THELEN ASSOCIATES, INC.
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January 5, 2005

Tetra Tech, Inc.
11156 Canal Road
Suite A
Cincinnati, Ohio 45241

Attn: Mr. Dennis Huber, P.E.

Re: Consulting Services
Boone County Rural Water Project
Phase II
Contract 2A
Boone County, Kentucky

Ladies and Gentlemen:

Summarized in this report are the results of the Geotechnical Consulting Services provided to Tetra Tech, Inc. (Tetra Tech) by Thelen Associates, Inc. (Thelen) for the proposed Boone County Rural Water Project, Phase II, Contract 2A in Boone County, Kentucky. The work was performed in accordance with our Proposal-Agreement K24209, and was authorized by Mr. Dennis Huber of Tetra Tech by signature of Tetra Tech Work Order No. 2 on September 29, 2004.

This report summarizes the results of the engineering reconnaissance and test borings performed along Big Bone Church Road, Kirby Lane, Forest View Drive, Michelle Drive and Brian Court, and a review of the Project Drawings prepared by Tetra Tech dated November, 2004 and received by Thelen on November 9, 2004. We have summarized our conclusions and recommendations for the location and depth of the water main based upon the reconnaissance, the results of the test borings and our analyses.

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

We appreciate the opportunity to provide our consulting services for the Boone County Rural Water Project, Phase II, Contract 2A. Should you have any questions concerning the information, conclusions or recommendations contained in this report, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

Bryan M. Scott

Bryan M. Scott, E.I.
Staff Geotechnical Engineer

Theodore W Vogelpohl

Theodore W. Vogelpohl, P.E.
Chief Geotechnical Engineer

STATE OF KENTUCKY
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PROFESSIONAL ENGINEER

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January 5, 2005

CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
PHASE II
CONTRACT 2A
BOONE COUNTY, KENTUCKY

1.0 INTRODUCTION

Presented in this report are the results of the geotechnical engineering reconnaissance and test boring exploration for the proposed 8-inch diameter water main to be installed along Big Bone Church Road, Kirby Lane, Forest View Drive, Michelle Drive and Brian Court in Boone County, Kentucky. The scope of our geotechnical services included a review of the existing topography and other land features along the alignment proposed on the plans prepared by Tetra Tech, Inc. (Tetra Tech). Areas of apparent or potential instability along the selected alignment were identified during the reconnaissance, and only these areas were explored by advancing test borings to determine the subsurface conditions. The conclusions and recommendations contained in this report regarding the water main alignment and depths are based on the engineering reconnaissance and the test boring information.

2.0 PROJECT CHARACTERISTICS

The project drawings referred to in this report are titled "Boone County Rural Water Project, Phase II, Contract 2A – Big Bone Church Road", dated November, 2004, marked as "Preliminary" and received by Thelen Associates, Inc. (Thelen) on November 9, 2004. The Phase II-A portion of the Boone County Rural Water Project will consist of roughly 8250 lineal feet of pipe along Big Bone Church Road, 1725 lineal feet of pipe along Kirby Lane, 2670 lineal feet of pipe along Forest View Drive, 1520 lineal feet of pipe along Michelle Drive and 665 lineal feet of pipe along Brian Court. All of the water main pipe is to be 8-inch diameter Class 50 ductile iron pipe.

The proposed water main along Big Bone Church Road will connect to an existing 8-inch diameter water main near the Big Bone Road intersection. The proposed water main along each of the other roads will connect to the proposed water main along Big Bone Church Road. The change in surface elevation along the water main alignment for Big Bone Church Road is 122 feet, with the lowest point located near the intersection of Kirby Lane (El. 748) and the highest point located at the east end of Big Bone Church Road near the intersection of Big Bone Road (El. 870). The change in surface elevation along the water main alignment for Kirby Lane is 94 feet, with the lowest point located at the creek crossing near the intersection of Big Bone Church Road (El. 736) and the highest point located at the south end of the proposed alignment (El. 830). The change in surface elevation along the water main alignment for Forest View Drive is 50 feet, with the lowest point located at the south end of Forest View Drive (El. 800) and the highest point located at the first southward bend of the alignment (El. 850). The change in surface elevation along the water main alignment for Michelle Drive is 72 feet, with the lowest point located near the intersection of Big Bone Church Road (El. 775) and the highest point located at the north end of Michelle Drive (El. 847). The change in surface elevation along the water main alignment for Brian Court is 27 feet, with the lowest point located at the southernmost culvert crossing (El. 824) and the highest point located at the end of the alignment (El. 851). The water main will follow the roadways, typically several feet off the upslope edge of the pavement. In general, the roadways have been constructed on relatively gentle to moderately sloping valley walls and ridgetops or fill embankments.

The depth of the new water main is expected to vary along the alignment due to constraints caused by local geology, existing utilities and other construction–related features. It is our understanding that the minimum cover at the crown of the pipe will be

4 feet. Throughout this report, "normal depth" refers to at least 4 feet of soil cover above the top of the pipe.

Specific descriptions of the proposed water main installation and our recommendations are provided on a section-by-section basis in the Conclusions and Recommendations Section of this report. This report only addresses the geotechnical issues for the water main project.

3.0 ENGINEERING RECONNAISSANCE

The reconnaissance of the alignment was made by the Engineer, during which evidence of soil and bedrock exposures, slope movement, pavement subsidence, steep existing slopes, fill embankments, erosion, etc., were noted. The project drawings by Tetra Tech were used as a reference during the reconnaissance.

The alignment begins on Big Bone Church Road at the intersection of Big Bone Road. The alignment begins on relatively gently sloping terrain and continues on the right hand (northwest) side of the road. The alignment passes several ponds that are on the left hand side of the road. Near the second of these ponds, the alignment crosses a low-lying marshy area. The alignment then crosses several moderately steep slopes on the uphill side of the road, before reaching Michelle Drive. Beyond Michelle Drive the alignment remains on the right (upslope) side of the road, before crossing to the downslope side of the road between Michelle Drive and Kirby Lane. The alignment then crosses a steep slope above the erosional bank of the creek below. Immediately beyond this slope, the alignment crosses Kirby Lane and continues on the downslope side of the road across relatively gentle slopes. The alignment continues along relatively gentle terrain, crossing several small fill embankments, and terminating near the intersection with Gum Branch Road.

Another section of the alignment follows Kirby Lane, beginning at its intersection with Big Bone Church Road. From the intersection the alignment descends a small slope, crosses the creek west of the culvert below Kirby Lane, then ascends a small slope and crosses to the east (upslope) side of Kirby Lane. The alignment then remains on the upslope side of the road past a culvert and crosses to the west side of the road. The alignment then passes Forest View Drive and continues on the west side of the road across relatively gentle terrain, and terminates approximately 300 feet beyond Forest View Drive.

The alignment also continues on Forest View Drive from the intersection with Kirby Lane. This section of the alignment begins on the north side of Forest View Drive as the road ascends a relatively gentle slope. Beyond this relatively gentle slope, the alignment crosses gentle terrain, remaining on the left side of the road, terminating at the end of Forest View Drive.

Another section of the alignment follows Michelle Drive from its intersection with Big Bone Church Road. The alignment follows the west side of Michelle Drive, crossing a relative gentle slope on the downslope side of the road, crossing a culvert, and remaining on the upslope side of the road as Michelle Drive ascends a relatively gentle slope. The alignment continues to follow the left (upslope) side of the road past Brian Court, and this section of the alignment terminates at the end of Michelle Drive.

The alignment also continues on Brian Court from the intersection with Michelle Drive. The alignment follows the north side of the road as Brian Court crosses relatively gentle terrain. This section of the alignment terminates at the end of Brian Court.

In general, the alignment crosses relatively gentle to moderately sloping terrain throughout the alignment. The proposed alignment does cross several areas of potential instability along the course of the alignment. Specific details of these areas are described on a section-by-section basis in the Conclusions and Recommendations Section of this report.

4.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Three test borings were made at locations selected by Thelen to explore the subsurface conditions along the alignment. The test borings are labeled BB-1 and BB-2 along Big Bone Church Road, and KL-1 along Kirby Lane. No test borings were completed along

Forest View Drive, Michelle Drive or Brian Court. The test boring locations and ground surface elevations were surveyed by Tetra Tech. The ground surface elevation at each test boring location was referenced to Mean Sea Level (MSL). The test boring locations are referenced to the centerline of the road by station and offset for this report. The locations are noted at the tops of the test boring logs and are summarized in Table 1, Test Boring Locations.

TABLE 1

Test Boring Locations

Test Boring No.	Road	Station	Offset
BB-1	Big Bone Church Road	12+30	7' R
BB-2	Big Bone Church Road	50+30	8' L
KL-1	Kirby Lane	6+81	4' R

The test borings were made with a truck-mounted drill rig advancing continuous flight augers. Two-inch outside diameter (O.D.) driven split-spoon samples were obtained at pre-selected intervals according to the procedures outlined in ASTM D1586. Recovered split-spoon samples were placed in glass jars to retain the samples at their in-situ moisture contents. The sample jars were appropriately marked in the field for proper identification.

Concurrent with the drilling operation, the Drilling Technician prepared the field test boring logs of the subsurface profile noting soil types and depths, standard penetration test resistances (N-values), soil and bedrock stratifications and ground water levels or the lack thereof. Following the completion of the test borings, the samples were returned to our Soil Mechanics Laboratory where they were reviewed and visually classified by the Project Geotechnical Engineer.

Representative samples from test borings were selected for laboratory tests, including moisture content tests and Atterberg limits tests. A tabulation of the moisture content and Atterberg limits tests is included in the Appendix to this report.

Final test boring logs were prepared based on the Drilling Technician's field logs and the Engineer's visual classification of the samples. Copies of these logs can be found in the Appendix along with a Soil Classification Sheet describing the terms and symbols used in their preparation.

The dashed lines on the test boring logs identify the changes between the soil and bedrock types, which were determined by interpolation between samples and should be considered approximate. Only changes that occur within samples can be precisely determined and are indicated by solid lines on the logs. The transition between soil and bedrock types may be abrupt or gradual.

5.0 GENERAL SUBSURFACE CONDITIONS

Specific subsurface conditions were identified only in limited areas along the alignment. The subsurface conditions in the explored areas are discussed on a section-by-section basis in the Conclusions and Recommendations section of this report.

The following is a discussion of the generalized subsurface conditions that are anticipated based on the observations during the engineering reconnaissance, published Geologic Maps, the test borings made along the alignment, and our general experience as Geotechnical Engineers in the Northern Kentucky Area. The general nature of the following discussion regarding the subsurface conditions in unexplored areas should be recognized, as should the possibility of encountering conditions along the alignment that vary from the generalized conditions.

A generalized profile of clays and silty clays overlying interbedded shale and limestone bedrock is expected throughout much of the alignment. Deeper deposits of glacial clays, silt and sands do exist in relatively long but isolated sections of the alignment. These glacial deposits often contain groundwater. In valleys and swales and at creek crossings, softer sediment overlying the soils or bedrock is likely to exist. Man-placed fill for road embankments, comprised of clays and shales with limestone floaters, is locally present over the native soils and bedrock along the alignment. The fill embankments are generally located on the downslope sides of the roadways and at creek and swale crossings.

Each of the test borings was performed within the pavement of the roadways and locally indicated somewhat thick pavements. Test Borings BB-1 and KL-1 encountered 6 to 8 inches of asphalt, while Test Boring BB-2 encountered 12 inches of asphalt.

The proposed water main will generally be placed within the drainage ditch on the upslope side of the roadway. The subsurface conditions along the water main alignment will generally consist of topsoil, a low-density near-surface silty clay or clay soil, then stiff to very stiff clay and silty clay above bedrock, interbedded shale and limestone. It is anticipated that some sections of the water main alignment will encounter bedrock at very shallow depths. It is also anticipated that alluvial and sediment deposits will be encountered at the creek and swale crossings.

The majority of the fill that will be encountered during the excavations for the water main has been placed during the construction of road embankments. The consistency of the fill may vary from soft to very stiff. The test borings completed along the alignment encountered medium stiff to very stiff fill.

The bedrock beneath the overburden soil is a system of Ordovician Age shale and limestone. This type of bedrock is typically classified into three zones identified by the extent of weathering of the shale portion of the bedrock. The uppermost zone is termed highly weathered shale and limestone, where the shale portion has virtually weathered to a brown silty clay or clay yet possesses horizontally aligned bedding characteristics of the bedrock system. The intermediate zone is described as weathered bedrock and is characterized by a shale component that is tougher, and generally at a lower moisture content, than the highly weathered zone above. The upper and intermediate zones have weathered from the third commonly accepted zone, the unweathered, gray, parent, interbedded shale and limestone. The limestone component of the highly weathered, weathered, and unweathered bedrock consists of horizontal beds, which are gray, crystalline, fossiliferous and hard. Highly weathered and weathered zones, locally, may or may not be present above the unweathered bedrock zone because of variable weathering and erosion conditions.

According to the United States Geological Survey (USGS) Union Quadrangle Map, the bedrock near the surface of the water main alignment is classified as one of three formations: the Bull Fork Formation, the Bellevue Tongue of the Grant Lake Limestone, and the Fairview Formation. The bedrock near the surface of the alignment for the easternmost approximately 1000 feet of the alignment along Big Bone Church Road, as well as the majority of the alignment along Brian Court is classified as the Bull Fork Formation. This Formation is usually comprised of approximately 50 to 60 percent limestone and 40 to 50 percent shale. The limestone layers are in even to irregular beds ranging in thickness from 1 to 8 inches and averaging 3 inches thick. The bedrock near the surface of the majority of the alignment along Forest View Drive is classified as the Bellevue Tongue of the Grant Lake Limestone. This Formation is predominantly limestone in thin, irregular, discontinuous beds with shale partings, and occasional regular beds of limestone.

The bedrock near the surface of the remainder of the alignment along Big Bone Church Road, the majority of the alignment along Michelle Drive, and the majority of the alignment along Kirby Lane is classified as the Fairview Formation. This Formation is usually comprised of 50 to 60 percent limestone and the remaining 40 to 50 percent shale. The limestone layers are in regular to irregular beds, averaging about 4 inches thick. The shale layers are commonly 1 to 8 inches thick, rarely more than 18 inches thick.

The bedrock was not cored as part of this project. There are exposures of bedrock in the creeks and some road cuts, giving an indication of the thickness and percentage of the shale and limestone.

All test borings were backfilled immediately after they were completed. Groundwater was first noted at depths of 7.0 and 8.5 feet in Test Borings BB-1 and BB-2, respectively. All test borings were dry at completion except Test Boring BB-1, which had water at a depth of 11.7 feet at completion. The water was typically noted within the sediment and within the bedrock. Experience has found that seepage water can occur along the layers of limestone within the bedrock, in glacial soil deposits, and at the soil/bedrock interface.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based upon our engineering reconnaissance of the water main alignment, the test borings, a visual examination of the samples, the laboratory test results, our understanding of the proposed construction, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the conclusions and make the recommendations in this report.

The conclusions and recommendations of this report have been derived by relating the general principles of the discipline of Geotechnical Engineering to the proposed construction outlined by the Project Characteristics section of this report. Because changes in surface, subsurface, climatic, and economic conditions can occur with time and location, we recommend for our mutual interest that the use of this report be restricted to this specific project.

Our understanding of the proposed design and construction is based on the documents provided to us at the time this report was prepared and which are referenced in the Project Characteristics section of this report. We recommend that our office be retained to review the final design documents, plans, and specifications to assess any impact changes, additions or revisions to these documents may have on the conclusions and recommendations of this Geotechnical Report. Any changes or modifications which are made in the field during the construction phase which alter the water main alignment or depths or other related site work should also be reviewed by our office prior to their implementation.

If conditions are encountered in the field during construction which vary from the facts of this report, we recommend that our office be contacted immediately to review the changed conditions in the field and make appropriate recommendations.

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.

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

The earthwork recommendations of this report presume that the earthwork will be monitored by an Engineering Technician under the direction of a Registered Professional Geotechnical Engineer. We recommend that the Owner contract these services directly with Thelen Associates, Inc.

Throughout the alignment, the proposed water main will cross many culverts below the roadways. At each of these culvert crossings, the water main should be installed below the existing culverts, unless the water main can be installed above the culvert while maintaining the minimum soil cover above the crown of the main and maintaining the minimum required clear distance between the water main and the culvert, considering the structural capacity of the culvert and any necessary soil cover between the culvert and the water main to provide protection from freezing.

The proposed water main installation and the terrain that exists along the alignment were reviewed on a Station-by-Station basis and are discussed individually in Sections 6.2 through 6.6 of this report. Section 6.7 contains general recommendations for placement and compaction of trench backfill.

6.2 Big Bone Church Road Water Main Alignment

6.2.1 Station 0+00 (Beginning of Alignment) to Station 47+00

A proposed 8-inch diameter ductile iron water main will connect to the existing 8-inch diameter water main immediately northwest of the intersection with Big Bone Road. The proposed 8-inch diameter water main will begin on the north side of Big Bone Church Road and continue on the north side, which is generally the upslope side of the road.

No evidence of current instability was noted in this section of the alignment. Test Boring BB-1 was completed in this section at Station 12+30, 7 Ft. Right. This test boring encountered 8 inches of asphalt overlying 6.3 feet of generally medium stiff fill, overlying 5.0 feet of medium stiff silty clay sediment, overlying 1.8 feet of wet medium stiff silty clay with limestone floaters, overlying unweathered interbedded shale and limestone bedrock.

Based on the available test boring information, we anticipate that medium stiff soils may be encountered at pipe subgrade elevation between approximately Station 10+00 and Station 14+50. We recommend that throughout this section, any soft to medium stiff soils encountered below pipe subgrade elevation be undercut to a depth of not less than two feet and replaced with compacted and tested backfill to provide a firm base for pipe support. The compacted and tested backfill could be select granular soils. We recommend that the extent of the soils to be undercut be reviewed by the Project Geotechnical Engineer or his representative.

In our opinion, after any unsuitable bearing materials are undercut and replaced, the water main can be installed at normal depths throughout this section.

6.2.2 Station 47+00 to Station 56+00

In this section, the water main begins on the upslope (north) side of the road, immediately crosses to the south side of the road, continues across a relatively steep slope below the road, crosses Kirby Lane, and remains on the downslope side of the road. No current instability was noted, however between approximately Station 50+00 and Station 52+00, the alignment is close to the erosive meander in the creek south of the alignment.

Test Boring BB-2 was completed at Station 50+30, 8 Ft. Left. This test boring encountered 12 inches of asphalt overlying 0.2 feet of stiff fill, overlying 4.8 feet of stiff to very stiff silty clay and clay with limestone floaters, overlying 1.5 feet of highly weathered shale and limestone bedrock, overlying unweathered shale and limestone bedrock.

We recommend that the water main be installed such that the top of the pipe is at least two feet below the top of the bedrock between Station 50+00 and Station 52+00. The main could be moved to below the upslope lane of the road in this section to reduce the depth to achieve the bedrock embedment. We recommend that the bedrock embedment be reviewed by the Project Geotechnical Engineer or his representative.

With the exception noted above between Station 50+00 and Station 52+00, in our opinion, the water main can be installed at normal depths throughout the remainder of this section.

6.2.3 Station 56+00 to Station 82+50 (End of Alignment)

In this section, the water main remains on the south side of the road, which is generally the downslope side of the road. The alignment crosses relatively gentle slopes above several houses. The alignment then crosses fill embankments over two drainage swales and ascends a relatively gentle slope to the end of the alignment near the intersection with Gum Branch Road.

No evidence of instability was noted throughout this section of the alignment. No test borings were completed in this section of the alignment.

We recommend that the water main be installed with at least six feet of soil cover between Station 66+00 and Station 67+00 in order to establish a more favorable relationship between the top of the pipe and the toe of the roadway fill embankment.

With the exception of the section across the fill embankment noted above, in our opinion, the water main can be installed at normal depths throughout the remainder of this section of the alignment.

6.3 Kirby Lane Water Main Alignment

6.3.1 Station 0+00 (Beginning of Alignment) to Station 17+25 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near Big Bone Church Road. The proposed 8-inch diameter water main will begin on the west side of Kirby Lane, cross the creek west of the culvert below Kirby Lane, ascend a small slope and cross to the east side of Kirby Lane. The alignment then remains on the east (upslope) side of Kirby Lane past a small culvert, crosses to the west side of the road, and remains on the west side of the road as Kirby Lane ascends a gentle slope to the end of the alignment.

Test Boring KL-1 was completed at Station 6+81, 4 Ft. Right. This test boring encountered 6 inches of asphalt overlying 9.0 feet of stiff to very stiff fill, overlying 2.5 feet of stiff silty clay, overlying highly weathered shale and limestone bedrock.

No evidence of instability was noted in this section. We recommend that the water main be gradually deepened at the beginning of this section such that the main is below creek elevation by the time it reaches the crest of the creek bank on the north side of the creek. Deepening should be used to eliminate upward-thrusting vertical bends. The main should be deep enough below the creek to allow adequate thrust restraint for the horizontal bends, and should be concrete encased for scour protection.

In our opinion, the water main can be installed at normal depths for the remainder of the alignment in this section after the creek crossing.

In our opinion, the water main can be installed at normal depths for the remainder of the alignment in this section after the creek crossing.

6.4 Forest View Drive Water Main Alignment

6.4.1 Station 0+00 (Beginning of Alignment) to Station 26+70 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Kirby Lane. The proposed 8-inch diameter water main will begin immediately northeast of the intersection, cross Kirby Lane, and remain on the north and east side of the Forest View Drive to the end of the alignment. Throughout this section the alignment crosses relatively gentle to nearly flat terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.5 Michelle Drive Water Main Alignment

6.5.1 Station 0+00 (Beginning of Alignment) to Station 15+20 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Big Bone Church Road. The proposed 8-inch diameter water main will begin immediately southwest of the intersection, and remain on the west side of the Michelle Drive to the end of the alignment. The west side of the road is the downslope side of the road for approximately 300 feet, where the alignment crosses a drainage swale, and the west side of the road becomes the upslope side of the road throughout the remainder of this section. Throughout this section the alignment crosses relatively gently sloping terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.6 Brian Court Water Main Alignment

6.6.1 Station 0+00 (Beginning of Alignment) to Station 6+65 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Michelle Drive. The proposed 8-inch diameter water main will begin immediately northwest of the intersection, and remain on the north and east side of the Brian Court to the end of the alignment. Throughout this section the alignment crosses relatively gentle terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.7 General Excavation and Backfilling Recommendations

The excavations throughout this project will encounter a variety of materials. Those materials will include roadway embankment fill, native clay and silty clay soils, glacial soils including clay, silt and sand, and interbedded shale and limestone bedrock. Experience indicates that the difficulty of completing the excavations in the bedrock usually far exceeds the difficulty of excavating in the fill materials and the native soils. The difficulty of making bedrock excavations is primarily related to the amount and thickness of the limestone layers in the bedrock as well as the degree of weathering. The Contractor should be aware of the presence of the bedrock and should be prepared for the difficulty that bedrock may present in the excavations.

The scope of this project involved subsurface explorations to define specific subsurface conditions only in critical areas, which represent a limited percentage of the total project length. Therefore, we recommend that the specifications for this project be based on unclassified excavation, not on separate cost items for soil excavation and bedrock excavation. The base bid for the project should include the cost of excavating the materials encountered within the specified water main depths, regardless of soil or bedrock characteristics.

It is difficult to shear limestone layers neatly in the sides of trench excavations. Frequently, when limestone layers are encountered at relatively shallow depths in trench excavations, the tendency is for the layers not to break even with the sides of the excavations, but rather to be pulled up in large chunks, which tend to heave and ravel the soils outside the limits of the intended trench. Where trench excavations will be made immediately adjacent to or within existing pavement with the intention of not disturbing the existing pavement beyond the trench limits, it should be anticipated that there will be some areas where there is heave and raveling due to removal of limestone layers that could damage pavement adjacent to the trench, and said pavement will have to be restored.

We expect that the excavated materials, exclusive of the thick limestone layers, can be used as backfill after the appropriate granular pipe bedding and backfill is installed. Fill materials should not include asphalt, concrete, trash, construction or demolition debris, topsoil or frozen material. Large pieces of limestone which tend to nest or retard compaction should be excluded from the backfill. Smaller pieces of limestone that can be broken up and dispersed so that they do not nest or retard compaction can be incorporated in the backfill provided that proper protection of the pipe from these pieces of limestone is provided.

The trench excavations for the project will extend a minimum of about 5 feet deep. In areas where mains are to be lowered, as discussed in the previous sections of this report, the excavations may extend up to 8 feet deep or more. The Contractor should be responsible for the stability and safety of all excavations and should exercise all necessary precautions to shore, slope or otherwise maintain stable trench excavations to protect workers, adjacent property, adjacent pavement and structures, and infrastructure. These trenches should be made and maintained in accordance with all Federal, State and Local regulations.

Normal and recommended utility construction practice is to bed and backfill pipes with granular fill to a specified height above the crown of the pipe. Compaction of trench backfill to a moist, firm, dense condition is important throughout the entire alignment of this project, because the alignment is generally beneath the existing pavement, immediately adjacent to existing pavement, near the toes of existing cut and fill slopes, or through existing roadway embankment slopes. We recommend that **all** backfill for this project be placed in shallow level layers, 6 to 8 inches in thickness, and be compacted to densities not less than 95 percent of the standard Proctor maximum dry density, ASTM D698. For soil backfill used within the road, the upper 8 inches of backfill beneath the pavement areas should be compacted to densities not less than 98 percent of the standard Proctor maximum dry density, ASTM D698. The backfill soils should be moisture-conditioned to within the range of 2 percent below to 3 percent above the optimum moisture at the time of compaction. All shale should be pulverized to a soil-like consistency and moisture conditioned the same as a soil. Where granular fill is used, it should be compacted to at least 75 percent of the maximum relative density obtained using ASTM D4253 and D4254 test methods. Density tests should be made in the backfill to document that the recommended degree of compaction is being achieved.

THELEN ASSOCIATES, INC.
1398 COX AVENUE
ERLANGER, KENTUCKY 41018-1002

CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
BOONE COUNTY, KENTUCKY
031308E

TABULATION OF LABORATORY TESTS

Boring Number	Sample Number	Depth, ft.		Moisture Content, %	Atterberg Limits, %			USCS Classification
		From	To		LL	PL	PI	
BB-1	4	7.5	9.0	26.1				
BB-2	2	2.5	4.0	18.2	40	20	20	CL
KL-1	2	2.5	4.0	21.2				
KL-1	3	5.0	6.5	23.1				

THELEN ASSOCIATES, INC.
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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING #: BB-1

PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB #: 031308E

LOCATION OF BORING: Big Bone Church Road, Station 12+30, 7 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet
824.7	SURFACE	0.0
824.0	ASPHALT (8 inches).	0.7
823.6	Mixed brown moist medium dense FILL, crushed limestone, some silty clay.	1.1
822.7	Mixed brown and greenish gray moist stiff to very stiff FILL, clay with limestone floaters.	2.0
820.2	Mixed dark gray moist medium stiff FILL, silty clay with organic material.	4.5
817.7	Mixed dark gray moist medium stiff FILL, silty clay, trace organic material.	7.0
812.7	Dark gray moist medium stiff SILTY CLAY, trace organic material (sediment).	12.0
810.9	Brown and gray wet medium stiff SILTY CLAY with limestone floaters, trace bedding planes.	13.8
810.7	Interbedded gray moist soft SHALE and hard LIMESTONE (bedrock).	14.0
	Bottom of test boring at 14.0 feet.	

SAMPLE

Cond	Blows/6"	No.	Type	Rec. Inches
D/I	17/9/7	1A 1B	DS	13
I	2/2/3	2	DS	10
I	2/2/2	3	DS	10
I	2/2/2	4	DS	18
I	2/2/3	5	DS	10
I	10/17/50	6A 6B	DS	17

Datum: MSL Hammer Wt.: 140 lbs. Hole Diameter: 5 in. Foreman: GB
Surf. Elev.: 824.7 ft. Hammer Drop: 30 in. Rock Core Dia.: ____ in. Engineer: BMS
Date Started: 12/3/04 Pipe Size: O.D. 2 in. Boring Method: CFA Date Completed: 12/3/04

SAMPLE CONDITIONS
D – DISINTEGRATED
I – INTACT
U – UNDISTURBED
L – LOST

SAMPLE TYPE
DS – DRIVEN SPLIT SPOON
PT – PRESSED SHELBY TUBE
CA – CONTINUOUS FLIGHT AUGER
RC – ROCK CORE

GROUND WATER DEPTH
FIRST NOTED 7.0 ft.
AT COMPLETION 11.7 ft.
AFTER – hrs. – ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
DC – DRIVING CASING
MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING #: BB-2

PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB #: 031308E

LOCATION OF BORING: Big Bone Church Road, Station 50+30, 8 Ft. Left

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet
754.5	SURFACE	0.0
753.5	ASPHALT (12 inches).	1.0
753.3	Mixed dark brown moist stiff FILL, silty clay with crushed limestone.	1.2
752.0	Brown moist very stiff SILTY CLAY with iron oxide stains and concretions and shale fragments.	2.5
750.0	Olive brown and brown moist very stiff SILTY CLAY with limestone floaters and shale fragments, trace bedding planes (CL).	4.5
748.5	Brown and gray moist very stiff CLAY with limestone floaters, trace iron oxide stains and bedding planes.	6.0
747.0	Interbedded brown and olive brown moist very soft very highly weathered SHALE and gray hard LIMESTONE (bedrock).	7.5
746.0	Interbedded gray moist soft SHALE and hard LIMESTONE (bedrock).	8.5
	Split spoon refusal and bottom of test boring at 8.5 feet.	

SAMPLE

Depth (feet)	Cond	Blows/6"	No.	Type	Rec. Inches
1.0–2.5	I	7/11/10	1A 1B	DS	9
2.5–4.0	I	8/22/17	2	DS	9
5.0–6.5	I	5/9/31	3A 3B	DS	15
8.0–8.5	I	50/6"	4	DS	4

Datum MSL — Hammer Wt. 140 lbs. — Hole Diameter 5 in. — Foreman GB

Surf. Elev. 754.5 ft. — Hammer Drop 30 in. — Rock Core Dia. ____ in. — Engineer BMS

Date Started 12/3/04 — Pipe Size O.D. 2 in. — Boring Method CFA — Date Completed 12/03/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED 8.5, trace ft.	HSA – HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA – CONTINUOUS FLIGHT AUGER
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING #: KL-1
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB #: 031308E
LOCATION OF BORING: Kirby Lane, Station 6+81, 4 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
770.1	SURFACE	0.5						
769.6	ASPHALT (6 inches).	1.0		I	25/17/10	1A	DS	12
		2.0				1B		
769.1	Mixed brown moist stiff FILL, silty clay with crushed limestone, trace coarse gravel.	4.5		I	4/5/7	2	DS	12
768.1	Mixed brown and gray moist very stiff FILL, silty clay with limestone fragments, trace iron oxide stains.	7.0	5	I	4/4/4	3	DS	18
765.6	Mixed brown moist very stiff FILL, silty clay, trace shale fragments.	9.5		I	15/16/14	4	DS	8
763.1	Mixed brown and dark brown miost very stiff FILL, silty clay, trace topsoil.	12.0	10	I	5/7/8	5	DS	12
760.6	Mixed dark brown moist stiff FILL, silty clay with limestone floaters, trace iron oxide stains.	15.1		I	19/50/6"	6	DS	6
758.1	Brown moist very stiff SILTY CLAY, little iron oxide stains.		15	L	50/1"	7	DS	0
755.0	Interbedded brown moist soft highly weathered SHALE and gray hard LIMESTONE (bedrock).							
	Bottom of test boring at 15.1 feet.		20					
			25					

Datum: MSL | Hammer Wt.: 140 lbs. | Hole Diameter: 5 in. | Foreman: GB
Surf. Elev.: 770.1 ft. | Hammer Drop: 30 in. | Rock Core Dia.: ____ in. | Engineer: BMS
Date Started: 12/3/04 | Pipe Size: O.D. 2 in. | Boring Method: CFA | Date Completed: 12/3/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

CONSULTING SERVICES

BOONE COUNTY RURAL WATER PROJECT

PHASE II

CONTRACT 2B/2D

BOONE COUNTY, KENTUCKY

Prepared for: **Tetra Tech, Inc.**

Thelen Project No.: **031308E**

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January 5, 2005

Tetra Tech, Inc.
11156 Canal Road
Suite A
Cincinnati, Ohio 45241

Attn: Mr. Dennis Huber, P.E.

Re: Consulting Services
Boone County Rural Water Project
Phase II
Contract 2B/2D
Boone County, Kentucky

Ladies and Gentlemen:

Summarized in this report are the results of the Geotechnical Consulting Services provided to Tetra Tech, Inc. (Tetra Tech) by Thelen Associates, Inc. (Thelen) for the proposed Boone County Rural Water Project, Phase II, Contract 2B/2D in Boone County, Kentucky. The work was performed in accordance with our Proposal-Agreement K24209, and was authorized by Mr. Dennis Huber of Tetra Tech by signature of Tetra Tech Work Order No. 2 on September 29, 2004.

This report summarizes the results of the engineering reconnaissance and test borings performed along East Bend Road, Emerald Drive, Possum Path Road, Kirby Drive, Wolfe Road and Locust Grove Road, and a review of the Project Drawings prepared by Tetra Tech dated November, 2004 and received by Thelen on November 9, 2004. We have summarized our conclusions and recommendations for the location and depth of the water main based upon the reconnaissance, the results of the test borings and our analyses.

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

We appreciate the opportunity to provide our consulting services for the Boone County Rural Water Project, Phase II, Contract 2B. Should you have any questions concerning the information, conclusions or recommendations contained in this report, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

Bryan M. Scott, E.I.
Staff Geotechnical Engineer

Theodore W. Vogelpohl, P.E.
Chief Geotechnical Engineer

STATE OF KENTUCKY
THEODORE W. VOGELPOHL
14509
LICENSED
PROFESSIONAL ENGINEER

BMS/TWV:df
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Copies submitted: 2 – Client

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January 5, 2005

CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
PHASE II
CONTRACT 2B
BOONE COUNTY, KENTUCKY

1.0 INTRODUCTION

Presented in this report are the results of the geotechnical engineering reconnaissance and test boring exploration for the proposed 8-inch and 12-inch diameter water mains to be installed along East Bend Road, Emerald Drive, Possum Path Road, Kirby Drive, Wolfe Road and Locust Grove Road in Boone County, Kentucky. The scope of our geotechnical services included a review of the existing topography and other land features along the alignment proposed on the plans prepared by Tetra Tech, Inc. (Tetra Tech). Areas of apparent or potential instability along the selected alignment were identified during the reconnaissance, and only these areas were explored by advancing test borings to determine the subsurface conditions. The conclusions and recommendations contained in this report regarding the water main alignment and depths are based on the engineering reconnaissance and the test boring information.

2.0 PROJECT CHARACTERISTICS

The project drawings referred to in this report are titled "Boone County Rural Water Project, Phase II, Contract 2B – KY 338, East Bend Road", dated November, 2004, marked as "Preliminary" and received by Thelen Associates, Inc. (Thelen) on November 9, 2004. The Phase II, Contract 2B portion of the Boone County Rural Water Project will consist of roughly 21310 lineal feet of 12-inch diameter pipe along East Bend Road, 2625 lineal feet of 8-inch diameter pipe along Emerald Drive, 4170 lineal feet of 8-inch diameter pipe along Possum Path Road, 2685 lineal feet of 8-inch diameter pipe along Kirby Drive, 1730 lineal feet of 8-inch diameter pipe along Wolfe Road, and 5410 lineal feet of 8-inch diameter pipe along Locust Grove Road. All of the water main pipe is to be Class 50 ductile iron pipe.

The proposed water main along East Bend Road will connect to an existing 16-inch diameter water main approximately 550 feet west of the Edgewood Drive intersection. The proposed water main along each of the other roads will connect to the proposed water main along East Bend Road. The change in surface elevation along the water main alignment for East Bend Road is 104 feet, with the lowest point located west if Howe Road (El. 788) and the highest point located near Blackbird Lane (El. 892). The change in surface elevation along the water main alignment for Emerald Drive is 32 feet, with the lowest point located near the end of the alignment (El. 828) and the highest point located near the northward bend in the road (El. 860). The change in surface elevation along the water main alignment for Possum Path Road is 126 feet, with the lowest point located near the intersection of Kirby Drive (El. 744) and the highest point located near the beginning of the alignment (El. 870). The change in surface elevation along the water main alignment for Kirby Drive is 88 feet, with the lowest point located near the intersection of Possum Path Road (El. 744) and the highest point located near the end of the alignment (El. 832). The change in surface elevation along the water main alignment for Wolfe Road is 28 feet, with the lowest point located near the beginning of the alignment (El. 830) and the highest point located near the end of the alignment (El. 858). The change in surface elevation along the water main alignment for Locust Grove Road is 38 feet, with the lowest point located near the beginning of the alignment (El. 830) and the highest point located near Station 19+00 (El. 868). The water mains will follow the roadways, typically several feet off the upslope edge of the pavement. In general, the roadways have been constructed on relatively gentle to moderately sloping valley walls, ridgetops or fill embankments.

The depth of the new water main is expected to vary along the alignment due to constraints caused by local geology, existing utilities and other construction–related features. It is our understanding that the minimum cover at the crown of the pipe will be 4 feet. Throughout this report, "normal depth" refers to at least 4 feet of soil cover above the top of the pipe.

Specific descriptions of the proposed water main installation and our recommendations are provided on a section-by-section basis in the Conclusions and Recommendations Section of this report. This report only addresses the geotechnical issues for the water main project.

3.0 ENGINEERING RECONNAISSANCE

The reconnaissance of the alignment was made by the Engineer, during which evidence of soil and bedrock exposures, slope movement, pavement subsidence, steep existing slopes, fill embankments, erosion, etc., were noted. The project drawings by Tetra Tech were used as a reference during the reconnaissance.

The alignment begins on East Bend Road approximately 540 feet west of the intersection of Edgewood Drive. The alignment passes Emerald Drive and remains on the generally upslope side of the road as it ascends a gentle slope. The alignment then follows East Bend Road along a ridgetop and crosses to the east side of the road near Blackbird Lane. Beyond Blackbird Lane the alignment remains on the ridgetop until descending and ascending gentle slopes near King Oak Drive. The alignment then crosses to the west side of the road near Snow Road and continues on the west side of the road beyond Possum Path Road. The alignment remains on the west side of the road as East Bend Road crosses relatively gentle ridgetop terrain beyond Beech Road. The alignment continues on ridgetop terrain, crossing a valley immediately beyond Greene Road and making a right turn at Howe Road. Beyond Howe Road, the alignment crosses another valley, then regains the ridgetop and continues on gentle terrain beyond Wolfe Road to the end of this section of the alignment at Locust Grove Road.

Another section of the alignment follows Emerald Drive, beginning at its intersection with East Bend Road. From the intersection the alignment follows the west side of Emerald Drive across relatively gentle terrain to the end of this section of the alignment at the end of Emerald Drive.

The alignment also continues on Possum Path Road from the intersection with East Bend Road. This section of the alignment begins on the northeast corner of the intersection and continues along the north side of Possum Path Road as the road descends a relatively gentle slope, crosses a culvert, and ascends a relatively gentle slope. The alignment then descends another slope, crosses another culvert, ascends a slope briefly, and continues to the bottom of the valley. The alignment then crosses a large culvert near Kirby Drive and continues on the relatively gentle valley bottom to the end of this section of the alignment approximately 700 feet beyond Kirby Drive.

Another section of the alignment follows Kirby Drive from its intersection with Possum Path Road. The alignment follows the west side of Kirby Drive across relatively gentle terrain, crossing an embankment dam, and ascending a slope to follow the road as it follows the valley wall to the end of Kirby Drive

The alignment also continues on Wolfe Road from the intersection with East Bend Road. The alignment follows the west side of the road as Wolfe Road crosses relatively gentle terrain. This section of the alignment terminates approximately 400 feet beyond the westerly turn in the road.

The alignment also continues on Locust Grove Road from the intersection with East Bend Road. The alignment follows the north and east side of the road until the road makes a southerly bend. At this point the alignment crosses to the west side of the road, where it remains until the end of the alignment. The alignment along Locust Grove Road crosses relatively gentle ridgetop terrain. This section of the alignment terminates at the end of Locust Grove Road.

In general, the alignment crosses relatively gentle to moderately sloping terrain throughout the alignment. The proposed alignment does cross several areas of potential instability along the course of the alignment. Specific details of these areas are described on a section-by-section basis in the Conclusions and Recommendations Section of this report.

4.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Four test borings were made at locations selected by Thelen to explore the subsurface conditions along the alignment. The test borings are labeled PP-1 and PP-2 along Possum Path Road, and KD-1 and KD-2 along Kirby Drive. No test borings were completed along East Bend Road, Emerald Drive, Wolfe Road or Locust Grove Road. The test boring locations and ground surface elevations were surveyed by Tetra Tech. The ground surface elevation at each test boring location was referenced to Mean Sea Level (MSL). The test boring locations are referenced to the centerline of the road by station and offset for this report. The locations are noted at the tops of the test boring logs and are summarized in Table 1, Test Boring Locations.

TABLE 1

Test Boring Locations

Test Boring No.	Road	Station	Offset
PP-1	Possum Path Road	6+75	6' R
PP-2	Possum Path Road	31+65	4' R
KD-1	Kirby Drive	8+15	8' R
KD-2	Kirby Drive	21+07	6' R

The test borings were made with a truck-mounted drill rig advancing continuous flight augers. Two-inch outside diameter (O.D.) driven split-spoon samples were obtained at pre-selected intervals according to the procedures outlined in ASTM D1586. Recovered split-spoon samples were placed in glass jars to retain the samples at their in-situ moisture contents. The sample jars were appropriately marked in the field for proper identification.

Concurrent with the drilling operation, the Drilling Technician prepared the field test boring logs of the subsurface profile noting soil types and depths, standard penetration test resistances (N-values), soil and bedrock stratifications and ground water levels or the lack thereof. Following the completion of the test borings, the samples were returned to our Soil Mechanics Laboratory where they were reviewed and visually classified by the Project Geotechnical Engineer.

Representative samples from the test borings were selected for laboratory tests, including moisture content tests and Atterberg limits tests. A tabulation of the moisture content and Atterberg limits tests is included in the Appendix to this report.

Final test boring logs were prepared based on the Drilling Technician's field logs and the Engineer's visual classification of the samples. Copies of these logs can be found in the Appendix along with a Soil Classification Sheet describing the terms and symbols used in their preparation.

The dashed lines on the test boring logs identify the changes between the soil and bedrock types, which were determined by interpolation between samples and should be considered approximate. Only changes that occur within samples can be precisely determined and are indicated by solid lines on the logs. The transition between soil and bedrock types may be abrupt or gradual.

5.0 GENERAL SUBSURFACE CONDITIONS

Specific subsurface conditions were identified only in limited areas along the alignment. The subsurface conditions in the explored areas are discussed on a section-by-section basis in the Conclusions and Recommendations section of this report.

The following is a discussion of the generalized subsurface conditions that are anticipated based on the observations during the engineering reconnaissance, published Geologic Maps, the test borings made along the alignment, and our general experience as Geotechnical Engineers in the Northern Kentucky Area. The general nature of the following discussion regarding the subsurface conditions in unexplored areas should be recognized, as should the possibility of encountering conditions along the alignment that vary from the generalized conditions.

A generalized profile of clays and silty clays overlying interbedded shale and limestone bedrock is expected throughout much of the alignment. Deeper deposits of glacial clays, silts and sands do exist in relative long but isolated sections of the alignment. These glacial deposits often contain groundwater. In valleys and swales and at creek crossings, softer saturated sediment overlying the soils or bedrock is likely to exist. Man-placed fill for road embankments, comprised of clays and shales with limestone floaters, is locally present over the native soils and bedrock along the alignment. The fill embankments are generally located on the downslope sides of the roadways and at creek and swale crossings.

Each of the test borings was performed within the pavement of the roadways. Test Borings PP-1 and PP-2 each encountered 6 inches of asphalt, while Test Borings KD-1 and KD-2 encountered 2 inches of asphalt overlying 5 to 6 inches of concrete.

The proposed water main will generally be placed within the drainage ditch on the upslope side of the roadway. The subsurface conditions along the water main alignment will generally consist of topsoil, a low-density near-surface silty clay or clay soil, then stiff to very stiff clay and silty clay above bedrock, interbedded shale and limestone. It is anticipated that some sections of the water main alignment will encounter bedrock at very shallow depths. It is also anticipated that alluvial and sediment deposits will be encountered at the creek and swale crossings.

The majority of the fill that will be encountered during the excavations for the water main has been placed during the construction of road embankments. Fill will also be encountered at the embankment dam along Kirby Drive. The consistency of the fill may vary from soft to very stiff. The test borings completed along the alignment encountered medium stiff to stiff fill.

The bedrock beneath the overburden soil is a system of Ordovician Age shale and limestone. This type of bedrock is typically classified into three zones identified by the extent of weathering of the shale portion of the bedrock. The uppermost zone is termed highly weathered shale and limestone, where the shale portion has virtually weathered to a brown silty clay or clay yet possesses horizontally aligned bedding characteristics of the bedrock system. The intermediate zone is described as weathered bedrock and is characterized by a shale component that is tougher, and generally at a lower moisture content, than the highly weathered zone above. The upper and intermediate zones have weathered from the third commonly accepted zone, the unweathered, gray, parent, interbedded shale and limestone. The limestone component of the highly weathered, weathered, and unweathered bedrock consists of horizontal beds, which are gray, crystalline, fossiliferous and hard. Highly weathered and weathered zones, locally, may or may not be present above the unweathered bedrock zone because of variable weathering and erosion conditions.

According to the United States Geological Survey (USGS) Burlington, Rising Sun and Union Quadrangle Maps, the bedrock near the surface of the water main alignment is classified as one of three formations: the Bull Fork Formation, the Bellevue Tongue of the Grant Lake Limestone, and the Fairview Formation. The bedrock near the surface of the alignment along East Bend Road, Emerald Drive, Wolfe Road, Locust Grove Road and the areas of Possum Path Road above approximately El. 800 is classified as the Bull Fork Formation. This Formation is usually comprised of approximately 50 percent limestone and 50 percent shale. The limestone layers are in even to irregular beds ranging in thickness from 1 to 6 inches and averaging 3 inches thick. The bedrock near the surface of the alignment along Possum Path Road between approximately El. 780 and El. 800 is classified as the Bellevue Tongue of the Grant Lake Limestone. This Formation is predominantly limestone in thin, irregular, discontinuous beds with shale partings.

The bedrock near the surface of the alignment along Possum Path Road below approximately El. 780, and along Kirby Drive is classified as the Fairview Formation. This Formation is usually comprised of approximately 50 percent limestone and 50 percent shale. The limestone layers are in even to irregular thin to medium beds, averaging about 4 inches thick. The shale layers are commonly 1 to 8 inches thick, rarely more than 18 inches thick.

The bedrock was not cored as part of this project. There are exposures of bedrock in the creeks and some road cuts, giving an indication of the thickness and percentage of the shale and limestone.

All test borings were backfilled immediately after they were completed. Groundwater was first noted at a depth of 17.0 feet in Test Boring KD-1. All test borings were dry at completion except Test Boring KD-1, which had water at a depth of 16.4 feet at completion. The water was noted at the interface of the fill placed for the embankment dam and the native soil. Experience has found that seepage water can also occur in glacial soil deposits, at the soil/bedrock interface and along the layers of limestone within the bedrock.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based upon our engineering reconnaissance of the water main alignment, the test borings, a visual examination of the samples, the laboratory test results, our understanding of the proposed construction, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the conclusions and make the recommendations in this report.

The conclusions and recommendations of this report have been derived by relating the general principles of the discipline of Geotechnical Engineering to the proposed construction outlined by the Project Characteristics section of this report. Because changes in surface, subsurface, climatic, and economic conditions can occur with time and location, we recommend for our mutual interest that the use of this report be restricted to this specific project.

Our understanding of the proposed design and construction is based on the documents provided to us at the time this report was prepared and which are referenced in the Project Characteristics section of this report. We recommend that our office be retained to review the final design documents, plans, and specifications to assess any impact changes, additions or revisions to these documents may have on the conclusions and recommendations of this Geotechnical Report. Any changes or modifications which are made in the field during the construction phase which alter the water main alignment or depths or other related site work should also be reviewed by our office prior to their implementation.

If conditions are encountered in the field during construction which vary from the facts of this report, we recommend that our office be contacted immediately to review the changed conditions in the field and make appropriate recommendations.

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.

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

The earthwork recommendations of this report presume that the earthwork will be monitored by an Engineering Technician under the direction of a Registered Professional Geotechnical Engineer. We recommend that the Owner contract these services directly with Thelen Associates, Inc.

Throughout the alignment, the proposed water main will cross many culverts below the roadways. At each of these culvert crossings, the water main should be installed below the existing culverts, unless the water main can be installed above the culvert while maintaining the minimum soil cover above the crown of the main, and maintaining the minimum required clear distance between the water main and the culvert, considering the structural capacity of the culvert and any necessary soil cover between the culvert and the water main to provide protection from freezing.

The proposed water main installation and the terrain that exists along the alignment were reviewed on a Station-by-Station basis and are discussed individually in Sections 6.2, through 6.7 of this report. Section 6.8 contains general recommendations for placement and compaction of trench backfill.

6.2 East Bend Road Water Main Alignment

6.2.1 Station 5+40 (Beginning of Alignment) to Station 142+50

A proposed 12-inch diameter ductile iron water main will connect to the existing 16-inch diameter water main approximately 540 feet west of the intersection of East Bend Road and Edgewood Drive. The proposed 12-inch diameter water main will begin on the north and west (right) side of East Bend Road, and follow the north and west side of East Bend Road past Emerald Drive, cross to the east (left) side of the road north of Blackbird Lane, and continue on the east side of the road past Blackbird Lane and King Oak Drive to the northeast corner of the intersection with Snow Road. The alignment will then cross to the west side of East Bend Road and cross Possum Path Road. Remaining on the west side of East Bend Road, the alignment will cross Beech Road and then Greene Road as East Bend Road turns to the southeast, and then will follow the south and west side of East Bend Road to the end of this section. Throughout this section, the alignment crosses relatively gentle to nearly flat terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.2.2 Station 142+50 to Station 197+00

In this section, the alignment continues on the west side of East Bend Road. Near the beginning of this section, the alignment passes between the road and a barn that is close to the road. The alignment then continues on the west side of the road to Howe Road, where East Bend Road turns to the west. The alignment then continues to follow the right hand side of the road, which becomes the north side of the road. The alignment proceeds down a relatively gentle slope, passes around a large culvert, ascends a relatively gentle slope, and continues on the north side of the road until East Bend Road makes a slight turn to the southwest. Near this bend, the alignment crosses to the southeast side of the road and continues to Wolfe Road at the end of this section. Throughout this section, the alignment crosses relatively gentle to nearly flat terrain. No test borings were completed in this section of the alignment.

Given the short distance between the barn near the beginning of this section and the road, consideration should be given to deflecting the main to within the pavement between Station 142+85 and Station 143+50 in order to reduce the possibility of damaging the barn foundations. If this is undesirable, the Contractor should be prepared to take additional measures in order to ensure that the barn foundations are not damaged during construction, or as a result of the installation of the water main.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.2.3 Station 197+00 to Station 218+50 (End of Alignment)

In this section, the alignment continues on the southeast side of East Bend Road, from the intersection with Wolfe Road. The alignment crosses to the northwest side of the road near Station 205+50, and continues on the northwest side of the road to the end of East Bend Road, then turns right to follow Locust Grove Road for approximately 100 feet, where it terminates at a plug at the end of the alignment. Throughout this section, the alignment crosses relatively gentle to nearly flat terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.3 Emerald Drive Water Main Alignment

6.3.1 Station 0+00 (Beginning of Alignment) to Station 26+25 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near East Bend Road. The proposed 8-inch diameter water main will begin on the northwest corner of the intersection, and follow the south and west side of Emerald Drive as it crosses relatively gentle terrain. This section of the alignment terminates at the end of Emerald Drive. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.4 Possum Path Road Water Main Alignment

6.4.1 Station 0+00 (Beginning of Alignment) to Station 7+50

A proposed 8-inch diameter ductile iron water main will connect to the proposed 12-inch diameter water main near the intersection with East Bend Road. The proposed 8-inch diameter water main will begin near the northwest corner of the intersection with East Bend Road. The alignment follows the north side of the Possum Path Road and descends a moderately steep slope, follows the steep fill embankment downslope of the road, and crosses a drainage swale near the end of this section of the alignment.

Test Boring PP-1 was completed in this section at Station 6+75, 6 Ft. Right. This test boring encountered 6 inches of asphalt overlying 6.5 feet of stiff to medium stiff silty clay fill, overlying 2.5 feet of stiff silty clay sediment, overlying medium stiff silty clay sediment.

Based on the available test boring information and the existing topography, there is, in our opinion, potential instability along the steep fill embankment on the downslope side of the road. We recommend that the main between Station 4+25 and Station 5+75 be deepened such that the top of the main is at least six feet below the existing ground surface.

Based on the available test boring information, we anticipate that soft to medium stiff soils may be encountered at pipe subgrade elevation between approximately Station 4+50 and Station 7+50. We recommend that throughout this section, any soft to medium stiff soils encountered below pipe subgrade elevation be undercut to a depth of not less than two feet and replaced with compacted and tested backfill to provide a firm base for pipe support. The compacted and tested backfill could be select granular soils. We recommend that the extent of the soils to be undercut be reviewed and recommended by the Project Geotechnical Engineer or his representative.

With the exception noted above for the fill embankment, no evidence of instability was noted in the remainder of this section of the alignment, and in our opinion, after any unsuitable bearing materials are undercut and replaced, the water main can be installed at normal depths throughout the remainder of this section.

6.4.2 Station 7+50 to Station 28+45

In this section, the alignment remains on the north side of Possum Path Road and ascends a moderately steep slope, and then crosses two fill embankments in drainage swales near Station 13+50 and Station 17+00. The alignment then descends a moderately steep slope on the upslope side of the road, and reaches the valley bottom near Station 25+00. The alignment then follows the north side of the road along the valley bottom to the end of this section of the alignment. No test borings were completed in this section of the alignment.

Instability was noted on the downslope (south) side of the fill embankments (side opposite the water main) near Stations 13+50 and 17+00. As the water main crosses these two fill embankments, we recommend that the water main be deepened in order to cross below the existing culverts with the required minimum clear distance between the water main and the culverts. We recommend that this depth be maintained for at least 30 feet on either side of each of the culverts. This recommendation may require depths greater than normal depth.

With the exception noted above for the fill embankments, no evidence of instability was noted in the remainder of this section of the alignment, and in our opinion, the water main can be installed at normal depths throughout the remainder of this section.

6.4.3 Station 28+45 to Station 34+35

In this section, the alignment begins on the north side of the road. Two potential alignments are shown on the plans. Option A continues on the north side of the road, crossing a 54-inch diameter culvert, and continues to the end of this section. Option B crosses to the south side of the road, passes around the culvert through a series of bends, and follows the south side of the road to the end of this section.

Test Boring PP-2 was completed in this section at Station 31+65, 4 Ft. Right. This test boring encountered 6 inches of asphalt overlying 2.0 feet of very stiff silty clay, overlying highly weathered shale and limestone bedrock.

For Option A, we recommend that the main be installed below the 54-inch diameter culvert, and be installed with at least 6 feet of soil cover or with a bedrock embedment of two feet from top of main to top of bedrock throughout this section. Based on Test Boring PP-2, bedrock is expected at relatively shallow depths.

For Option B, we recommend that the main be installed with a bedrock embedment of at least one foot from top of main to top of bedrock. We also recommend that any bends that will thrust toward the culvert be installed deep enough that thrust blocks are resisted by bedrock below culvert invert level.

6.4.4 Station 34+35 to Station 41+70 (End of Alignment)

In this section, the alignment begins on the north side of the intersection with Kirby Drive and continues west along the north side of Possum Path Road, crossing the creek and ascending a relatively gentle slope to the end of Possum Path Road. Throughout this section the alignment crosses relatively gentle terrain and no test borings were completed in this section of the alignment.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.5 Kirby Drive Water Main Alignment

6.5.1 Station 0+00 (Beginning of Alignment) to Station 26+85 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Possum Path Road. The proposed 8-inch diameter water main will begin immediately northwest of the intersection, and remain on the west side of the Kirby Drive to the end of the alignment. The alignment crosses an embankment dam near Station 9+00 and ascends a relatively gentle slope as Kirby Drive follows the valley wall. Throughout this section the alignment crosses relatively gently sloping terrain.

Two test borings were completed along this section of the alignment. Test Boring KD-1 was completed at Station 8+15, 8 Ft. Right. This test boring encountered 2 inches of asphalt overlying 5 inches of concrete, overlying 6.4 feet of stiff silty clay fill with limestone floaters, overlying 7.5 feet of medium stiff to stiff silty clay fill with traces of wood and organic matter, overlying 2.5 feet of stiff silty clay fill, overlying very stiff silty clay with limestone floaters. Test Boring KD-2 was completed at Station 21+07, 6 Ft. Right. This test boring encountered 2 inches of asphalt overlying 6 inches of concrete, overlying very stiff clay and silty clay. The test boring was terminated at a depth of 11.5 feet.

No current evidence of instability in the dam embankment was noted, but based on the available test boring information, there is, in our opinion, potential instability at the location of the embankment dam. The risk of the potential instability cannot be quantified without a complete analysis of the embankment dam, including a seepage analysis and overflow analysis. If Boone County Water is willing to accept some unknown risk associated with the dam, then the main can be installed along the edge of the road at the crest of the dam. If such risk is unacceptable to Boone County Water, then we recommend that the alignment be moved from the embankment dam to a location below the toe of the embankment dam. We recommend that the alignment be revised to descend the natural slope to the west beginning at Station 6+50 to a point where the revised alignment is at least 15 feet beyond the toe of the embankment dam. At this point we recommend that the alignment turn to the south and cross below the toe of embankment dam, remaining at least 15 feet beyond the toe of the embankment dam. Once beyond the toe of the dam, we recommend that the alignment turn to the east and ascend the natural slope of the valley wall and rejoin the proposed alignment near Station 10+50.

With the exception noted above, no evidence of instability was noted in the remainder of this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.6 Wolfe Road Water Main Alignment

6.6.1 Station 0+00 (Beginning of Alignment) to Station 17+30 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 12-inch diameter water main near the intersection with East Bend Road. The proposed 8-inch diameter water main will begin immediately south of the intersection, cross East Bend Road, and remain on the west side of the Wolfe Road to the end of the alignment. Throughout this section the alignment crosses relatively gentle terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.7 Locust Grove Road Water Main Alignment

6.7.1 Station 0+00 (Beginning of Alignment) to Station 54+10 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 12-inch diameter water main near the intersection with East Bend Road. The proposed 8-inch diameter water main will begin immediately north of the intersection, and remain on the north and east side of the Locust Grove Road to the southerly bend in the road. At this point the alignment crosses to the west side of the road and remains on the west side of the road to the end of the alignment. Throughout this section the alignment crosses relatively gentle ridgetop terrain. The alignment crosses several small fill embankments throughout this section. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.8 General Excavation and Backfilling Recommendations

The excavations throughout this project will encounter a variety of materials. Those materials will include roadway embankment fill, native clay and silty clay soils, glacial soils including clay, silt and sand, and interbedded shale and limestone bedrock. Experience indicates that the difficulty of completing the excavations in the bedrock usually far exceeds the difficulty of excavating in the fill materials and the native soils. The difficulty of making bedrock excavations is primarily related to the amount and thickness of the limestone layers in the bedrock as well as the degree of weathering. The Contractor should be aware of the presence of the bedrock and should be prepared for the difficulty that bedrock may present in the excavations.

The scope of this project involved subsurface explorations to define specific subsurface conditions only in critical areas, which represent a limited percentage of the total project length. Therefore, we recommend that the specifications for this project be based on unclassified excavation, not on separate cost items for soil excavation and bedrock excavation. The base bid for the project should include the cost of excavating the materials encountered within the specified water main depths, regardless of soil or bedrock characteristics.

It is difficult to shear limestone layers neatly in the sides of trench excavations. Frequently, when limestone layers are encountered at relatively shallow depths in trench excavations, the tendency is for the layers not to break even with the sides of the excavations, but rather to be pulled up in large chunks, which tend to heave and ravel the soils outside the limits of the intended trench. Where trench excavations will be made immediately adjacent to or within existing pavement with the intention of not disturbing the existing pavement beyond the trench limits, it should be anticipated that there will be some areas where there is heave and raveling due to removal of limestone layers that could damage pavement adjacent to the trench, and said pavement will have to be restored.

We expect that the excavated materials, exclusive of the thick limestone layers, can be used as backfill after the appropriate granular pipe bedding and backfill is installed. Fill materials should not include asphalt, concrete, trash, construction or demolition debris, topsoil or frozen material. Large pieces of limestone which tend to nest or retard compaction should be excluded from the backfill. Smaller pieces of limestone that can be broken up and dispersed so that they do not nest or retard compaction can be incorporated in the backfill provided that proper protection of the pipe from these pieces of limestone is provided.

The trench excavations for the project will extend a minimum of about 5 feet deep. In areas where mains are to be lowered, as discussed in the previous sections of this report, the excavations may extend up to 8 feet deep or more. The Contractor should be responsible for the stability and safety of all excavations and should exercise all necessary precautions to shore, slope or otherwise maintain stable trench excavations to protect workers, adjacent property, adjacent pavement and structures, and infrastructure. These trenches should be made and maintained in accordance with all Federal, State and Local regulations.

Normal and recommended utility construction practice is to bed and backfill pipes with granular fill to a specified height above the crown of the pipe. Compaction of trench backfill to a moist, firm, dense condition is important throughout the entire alignment of this project, because the alignment is generally beneath the existing pavement, immediately adjacent to existing pavement, near the toes of existing cut and fill slopes, or through existing roadway embankment slopes. We recommend that **all** backfill for this project be placed in shallow level layers, 6 to 8 inches in thickness, and be compacted to densities not less than 95 percent of the standard Proctor maximum dry density, ASTM D698. For soil backfill used within the road, the upper 8 inches of backfill beneath the pavement areas should be compacted to densities not less than 98 percent of the standard Proctor maximum dry density, ASTM D698. The backfill soils should be moisture-conditioned to within the range of 2 percent below to 3 percent above the optimum moisture at the time of compaction. All shale should be pulverized to a soil-like consistency and moisture conditioned the same as a soil. Where granular fill is used, it should be compacted to at least 75 percent of the maximum relative density obtained using ASTM D4253 and D4254 test methods. Density tests should be made in the backfill to document that the recommended degree of compaction is being achieved.

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CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
BOONE COUNTY, KENTUCKY
031308E

TABULATION OF LABORATORY TESTS

Boring Number	Sample Number	Depth, ft.		Moisture Content, %	Atterberg Limits, %			USCS Classification
		From	To		LL	PL	PI	
PP-1	4	7.5	9.0	27.1				
PP-2	3	5.0	6.5	16.2				
KD-1	3	5.0	6.5	18.3				
KD-1	4	7.5	9.0	31.0				
KD-1	8	17.5	19.0	23.2				
KD-2	2	2.5	4.0	19.9	56	20	36	CH

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # KD-1
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Kirby Drive, Station 8+15, 8 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	Cond	Blows/6"	No.	Type	Rec. Inches
777.4	SURFACE	0.2					
777.2	ASPHALT (2 inches).	0.6	I	10/12/12	1	DS	12
776.8	CONCRETE (5 inches).	4.5	I	3/2/2	2	DS	18
772.9	Mixed brown and greenish gray, some dark brown moist stiff FILL, silty clay.	7.0	I	4/6/4	3	DS	18
770.4	Mixed gray, some dark brown moist stiff FILL, silty clay with limestone floaters, trace shale fragments.	9.5	I	3/2/2	4	DS	18
767.9	Mixed gray, some dark brown very moist medium stiff FILL, silty clay with limestone floaters, trace shale fragments.	12.0	I	3/4/3	5	DS	18
765.4	Mixed brown moist medium stiff to stiff FILL, silty clay, trace wood and organic matter.	14.5	I	4/5/16	6	DS	18
762.9	Mixed dark brown, some brown moist stiff to medium stiff FILL, silty clay and topsoil, little organic material, trace wood.	17.0	I	3/6/12	7	DS	18
760.4	Mixed brown and dark brown moist stiff FILL, silty clay, little wood and organic material.	18.4	I	11/50/5"	8	DS	9
759.0	Olive brown, trace gray moist very stiff SILTY CLAY with limestone floaters, trace hairlike roots.						
	Bottom of test boring at 18.4 feet.						

Depth scale (feet): 5, 10, 15, 20, 25

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 777.4 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/3/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/3/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED 17.0 ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION 16.4 ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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LOG OF TEST BORING

BORING # KD-2

CLIENT: Tetra Tech, Inc.

PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E

LOCATION OF BORING: Kirby Drive, Station 21+07, 6 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
831.0	SURFACE	0.2						
830.8	ASPHALT (2 inches).	0.7		I	2/4/6	1	DS	18
830.3	CONCRETE (6 inches).	2.5		I	6/9/13	2	DS	12
828.5	Brown moist very stiff CLAY, trace iron oxide stains and hairlike roots.		5	I	7/9/12	3	DS	18
824.0	Brown, trace gray moist very stiff CLAY, trace fine sand and iron oxide concretions (CH).	7.0		I	6/10/10	4	DS	18
821.5	Brown moist very stiff sandy SILTY CLAY, trace fine gravel.	9.5	10	I	14/10/15	5	DS	18
819.5	Brown moist very stiff SILTY CLAY, some fine to coarse sand.	11.5						
	Bottom of test boring at 11.5 feet.		15					
			20					
			25					

Datum MSL — Hammer Wt. 140 lbs. — Hole Diameter 5 in. — Foreman BR

Surf. Elev. 831.0 ft. — Hammer Drop 30 in. — Rock Core Dia. ___ in. — Engineer BMS

Date Started 12/3/04 — Pipe Size O.D. 2 in. — Boring Method CFA — Date Completed 12/3/04

SAMPLE CONDITIONS
D – DISINTEGRATED
I – INTACT
U – UNDISTURBED
L – LOST

SAMPLE TYPE
DS – DRIVEN SPLIT SPOON
PT – PRESSED SHELBY TUBE
CA – CONTINUOUS FLIGHT AUGER
RC – ROCK CORE

GROUND WATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER – hrs. – ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
DC – DRIVING CASING
MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # PP-1
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Possum Path Road, Station 6+75, 6 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	Cond	Blows/6"	No.	Type	Rec. Inches
831.5	SURFACE	0.0					
831.0	ASPHALT (6 inches).	0.5	I	3/3/4	1	DS	18
829.5	Mixed greenish gray moist stiff FILL, silty clay with limestone floaters.	2.0	I	3/3/3	2	DS	18
827.0	Mixed dark brown moist medium stiff FILL, silty clay with limestone floaters, trace shale fragments.	4.5	I	4/3/4	3	DS	18
824.5	Mixed dark brown and greenish gray moist stiff FILL, silty clay with limestone floaters, trace organic material.	7.0	I	3/5/4	4	DS	18
822.0	Dark brown moist stiff SILTY CLAY, trace fine gravel and organic material (sediment).	9.5	I	3/2/3	5	DS	18
820.0	Dark brown moist medium stiff SILTY CLAY (sediment).	11.5					
	Bottom of test boring at 11.5 feet.						

Depth scale (feet): 5, 10, 15, 20, 25

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 831.5 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/3/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/3/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER -- hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # PP-2
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Possum Path Road, Station 31+65, 4 Ft. Right

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
764.6	SURFACE	0.0						
764.1	ASPHALT (6 inches).	0.5		I	7/7/9	1	DS	18
762.1	Brown moist very stiff SILTY CLAY, trace hairlike roots.	2.5		I	13/50/4"	2	DS	3
758.1	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	6.5	5	I	18/31/50	3	DS	14
	Bottom of test boring at 6.5 feet.		10					
			15					
			20					
			25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 764.6 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/3/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/3/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Density	
Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

CONSULTING SERVICES

BOONE COUNTY RURAL WATER PROJECT

PHASE II

CONTRACT 2C

BOONE COUNTY, KENTUCKY

Prepared for: **Tetra Tech, Inc.**

Thelen Project No.: **031308E**

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers

1398 Cox Avenue / Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408
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January 5, 2005

Tetra Tech, Inc.
11156 Canal Road
Suite A
Cincinnati, Ohio 45241

Attn: Mr. Dennis Huber, P.E.

Re: Consulting Services
Boone County Rural Water Project
Phase II
Contract 2C
Boone County, Kentucky

Ladies and Gentlemen:

Summarized in this report are the results of the Geotechnical Consulting Services provided to Tetra Tech, Inc. (Tetra Tech) by Thelen Associates, Inc. (Thelen) for the proposed Boone County Rural Water Project, Phase II, Contract 2C in Boone County, Kentucky. The work was performed in accordance with our Proposal-Agreement K24209, and was authorized by Mr. Dennis Huber of Tetra Tech by signature of Tetra Tech Work Order No. 2 on September 29, 2004.

This report summarizes the results of the engineering reconnaissance and test borings performed along Petersburg Road, Anson Lane, Brewer Lane and Caribou Drive, and a review of the Project Drawings prepared by Tetra Tech dated November, 2004 and received by Thelen on November 9, 2004. We have summarized our conclusions and recommendations for the location and depth of the water main based upon the reconnaissance, the results of the test borings and our analyses.

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

We appreciate the opportunity to provide our consulting services for the Boone County Rural Water Project, Phase II, Contract 2C. Should you have any questions concerning the information, conclusions or recommendations contained in this report, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

Bryan M. Scott

Bryan M. Scott, E.I.
Staff Geotechnical Engineer

Theodore W. Vogelpohl

Theodore W. Vogelpohl, P.E.
Chief Geotechnical Engineer

STATE OF KENTUCKY
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LICENSED
PROFESSIONAL ENGINEER

BMS/TWV:df
031308E
Copies submitted: 2 – Client

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January 5, 2005

CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
PHASE II
CONTRACT 2C
BOONE COUNTY, KENTUCKY

1.0 INTRODUCTION

Presented in this report are the results of the geotechnical engineering reconnaissance and test boring exploration for the proposed 8-inch diameter water main to be installed along Petersburg Road, Anson Lane, Brewer Lane and Caribou Drive in Boone County, Kentucky. The scope of our geotechnical services included a review of the existing topography and other land features along the alignment proposed on the plans prepared by Tetra Tech, Inc. (Tetra Tech). Areas of apparent or potential instability along the selected alignment were identified during the reconnaissance, and only these areas were explored by advancing test borings to determine the subsurface conditions. The conclusions and recommendations contained in this report regarding the water main alignment and depths are based on the engineering reconnaissance and the test boring information.

2.0 PROJECT CHARACTERISTICS

The project drawings referred to in this report are titled "Boone County Rural Water Project, Phase II, Contract 2C – KY 20, Petersburg/Belleview Roads", dated November, 2004, marked as "Preliminary" and received by Thelen Associates, Inc. (Thelen) on November 9, 2004. The Phase II, Contract 2C portion of the Boone County Rural Water

Project will consist of roughly 7515 lineal feet of pipe along Petersburg Road, 3585 lineal feet of pipe along Anson Lane, 1050 lineal feet of pipe along Brewer Lane and 2160 lineal feet of pipe along Caribou Drive. All of the water main pipe is to be 8-inch diameter Class 50 ductile iron pipe.

The proposed water main along Petersburg Road will connect to an existing 12-inch diameter water main near the Idlewild Road intersection. The proposed water main along each of the other roads will connect to the proposed water main along Big Bone Church Road. The change in surface elevation along the water main alignment for Petersburg Road is 58 feet, with the lowest point located near the middle of the alignment (El. 788) and the highest point located near the beginning of the alignment (El. 840). The change in surface elevation along the water main alignment for Anson Lane is 52 feet, with the lowest point located along the private extension to Anson Lane (El. 756) and the highest point located near the intersection with Brewer Lane (El. 808). The change in surface elevation along the water main alignment for Brewer Lane is 16 feet, with the lowest point located at the south end of Brewer Lane (El. 804) and the highest point located at the north end of Brewer Lane (El. 820). The change in surface elevation along the water main alignment for Caribou Drive is 42 feet, with the lowest point located at the east end of Caribou Drive (El. 770) and the highest point located at the beginning of the alignment (El. 812). The water main will follow the roadways, typically several feet off the upslope edge of the pavement. In general, the roadways have been constructed on relatively gentle to moderately sloping valley walls and ridgetops or fill embankments.

The depth of the new water main is expected to vary along the alignment due to constraints caused by local geology, existing utilities and other construction–related features. It is our understanding that the minimum cover at the crown of the pipe will be 4 feet. Throughout this report, "normal depth" refers to at least 4 feet of soil cover above the top of the pipe.

Specific descriptions of the proposed water main installation and our recommendations are provided on a section-by-section basis in the Conclusions and Recommendations

Section of this report. This report only addresses the geotechnical issues for the water main project.

3.0 ENGINEERING RECONNAISSANCE

The reconnaissance of the alignment was made by the Engineer, during which evidence of soil and bedrock exposures, slope movement, pavement subsidence, steep existing slopes, fill embankments, erosion, etc., were noted. The project drawings by Tetra Tech were used as a reference during the reconnaissance.

The alignment begins on Petersburg Road at the intersection of Idlewild Road. The alignment begins on relatively gently sloping terrain near the drainage ditch located approximately 30 feet from the road. The alignment continues on the right hand (northwest) side of the road past Anson Lane, and transitions from 30 feet from the road to 4 feet from the road between Anson Lane and Second Creek Road. The alignment remains on the northwest side of the road, crossing relatively gently sloping terrain to the end of this section of the alignment near the intersection with Ashby Fork Drive.

Another section of the alignment follows Anson Lane, beginning at its intersection with Petersburg Road. From the intersection the alignment descends a slope, following the upslope side of the roadway, and crosses an embankment dam on the downslope side of the road. The alignment then ascends a gentle slope and remains on the upslope side of the road as Anson Lane follows a ridgetop to the end of this section of the alignment.

The alignment also continues on Brewer Lane from the intersection with Anson Lane. This section of the alignment begins on the south side of Anson Lane and follows the east side of Brewer Lane. The alignment crosses relatively gentle to nearly flat terrain, terminating at the end of Brewer Lane.

Another section of the alignment follows Caribou Drive from its intersection with Petersburg Road. The alignment follows the east side of Caribou Drive, descending a relative gentle slope, crossing a drainage swale on the downslope side of the road, and remaining on the upslope side of the road as Caribou Drive ascends a relatively gentle slope. The alignment continues to follow the left side of the road as Caribou Drive follows gentle ridgetop terrain.

In general, the alignment crosses relatively gentle to moderately sloping terrain throughout the alignment. The proposed alignment crosses one area of apparent instability along the course of the alignment. Specific details of this and other areas are described on a section-by-section basis in the Conclusions and Recommendations Section of this report.

4.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Four test borings were made at locations selected by Thelen to explore the subsurface conditions along the alignment. The test borings are labeled AN-1 through AN-3 along Anson Lane, and CB-1 along Caribou Drive. No test borings were completed along Petersburg Road or Brewer Lane. The test boring locations and ground surface elevations were surveyed by Tetra Tech. The ground surface elevation at each test boring location was referenced to Mean Sea Level (MSL). The test boring locations are referenced to the centerline of the road by station and offset for this report. The locations are noted at the tops of the test boring logs and are summarized in Table 1, Test Boring Locations.

TABLE 1

Test Boring Locations

Test Boring No.	Road	Station	Offset
AN-1	Anson Lane	4+78	8' L
AN-2	Anson Lane	24+52	4' L
AN-3	Anson Lane	29+46	3' L
CB-1	Caribou Drive	6+57	11' L

The test borings were made with a truck-mounted drill rig advancing either continuous flight augers or hollow stem augers. Two-inch outside diameter (O.D.) driven split-spoon samples were obtained at pre-selected intervals according to the procedures outlined in ASTM D1586. Recovered split-spoon samples were placed in glass jars to retain the samples at their in-situ moisture contents. The sample jars were appropriately marked in the field for proper identification.

Concurrent with the drilling operation, the Drilling Technician prepared the field test boring logs of the subsurface profile noting soil types and depths, standard penetration test resistances (N-values), soil and bedrock stratifications and ground water levels or the lack thereof. Following the completion of the test borings, the samples were returned to our Soil Mechanics Laboratory where they were reviewed and visually classified by the Project Geotechnical Engineer.

Representative samples from the test borings were selected for laboratory tests, including moisture content tests and Atterberg limits tests. A tabulation of the moisture content and Atterberg limits tests is included in the Appendix to this report.

Final test boring logs were prepared based on the Drilling Technician's field logs and the Engineer's visual classification of the samples. Copies of these logs can be found in the Appendix along with a Soil Classification Sheet describing the terms and symbols used in their preparation.

The dashed lines on the test boring logs identify the changes between the soil and bedrock types, which were determined by interpolation between samples and should be considered approximate. Only changes that occur within samples can be precisely determined and are indicated by solid lines on the logs. The transition between soil and bedrock types may be abrupt or gradual.

5.0 GENERAL SUBSURFACE CONDITIONS

Specific subsurface conditions were identified only in limited areas along the alignment. The subsurface conditions in the explored areas are discussed on a section-by-section basis in the Conclusions and Recommendations section of this report.

The following is a discussion of the generalized subsurface conditions that are anticipated based on the observations during the engineering reconnaissance, published Geologic Maps, the test borings made along the alignment, and our general experience as Geotechnical Engineers in the Northern Kentucky Area. The general nature of the following discussion regarding the subsurface conditions in unexplored areas should be recognized, as should the possibility of encountering conditions along the alignment that vary from the generalized conditions.

A generalized profile of clays and silty clays overlying interbedded shale and limestone bedrock is expected throughout the majority of the alignment. In valleys and swales and at creek crossings, softer sediment overlying the soils or bedrock is likely to exist. Man-placed fill for road embankments and embankment dams, comprised of clays and shales with limestone floaters, is locally present over the native soils and bedrock along the alignment. The fill embankments are generally located on the downslope sides of the roadways and at creek and swale crossings.

Each of the test borings was performed within the pavement of the roadways. Each of the test borings encountered between 4 and 7 inches of asphalt.

The proposed water main will generally be placed within the drainage ditch on the upslope side of the roadway. The subsurface conditions along the water main alignment will generally consist of topsoil, a low-density near-surface silty clay or clay soil, then stiff to very stiff clay and silty clay above bedrock, interbedded shale and limestone. It is anticipated that some sections of the water main alignment will encounter bedrock at shallow depths. It is also anticipated that alluvial and sediment deposits will be encountered at the creek and swale crossings.

The majority of the fill that will be encountered during the excavations for the water main alignment has been placed during the construction of road embankments. Fill will also be encountered at the embankment dam along Anson Lane. The consistency of the fill may vary from soft to very stiff. The test borings completed along the alignment encountered medium stiff to very stiff fill.

The bedrock beneath the overburden soil is a system of Ordovician Age shale and limestone. This type of bedrock is typically classified into three zones identified by the extent of weathering of the shale portion of the bedrock. The uppermost zone is termed highly weathered shale and limestone, where the shale portion has virtually weathered to a brown silty clay or clay yet possesses horizontally aligned bedding characteristics of the bedrock system. The intermediate zone is described as weathered bedrock and is characterized by a shale component that is tougher, and generally at a lower moisture content, than the highly weathered zone above. The upper and intermediate zones have weathered from the third commonly accepted zone, the unweathered, gray, parent, interbedded shale and limestone. The limestone component of the highly weathered, weathered, and unweathered bedrock consists of horizontal beds, which are gray, crystalline, fossiliferous and hard. Highly weathered and weathered zones, locally, may or may not be present above the unweathered bedrock zone because of variable weathering and erosion conditions.

According to the United States Geological Survey (USGS) Lawrenceburg Quadrangle Map, the bedrock near the surface of the water main alignment is classified as one of three formations: the Bull Fork Formation, the Bellevue Tongue of the Grant Lake Limestone, and the Fairview Formation. The bedrock near the surface of the alignment along all of Petersburg Road, all of Brewer Lane, and areas of Anson Lane and Caribou Drive above approximately El. 780 is classified as the Bull Fork Formation. This Formation is usually comprised of approximately 50 percent limestone and 50 percent shale. The limestone layers are in even to irregular beds generally less than 4 inches thick. The bedrock near the surface of the alignment along Anson Lane and Caribou Drive between approximately El. 770 and El. 780 is classified as the Bellevue Tongue of the Grant Lake Limestone. This Formation is predominantly limestone in thin, very irregular, discontinuous beds with shale partings.

The bedrock near the surface of the remainder of the alignment along Anson Lane and Caribou Drive below approximately El. 770 is classified as the Fairview Formation. This Formation is usually comprised of 50 to 60 percent limestone and the remaining 40 to

50 percent shale. The limestone layers are in regular to irregular thin to medium beds, averaging about 4 inches thick.

The bedrock was not cored as part of this project. There are exposures of bedrock in the creeks and some road cuts, giving an indication of the thickness and percentage of the shale and limestone.

All test borings were backfilled immediately after they were completed. Groundwater was first noted at depths of 2.5 and 24.5 feet in Test Boring AN-1. All test borings were dry at completion. The water was noted within the fill placed for the embankment dam. Experience has found that seepage water can also occur at the soil/bedrock interface and along the layers of limestone within the bedrock.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based upon our engineering reconnaissance of the water main alignment, the test borings, a visual examination of the samples, the laboratory test results, our understanding of the proposed construction, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the conclusions and make the recommendations in this report.

The conclusions and recommendations of this report have been derived by relating the general principles of the discipline of Geotechnical Engineering to the proposed construction outlined by the Project Characteristics section of this report. Because changes in surface, subsurface, climatic, and economic conditions can occur with time and location, we recommend for our mutual interest that the use of this report be restricted to this specific project.

Our understanding of the proposed design and construction is based on the documents provided to us at the time this report was prepared and which are referenced in the Project Characteristics section of this report. We recommend that our office be retained to review the final design documents, plans, and specifications to assess any impact changes, additions or revisions to these documents may have on the conclusions and recommendations of this Geotechnical Report. Any changes or modifications which are made in the field during the construction phase which alter the water main alignment or depths or other related site work should also be reviewed by our office prior to their implementation.

If conditions are encountered in the field during construction which vary from the facts of this report, we recommend that our office be contacted immediately to review the changed conditions in the field and make appropriate recommendations.

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.

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

The earthwork recommendations of this report presume that the earthwork will be monitored by an Engineering Technician under the direction of a Registered Professional Geotechnical Engineer. We recommend that the Owner contract these services directly with Thelen Associates, Inc.

Throughout the alignment, the proposed water main will cross many culverts below the roadways. At each of these culvert crossings, the water main should be installed below the existing culverts, unless the water main can be installed above the culvert while maintaining the minimum soil cover above the crown of the main and maintaining the minimum required clear distance between the water main and the culvert, considering the structural capacity of the culvert and any necessary soil cover between the culvert and the water main to provide protection from freezing.

The proposed water main installation and the terrain that exists along the alignment were reviewed on a Station-by-Station basis and are discussed individually in Sections 6.2 through 6.5 of this report. Section 6.6 contains general recommendations for placement and compaction of trench backfill.

6.2 Petersburg Road Water Main Alignment

6.2.1 Station 0+00 (Beginning of Alignment) to Station 34+20

A proposed 8-inch diameter ductile iron water main will connect to the existing 8-inch diameter water main immediately north of the intersection with Idlewild Road. The proposed 8-inch diameter water main will begin approximately 30 feet north of Petersburg Road and continue north of the road past Stevens Road and Anson Lane, crossing relatively gentle terrain and several moderately steep roadway embankment slopes. From Station 30+00 to the end of this section, the alignment gradually becomes closer to the road, becoming 4 feet from the road at the end of this section of the alignment.

No evidence of instability was noted along this section of the alignment, and no test borings were completed in this section of the alignment.

In our opinion, the water main can be installed at normal depths throughout this section.

6.2.2 Station 34+20 to Station 75+15 (End of Alignment)

In this section, the water main follows the north side of the road through relatively gentle terrain, crossing several roadway fill embankments.

No evidence of instability was noted along this section of the alignment, and no test borings were completed in this section of the alignment.

In our opinion, the water main can be installed at normal depths throughout this section.

6.3 Anson Lane Water Main Alignment

6.3.1 Station 0+00 (Beginning of Alignment) to Station 6+50

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near Petersburg Road. The proposed 8-inch diameter water main will begin on the west side of Anson Lane, descend a slope on the upslope side of the road, and cross an embankment dam on the downslope side of the dam.

Test Boring AN-1 was completed at Station 4+78, 8 Ft. Left. This test boring encountered 7 inches of asphalt overlying 1.9 feet of medium dense crushed limestone fill, overlying 7.0 feet of stiff silty clay fill with limestone fragments, overlying 7.5 feet of generally very moist medium stiff silty clay and clay fill, overlying 7.5 feet of medium stiff to stiff silty clay fill with shale fragments and limestone floaters, overlying 5.0 feet of stiff to very stiff silty clay fill with shale fragments and limestone floaters, overlying highly weathered shale and limestone bedrock.

Instability was noted on the downslope (west) slope of the embankment dam. Instability of the embankment dam was also indicated by the presence of pavement cracking and a pavement overlay consistent with slope instability. The presence of very moist medium stiff soils in the test boring further indicates potential slope instability at the location of the test boring.

Based on the observed instability and potential future instability, we recommend that the alignment be moved from the embankment dam to a location below the toe of the embankment dam. We recommend that the alignment be revised to descend the natural slope beginning at Station 3+50 to a point where the revised alignment is at least 15 feet beyond the toe of the embankment dam. At this point we recommend that the alignment turn to the north and cross the valley, remaining at least 15 feet beyond the toe of the embankment dam. After crossing the valley, we recommend that the alignment turn to the north and ascend the natural slope of the valley wall and rejoin the proposed alignment near Station 6+50. We recommend that the main be deep enough that the pipe and thrust blocks are embedded in the bedrock as the main crosses the valley bottom.

No instability was noted between Station 0+00 and Station 3+50, and in our opinion, the water main can be installed at normal depths in this section.

6.3.2 Station 6+50 to Station 35+85 (End of Alignment)

The proposed 8-inch diameter water main will continue on the west side of Anson Lane, ascend a relatively gentle slope, pass Brewer Lane, and continue on the west side of Anson Lane as the road follows ridgetop terrain.

Two test borings were completed along this section of the alignment. Test Boring AN-2 was completed at Station 24+52, 4 Ft. Left. This test boring encountered 4 inches of asphalt overlying 4.2 feet of very stiff silty clay, overlying weathered shale and limestone bedrock. Test Boring AN-3 was completed at Station 29+46, 3 Ft. Left. This test boring encountered 5 inches of asphalt overlying 4.1 feet of very stiff clay, overlying 2.5 feet of highly weathered shale and limestone bedrock, overlying weathered shale and limestone bedrock.

No evidence of instability was noted in this section and, in our opinion, the water main can be installed at normal depths.

6.4 Brewer Lane Water Main Alignment

6.4.1 Station 0+00 (Beginning of Alignment) to Station 10+50 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Anson Lane. The proposed 8-inch diameter water main will begin immediately south of the intersection, cross Anson Lane, and remain on the east side of the Brewer Lane to the end of the alignment. Throughout this section the alignment crosses relatively gentle to nearly flat terrain. No test borings were completed in this section of the alignment.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths.

6.5 Caribou Drive Water Main Alignment

6.5.1 Station 0+00 (Beginning of Alignment) to Station 21+60 (End of Alignment)

A proposed 8-inch diameter ductile iron water main will connect to the proposed 8-inch diameter water main near the intersection with Petersburg Road. The proposed 8-inch diameter water main will begin immediately north of the intersection, and remain on the east side of the Caribou Drive to the end of the alignment. The alignment descends a relatively gentle slope, crosses a fill embankment and culvert, ascends a relatively gentle slope, and follows ridgetop terrain to the end of this section of the alignment.

Test Boring CB-1 was completed at Station 6+57, 11 Ft. Left. This test boring encountered 6 inches of asphalt overlying 11.0 feet of stiff to very stiff silty clay and clay fill. The test boring was terminated in this fill.

No evidence of instability was noted in this section of the alignment, and in our opinion, the water main can be installed at normal depths except between Stations 6+00 and 7+00, where we recommend that the main be deepened to 6 feet of cover due to the steepness of the fill slope at that location.

6.6 General Excavation and Backfilling Recommendations

The excavations throughout this project will encounter a variety of materials. Those materials will include roadway embankment fill, native clay and silty clay soils, and interbedded shale and limestone bedrock. Experience indicates that the difficulty of completing the excavations in the bedrock usually far exceeds the difficulty of excavating in the fill materials and the native soils. The difficulty of making bedrock excavations is primarily related to the amount and thickness of the limestone layers in the bedrock as well as the degree of weathering. The Contractor should be aware of the presence of the bedrock and should be prepared for the difficulty that bedrock may present in the excavations.

The scope of this project involved subsurface explorations to define specific subsurface conditions only in critical areas, which represent a limited percentage of the total project length. Therefore, we recommend that the specifications for this project be based on unclassified excavation, not on separate cost items for soil excavation and bedrock excavation. The base bid for the project should include the cost of excavating the materials encountered within the specified water main depths, regardless of soil or bedrock characteristics.

It is difficult to shear limestone layers neatly in the sides of trench excavations. Frequently, when limestone layers are encountered at relatively shallow depths in trench excavations, the tendency is for the layers not to break even with the sides of the excavations, but rather to be pulled up in large chunks, which tend to heave and ravel the soils outside the limits of the intended trench. Where trench excavations will be made immediately adjacent to or within existing pavement with the intention of not disturbing the existing pavement beyond the trench limits, it should be anticipated that there will be some areas where there is heave and raveling due to removal of limestone layers that could damage pavement adjacent to the trench, and said pavement will have to be restored.

We expect that the excavated materials, exclusive of the thick limestone layers, can be used as backfill after the appropriate granular pipe bedding and backfill is installed. Fill materials should not include asphalt, concrete, trash, construction or demolition debris, topsoil or frozen material. Large pieces of limestone which tend to nest or retard compaction should be excluded from the backfill. Smaller pieces of limestone that can be broken up and dispersed so that they do not nest or retard compaction can be incorporated in the backfill provided that proper protection of the pipe from these pieces of limestone is provided.

The trench excavations for the project will extend a minimum of about 5 feet deep. In areas where mains are to be lowered due to existing culverts and utilities, the excavations may extend to greater depths. The Contractor should be responsible for the stability and safety of all excavations and should exercise all necessary precautions to shore, slope or otherwise maintain stable trench excavations to protect workers, adjacent property, adjacent pavement and structures, and infrastructure. These trenches should be made and maintained in accordance with all Federal, State and Local regulations.

Normal and recommended utility construction practice is to bed and backfill pipes with granular fill to a specified height above the crown of the pipe. Compaction of trench backfill to a moist, firm, dense condition is important throughout the entire alignment of this project, because the alignment is generally beneath the existing pavement, immediately adjacent to existing pavement, near the toes of existing cut and fill slopes, or through existing roadway embankment slopes. We recommend that **all** backfill for this project be placed in shallow level layers, 6 to 8 inches in thickness, and be compacted to densities not less than 95 percent of the standard Proctor maximum dry density, ASTM D698. For soil backfill used within the road, the upper 8 inches of backfill beneath the pavement areas should be compacted to densities not less than 98 percent of the standard Proctor maximum dry density, ASTM D698. The backfill soils should be moisture-conditioned to within the range of 2 percent below to 3 percent above the optimum moisture at the time of compaction. All shale should be pulverized to a soil-like consistency and moisture conditioned the same as a soil. Where granular fill is used, it should be compacted to at least 75 percent of the maximum relative density obtained using ASTM D4253 and D4254 test methods. Density tests should be made in the backfill to document that the recommended degree of compaction is being achieved.

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CONSULTING SERVICES
BOONE COUNTY RURAL WATER PROJECT
BOONE COUNTY, KENTUCKY
031308E

TABULATION OF LABORATORY TESTS

Boring Number	Sample Number	Depth, ft.		Moisture Content, %	Atterberg Limits, %			USCS Classification
		From	To		LL	PL	PI	
AN-1	2	2.5	4.0	25.6	49	18	31	CL
AN-1	4	7.5	9.0	26.3				
AN-1	5	10.0	11.5	29.0				
AN-2	2	2.5	4.0	22.9				
AN-3	2	2.5	4.0	25.2				
CB-1	2	2.5	4.0	23.8				
CB-1	3	5.0	6.5	27.2				

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # AN-1
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Anson Lane, Station 4+78, 8 Ft. Left

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	Cond	Blows/6"	No.	Type	Rec. Inches
765.3	SURFACE	0.0					
764.7	ASPHALT (7 inches).	0.6	I	20/11/3	1	DS	18
762.8	Mixed gray moist medium dense FILL, crushed limestone with silty clay.	2.5	I	3/4/5	2	DS	18
755.8	Mixed brown, trace gray moist stiff FILL, silty clay, trace iron oxide stains and limestone fragments (CL).	9.5	I	4/3/4	3	DS	18
			I	8/4/3	4	DS	18
753.3	Mixed brown, trace gray very moist medium stiff FILL, clay, trace shale fragments and iron oxide stains.	12.0	I	2/4/3	5	DS	18
750.8	Mixed brown, trace gray very moist medium stiff FILL, silty clay, trace organic material and topsoil.	14.5	I	3/3/5	6	DS	18
748.3	Mixed brown, trace gray moist medium stiff FILL, silty clay, trace shale and limestone fragments.	17.0	I	3/5/5	7	DS	18
745.8	Mixed brown moist medium stiff to stiff FILL, silty clay with limestone floaters.	19.5	I	6/5/5	8	DS	7
740.8	Mixed brown moist stiff to medium stiff FILL, silty clay with limestone floaters, trace roots and shale fragments.	24.5	I	5/6/11	9	DS	12
738.3	Mixed brown, olive brown and gray moist stiff FILL, silty clay with shale fragments and limestone floaters.	27.0	I	6/7/9	10	DS	18
735.8	Mixed dark brown and olive brown moist stiff to very stiff FILL, silty clay with limestone floaters and shale fragments.	29.5		Note: Scale Change			
			I	11/9/9	11	DS	12
			I	17/9/20	12	DS	18
734.3	Interbedded brown and gray moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	31.0	I	26/50/6"	13	DS	7
	Bottom of test boring at 31.0 feet.						

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 6 in. Foreman BR
Surf. Elev. 765.3 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/2/04 Pipe Size O.D. 2 in. Boring Method 2-1/4 HSA Date Completed 12/2/04

SAMPLE CONDITIONS
D – DISINTEGRATED
I – INTACT
U – UNDISTURBED
L – LOST

SAMPLE TYPE
DS – DRIVEN SPLIT SPOON
PT – PRESSED SHELBY TUBE
CA – CONTINUOUS FLIGHT AUGER
RC – ROCK CORE

GROUND WATER DEPTH
FIRST NOTED 2.5/24.5 ft.
AT COMPLETION – ft.
AFTER – hrs. – ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA– HOLLOW STEM AUGERS
CFA– CONTINUOUS FLIGHT AUGERS
DC – DRIVING CASING
MD – MUD DRILLING

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
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2140 Waycross Road / Cincinnati, Ohio 45240-2719 / 513-825-4350 / Fax 513-825-4756
www.thelenassoc.com

LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # AN-2
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Anson Lane, Station 24+52, 4 Ft. Left

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
756.0	SURFACE	0.3						
755.7	ASPHALT (4 inches).			I	8/8/44	1	DS	18
751.5	Brown moist very stiff SILTY CLAY with limestone floaters, trace iron oxide stains.	4.5		I	7/8/12	2	DS	18
	Interbedded brown, trace gray moist soft weathered SHALE and gray hard LIMESTONE (bedrock).		5	I	10/14/15	3	DS	18
747.0		9.0		I	12/14/13	4	DS	18
	Bottom of test boring at 9.0 feet.		10					
			15					
			20					
			25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 756.0 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/2/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/2/04

SAMPLE CONDITIONS
D – DISINTEGRATED
I – INTACT
U – UNDISTURBED
L – LOST

SAMPLE TYPE
DS – DRIVEN SPLIT SPOON
PT – PRESSED SHELBY TUBE
CA – CONTINUOUS FLIGHT AUGER
RC – ROCK CORE

GROUND WATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER – hrs. – ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA– HOLLOW STEM AUGERS
CFA– CONTINUOUS FLIGHT AUGER
DC – DRIVING CASING
MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # AN-3
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Anson Lane, Station 29+46, 3 Ft. Left

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
763.2	SURFACE	0.4						
762.8	ASPHALT (5 inches).			I	4/5/17	1	DS	9
	Brown moist very stiff CLAY, some iron oxide stains.			I	7/8/12	2	DS	18
758.7		4.5	5					
	Interbedded olive brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	7.0		I	15/4/7	3	DS	7
756.2	Interbedded brown moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	9.0		I	26/29/41	4	DS	18
754.2			10					
	Bottom of test boring at 9.0 feet.							
			15					
			20					
			25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 763.2 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/2/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/2/04

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

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LOG OF TEST BORING

CLIENT: Tetra Tech, Inc. BORING # CB-1
PROJECT: Consulting Services, Boone County Rural Water Project, Boone County, Kentucky JOB # 031308E
LOCATION OF BORING: Caribou Drive, Station 6+57, 11 Ft. Left

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
780.1	SURFACE	0.0						
779.6	ASPHALT (6 inches).	0.5 / 2.0		I	4/3/5	1	DS	18
778.1	Mixed brown moist stiff FILL, clay.			I	6/7/7	2	DS	18
775.6	Mixed dark gray moist very stiff FILL, silty clay.	4.5	5	I	3/2/5	3	DS	18
773.1	Mixed brown moist stiff FILL, clay.	7.0		I	4/7/11	4	DS	18
770.6	Mixed brown and gray moist very stiff FILL, silty clay with shale fragments.	9.5	10					
768.6	Mixed brown and gray moist very stiff FILL, clay.	11.5		I	3/3/6	5	DS	12
	Bottom of test boring at 11.5 feet.		15, 20, 25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman BR
Surf. Elev. 780.1 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer BMS
Date Started 12/2/04 Pipe Size O.D. 2 in. Boring Method CFA Date Completed 12/2/04

SAMPLE CONDITIONS
D – DISINTEGRATED
I – INTACT
U – UNDISTURBED
L – LOST

SAMPLE TYPE
DS – DRIVEN SPLIT SPOON
PT – PRESSED SHELBY TUBE
CA – CONTINUOUS FLIGHT AUGER
RC – ROCK CORE

GROUND WATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER – hrs. – ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA– HOLLOW STEM AUGERS
CFA– CONTINUOUS FLIGHT AUGERS
DC – DRIVING CASING
MD – MUD DRILLING

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SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density	
Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties	
Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification		
Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

Important Information About Your
Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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