REPORT OF GEOTECHNICAL EXPLORATION SOUTH INDUSTRIAL DRIVE PAVEMENT AND SUBGRADE REHABILITATION ERWIN, TENNESSEE

S&ME Project No. 1401-10-125

Prepared For:
Tysinger Hampton and Partners, Inc.
Johnson City, Tennessee

Prepared By:
S&ME, Inc.
644 Eastern Star Road
Kingsport, Tennessee 37663

September 9, 2010

S&ME

S&ME

September 9, 2010

Tysinger Hampton and Partners, Inc.
3428 Bristol Highway
Johnson City, Tennessee 37601

Attention: Mr. Tom Patton, P.E.

Reference: Report of Pavement and Subgrade Evaluation
South Industrial Drive
Erwin, Tennessee
S&ME Project No. 1401-10-125

Dear Mr. Patton:

The enclosed report presents the results of the pavement and subgrade evaluation by S&ME, Inc. for the proposed rehabilitation of South Industrial Drive in Erwin, Tennessee. The work was performed in general accordance with our proposal (S&ME Proposal Number 1401-10-125) dated July 15, 2010.

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning subgrade, pavement design, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations.

S&ME, Inc. appreciates this opportunity to be of service to you. Please call if you have questions concerning this report or any of our services.

Respectfully submitted,

S&ME, Inc.

Daniel R Boles, P.E.
Branch Manager
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DRB/KCD/mc

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EXECUTIVE SUMMARY

This summary is presented for the convenience of the reader. However, the full report text should be studied and understood before preparing any estimation of quantities or designs based on this report.

Project information has been supplied through phone conversations, emails and an on site meeting on August 2, 2010. We understand the proposed project consists of rehabilitation of South Industrial Drive and may consist of overlays, full pavement section replacement and pavement section replacement with subgrade remediation. South Industrial Drive is severely degraded and has alligator cracking, rutting, potholes and longitudinal cracking. The distress appears to be greatest at the locations where the local industry trucks enter and exit the roadway. In addition, it is our understanding there may have been issues in the past with a high water table in the roadway area.

The borings performed at the site encountered very thin pavement sections underlain by fill soils and alluvial soils. The fill soils immediately below the pavement section have moderate consistency to medium dense relative density but are interspersed with debris and wood.

On the basis of this geotechnical exploration, the anticipated pavement loads, and our engineering evaluation of the subsurface conditions at this site, we believe the majority of the pavement failure is due to inadequate pavement sections for the actual traffic load being placed on the pavement. These factors combined with a variable subgrade have contributed to the pavement failure.

We believe the majority of the pavement will require removal and replacement. The only area that may be adequate for overlay would be between the area about 300 feet from the intersection with Jackson-Love Highway to the area about 1,100 feet from Jackson-Love Highway. It is likely this area may become further degraded due to heavy construction traffic during the rehabilitation process and may ultimately require removal and replacement, also.

The existing fill materials should be evaluated once construction commences to determine whether they are acceptable for use as pavement support. This evaluation should be accomplished by performing thorough proofrolling at the time of construction. Areas which do not perform well under proofrolling should be undercut and replaced with structural fill or dense-grade stone. We anticipate, based on the variable material within the fill materials (i.e. debris, wood, brick), undercutting will be required to develop stable subgrades in numerous areas.

Given the condition of the soil encountered, performance of the work in a period of dry weather should help reduce the amount of undercut required to prepare the subgrade. Areas that currently displayed rutting, regardless of depth, or that have an unsatisfactory proofroll are the areas most likely to require significant undercutting in order to prepare the subgrade.

Areas that display alligator cracking are not good candidate areas for an asphalt overlay because the cracking tends to reflect through the new pavement and can significantly limit the life of the overlay. It is recommended the pavement in these areas be removed and after the subgrade is properly prepared, the pavement be replaced.

Risks and challenges associated with the development of this site include the potential for wet, soft soils to be encountered in excavations at or near current grades which can be easily aggravated by the prevailing weather conditions and the presence of existing fill soils. These risks and challenges are discussed in greater detail in the following sections of this report.

1. INTRODUCTION

S&ME, Inc. has completed the pavement and subgrade evaluation at the location of the proposed rehabilitation of South Industrial Drive in Erwin, Tennessee. The work was performed in general accordance with our proposal dated July 15, 2010. The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning subgrade and pavement thicknesses.

On August 9 and 10, 2010, a total of five (5) soil test borings were drilled to obtain subsurface information in the area of the proposed rehabilitation. Additional discussion of the subsurface conditions encountered is provided in subsequent sections of this report. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations. A Site & Boring Location Plan (Figures 1 and 2) showing the approximate boring locations, a discussion of the field investigative procedures, a legend to soil classification and symbols, Soil Test Boring Records, laboratory test results, and recommended pavement design are presented in the Appendix of this report.

2. SITE AND PROJECT DESCRIPTION

Project information has been supplied through phone conversations, emails and an on site meeting on August 2, 2010. We understand the proposed project will consist of rehabilitation of South Industrial Drive and may consist of overlays, full pavement section replacement and pavement section replacement with subgrade remediation. South Industrial Drive is severely degraded and has alligator cracking, rutting, potholes and longitudinal cracking. The distress appears to be greatest at the locations where the local industry trucks enter and exit the roadway. In addition, it is our understanding there may have been issues in the past with a high water table in the area of South Industrial Drive.

We have not been provided traffic volumes, but based on the industry located along the roadway, we have assumed the total loading for a 20-year design life will be approximately 1,000,000 equivalent single-axle loads (ESALs).

3. REGIONAL GEOLOGY

The Erwin area, as well as most of northeastern Tennessee, is located within the Valley and Ridge Physiographic Province. This province is characterized by elongated, northeasterly-trending ridges formed on more resistant limestones and shales. Between ridges, broad valleys and rolling hills are formed on less-resistant limestones, dolomites, and shales.

Published geologic information and our experience in the area indicate the site is underlain by the Rome formation. Soils weathered from these types of bedrock are generally brown and tan silty clays and/or clayey silts. These soils are underlain by sandstone, siltstone, shale, dolomite and limestone. The subsurface conditions at this site appear to be more influenced by the presence of man-placed fills and alluvial soils deposited by the Nolichucky River.

4. SUBSURFACE CONDITIONS

4.1 Field Exploration Procedures

On August 9 and 10, 2010, five (5) soil test borings (designated B-1 through B-5) were drilled to obtain subsurface information along South Industrial Drive. Additionally, on August 9, 2010, five (5) asphalt cores were performed for drilling access and to provide measurement of existing asphalt thickness in the area. The borings and cores were located in the field prior to drilling by S&ME, Inc. Since surveying procedures were not used to locate the borings or asphalt cores, the locations presented should be considered approximate. The locations of the borings and asphalt cores are shown on the Site & Boring Location Plan, Figure 1, in the Appendix of this report.

The borings were advanced to depths ranging from about 5.5 to 10.0 feet below the existing asphalt surface, using hollow-stem augering techniques. A Dietrich D-50 track-mounted drill rig was used for advancement of the borings.

During advancement of the soil test boring operations, samples were obtained from the encountered soils using standard penetration tests (ASTM D1586). The standard penetration test provides a split-spoon sample of the tested soil and a resulting standard penetration resistance value, which gives an indication of the density and consistency of the in-place soils. Standard penetration resistance values can be utilized with empirical correlations to estimate physical properties and engineering characteristics for most soils. Relatively undisturbed samples were attempted as well. Each of the Shelby tubes was crushed during the attempts to obtain the samples. In addition, bulk samples were to be obtained, but no significant auger cuttings were returned from the borings.

All of the samples collected during the field exploration were returned to the laboratory for visual examination and selected laboratory testing. The obtained samples were visually examined, classified, and logged by the Geotechnical Engineer. Our logs of the exploratory borings are included in the Appendix.

4.2 Soil Stratification

Prior to the actual exploratory drilling, asphalt coring was performed at the five boring locations on South Industrial Drive. Once coring was completed, asphalt thicknesses were recorded. The results of asphalt and basestone thickness measurements are depicted in the table below:

Boring / Core Number	Asphalt Thickness (ft.)	Basestone Thickness (ft.)
B-1	0.3	0.3
B-2	0.45	0.35
B-3	0.3	0.1
B-4	0.3	0.1
B-5	0.2	0.1

Beneath the existing pavement system layer, fill soils were encountered to depths ranging from about 3 to 10 feet below the existing ground surface. Fill soils are soils transported to their present location by man. The fill soils generally consisted of stiff to very hard dark brown to red brown silty clays and fine sandy silty clays. In addition, dark brown to brown loose to medium dense clayey sands and silty sands were encountered. SPT N-values ranged between 10 blows per foot (bpf) to greater than 50 bpf for the fine-grained soils and from 6 to 13 bpf for the coarse-grained soils. Therefore the fine-grained soils are classified as stiff to very hard consistency and the coarse-grained soils are classified as loose to medium dense relative density. Several of the samples contained varying mica content, small river cobbles or rock fragments and debris such as brick fragments, plastic, wood or glass. It is likely some of the N-values were increased due to debris or wood in the samples. Based on our observation of the samples, they were likely placed in an uncontrolled manner. Borings B-3 and B-5 were terminated in the fill soils at a depth of 10 feet.

Beneath the layer of fill in Borings B-1, B-2 and B-4, alluvial soils were encountered to boring termination at depths ranging from about 5.5 feet to 10 feet. Alluvial soils are soils transported to their present location by water. The alluvial soils were typically brown to tan brown silty fine sands and tan to tan brown fine sandy silty clay with varying mica content and small river cobbles. SPT N-values ranged between 3 bpf and 13 bpf within the coarse-grained alluvial layers indicating a very loose to medium dense relative density. SPT N-values ranged from 5 to 12 in the fine-grained soils indicating a firm to stiff material consistency. Boring B-2 encountered refusal to auger advancement at a depth of about 5.5 feet. Borings B-1 and B-4 were terminated at the predetermined depth of 10 feet.

4.3 Water Levels

The boreholes were observed for the presence of water at time of boring (TOB). Water was not observed in any of the borings at TOB. Due to safety concerns, all borings were backfilled upon completion of drilling; therefore, long-term water levels were not recorded. It should be noted that water levels tend to fluctuate with seasonal, climatic, and environmental changes. Therefore, water could be encountered in excavations below the existing ground surface at this site. Furthermore, perched water could be encountered in zones of increased permeability, such as loose fill or within gravel or sands. We would anticipate if water is encountered within excavations, it could be controlled using typical methods such as pumping provided excavation remains relatively shallow.

5. LABORATORY TESTING

Laboratory testing was conducted on selected split-spoon samples and included natural moisture contents and Atterberg limits testing. The following paragraphs outline the results of this testing. Details of the specific test results are included in the attached laboratory data.

Natural moisture contents were performed on all eighteen (18) samples obtained during drilling using ASTM D2216. The natural moisture content of these samples ranged from 8.1 percent to 26.3 percent. The overall average moisture content of all samples combined was 16.5 percent.

Atterberg limits (ASTM D4318) were performed on split-spoon samples obtained from borings B-1 and B-4. The liquid limits of the samples were 42 and 38 percent, with plastic limits of 19 and 20 percent, resulting in plasticity indices of 23 and 18, respectively. These soils classify as a CL (lean clay) in accordance with the Unified Soil Classification System (USCS).

California Bearing Ratio and standard Proctors were not performed because samples were unable to be obtained.

6. ASSESSMENT

On the basis of this geotechnical exploration, the anticipated pavement loads, and our engineering evaluation of the subsurface conditions at this site, we believe the majority of the pavement failure is due to inadequate pavement sections for the actual traffic load being placed on the pavement. These factors combined with a variable subgrade have contributed to the pavement failure. Even though fill materials encountered contained varying materials, the soil consistencies and relative densities were generally sufficient enough, in many cases, to support the traffic loading with a pavement section of proper design thickness. The existing pavement thickness at the boring locations ranged from about 2.5 to 5.5 inches and the basestone thickness ranged from 1 to 4 inches. These pavement sections are inadequate to support anything but very low traffic loading scenarios on very high consistency soils. In spite of the variable fill soils, we believe the site is adaptable for the proposed rehabilitation project with some risk. In order to rehabilitate the pavement, the following risks and challenges should be understood during the design phases of the project. The following paragraphs discuss in greater detail the challenges associated with development of the proposed site.

We believe the majority of the pavement will require removal and replacement. The only area that may be adequate for overlay would be between the area about 300 feet from the intersection with Jackson-Love Highway and the area about 1,100 feet from Jackson-Love Highway. It is likely this area may become further degraded due to heavy construction traffic during the rehabilitation process and may ultimately require removal and replacement.

Fill materials were encountered at multiple boring locations. These fill materials should be evaluated once construction commences to determine whether they are acceptable for use as pavement support. This evaluation should be accomplished by performing thorough proofrolling at the time of construction. Areas which do not perform well under proofrolling should be undercut and replaced with structural fill or dense-grade stone. We anticipate, based on the variable material within the fill materials (i.e. debris, wood, brick), undercutting will be required to develop stable subgrades in numerous areas. In addition, moisture content of the subgrade soils will play a factor in rehabilitation of the pavement. Areas which receive poor drainage should be regraded or adequate piping should be provided to carry water away from the roadway. Additionally, weather can be a factor in the performance of the pavement from the variation of the groundwater level to the saturation of an acceptable subgrade due to precipitation. Therefore, the proofrolling should be performed at a time of dry weather and immediately prior to placement of the basestone. Given the condition of the soil encountered, performance of the work in a period of dry weather should help reduce the amount of undercut required to prepare the subgrade. Areas that currently displayed rutting, regardless of depth, or that have an unsatisfactory proofroll are the areas most likely to require significant undercutting in order to prepare the subgrade.

Areas that display alligator cracking are not good candidate areas for an asphalt overlay because the cracking tends to reflect through the new pavement and can significantly limit the life of the overlay. It is recommended the pavement in these areas be removed after the subgrade is properly prepared and the pavement section be replaced.

Provided the risks and challenges associated with this site are understood, we anticipate the proposed alignment may be supported by approved existing soils or properly compacted structural fill. The following sections of this report provide the design recommendations that can be used for the proposed project. Our recommendations are contingent on S&ME providing observations and associated testing of the construction at the project site.

7. DESIGN RECOMMENDATIONS

7.1 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations contained in this report are based on applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, expressed or implied, is made.

The analyses and recommendations submitted herein are based, in part, on the data obtained from the subsurface exploration. The nature and the extent of variations between the widely-spaced borings will not become evident until the time of construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or elevation of the pavement systems are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions verified or modified in writing.

We strongly recommend that S&ME be provided the opportunity to review the final design plans and specifications in order that earthwork and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME, Inc.'s observation and monitoring of grading and construction activities.

7.2 Flexible Pavement Design

AASHTO flexible pavement design methods have been utilized for pavement recommendations. Our recommendations are based on the assumptions that the subgrade has been properly prepared as recommended in this report. We have not been provided traffic volumes, but based on the industry located along the roadway, we have assumed the total loading for a 20-year design life will be approximately 1,000,000 equivalent single- axle loads (ESALs). If pavement loading is found to be substantially greater than or less than the values indicated, it is recommended the design be re-evaluated. Additionally, we have assumed a design CBR value for pavement design. This value should be confirmed prior to construction once borrow soils are located or design elevations confirmed. The following criteria were used for the design of the flexible pavement sections:

- 20-year design life
- 1,000,000 total ESALs assumed traffic loading
- 85 percent reliability
- 4.2 initial serviceability index
- 2.0 terminal serviceability index
- An assumed design CBR value of 4.0

Based on these criteria, we recommend the following heavy-duty flexible pavement section:

Flexible Pavement Section Recommended Thickness	
Pavement Materials	**Layer Thickness (inches)**
Bituminous Asphalt Surface Mix	2.0
Bituminous Asphalt Base Mix	4.0
Compacted Crushed Aggregate Base	8.0

We recommend a base stone equivalent to a Type A, Class A and Grading D in accordance with Section 903.05 of the Tennessee Department of Transportation specifications. The bituminous asphalt pavement should be Grading "E" as per Section 411 for the surface mix and Grading "B" as per section 307 for the binder mix. Compaction requirements for the crushed aggregate base and the bituminous asphalt pavement should generally follow Tennessee Department of Transportation specifications. To confirm the base course has been uniformly compacted, in-place field density tests should be performed by an Engineering Technician under the direction of a Geotechnical Engineer.

7.3 Rigid Pavement Design

AASHTO rigid pavement design methods have been utilized for pavement recommendations. Our recommendations are based on the assumptions the subgrade has been properly prepared as recommended in this report. The following criteria was used for the design of the rigid pavement sections:

- 20-year design life
- 1,000,000 total ESALs assumed traffic loading
- 85 percent reliability
- 4.2 initial serviceability index
- 2.0 terminal serviceability index
- 0.35 standard deviation
- Modulus of subgrade reaction value of 100 pci with 4-inch stone layer
- 28-day concrete compressive strength of 4,000 psi
- Allowable flexural working stress of 340 psi

We recommend the following rigid pavement section:

Recommended Thickness (Inches)	
Pavement Materials	**Layer Thickness (Inches)**
4,000 psi Type I Concrete	9.0
Compacted Crushed Aggregate Base	4.0

Concrete should be reinforced with welded wire fabric or reinforcing bars to assist in controlling cracking from drying shrinkage and thermal changes. Sawed or formed control joints should be included for each 225 square feet of area or less (15 feet by 15 feet). Saw cuts should not cut through the welded wire fabric or reinforcing steel, and dowels should be utilized at formed and/or cold joints. Pavement design calculations are included in the Appendix of this report.

Our recommendations are based upon the assumption that the subgrade has been properly prepared as recommended in this report and any off-site soil borrow to be used to backfill to the final subgrade meets the requirements of Section 8.2 for structural fill.

All paved areas should be constructed with positive drainage to direct water off-site and to minimize surface water seeping into the pavement subgrade. The subgrade should have a minimum slope of 1 percent. In down grade areas, the basestone should extend through the slope to allow any water entering the basestone to exit. For rigid pavements, water-tight seals should also be provided at formed construction and expansion joints.

8. CONSTRUCTION CONSIDERATIONS

8.1 Site Preparation

Site preparation should be initiated by clearing all deleterious materials, such as debris, wood and organic material from the site. Areas at grade or requiring fill should be proof-rolled with a loaded dump truck or similar piece of heavy pneumatic-tired equipment under the direction of qualified personnel. Portions of the subgrade that deflect excessively during proof-rolling and that cannot be densified by continued rolling should be undercut to stable material. The resulting undercut area should then be backfilled with structural fill or compacted crushed stone. As previously stated, some amounts of existing fill are present on the site. These existing fill materials should be evaluated through proofrolling and if necessary remediated through undercutting or re-compaction. Areas that do not encounter stable material within two feet should be remediated with a geogrid such as Tensar BX-1100 or equivalent prior to backfill.

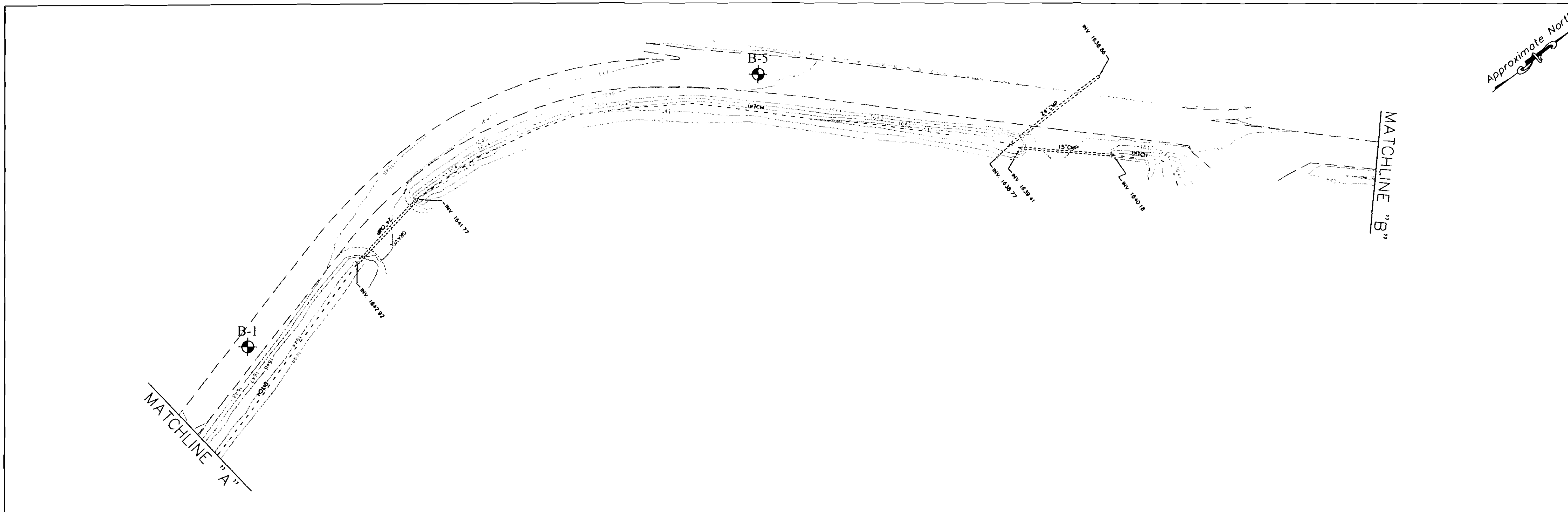
8.2 Structural Fill Placement

Structural soil fill should have a maximum dry unit weight of at least 90 pcf and a PI of 30 or less. The maximum rock particle size in structural fill should not exceed four inches in diameter when compaction is completed. If shale is selected as the borrow material for site grading, all material should consist of brown weathered shale. Dark gray, or black, unweathered shale should not be used as structural fill. During placement of this material, the shale particles should break down under the compactor. Our experience indicates that often, watering is required to maintain proper compaction moisture when weathered shale is used as structural fill. Structural soil fill should be placed in 8-inch thick loose lifts and compacted to at least 98 percent of the soils' maximum dry density as compared to standard Proctor. The moisture content at the time of placement and compaction should be within –1 percent to + 3 percent of the corresponding optimum moisture content.

8.3 Excavation Safety

Excavations should be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29CFR Part 1926) excavation safety standards. It should be noted that the Contractor is solely responsible for site safety. This information is provided only as a service and under no circumstances should S&ME be assumed to be responsible for construction site safety. The fill or alluvial soils at this site are generally classified as Type C according to the OSHA standards, which should be sloped at 1.5(H) to 1(V) or flatter in excavations less than 20 feet deep. Each excavation should be observed and classified by an OSHA-competent person.

APPENDIX



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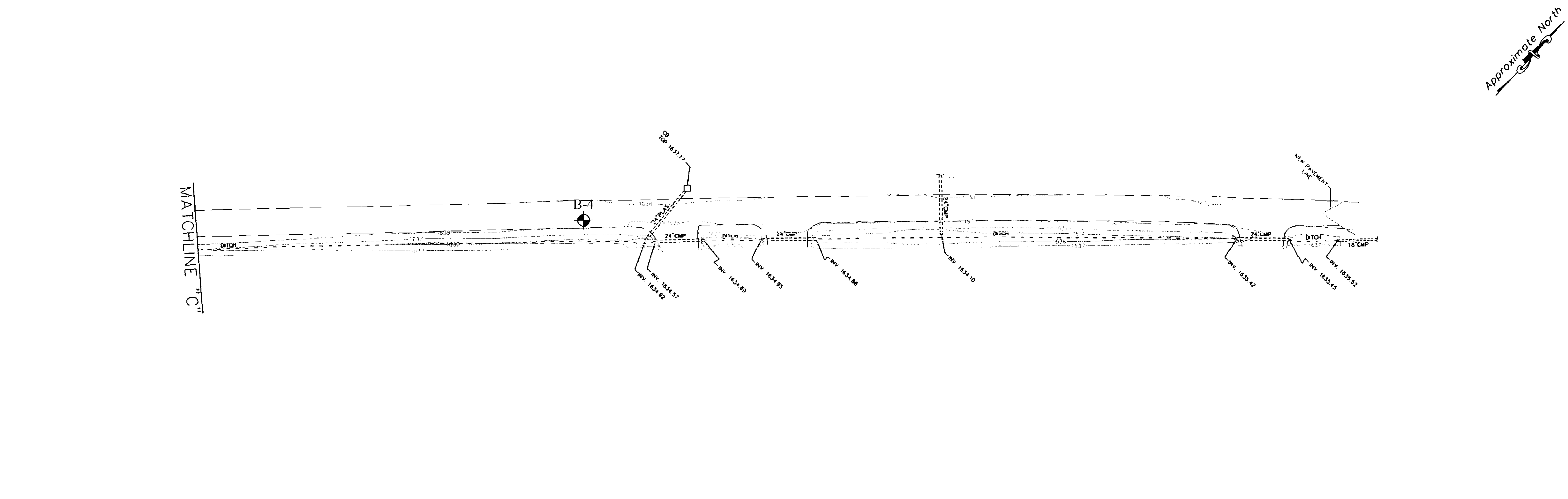
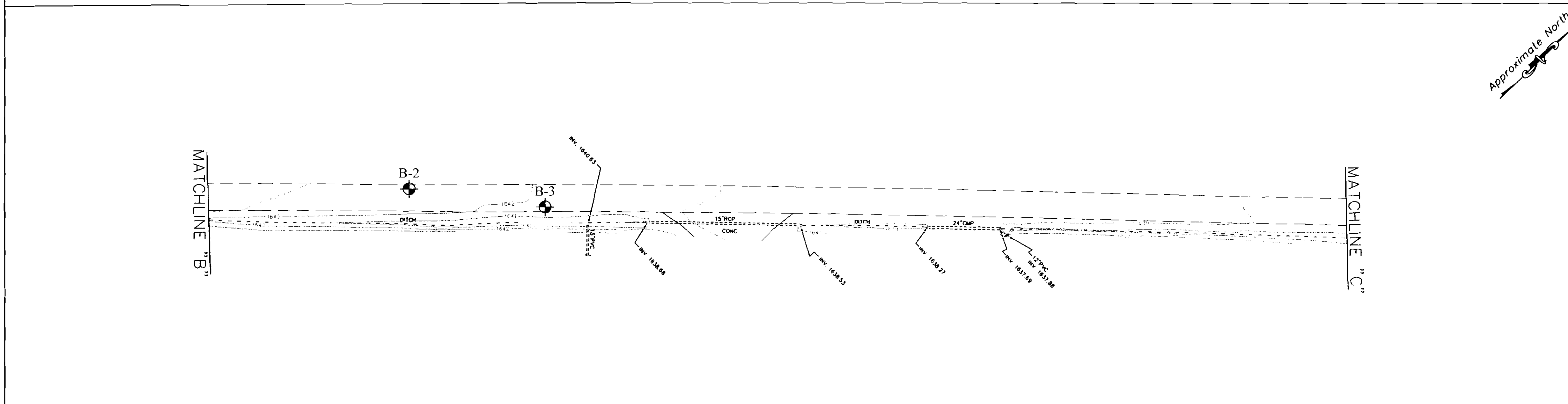
NOTES:	LEGEND:
Drawing was derived from a site plan provided by Tysinger Hampton and Partners.	⯐ - Represents approximate location of soil test borings.

PROJECT:	SCALE:	PROJECT NUMBER:
Site & Boring Location Plan South Industrial Drive Improvements Erwin, Tennessee	1" = 80'	1401-10-125
	DRAWN BY: D.H.S.	DATE: 9/8/2010
	APPROVED BY: D.R.B.	DRAWING NUMBER: FIGURE 1





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	DRAWN BY: D.H.S.	DATE: 9/8/2010
	APPROVED BY: D.R.B.	DRAWING NUMBER: FIGURE 2

FIELD INVESTIGATIVE PROCEDURES

SOIL TEST BORINGS

All boring and sampling operations were conducted in accordance with ASTM Specifications. Borings were advanced into the ground using a hollow-stem auger. At selected intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-spoon sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot was recorded and is designated the "Standard Penetration Resistance". This procedure is described by ASTM Specification D1586. The penetration resistance, when properly evaluated, is an indicator of soil strength, density, and ability to support foundations.

Where required for more quantitative evaluations, a thin-walled tube of 16-gage steel was forced into the ground at the appropriate depth. The disturbance of the soil sample obtained in this manner is relatively low as compared to most other methods of obtaining a sample. The tubing and encased soil are returned to the surface and both ends are sealed with microcrystalline wax to prevent loss of moisture and to reduce sample disturbance while the sample is being transported to our laboratory. This technique is described by ASTM Specification D1587.

KEY TO CLASSIFICATIONS

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	NO. OF BLOWS,N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
	5 - 10	LOOSE
SANDS	11 - 20	FIRM
	21 - 30	VERY FIRM
	31 - 50	DENSE
	OVER 50	VERY DENSE

	NO. OF BLOWS,N	CONSISTENCY
	0 - 2	VERY SOFT
	3 - 4	SOFT
SILTS AND CLAYS	5 - 8	FIRM
	9 - 15	STIFF
	16 - 30	VERY STIFF
	31 - 50	HARD
	OVER 50	VERY HARD

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPES

(Shown in Graphic Log)

- Fill
- Asphalt
- Concrete
- Topsoil
- Gravel
- Sand
- Silt
- Clay
- Organic
- Silty Sand
- Clayey Sand
- Sandy Silt
- Clayey Silt
- Sandy Clay
- Silty Clay
- Partially Weathered Rock
- Cored Rock

WATER LEVELS

(Shown in Water Level Column)

- ▽ = Water Level At Termination of Boring
- ▼ = Water Level Taken After 24 Hours
- ◄ = Loss of Drilling Water
- HC = Hole Cave

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Soft	0 to 2
Soft	3 to 4
Firm	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	31 to 50
Very Hard	Over 50

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50

SAMPLER TYPES

(Shown in Samples Column)

- Shelby Tube
- ☒ Split Spoon
- I Rock Core
- ☐ No Recovery

TERMS

Standard Penetration Resistance - The Number of Blows of 140 lb. Hammer Falling 30 in. Required to Drive 1.4 in. I.D. Split Spoon Sampler 1 Foot. As Specified in ASTM D-1586.

REC - Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100%.

RQD - Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks excluded) Divided by the Total Length of the Core Run Times 100%.

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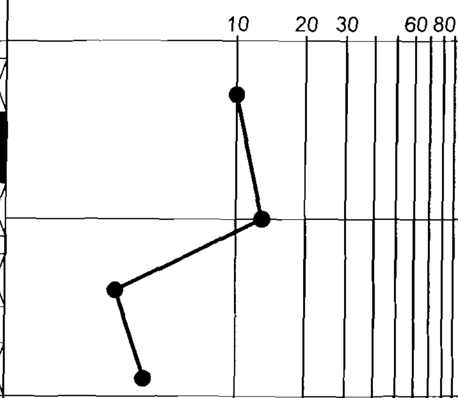
PROJECT: South Industrial Drive Pavement Rehabilitation Erwin, Tennessee S&ME Project No. 1401-10-125	BORING LOG **B-1**

DATE DRILLED: **8/9/10**	ELEVATION: **1648.0**	NOTES: **Elevation estimated from topographic drawing provided by TH&P.**
DRILLING METHOD: **3¼" H.S.A.**	BORING DEPTH: **10.0**	
LOGGED BY: **D.R.B.**	WATER LEVEL: **Dry @ TOB**	
DRILLER: **L. Morrison**	DRILL RIG: **Diedrich D-50**	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft)	N VALUE
		ASPHALT			1			10
		BASESTONE						
		FILL - Stiff Red Brown to Tan Brown Silty **CLAY** (CL) with Brick Fragments						
5		**FILL** - Medium Dense Dark Brown Clayey Fine **SAND** (SM) with Trace Organics		1643.0	2			13
		ALLUVIUM- Very Loose Brown Slightly Micaeous Silty Fine **SAND** (SM)			3			3
10		**ALLUVIUM**- Very Loose Brown Slightly Micaeous Silty Fine **SAND** (SM) with River Cobbles		1638.0	4			4
		BORING TERMINATED AT 10.0 FEET						



BORING LOG GINT.GPJ S&ME.GDT 9/9/10

NOTES:

1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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ENVIRONMENTAL SERVICES

PROJECT: **South Industrial Drive Pavement Rehabilitation** **Erwin, Tennessee** **S&ME Project No. 1401-10-125**	**BORING LOG B-2**

DATE DRILLED: **8/10/10**	ELEVATION: **1643.0**	NOTES: **Elevation estimated from topographic drawing provided by TH&P.**
DRILLING METHOD: **3¼" H.S.A.**	BORING DEPTH: **5.5**	
LOGGED BY: **D.R.B.**	WATER LEVEL: **Dry @ TOB**	
DRILLER: **L. Morrison**	DRILL RIG: **Diedrich D-50**	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft) 10 20 30 60 80	N VALUE
		ASPHALT			1			41
		BASESTONE						
		FILL - Hard Tan Brown Slightly Micaeous Silty **CLAY** (CL)			1			
5		**ALLUVIUM**- Stiff Tan Brown Slightly Micaeous Silty **CLAY** (CL) with River Cobble		1638.0	2			13
		AUGER REFUSAL AT 5.5 FEET						

BORING LOG GINT.GPJ S&ME.GDT 9/9/10

NOTES:

1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

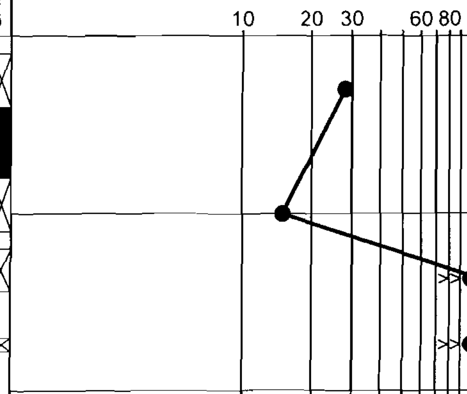
S&ME
ENGINEERING • TESTING
ENVIRONMENTAL SERVICES

PROJECT: South Industrial Drive Pavement Rehabilitation Erwin, Tennessee S&ME Project No. 1401-10-125	BORING LOG B-3

DATE DRILLED: 8/10/10	ELEVATION: 1642.0	NOTES: Elevation estimated from topographic drawing provided by TH&P. * - SPT N-Value likely inflated due to the presence of rock fragments.
DRILLING METHOD: 3¼" H.S.A.	BORING DEPTH: 10.0	
LOGGED BY: D.R.B.	WATER LEVEL: Dry @ TOB	
DRILLER: L. Morrison	DRILL RIG: Diedrich D-50	

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	N VALUE
	ASPHALT			1	28
	BASESTONE			1	
5	**FILL** - Very Stiff Tan Brown Slightly Micaeous Fine Sandy Silty **CLAY** (CL) with River Cobble		1637.0	2	15
	FILL - Stiff Tan Brown Slightly Micaeous Fine Sandy Silty **CLAY** (CL) with Rock Fragments			3*	50/3
	FILL - Very Hard Dark Brown to Tan Brown Slighlty Micaeous Silty **CLAY** and **WOOD**			4*	50/4
10	*BORING TERMINATED AT 10.0 FEET*		1632.0		

STANDARD PENETRATION TEST DATA (blows/ft)



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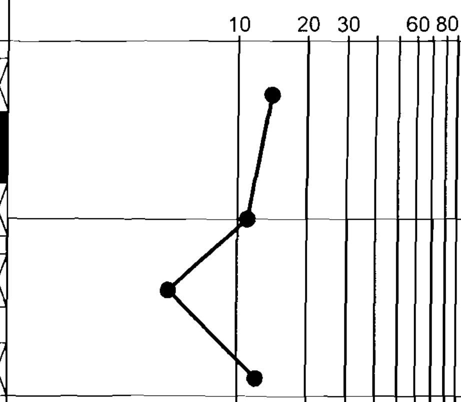
PROJECT: **South Industrial Drive Pavement Rehabilitation** **Erwin, Tennessee** **S&ME Project No. 1401-10-125**	**BORING LOG B-4**

DATE DRILLED: **8/10/10**	ELEVATION: **1638.0**	NOTES: **Elevation estimated from topographic drawing provided by TH&P.**
DRILLING METHOD: **3¼" H.S.A.**	BORING DEPTH: **10.0**	
LOGGED BY: **D.R.B.**	WATER LEVEL: **Dry @ TOB**	
DRILLER: **L. Morrison**	DRILL RIG: **Diedrich D-50**	

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	N VALUE
	ASPHALT			1	14
	BASESTONE			1	
5	**FILL** - Stiff Brown Slightly Micaeous Fine Sandy Silty **CLAY** (CL)		1633.0	2	11
	ALLUVIUM- Firm Tan Brown Slightlly Micaeous Silty **CLAY** (CL)			3	5
10	**ALLUVIUM**- Stiff Tan Slightly Micaeous Fine Sandy Silty **CLAY** (CL)		1628.0	4	12
	BORING TERMINATED AT 10.0 FEET				

STANDARD PENETRATION TEST DATA (blows/ft)



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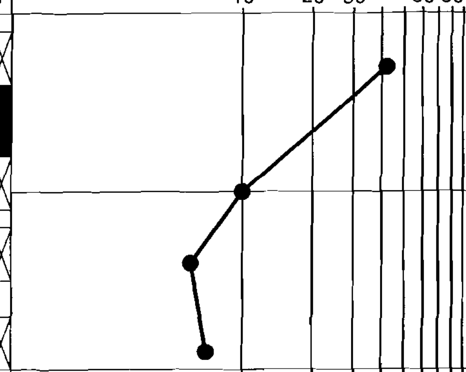
PROJECT: South Industrial Drive Pavement Rehabilitation Erwin, Tennessee S&ME Project No. 1401-10-125	**BORING LOG B-5**

DATE DRILLED: 8/9/10	ELEVATION: 1646.0	NOTES: Elevation estimated from topographic drawing provided by TH&P.
DRILLING METHOD: 3¼" H.S.A.	BORING DEPTH: 10.0	* - SPT N-Value likely inflated due to the presence of rock fragments.
LOGGED BY: D.R.B.	WATER LEVEL: Dry @ TOB	
DRILLER: L. Morrison	DRILL RIG: Diedrich D-50	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. / SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft) 10 20 30 60 80	N VALUE
		ASPHALT			1*		42
		BASESTONE			1		
		FILL - Hard Red Brown to Brown Slightly Micaeous Silty **CLAY** (CL) with Rock Fragments					
5		**FILL** - Stiff Red Brown to Brown Slightly Micaeous Silty **CLAY** (CL) with Plastic		1641.0	2		10
					3		6
		FILL - Loose Brown Slightly Micaeous Silty Fine **SAND** (SM) with Glass and River Cobble			4		7
10		**FILL** - Loose Brown Slightly Micaeous Clayey Silty Fine **SAND** (SM) with River Cobble		1636.0			
		BORING TERMINATED AT 10.0 FEET					



BORING LOG GINT GPJ S&ME GDT 9/9/10

NOTES:

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3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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ENVIRONMENTAL SERVICES

LABORATORY TEST RESULTS

Moisture Content Data
(ASTM D 2216)

S&ME

S&ME Project #: 1401-10-125
Project Name: South Industrial Drive Pavement Rehabilitation
Project Address: Erwin, Tennessee
Client Name: Tysinger Hampton and Partners, Inc.
Client Address: Johnson City, Tennessee
Date: 8/25/2010

Boring No.	Sample No.	Depth	CAN NO.	CAN WT.	INITIAL WT.	FINAL WT.	% MOISTURE
B-1	S-1	0.5' - 2.0'	N18	6.60	45.17	38.19	22.1
B-1	S-2	4.0' - 5.5'	N41	6.62	206.91	188.49	10.1
B-1	S-3	6.0' - 7.5'	N43	6.63	192.10	178.27	8.1
B-1	S-4	8.5' - 10.0'	N4	6.59	155.94	142.98	9.5
B-2	S-1	0.5' - 2.0'	N59	6.72	210.15	188.91	11.7
B-2	S-2	4.0' - 5.5'	N65	6.66	147.74	130.98	13.5
B-3	S-1	0.5' - 2.0'	N72	6.67	203.67	182.77	11.9
B-3	S-2	4.0' - 5.5'	N93	6.80	118.84	107.57	11.2
B-3	S-3	6.0' - 7.5'	N76	6.71	87.43	73.96	20.0
B-3	S-4	8.5' - 10.0'	N86	6.61	103.86	87.05	20.9
B-4	S-1	0.5' - 2.0'	N6	6.64	177.83	146.95	22.0
B-4	S-2	4.0' - 5.5'	N44	6.85	183.94	156.31	18.5
B-4	S-3	6.0' - 7.5'	N55	6.70	64.80	52.70	26.3
B-4	S-4	8.5' - 10.0'	N92	6.82	201.28	169.35	19.6
B-5	S-1	0.5' - 2.0'	N83	6.61	200.11	173.55	15.9
B-5	S-2	4.0' - 5.5'	N64	6.63	217.90	184.85	18.5
B-5	S-3	6.0' - 7.5'	N46	6.65	208.35	174.77	20.0
B-5	S-4	8.5' - 10.0'	N54	6.69	213.72	183.73	16.9

SOIL PLASTICITY TESTS (ATTERBERG LIMITS TESTS)

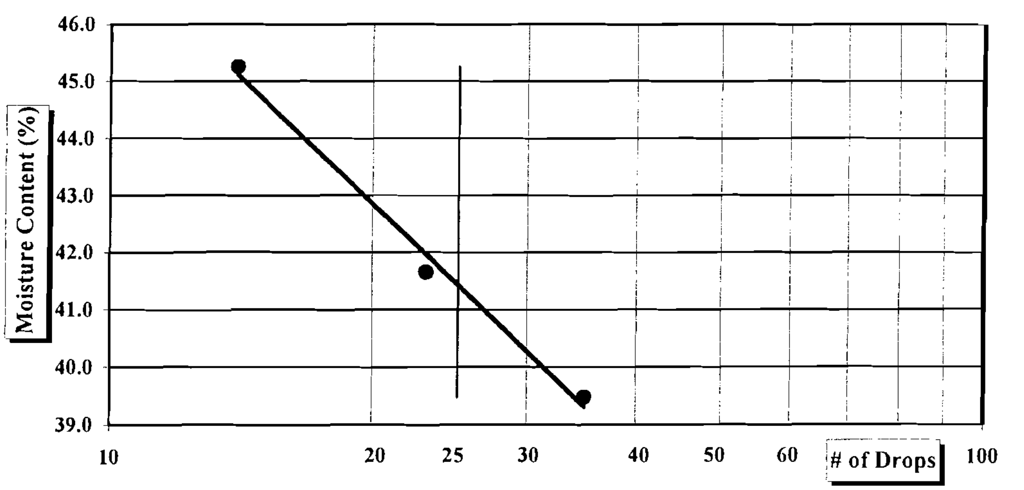
Representative samples of the soils were selected for Atterberg Limits testing to determine the soil plasticity characteristics. The soil's Plastic Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. These soil plasticity characteristics are determined in accordance with ASTM D-4318. The data obtained are presented on the attached Soil Data Summary Sheet.

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: 1401-10-125
Report Date: August 27, 2010
Project Name: South Industrial Drive Pavement Rehabilitation
Test Date(s): August 26, 2010
Client Name: Tysinger Hampton and Partners, Inc.
Client Address: Johnson City, Tennessee
Boring #: B-1
Sample #: S-1
Sample Date: August 9, 2010
Location:
Offset:
Depth: 0.5'-2.0'
Sample Description: Brown Clay - CL

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	C83	C13	C18				C89	15	
A	Tare Weight	22.02	30.49	30.38				22.02	20.07	
B	Wet Soil Weight + A	47.46	58.41	56.86				29.44	27.65	
C	Dry Soil Weight + A	40.26	50.20	48.61				28.26	26.45	
D	Water Weight (B-C)	7.20	8.21	8.25				1.18	1.20	
E	Dry Soil Weight (C-A)	18.24	19.71	18.23				6.24	6.38	
F	% Moisture Content (D/E)*100	39.5	41.7	45.3				18.9%	18.8%	
N	# OF DROPS	35	23	14						
LL	LL = F * FACTOR									
Ave.	Average							18.86%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:
Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation	☐	Dry Preparation	☐	Air Dried	☒
Liquid limit Test:	Multipoint Method	☒	One-point Method	☐		
Classification:	ASTM D 2487	☒	AASHTO M 145	☐		
Liquid limit Test:	ASTM D 4318	☒	AASHTO T 89	☐		
Plastic limit Test:	ASTM D 4318	☒	AASHTO T 90	☐		

NP, Non-Plastic ☐
Liquid Limit 42
Plastic Limit 19
Plastic Index 23
Group Symbol CL

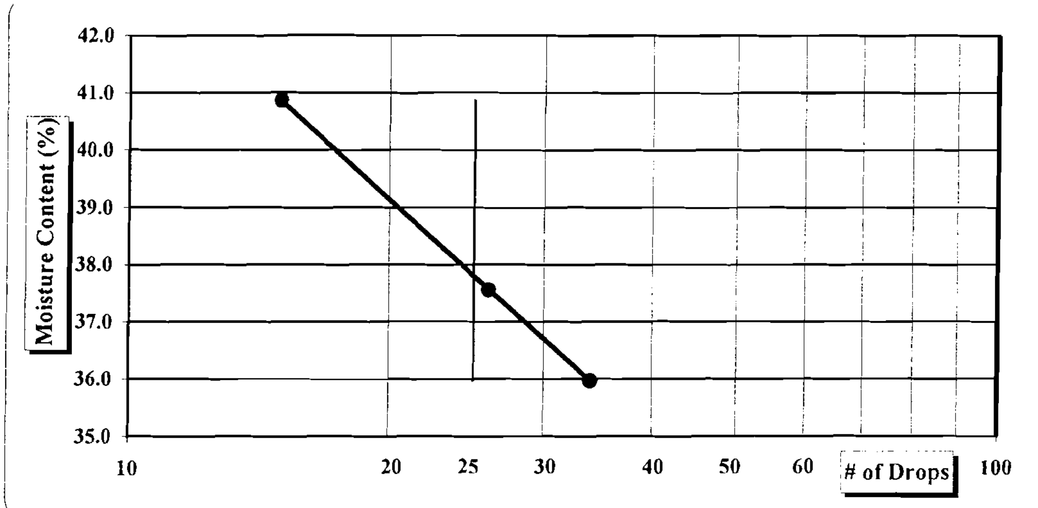
Technician Name / Certification #: Bill Kendall / NICET #90145
Technical Responsibility / Position: Mark Surgenor, Engineering Manager

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: 1401-10-125 Report Date: August 27, 2010
Project Name: South Industrial Drive Pavement Rehabilitation Test Date(s): August 26, 2010
Client Name: Tysinger Hampton and Partners, Inc.
Client Address: Johnson City, Tennessee
Boring #: B-4 Sample #: S-3 Sample Date: August 9, 2010
Location: Offset: Depth: 6.0'-7.5'
Sample Description: Brown Slightly Sandy Clay - CL

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	C2	C32	C64				C10	C23	
A	Tare Weight	30.30	31.10	31.57				30.42	30.47	
B	Wet Soil Weight + A	54.04	56.26	61.94				38.69	37.31	
C	Dry Soil Weight + A	47.76	49.39	53.13				37.31	36.16	
D	Water Weight (B-C)	6.28	6.87	8.81				1.38	1.15	
E	Dry Soil Weight (C-A)	17.46	18.29	21.56				6.89	5.69	
F	% Moisture Content (D/E)*100	36.0	37.6	40.9				20.0%	20.2%	
N	# OF DROPS	34	26	15						
LL	LL = F * FACTOR									
Ave.	Average							20.12%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation	☐	Dry Preparation	☐	Air Dried	☒	NP, Non-Plastic	☐
Liquid limit Test:	Multipoint Method	☒	One-point Method	☐			Liquid Limit	38
Classification:	ASTM D 2487	☒	AASHTO M 145	☐			Plastic Limit	20
Liquid limit Test:	ASTM D 4318	☒	AASHTO T 89	☐			Plastic Index	18
Plastic limit Test:	ASTM D 4318	☒	AASHTO T 90	☐			Group Symbol	CL

Technician Name / Certification #: Bill Kendall / NICET #90145
Technical Responsibility / Position: Mark Surgenor Engineering Manager

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Flexible Structural Design Module

FLEXIBLE PAVEMENT DESIGN
South Industrial Drive Pavement Rehabilitation - Erwin, Tennessee
S&ME Project No.: 1401-10-125

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	1,000,000
Initial Serviceability	4.2
Terminal Serviceability	2
Reliability Level	85 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	6,000 psi
Stage Construction	1
Calculated Design Structural Number	3.46 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	AC Surface	0.42	1	2	24	0.84
2	AC Binder	0.4	1	4	24	1.60
3	Aggregate Base	0.13	1.05	8	24	1.09
Total	-	-	-	14.00	-	3.53

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Rigid Structural Design Module

HEAVY DUTY RIGID PAVEMENT DESIGN
South Industrial Drive Pavement Rehabilitation - Erwin, Tennessee
S&ME Project No.: 1401-10-125

Rigid Structural Design

Pavement Type	JRCP
18-kip ESALs Over Initial Performance Period	1,000,000
Initial Serviceability	4.2
Terminal Serviceability	2
28-day Mean PCC Modulus of Rupture	500 psi
28-day Mean Elastic Modulus of Slab	3,600,000 psi
Mean Effective k-value	100 psi/in
Reliability Level	85 %
Overall Standard Deviation	0.35
Load Transfer Coefficient, J	3.8
Overall Drainage Coefficient, Cd	1
Calculated Design Thickness	8.64 in

Layer Information

Layer	Material Description	Thickness (in)	One Dir Width (ft)
1	JRCP	8.636743	24
2	;Aggregate Base	4	24
Total	-	12.64	-