about 30 percent of the study area primarily within the area along the intermittent blueline stream and Binter Court. The remaining portions of the site consist of small area of Vertrees Silt Loam in the northwest portion of the site and Crider Silt Loam in a thin strip next to Wilson Road.

According to the USDA soil survey, the Vertrees Silty Clay Loam soil series is described as moderately steep, deep, well-drained soils on ridgetops and hillsides and in karst areas. This soil has lost most of its original surface layer through erosion. A representative profile of this soil includes from 0 to 6 inches of yellowish-brown to reddish-brown silty clay loam and from 6 to 66 inches of yellowish-red, firm, plastic clay with occasional chert and soft shale fragments. This soil is reported to have low to moderate shrink-swell potential.

According to the USDA soil survey, the Nolin Silt Loam soil series described as well-drained, nearly level soil in strips 175 to 2,000 feet wide on flood plains and in circular and oval areas in depressions. In the depressions, surface water drains through openings into underground streams. Without drainage improvements, this soil is subject to frequent flooding from November to May. In a representative profile this soil includes from 0 to 10 inches of dark grayish-brown silt loam and from 10 to 66 inches of brown silt loam. This soil is reported to have low shrink-swell potential.

According to the USDA soil survey, the Vertrees Silt Loam soil series is described as well-drained, deep, moderately steep soil is found on hillsides and in karst areas. Areas on hillsides are in bands 150 to 1,000 feet wide that wind around the points of ridges and the heads of ravines. In karst areas, depressions are common and slopes are irregular. A representative soil profile includes from 0 to 7 inches of brown silt loam with occasional chert fragments, from 7 to 24 inches and is yellowish-brown mottled red, firm, plastic clay. From 24 to 70 inches it is red, firm, plastic clay with occasional chert fragments.

According to the USDA soil survey, the Crider Silt Loam soil series is described as well-drained, deep, gently sloping soil in bands, blocks and ovals areas on ridgetops and in karst areas. In karst areas, drainageways are dismembered and lead through openings in depressions into underground streams. A representative soil profile includes from 0 to 8 inches of brown silt loam, from 8 to 26 inches it is brown silt loam and from 26 to 48 inches it is reddish-brown silty clay loam. Below this horizon is red, very firm, plastic silty clay.

General Bedrock Information

A review of published geologic literature indicates the site is underlain by the Mississippian Age St. Louis Limestone (map unit Msl) as shown on Figure 2. This formation, which is greater than 230 feet thick, consists of limestone, dolomite and shale. Limestone is yellowish-gray, light olive gray and dark gray to medium bluish-gray and weathers light gray to olive gray. It is very fine to medium grained with beds commonly less than 1 foot thick. Dolomite is medium dark gray to olive gray and weathers to yellowish-gray. It is very fine to fine grained with beds from 0.1 to 5 feet thick. The parent rock weathers to silty and cherty reddish-orange clay residuum generally 30 to 40 feet thick. This deep weathering and numerous sinks typify the unit. Bedrock dips gently to the east at a rate of approximately 20 feet per mile based on mapped information for the Salem Limestone which lies stratigraphically below the St. Louis Limestone about elevation 198.1 m, MSL (650 feet, MSL).

Site Exploration Results

A total of 49 borings were performed on a relatively even spacing across the study site at the locations shown on Figure 3. Soils encountered in the upper intervals or horizons generally match reasonably well with published soil survey information. Some areas of the golf course as well as the developed areas have been graded to achieve the current contours; however, soil types encountered in these areas generally reflected site soils in presumably undisturbed areas. At sixty percent of the boring locations bedrock was not encountered within the upper 15 feet. Six borings

encountered rock at depths of 10 feet or less(as shallow as six feet); however, rock was generally encountered at depths greater than 10 feet below existing ground surface.

Based on the exploratory data, the general soil profile consists of a surficial interval of topsoil. This interval generally consisted of organic silty clay and was predominantly six inches thick. At a few notable locations (Borings B-42 through B-44 and B-46) topsoil was found to be approximately 12 inches thick and at several other locations (Borings B-39 and B-45) topsoil was found to be only one to two inches thick. Below the surficial interval the borings encountered generally firm to stiff, brown to dark brown, reddish-brown and reddish-brown to mottled tan, moderate to high plastic, slightly silty to silty clay to low plastic, clayey silt. Variable amounts of black nodules, chert and rock fragments were found throughout. This interval varied in thickness from several feet thick to greater than 15 feet. Intervals of gray silt were noted in several borings and where the borings approached the highly weathered underlying bedrock, soils were significantly lighter in color, tended to be more silty to sandy and contained substantial amounts of shale and limestone rock fragments. It is noted that rock fragments contained within the soil matrix generally inflated SPT blow counts. Soils with a consistency of soft to very soft soils were encountered at several locations as listed below:

- Boring B-6 to a depth of 8.0 feet,
- Boring B-9 to a depth of 4.0 feet,
- Boring B-14 to a depth of 3.0 feet,
- Boring B-18 to a depth of 4.0 feet,
- Boring B-19 to a depth of 6.0 feet,
- Boring B-27 between 3.0 and 10.0 feet, and
- Boring B-28 between 3.0 and 8.0 feet.

The soils encountered during this exploration appear to be residual in nature, derived from the underlying limestone, dolomite and shale bedrock. Each boring was checked for water upon completion and was found to be dry. Despite the lack of water present in the borings, stabilized or temporary perched groundwater levels may potentially be found within the depths explored at other times of the year.

Bulk over burden soils from five locations were tested for a variety of soil characteristics and corrosion potential. All lab data is summarized in Appendix 3 and several aspects are discussed here. The table below presents the soil corrosivity parameters tested compared with reference critical values.

Test	Units	Reference Critical Values	Lab Results				
			B-6 1' - 5'	B-19 1' - 5'	B-23 1' - 5'	B-29 2' - 5'	B-35 10' - 15'
Resistivity	ohm - cm	<2,000	20,400	5,710	8,500	8,600	47,800
Sulphate Content	ppm	>500	18.4	ND	16.1	ND	ND

Notes: ND - Below detection level of test method.

Based on the above data the soil constituents tested for the selected soils appear to fall well below the level of concern for sulphate content and well above the minimum value for resistivity.

CBR values were determined for five bulk soil samples obtained from the site and test reports are included for these samples in Appendix 3. CBR values obtained at 95% of the modified proctor maximum dry density and optimum moisture ranged from 1.4 to 12.5. The low value of 1.4 (from Boring B-2; 1' - 5') was determined from a high plastic clay sample with a liquid limit of 69 (plasticity index of 43). The remaining four samples with plasticity indices between 9 and 18 resulted in CBR values between 6.0 and 12.5, averaging 9.3. The CBR test from Boring B-2 also indicated some moderate swell potential of about 1.5% for samples at or above optimum moisture.

Bedrock cores revealed variable bedrock materials including predominantly limestone and dolomite with some shale. Core recovery was quite variable with values ranging from 27% to 100% and averaging about 85%. Rock Quality Designation (RQD) was determined for each core with values ranging from 0% to 89% and averaging 62%. Rock quality varied from very poor to excellent with the majority ranging from fair to good. Rock exhibiting poor rock quality were highly weathered throughout the length of core obtained, while rock exhibiting good to fair quality showed highly weathered rock generally within the upper one to three feet.

SEISMICITY

Based on the Kentucky Building Code (Edition 8, 2002), Section 1615.0, and the Army's Technical Instruction 809-04 a "C" Site Classification should be used for design of structures considering the firm to stiff soils and presence of relatively shallow underlying bedrock. Furthermore, the design of the structures should conform to seismic requirements stipulated by the local building code. In accordance with the Kentucky Building Code, Hardin County, Kentucky area has Spectral Response Acceleration Coefficients, S_s and S_1, of 0.25 and 0.14, respectively.

PRELIMINARY COMMENTS AND RECOMMENDATIONS

General

We have not been provided with a site grading plan, proposed building elevations, or specific design information for the proposed development. Therefore, the comments and recommendations contained in this preliminary geotechnical site assessment are provided as guidelines that are applicable, in general, to the overall study area but not necessarily applicable to any particular location. Our comments and recommendations must be reviewed, and if necessary, updated or revised in light of the final development design. A site specific geotechnical study should be performed for each structure or pavement once the locations, elevations, structural loadings and limits of those features are determined.

Based on the provided information we understand building development will consist of 1- to 2-story wood-framed, single or multi-family houses or town homes. For typical buildings with maximum column loads of less than about 75 kips and wall loads of less than about 2.0 kips/ft., we expect shallow spread foundations bearing upon stable natural soil or properly compacted engineered fill can be utilized. If significantly heavier structures are anticipated, or extremely settlement-sensitive structures, are planned, it should be possible to extend foundations to contact bedrock as either over-excavated spread footings or as drilled piers.

Although the number of karst features in the southwest portion of the study area is significant, developments within similar geologic settings have been successfully completed in the past when adequate engineering and remedial efforts are employed. Therefore, provided the Owner is willing to accommodate some karst features in an undisturbed state within the site plan and repair existing sinkholes where that is a reasonable approach, we are of the opinion the site can be developed.

With the exception of areas within and adjacent to closed depressions, and along drainage channels or perennial streams, the site conditions are judged to be fairly straightforward in regard to geotechnical issues associated with site development for residential-type structures. The upper interval of topsoil will require stripping operations to achieve a stable subgrade. Excavations deeper than about 10 feet have an increased chance to encounter bedrock. The non-organic soils on-site are generally suitable for use as engineered fill, although some very silty soils may be somewhat difficult to compact if the work is conducted during periods of wet weather and high plastic clay soils may present difficulty with handling and achieving a firm surface. Additionally, soft zones encountered within building footprints will require undercut and repair.

Site Preparation

Within the areas proposed for construction, topsoil, organic materials, fill materials and other deleterious materials will need to be removed from the proposed building areas. Stripping operations should extend a distance of at least 10 feet beyond the limits of the proposed structures where possible. Based on the exploration, stripping operations to remove the topsoil will be on the order of 6 to 8 inches. Existing utilities, structures, foundations, pavements and other buried infrastructure within the proposed building footprints should be completely removed or protected from damage. Any excavations resulting from abandoning utilities should be inspected to confirm all deleterious materials have been removed and that abandoned underground pipes and other utilities have been completely removed or properly severed, capped and/or grouted.

Subsequent to the stripping operations, the exposed subgrade within areas to receive fill should be systematically and thoroughly proofrolled with heavy, rubber-tired equipment in order to detect soft or unstable soils. If soft areas are detected during grading operations, the areas should be undercut to firm ground and the resulting excavation backfilled with engineered fill. Alternatively, if approved by the geotechnical engineer, unstable areas may be aerated and re-compacted. Upon reaching the required subgrade elevation within excavated areas, the soil subgrade should also be proofrolled and repaired as previously described.

Organic-free, low plasticity, soils derived from on-site excavations will be suitable for use as engineered fill. (It is noted that a percentage of the onsite soils do characterize as high plastic soils.) If used, off-site borrow should consist of organic-free soil with a plasticity index of less than 25. Borrow soil should be free of large rock fragments (i.e., greater than about 4 inches) or other deleterious materials. Engineered soil fill placed within the proposed building footprints should be compacted to at least 98% of the soil's maximum dry density in accordance with ASTM D 698 (standard *Proctor*) and placed in lifts not exceeding 8 inches in uncompacted thickness when heavy, self-propelled compaction equipment is utilized. When hand operated compaction equipment is used the loose lift thickness should be reduced to one-half this value. In order to reduce the potential for volume changes in response to changes in moisture content, the moisture content of engineered fill should be controlled to within ±2% of the standard Proctor optimum moisture content during placement. Extremely silty or high plastic soils should be controlled to within 1% of the standard Proctor optimum moisture content during placement. Contractors should be prepared to reduce or increase the soil’s moisture, as required, to maintain the soil’s moisture content within the specified range.

The site should be maintained in a well-drained condition, both during and after construction, to inhibit water from ponding on soil subgrades. Ponding water could lead to deterioration of the soil subgrade, necessitating over-excavation of the softened soil.

Embankment/Cut Slopes

We are unaware of any new, fill slopes (i.e., greater than about 10 feet) that will need to be constructed as part of the construction. Therefore, the following is provided as general information. If the final site design includes new slopes in excess of about 10 feet in height, we request we be given the opportunity to review the subsurface data and our recommendations in the light of the proposed design.

Cut and fill slopes in soil, should be designed no steeper than 2H:1V for slopes up to a maximum height of 10 feet; the limits of structures, as measured from the outside face of the nearest foundation member, must be at least 10 feet from the slope crest. Fill placed in the zone between the edge of the building and the slope crest should be compacted to engineered fill requirements. Higher slopes should be individually studied in light of the completed grading plan. If it is desirable for fill slopes to be maintained as landscaped areas, we recommend inclinations no steeper than 3H:1V be used. The Contractor must properly plan and design his excavations with safety in mind. All applicable OSHA requirements relating to braced and unbraced excavations must be followed.

During construction, isolated zones of perched water, as well as wet weather springs, may be encountered within the overburden. It is our opinion water emerging from cut slopes as groundwater seeps can be controlled by a perimeter drainage system consisting of surface ditches installed below the cut slope. Depending on the configuration of the slope(s) and the amount of water encountered, subsurface drains installed along the perimeter of the proposed building may also be needed. Intercepting surface water and shallow subsurface water flows in this manner should eliminate the need for under-floor-slab drains beneath the building.

Foundation Design

Provided the site is prepared in accordance with the recommendations stated previously, the proposed 1- and 2-story residential buildings can be supported by means of a conventionally designed, shallow foundation system bearing on either engineered fill, stable, natural soil or a combination of those mediums. Based on a safety factor of at least 3 with respect to general shear failure, we believe allowable load bearing capacities of 3.0 KSF and 2.5 KSF for individual and

continuous building footings, respectively will be applicable for both the stable, natural soil and properly compacted, engineered fill. A minimum footing width of 24 inches should be specified for all foundations, regardless of loading, in order to accommodate minor subgrade inconsistency and to resist punching failure. Further, the perimeter building footings should be designed to bear at least 24 inches below exterior, finished grades to provide frost protection, adequate confinement, and to provide a bearing level that is below the depth of significant seasonal moisture change.

If foundation excavations within the proposed building areas encounter bedrock, we recommend rock be removed to a minimum depth of 2 feet below the proposed foundation bearing surface and that the over-excavation be replaced with engineered fill compacted to a minimum of 98% of the standard Proctor maximum dry density.

Based on the available data, we estimate the maximum, total, long-term settlement of the proposed structures will be less than 1.0 inch. Differential settlement between adjacent foundation members could approach 50% of the total amount. Because measurable increments of settlement are likely to occur, the design of masonry walls should include provisions for liberally spaced, vertical control joints to minimize “cosmetic” cracking.

Based on the water level data generated during this study (borings were dry upon completion), we do not expect shallow foundation excavations will experience groundwater inflows. However, depending on the depth of excavation, as well as weather conditions immediately prior to and during the construction, perched zones of groundwater could be encountered. In that event, it will be necessary for the Contractor to maintain excavations in a dewatered condition during inspection and prior to concrete placement. The bottom of the excavations must be thoroughly cleaned prior to concrete placement; concrete should not be cast if the excavation contains loose soil, debris or water. Whenever possible, foundation excavations should be opened and cast with concrete on the same day. If soil excavations must remain open for an extended period, they must be protected from rainfall, excessive drying, and surface water infiltration. Footing excavations should be examined by a representative of the geotechnical engineer immediately prior to the placement of concrete in order to assess the condition of the bearing surface.

Slabs-On-Grade

Concrete slabs-on-grade are expected to perform satisfactorily if founded on a properly prepared subgrade as described above. A free-draining, granular base, at least 4-inches thick, and a vapor barrier should be incorporated into the slab design. Slab thickness and reinforcing requirements

can be preliminarily designed based on an estimated subgrade reaction modulus of 120 PCI. An appropriate number of control joints should be included in the slab design to accommodate minor differential settlement that may occur.

Pavement Design

For the purposes of pavement design recommendations, we have considered light duty loading resulting from approximately 200 cars per day. If the subgrade is prepared as previously recommended, we estimate the soil subgrade should develop minimum support characteristics equal to a California Bearing Ratio (CBR) of approximately 5. Based on that degree of subgrade support, we offer the follow flexible pavement section for your consideration. Once the material is identified which will provide support for the pavement, the material's CBR should be determined. Based on that test, the pavement thickness provided may need to be revised. The following design scenarios were developed considering an effective design life of 20 years.

FLEXIBLE PAVEMENT

Material	Light-Duty (200 Cars Per Day) Thickness (inches)
Asphalt Surface (Hot Mix)	1.0
Asphalt Binder (Hot Mix)	2.0
Mineral Aggregate Base Course compacted to at least 98% of its maximum modified *Proctor* (ASTM D1556) dry density	6.0
Total Pavement Thickness	**9.0**

Pavement materials and methods should conform to the requirements of the latest edition the state Department of Highway's *Standard Specifications for Road and Bridge Construction.*

To prevent the aggregate base course from becoming saturated, and thereby reducing the support capabilities of the subgrade, we recommend the subgrade be graded to provide positive drainage away form the paved areas. Immediately prior to the placement of the base course stone, the subgrade should be proofrolled in order to verify its stability. During the placement of the aggregate base, field compaction tests and thickness determinations should be performed in order to confirm compliance with the project documents. If a significant amount of time elapses between placement of the aggregate base and the asphalt courses, the surface of the stone should be proofrolled immediately prior to paving operations to confirm that its stability has been maintained.

Karst Planning Issues

Based on the presence of active karst activity within significant portions of the site, and in keeping with good practice, planning for developments in karst areas should consider: how to protect the public from hazards associated with karst activity; how to avoid or mitigate impacts to groundwater supplies; and, how to avoid negative impacts to surface water drainage. The discussion below considers how each of the above items can be addressed, and other issues you should consider as planning progresses.

The present state of the art of geotechnical engineering does not allow us to predict, with certainty, when or where future sinkholes might occur. However, in general, the highest potential risk of subsidence occurs within areas that have already experienced subsidence. Therefore, the most obvious approach in development to reduce potential risk is to avoid locating structures or roadways within existing closed depressions or in areas heavily impacted by karst features. However, given proper geotechnical study, a specific plan may be developed to address development in impacted areas.

Another issue to consider is the potential effect on groundwater created by development in karst areas. The blockage of existing sinkholes previously accepting surface water can influence the groundwater table. If water wells are being used in areas close to “plugged” sinkholes, the groundwater level could drop to make those wells less productive. Routing surface water runoff into sinkholes may raise groundwater contamination concerns or even accelerate the development of a sinkhole.

With specific application to the subject site, the observed karst features do influence surface and groundwater flow to some degree. Specifically, surface water is intercepted by the karst features, which diverts surface water underground; however, based on our site review, we believe most, if not all, of this water likely reappears in the form of springs located within the military reservation limits. Protection of the quality of water entering the sinkholes should be considered in any development plan. Runoff from paved areas should not be allowed to flow directly into sinkholes without some level of treatment. Treatment may include natural filtration through use of buffer zones or stormwater detention ponds to trap sediment and prevent floating contaminants such as oils from being discharged directly to sinkholes.

Federal EPA regulations may require certain precautions be employed when surface runoff is directed underground via use of "injection wells"; areas where surface runoff is directed underground through sinkholes are included in EPA's definition of injection wells. Specific things that can be included within the site development plans in order to incorporate reasonable, prudent precautions to reduce impacts to water quality and the natural karst setting, and to limit the risk of karst-related damage to future structures, include the following:

1. Any sinkhole which accepts surface runoff and serves as part of a regional system to detain and/or transport surface runoff should be left in its natural condition. Several of the depressions in the southwest portion of the study area appear to accept surface water flow from beyond the limits of their specific closed depression. Sinkhole S-100 appears to be located at the downstream end of a natural drainage feature and receives surface water from a upstream areas.
2. Existing surface drainage should continue to flow to the same locations as exists now. If surface runoff beyond what currently occurs must be directed into sinkholes, a detailed geologic/hydrologic analysis should be completed to determine that the sinkhole is capable of transmitting that volume of runoff without causing flooding at that location, or within areas that are inter-connected to the sinkhole by bedrock channels. In addition, the quality of the surface water must be maintained. If the runoff crosses paved areas or can otherwise pick up debris or oily residue created by the development, pre-treatment should be included prior to discharge into the sinkhole. An injection well permit from the State will be required for any changes in the water quantity or quality entering sinkholes.
3. The naturally occurring bedrock channels, voids and joints which currently serve to transmit groundwater downward or laterally, should be left in such a condition to allow groundwater to follow its natural course. If structural repairs to bedrock cavities are required to prepare the subgrade for development, an open-graded stone should be employed to fill bedrock defects so that groundwater can continue to move through those channels. Concrete should not be allowed to plug any bedrock channels.
4. In areas where karst features are to be avoided to help reduce costs for sinkhole remediation, the building limit line should be located on the order of 50 feet beyond the limits of the sinkhole(s).
5. If surface water/groundwater interconnections will be altered, the location of water wells that may intercept groundwater channels influenced by water entering sinkholes on this property should be identified. If justified, studies should be completed to determine if the quantity or quality of water within those wells will be adversely affected by altering flows into sinkholes

on this site. Our preliminary data indicates it is unlikely offsite properties will be affected even if groundwater flows on this site are altered; however, we are not aware of the proximity of any offsite wells.

Sinkhole Remediation

The development of property underlain by solution-prone bedrock with a known history of sinkhole formation is always associated with risk. Specifically, existing sinkholes can continue to subside or enlarge in area, or new sinkholes may develop within presently stable areas. While engineering analysis and sinkhole remedial efforts can substantially lessen the potential risk of subsidence, some potential risk of future subsidence is inherent with the geologic setting of the property.

Where structures and roadways cannot be located to avoid closed depressions, proper measures must be taken to repair closed depressions that are in the path of development. Ideally, the relatively larger sinkholes or sinkhole areas could be incorporated for use in landscape areas.

Our general concerns regarding sinkholes, particularly those with soil subgrade, in this geological setting are as follows:

1. Soft, unconsolidated soils may consolidate and voids may collapse under the weight of fill materials or structures.
2. The continued movements of surface water into the subsurface via the existing depressions will continue to weather the bedrock and move soil downward into the bedrock, increasing the potential risk of subsidence.

Therefore, in order to reduce the potential risk of subsidence, we recommend, where applicable, unconsolidated soils be excavated from each closed depression and the bedrock surface exposed to locate openings into the bedrock. Subsequently, the openings should be plugged with an inverted stone filter (this stone filter can then be capped with concrete provided it does not plug groundwater channels) and the remaining excavation then backfilled with appropriate engineering fill. Repairs should be accomplished under the direction of the geotechnical engineer or his representative.

Ponding and subsequent infiltration of water into the subgrade is a significant factor in initiating ground subsidence and sinkhole formation. Therefore, preventing additional surface water from concentrating or ponding in sinkhole prone areas is essential. If possible, additional surface water

runoff should be piped away from structures and discharged directly into existing drainage channels or storm sewers when possible. If any incipient sinkholes are noted during construction, they should be repaired, as determined by the geotechnical engineer, on a case-by-case basis.

In summary, due to the presence of limestone and/or dolomite bedrock, the potential risk of sinkhole development at any location cannot be entirely eliminated. Within areas with active karst features, additional engineering and remediation efforts will be needed to provide site by site recommendations for development. Provided sinkhole remediation efforts are properly completed, we believe the long-term potential risk is similar to the potential risk associated with developing most other properties in this part of Hardin County which are underlain by limestone and/or dolomite bedrock.

CONSTRUCTION MONITORING

The satisfactory, long-term performance of the foundations, slabs and pavements will be dependent on the workmanship and adherence to details contained in our recommendations and the project specifications. We strongly recommend all aspects of the site preparation and foundation installation be monitored by a qualified technician responsible to the geotechnical engineer and the Owner. In particular, the proofrolling operations and any remedial treatment to the subgrade and sinkholes, and the placement and compaction of engineered fill should be continuously monitored and periodically tested. Additionally, prior to concrete placement, the geotechnical engineer or his representative should review all foundation excavations.

CLOSURE

We emphasize the information contained is this report represents a preliminary study, which while it provides appropriate design guidelines and recommendations for the anticipated development may not meet the standard of care both in the concentration of borings and site specific recommendations for specific design of any particular building. In order to develop detailed comments and recommendations concerning the geotechnical aspects of design and construction for specific structures, it will be necessary for us to perform a more detailed study that addresses site specific items relating to the proposed facilities.

AMEC appreciates this opportunity to be of service to RSP Architects and Ft. Knox Military Reservation. At your convenience, we are available to discuss the details of this report and any questions you may have, and the in-depth comprehensive geotechnical investigation that will be required to develop final recommendations.

Yours truly,

David G. Sawitzki, PE
Geotechnical Engineer

James B. Shepard, Vice President
General Manager - Louisville Office

Enclosures

APPENDIX 2

BORING LOGS

THE BORING LOGS SHOWN DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS DRILLED AND AT THE PARTICULAR TIMES, WHICH MAY BE DESIGNATED ON THE LOGS. SUBSURFACE CONDITIONS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURING AT THESE BORING LOCATIONS. ALSO, THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE SUBSURFACE CONDITIONS AT THE BORING LOCATIONS DRILLED

APPENDIX 3

LABORATORY TEST DATA SUMMARY,
GRAIN SIZE DISTRIBUTION CURVES,
MOISTURE DENSITY RELATIONSHIPS (PROCTORS)
UNCONFINED COMPRESSION TESTS AND
CBR TEST REPORTS