

**PHASE I - TEST AREA INVESTIGATION REPORT**

**MINE RESEARCH PROJECT GUE-70-14.10**

**PID NO. 18459**

**GUERNSEY CO., OHIO**

**Report to**

**OHIO DEPARTMENT OF TRANSPORTATION**

**Prepared by**

**BBC&M ENGINEERING, INC.**

**DUBLIN, OHIO**

**January 2001**

## TABLE OF CONTENTS

<b>Part I</b>	<b>EXECUTIVE SUMMARY .....</b>	<b>I-1</b>
<b>Part II</b>	<b>RECOMMENDATIONS FOR PHASE II INVESTIGATION .....</b>	<b>II-1</b>
2.1	SUMMARY OF RECOMMENDATIONS .....	II-1
2.2	RECOMMENDATION DETAILS .....	II-4
2.2.1	Groundwater Investigation .....	II-4
2.2.1.1	Horizontal Flow .....	II-4
2.2.1.2	Vertical Flow .....	II-7
2.2.1.3	Groundwater Quality .....	II-8
2.2.1.4	Loadings .....	II-9
2.2.1.5	Field Permeability Testing .....	II-10
2.2.2	Geotechnical Investigation .....	II-10
2.2.2.1	Stratigraphic Identification and Void Detection .....	II-10
2.2.2.2	Field Testing .....	II-11
2.2.2.3	Laboratory Testing .....	II-11
2.2.2.4	Pressure Ratio .....	II-11
2.2.2.5	Piping Potential .....	II-12
2.2.3	Geophysical Investigation .....	II-12
2.2.3.1	Conclusions .....	II-12
2.2.3.2	Recommendations .....	II-13
2.2.4	Integration of Investigations .....	II-15
<b>Part III</b>	<b>INVESTIGATIVE METHODS: PROCEDURES, FINDINGS, AND EVALUATION .....</b>	<b>III-1</b>
3.1	GROUNDWATER INVESTIGATION .....	III-1
3.1.1	Purpose .....	III-1
3.1.2	Regional Hydrogeology .....	III-2
3.1.3	Site Stratigraphy .....	III-4
3.1.4	Monitoring Well Locations .....	III-7
3.1.5	Field Work .....	III-8
3.1.5.1	Water Level Measurements .....	III-8
3.1.5.2	Aquifer Permeability Testing .....	III-8
3.1.5.3	Groundwater Sample Collection and Analysis .....	III-10
3.1.6	Grout Characterization .....	III-11
3.1.6.1	Geochemical Analysis .....	III-11
3.1.6.2	Dissolution Testing .....	III-11
3.1.6.3	Manufacturer Data .....	III-12

3.1.7	Interpretation and Analysis .....	III-12
3.1.7.1	Permeability Determination .....	III-12
3.1.7.2	Groundwater Flow .....	III-15
3.1.7.3	Groundwater Quality .....	III-20
3.1.7.4	Loadings .....	III-22
3.2	GEOTECHNICAL INVESTIGATION .....	III-24
3.2.1	Background Data Review .....	III-24
3.2.2	Field Testing .....	III-24
3.2.3	Laboratory Testing .....	III-25
3.2.4	Stratigraphy Characterization .....	III-26
3.2.4.1	Soils .....	III-26
3.2.4.2	Bedrock .....	III-26
3.2.5	Interpretation and Analysis .....	III-27
3.2.5.1	Pressure Ratios .....	III-27
3.2.5.2	Piping Potential .....	III-28
3.2.5.3	Consolidation Testing .....	III-28
3.2.5.4	Triaxial Testing .....	III-29
3.2.5.5	Effectiveness of Grouting Program .....	III-29
3.2.6	3D Stratigraphic Computer Model .....	III-30
3.3	GEOPHYSICAL INVESTIGATION .....	III-31
3.3.1	Borehole Geophysical .....	III-32
3.3.2	Surface GPR .....	III-32
3.3.3	Side Looking Underground Radar .....	III-33
3.3.4	Cross-Hole GPR .....	III-33
3.3.5	Spectral Analysis of Surface Waves (SASW) .....	III-34
3.3.6	Cross-Hole Seismic Tests .....	III-34
3.3.7	High Resolution Surface Seismic Reflection .....	III-35
3.4	DRILLING, SAMPLING, AND INSTRUMENTATION INSTALLATION .....	III-37
3.4.1	Drilling and Sampling Methods .....	III-37
3.4.2	Geophysical Drilling and Instrumentation Installation .....	III-38
3.4.3	Groundwater Monitoring Well Drilling and Installation .....	III-39
3.4.4	Locations and Elevations .....	III-40
<b>Part IV</b>	<b>COMPARATIVE ANALYSIS .....</b>	<b>IV-1</b>

## APPENDICES

### **Appendix A:** **Maps and Cross Sections**

	<u>Plate No.</u>
Vicinity Map .....	1
Plan of Monitoring Wells .....	2
Site Plan .....	3
Schematic Subsurface Sections .....	4 to 7
Static Water Levels .....	8
Potentiometric Maps .....	9 to 35

### **Appendix B:** **Groundwater Quality Data**

Summary of Chemical Analysis, Average Results for all Events .....	1
Average Concentrations, Graphic Plots of Constituent Concentrations .....	2 to 9
Summaries of Chemical Analysis by Sampling Event .....	10 to 13
Summaries of Chemical Analysis by Parameter .....	14 to 21
Groundwater Sampling Field Sheets .....	22 to 30
Chain of Custody Records .....	31 to 36
Laboratory Reports of Analysis Results .....	37 to 74
Geochemical Analysis, Grout Mix Composition .....	75 to 77
Preliminary Grout Dissolution Test Results .....	78 to 86
Leachate Characterization Data for Conesville Fly Ash Grout Mixtures .....	87
Fly Ash Grout Strength Testing Program Results .....	88 to 91
Grout Mixes .....	92
Material Safety Data Sheet for Coal Ash .....	93 to 94

### **Appendix C:** **Soils Laboratory Test Results**

Summary of Laboratory Test Results .....	1 to 4
Gradation Curves .....	5 to 10
Pressure - Void Ratio Curves .....	11 to 18
Consolidation Time Curves .....	19 to 32
Triaxial Compression Test Results .....	33 to 42

**Appendix D:**  
**Hydrogeologic and Geotechnical Calculations**

Summary of Field Permeability Test Results, Granular and Bedrock Deposits .....	1
Comparison of Permeability Results, Field Test Methods .....	2
Specific Capacity Calculations .....	3 to 4
Permeability Estimates from Specific Capacity .....	5 to 6
Modified Slug Test Permeability Calculations, Aquifer Characteristics .....	7 to 9
Modified Slug Test Data, Plots of Residual Drawdown vs. Time .....	10 to 37
Permeability Calculations from Consolidation Testing, Cohesive Samples .....	38 to 40
Vertical Gradient Calculations .....	41 to 42
Vertical Flow Calculations .....	43 to 45
Horizontal Groundwater Flow Calculations, Darcy Equations .....	46 to 51
Horizontal Groundwater Flow Calculations, Flow Net Methods .....	52 to 53
Horizontal Groundwater Flow Calculations, Open Passageways .....	54
Average Groundwater Loading Increases for Coal Zone .....	55
Groundwater Constituent Loading Calculations .....	56 to 60
Constituent Loading Comparison, Method Comparison Zone Flow to Balanced Flow ..	61 to 67
Dissolution Time Calculation .....	68
Ratio of Buoyant Pressure to Soil Pressure Calculations .....	69 to 74
Piping Potential Calculations .....	75 to 76
Composite Gradation Curves .....	77 to 80

**Appendix E:**  
**Boring Logs -Post Grout**

Explanations of Terms and Symbols used on Boring Logs .....	1 to 2
Boring Logs .....	3 to 128

**Appendix F:**  
**Well Completion Diagrams**

Geophysical Boring Information, Summary of Completion Diagrams .....	1
Monitoring Well Construction Data, Summary of Completion Diagrams .....	2
Geophysical Casing Installation Diagrams .....	3 to 20
Monitoring Well Completion Diagrams .....	21 to 38

**Appendix G:**  
**Historic and Background Data**

Household Water Well Locator Map .....	1
Summary of Household Water Well Data .....	2
Household Water Well Logs .....	3 to 15
Static Water Levels, Old Monitoring Wells .....	16
Plot of Static Water Levels, Coal Zone Old Monitoring Wells .....	17
Plot of Static Water Levels, Shallow Old Monitoring Wells .....	18
Construction Grout Takes, Graphic Plot by Station .....	19
Construction Grout Takes, by Location .....	20 to 25
Boring Logs, Pre-Construction Grouting .....	26 to 59

**Appendix H:**  
**Overview of Geophysical Testing**

Introduction .....	2
Geology of the Drill Holes .....	4

**Appendix I:**  
**Borehole Geophysical Well Logs**

Introduction .....	2
Natural Gamma Ray Geophysical Log .....	3
Resistivity Geophysical Well Log .....	4
Interpretation of Gamma Ray and Resistivity Logs in the Geophysical Test Holes .....	5

**Appendix J:**  
**Surface Ground Penetrating Radar (GPR) Measurements**

Introduction .....	2
Measurements over the Road Surface .....	3
GPR Lines off the Road .....	15
Conclusions on Surface GPR Data .....	17

**Appendix K:**  
**Side Looking GPR (SLUR)**

Introduction ..... 2  
Summary of SLUR Measurements ..... 14

**Appendix L:**  
**Cross-Hole GPR**

Overview of Cross-Hole GPR ..... 2

**Appendix M:**  
**Spectral Analysis of Surface Waves (SASW)**

Introduction ..... 2  
Summary Comments on the SASW Test ..... 9

**Appendix N:**  
**Cross Hole Seismic Studies**  
**(Cross-Well Velocity and Shear Wave Tomography)**

Cross-Hole Seismic Logging (CSL) ..... 2  
Cross-Hole Shear Wave Tomography Measurements ..... 10  
Conclusions on Cross-Hole Seismic Velocity and Tomographic Analysis ..... 26  
References ..... 27

**Appendix O:**  
**Surface Seismic Results**

Surface Seismic Results ..... 2  
Data and Interpretation ..... 5

## **PART 1: EXECUTIVE SUMMARY**

The GUE-70-14.10 Mine Research Project is the investigation of a 2,200-foot-long section of Interstate 70 in Guernsey County, Ohio. Portions of the project area pavement were damaged as a result of mine subsidence. The damaged areas were remediated in 1995 and concerns are present regarding the post-remediation condition of the soils and bedrock beneath the project. The objective of the investigation is to detect the presence and ongoing formation of voids or anomalies beneath the pavement using various geophysical, geotechnical, and groundwater investigative methods.

The project is divided into three phases. The primary purpose of the Phase I Test Area Investigation was to field test and evaluate, on a small scale, the various field and analytical methods proposed for the project during the full site investigation (Phase II). The small test area consisted of the eastbound lanes between Stations 483+00 and 485+00.

The work completed for the test area included an evaluation of methods for a groundwater investigation, geophysical investigations, and geotechnical analysis. For the groundwater investigation wells were installed into the coal zone aquifer and several unconsolidated granular water bearings strata. From the wells, groundwater flow and quality data were obtained and evaluated. The geophysical investigation included the completion of surfaces seismic and ground penetrating radar investigations, down-hole seismic and ground penetrating radar investigations, and down-hole geophysical logging. The geotechnical investigation included the evaluation of stratigraphy, soil and bedrock strengths, and an evaluation of piping potential.

Phase I was successful in identifying the relative merits of the test methods used. Based on the findings of the Test Area Investigation, it is recommended that Phase II of the Investigation be generally conducted as follows:

- 1) A groundwater investigation should be completed to identify the vertical and horizontal flows (direction, gradient, velocity, and quantity) of groundwater within the grouted portion of the coal zone and saturated granular strata above the grouted portion of the mine complex. Beyond the limits of the grout curtain and within the boundaries of the right-of-way, static levels of the coal zone aquifer would be determined in order to verify the direction of flow within the grout curtain, to permit the collection of groundwater quality data for the comparisons of loadings from up-gradient to down-gradient across the grout curtain, and to permit the calculation of groundwater flow through the grout curtain using flow net methods. These investigations will assist in the assessment of ongoing formation of voids below the pavement due to groundwater movement and dissolution of grout.

- 2) Surface seismic and ground penetrating radar geophysical methods would be used to investigate the entire 2,200 feet of roadway. Seismic work would be completed on the north and south shoulders of both the eastbound and west bound lanes. Ground penetrating radar work would be completed in and along the traveled lanes. Lane closure would be required for the radar work.
- 3) Anomalous or unique areas identified during the full site geophysical investigation would be further investigated using additional surface and sub-surface (cross-hole ground penetrating radar and seismic) geophysical methods. This work would attempt to confirm the presence of the anomalies, determine if the anomalous areas extend beneath the pavement, and delineate the vertical and horizontal extent of the areas. Temporary lane closure may be required to complete this work.
- 4) Geotechnical drilling, sampling, testing, and analysis would be performed in the confirmed anomalous areas to further characterize the subsurface conditions. This work would include drilling borings at or near the margins of the anomalies. The work might require temporary closure of traveled lanes.

The details and the actual scope of work for Phase II will be presented in a Work Plan which will be submitted for approval prior to the initiation of the Phase II field work. Alternate methods of field investigation may be considered for implementation in Phase II which were not used during Phase I. Potential alternate methods are briefly discussed in this report. The scope of items actually completed may be limited by budgetary constraints. On this basis, the types of data to be collected and number and extent of anomalous areas to be evaluated will be prioritized. A more complete discussion of applicability, costs, and recommendations as to use of alternate methods will be included within the Work Plan for Phase II.

During Phase I of the investigation, two anomalous areas were identified. Geophysical methods identified a soil “slump” feature beneath the traveled lanes near Station 484+00 (beneath and east of the original subsidence feature from 1995). Geotechnical drilling and sampling identified a depression in the bedrock surface and weak soils north of the eastbound lanes near Station 483+50 (west and north of the original subsidence feature). The conditions below these may be further investigated during Phase II of the Investigation. Lane closure may be required for this work.

The Phase I groundwater investigation determined groundwater flow in the coal zone within the test area to be generally to the northwest beneath the roadway. Horizontal and vertical velocities were generally very low and below values which would permit physical transport of any significant amount of soil material. Constituent concentrations generally increased from up-gradient to down-gradient of the grout curtain; however, high concentrations of constituent minerals and Total Dissolved Solids typically associated with significant dissolution of grout were not observed. Dissolution of the grout curtain is believed to be occurring, but at a rate low enough that the effectiveness of the grouting program as a result of dissolution is not believed to be a significant concern.

It should be noted that the relatively benign findings and recommendations regarding the groundwater investigation are based on the current static water levels and resulting groundwater flows. If ground water levels were to drop due to future mine dewatering, groundwater gradients, soil transport, and dissolution rates of the grout would increase.

**PART 2:**  
**RECOMMENDATIONS FOR PHASE II INVESTIGATION**

**2.1 SUMMARY OF RECOMMENDATIONS**

The intent of the Phase I investigation was to identify successful candidate investigative methods for implementation in Phase II. The decision process for evaluating geotechnical and groundwater investigative methods used in Phase I included the following steps:

- 1) Did the method yield credible data that could be relied upon for evaluation regardless of its applicability to potential pavement failure? If not, the method is not recommended for Phase II; if yes continue to step 2.
- 2) Does the method generally yield data which is useful or applicable for the evaluation of subsurface conditions and/or potential pavement failure? If not, the method is not recommended for Phase II.

Geophysical methods were evaluated relative to the subsurface conditions encountered., the results are presented in the two tables on the following pages.



By their very nature, each geophysical method is affected by different physical properties. Electrical methods respond to contrasts in electrical properties (conductivity/resistivity and permittivity), seismic techniques respond to variations in density and seismic (compressional and shear) velocities. These physical properties are in-turn related indirectly to bulk geotechnical properties of density and porosity. Geophysical methods are necessarily interpretive, since they can be affected by many different physical factors. All of the methods tested were successful in some aspect of the investigation. The methods recommended for use during Phase II are based on primary anticipated targets. The following table is an attempt, for report purposes, to provide a simplistic comparison.

**Response of geophysical methods to different subsurface conditions**

<b>Method</b>	<b>Near surface voids</b>	<b>Strata boundary</b>	<b>Deep voids</b>	<b>other geologic features</b>	<b>water table</b>	<b>slumps under highway</b>
<b>seismic reflection</b>	No, but depends on size of voids	Yes	Depends on size of voids	Yes, channel sands, faults, etc.	Not under most conditions	Yes
<b>gamma ray well logs</b>	Indirectly Large Voids	Yes, sand/ shale/ limestone/ coal	Indirectly, Large voids	Yes, any changes in shale content	No	No
<b>conductivity well logs</b>	Yes, contrast opposite in dry and wet holes	Yes	Yes, contrast opposite in dry and wet holes	Yes	Yes	
<b>SLUR</b>	Yes, depends on the size and location	No	Yes, depends on the size and location	Yes	No	Under the right conditions
<b>SASW</b>	Yes, depends on the size and depth of void	Yes	Yes, depends on the size and depth of void	Yes	No	If large, and velocity related
<b>cross-hole seismic velocity</b>	Yes, if boreholes straddle voids	Yes	Yes, if boreholes straddle voids	Yes	Yes, if P- and S-waves used together	If velocity related
<b>cross-hole seismic tomography</b>	Yes	Yes	Yes, if boreholes straddle voids	Yes, slumps, faults, etc.	Yes, if P- and S-waves used together	Yes
<b>surface GPR</b>	Yes	Shallow	No	Yes	Shallow	Yes, if shallow

## **2.2 RECOMMENDATION DETAILS**

### **2.2.1 Groundwater Investigation**

The findings, conclusions, and recommendations related to the groundwater investigation are based on the current site conditions, including primarily, the measured static levels, calculated gradients, and measured constituent concentrations. Significant changes from the current conditions, such as mine dewatering, could increase the gradients and resulting velocities which would increase the piping potential. Full dewatering of the mine could also cause the introduction of air into the mine, which could cause metals to begin to precipitate out of the water, thus increasing the acidity and dissolution rate of the grout.

#### **2.2.1.1 Horizontal Flows**

The grout curtain appears to be acting as a somewhat porous groundwater dam. During the period of the test area investigation, groundwater flow in the coal zone in the vicinity of the test area was generally southeast to northwest beneath the roadway. Gradients southeast (up-gradient) and northwest (down-gradient) of the grout curtain were relatively flat. Substantially steeper gradients were observed within the grout curtain. The static levels up-gradient (southeast) were consistently 1.5 to 2 feet higher than water levels down-gradient (northwest) of the grout curtain. The steepest gradients measured were those within the grout curtain. These observations are consistent with the grout curtain obstructing groundwater flow, which causes groundwater flow through the grout curtain to be generally perpendicular to the roadway.

Historic measurements of groundwater flow within the coal zone indicate that between June of 1995 and March of 1998, local flow may have been from northwest to southeast (opposite of the direction of flow during the test area investigation). Inadequate data is available to determine the direction of flow prior to June of 1995; based on data collected by ODOT, the reversal in flow appears to have occurred between March and June of 1998. All data collected between June of 1998 and August of 2000 indicates a direction of flow from southeast to northwest for flow within the grouted portion of the mine. At this time, the local flow direction and the cause of the reversal are unknown. It is possible the historic local direction of flow was to the southeast and the reversal during the Spring of 1998 was caused by the formation of a recharge source southeast of the test area and a discharge point northwest of the test area. Alternatively, the historic direction of flow could be to the northwest and it is possible the placement of grout (April to June 1995) caused a localized temporary disruption of the flow directions which returned to the historic direction during the Spring of 1998.

Because this research project is focused on potential impacts to the roadway, it is recommended that Phase II funds be used to continue investigation of conditions in the immediate vicinity of the roadway, rather than broaden the area of investigation to determine conditions which do not have a direct potential impact on the pavements. It is noted that regardless of the regional and local directions of flow beyond the project limits, it is the flow in the immediate proximity of the pavements which are critical and comparisons of data from up-gradient to down-gradient must be made based on the directions of flow at the time and location from which the data was obtained.

#### Coal Zone Inside Grout Curtain

The flow of the groundwater within the grouted portion of the coal zone was well defined by the test methods used. Both the direction and gradient were consistent with the grout acting as a dam within the coal/mine complex. Horizontal velocities were low (1.0 ft/min) and not in a range which would generally permit the physical transport of significant quantities of material; however, because material transport most likely did occur prior to grouting, it is recommended that horizontal flow within the grout curtain be evaluated for Phase II.

#### Coal Zone Across Grout Curtain Margins

The gradient across both the up-gradient (southeast) and down-gradient (northwest) margins of the grout curtain were well defined in the test area. The gradients were generally as expected if the grout curtain is acting as a dam. On the upstream side (Wells P-222A to P-001A), the gradient was generally low (approximately 6 ft/mi). On the downstream side (Wells P-223A to P-227A), the gradient was somewhat steeper (approximately 14 ft/mi). Because the mine is fully inundated, has a potentiometric surface well above the upper surface of the coal, and gradients are relatively low, the potential is low that water discharging from the down-gradient face of the grout curtain is causing substantial erosion (physical removal of material) of the grout as a result of excessive discharge velocities at the exposed face of the grout. The head differential across the margins confirmed the direction of flow within the grout curtain and the differential static level across the grout curtain permits the calculation of flow within the grout curtain using a flow net independent of the wells installed inside the grout curtain. It is therefore recommended that a limited number of paired wells be installed across the grout curtain margin within during Phase II.

## Coal Zone Outside Grout Curtain

Very low gradients are present outside the grout curtain on both the up-gradient and down-gradient sides. These low gradients cause the direction, velocity, travel time, quantity, and loadings calculations to be extremely sensitive to small (0.01') differences of groundwater elevations. Acceptable precision in surveying (+/-0.01') and in water level measurements (+/-0.01') could yield variations in flow directions of 90 to 180 degrees with error in gradients in the 50 to 100 percent range. This precision level can cause the calculation of flow quantities and loadings to have error ranges from 50 to 100 percent. The permeability of the coal zone in the areas outside the grout curtain is also much more variable than inside the grout curtain. The variability is caused by the presence of open passageways, in-place coal, and collapsed passageways, compared to in-place coal, grouted passageways, and grouted collapsed material present within the grout curtain. While flow was determined using Darcy equations, it is believed the gradient sensitivity is unacceptable for making reliable comparisons from up-gradient to down-gradient based on the flow estimates from outside the grout curtain. Because similar conditions are anticipated for the full site, it is not recommended that flow be determined outside the grout curtain using this method for Phase II; an alternate method for estimating flow and calculating loadings which was successful during Phase I and is recommended for Phase II is discussed below.

Groundwater flow calculations indicate that approximately 2,000 gallons per day discharge horizontally through the grout curtain within the test area and less than 1 gallon per day is recharged to the coal zone from the overlying sands. Because of the substantially different quantities, it can be assumed that the water budget should be nearly in balance for horizontal flow and that the quantity of water flowing horizontally into the grout should be nearly equal to the quantity of water flowing through the grout which should be nearly equal to the quantity of water flowing out of the grout (at least in the immediate vicinity of the grout curtain). Because the horizontal flow is well defined within the grout curtain, it is believed that applying that quantity of flow to the areas beyond the grout curtain (both up-gradient and down-gradient) is appropriate for estimating the quantity of flow in these areas and for calculating loadings. The relative direction of flow (toward or away from the grout curtain) can be determined by wells installed within or immediately outside the limits of the grout curtain. It is recommended that quantity of flow outside the grout curtain be determined during Phase II by assuming a balanced water budget, and the relative direction be determined by installing wells immediately beyond the limits of the grout curtain. Gradients of flow beyond the limits of the grout curtain would not be determined. These gradients are not necessary to complete the critical aspects of the investigation (determination of velocity beneath the pavement areas and estimation of dissolution rate of the grout curtain).

Alternative methods for determining velocities not used in Phase I include use of heat pulse flow meters or aqua vision techniques. Depending upon cost and site specific applicability, methods such as these may be used in Phase II.

## Granular Zones

Groundwater flow within the saturated granular zones was adequately defined in Phase I of the investigation. The investigation did not find velocities (or other characteristics) significant enough to conclude that active physical transport of material is taking place within the zones identified. However, because these zones may have played an important role in the transport of material prior to collapse (and potential future failure), it is recommended that granular zones encountered above the grouted portion of the mine be evaluated during Phase II.

### **2.2.1.2 Vertical Flows**

The methods used during Phase I of the investigation for the determination of vertical flow between sand zones, and between sand zones and the coal zone, were adequate to characterize the vertical flow. Vertical flow data did not indicate a high potential that active transport of material through the confining materials was occurring. The gradient from the upper sand to the lower sand is high which indicates a potential for piping. However, because of the low permeability of the cohesive soil between the sand zones, the velocity of flow is low and therefore the potential for active transport of material is also low. Vertical flow can be a significant cause of material loss which could result in collapse. It is recommended that during Phase II, vertical flows be determined where multiple water bearing zones are encountered inside the grout curtain. Because of the distance from the roadway, full inundation of the mine complex, and the low potential for transport as discovered in Phase I, it is not believed that characterization of vertical flows outside the grout curtain are necessary for Phase II.

UngROUTED boreholes were not detected during the Phase I investigation, and it is not known if any exist in the area studied. If ungrouted boreholes are present under the roadway and they are not specifically detected in the Phase II investigation, the methods used in Phase I will not be effective in determining the hazards associated with piping at the specific locations of the ungrouted boreholes. Vertical velocities at the location of ungrouted boreholes could be high enough to cause piping and transport of soil materials.

### 2.2.1.3 Groundwater Quality

#### Coal Zones

Groundwater flowing through the grouted portion of the coal zone comes in contact with a variety of generally randomly distributed materials including, coal, cement, flyash, and shale. As a result of the contacts, spatial variation of the quality of the water may be present. Because of the limited data that is and will be available, statistical analysis of the data is not appropriate and was therefore not proposed for either Phase I or Phase II of the investigation. The comparisons made in Phase I and proposed for Phase II are and comparisons based on simple mean and standard deviation.

A comparison of groundwater quality from the up-gradient (southeast) side of the grout curtain to the down-gradient (northwest) side of the grout curtain can be made based on the groundwater quality data. In general, the data indicates TDS, calcium, sulfates, hardness, and alkalinity concentrations increase as the water moves northwest, but there were exceptions for calcium and hardness at specific sampling events. For all constituents, the concentrations were unexceptional. The ranges of concentrations were generally within the current or proposed EPA Drinking Water Standards and published values for “poor” general household well use. High concentrations of constituent minerals and TDS associated with significant dissolution of grout were not observed. Because the conclusions concerning dissolution will be based upon comparisons of up-gradient to down-gradient wells and because of the potential spatial variability of the wells installed within the grouted portion of the mine, it is recommended that only the wells installed beyond (up-gradient and down-gradient) the grout curtain be sampled for groundwater quality during Phase II.

#### Sand Zones

The groundwater quality of the granular zones was generally well defined during Phase I of the investigation. The primary use of the water quality data collected from the granular zones was to determine if mixing between zones is likely occurring; however, this can also be estimated from horizontal and vertical gradients. The general water quality of the coal and sand zones was determined to be dissimilar. It is believed, therefore, that substantial mixing of the waters is not occurring. On this basis, it is believed that determination of the groundwater quality of the granular zones is not necessary for Phase II.

#### **2.2.1.4 Loadings**

Constituent loadings are calculated based upon calculated quantities of flow and measured concentrations of groundwater quality parameters. Two methods were used to calculate loadings; the difference between the methods used is the procedure used to calculate quantity of flow. One of the methods is referred to as “zone flow”; this approach determines flow within each zone (i.e. coal zone up-gradient, grouted coal zone, and coal zone down-gradient) independently by using Darcy’s equations. The other method is referred to as “balanced flow”; this method determines flow within the grouted portion of the coal zone and applies that quantity of flow both up-gradient and down-gradient of the grout curtain.

Loading for the coal zone was calculated using both methods, and based on the consistency of the results, it is believed that determination of flow based on “balanced flow” is the most representative method for estimating actual site loading changes. As indicated in the water quality discussion, concentrations of most of the groundwater parameters are relatively low both up-gradient (southeast) and down-gradient (northwest) of the grout curtain. However, a comparison of up-gradient loadings to down-gradient loadings indicates relatively large percentage increases for most of the groundwater indicator parameters as water passes through the grout curtain. Total dissolved solids increases 69%; sulfate increases 463%; iron increases 103%; and, alkalinity increases 33%. A net decrease of 11% was calculated for the hardness load.

These changes may indicate that dissolution of the grout curtain is occurring as groundwater moves through the grouted portion of the coal zone. The calculated rate of dissolution, however, is very small. The total dissolved solids load increase is believed to best represent the total net dissolution of the grout placed in the mine. Based on the TDS load increase, it is believed that the grout curtain within the test area is being dissolved at a rate of 3.3 pounds per day (0.017 pounds per linear foot per day). At this rate the grout curtain within the test area will be fully dissolved in 8,900 years. Because dissolution of the grout curtain is occurring, it is recommended that the loading comparisons from up-gradient to down-gradient in the coal zone be completed during Phase II. It is recommended that the loadings be based on constituent concentrations obtained from wells installed outside the grout curtain and that flows be based on flow through the grout curtain (balanced water budget). It is believed that loadings in other zones are not significant regarding potential formation of voids beneath the pavement, and it is therefore not recommended that loadings be calculated for the sand zones or the coal zone within the grout curtain.

### **2.2.1.5 Field Permeability Testing**

Permeability determination is required for calculation of groundwater flow. The field permeability testing (specific capacity and slug tests) generally yielded results which are believed to be in a range typical for the types of deposits present at the site. Therefore, on the whole, it is believed that the field permeability testing was successful and testing should be continued during Phase II. It should be noted that there are limitations to the permeability range which can be evaluated and some wells could not be tested as a result of high transmissivities. Specifically, the pump tests attempted in Well PW-1 were not successful at determining permeability using Theis, Cooper-Jacobs, Chow, or distance drawdown methods because of insufficient induced drawdown. It is believed that PW-1 derives water from a relatively extensive mine void beyond the limits of the grout curtain. Further pump tests are not recommended for Phase II. Two other methods were used to determine permeability: modified slug tests and specific capacity tests. Because specific capacity tests are an estimate of permeability, it is recommended that they not be used unless the transmissivity is so high that slug tests cannot be completed.

## **2.2.2 Geotechnical Investigation**

### **2.2.2.1 Stratigraphic Identification and Void Detection**

The drilling, sampling, and logging procedures used during the Phase I Investigation were adequate to define the naturally deposited stratigraphic units present in the Test Area and should be continued during Phase II. Some difficulty was encountered distinguishing between grout and shale when the borings were advanced into bedrock using a tricone bit (cuttings identification) rather than coring the bedrock. Use of a tricone bit in lieu of coring is recommended for Phase II only during the following circumstances:

- 1) Borings are closely spaced; or
- 2) Primary purpose of boring is void detection.

An alternate methods for qualitative stratigraphic identification and detection of voids and fracture zone is to complete down-hole camera work such as acoustic televiewer in some of the borings. This may be attempted in Phase II of the investigation, if we determine it is more expedient or economical than rock coring.

#### **2.2.2.2 Field Testing**

Field soils strength tests such as Standard Penetration Tests and Hand Penetrometer Measurements were useful in determining the general soil strength characteristics by comparison to published data. It is recommended that these tests be continued during Phase II.

#### **2.2.2.3 Laboratory Testing**

The type and quantity of classification tests completed during Phase I are believed to have been adequate to classify the various stratigraphic units present within the test area and additional testing should be completed as part of Phase II. Laboratory permeability testing is believed to have been adequate and additional testing is recommended during Phase II.

Generally speaking, the naturally deposited cohesive soils at the site are comprised of two lacustrine clay (silty clay) deposits which are intermittently separated by granular deposits. It is believed that adequate triaxial strength testing and consolidation testing were completed during the test area to investigation to characterize these soils. Additional strength and consolidation characterization testing may be necessary during Phase II in different soil types are encountered or during the investigation of anomalous areas.

#### **2.2.2.4 Pressure Ratios**

Pressure ratio is the ratio of the phreatic pressure (buoyant uplift forces) to the total weight of the soil column. As the pressure ratio approaches unity, the soil particles approach a “quick” condition and become buoyant or mobile. Adequate data was collected in the test area investigation so that pressure ratios could be calculated at critical depths. The ratios in the test area are below 0.45 and do not indicate that critical ratios are present. However, because the ratios can be indicative of conditions which may be conducive to soil void formation, it is recommended that pressure ratios be determined during Phase II. Field data necessary to compute ratios is readily available as part of the groundwater investigation.

### **2.2.2.5 Piping Potential**

Laboratory testing and field data were used to determine the piping potential at the granular cohesive soil contact using classical (Terzaghi) filter criteria. The results indicated that the filtering potential of the granular deposits was marginal and that under higher velocity conditions (>1 ft/min), movement of cohesive material into the granular deposits could occur. Vertical gradients between the upper and lower sand are relatively high (0.7 ft/ft). Within the test area, however, the groundwater flow velocities between the sands were low enough (3.7 ft/dy) that piping is not expected. It is recommended that evaluation of the piping potential using the same methods used in Phase I be included as part of the Phase II work.

### **2.2.3 Geophysical Investigation**

During the Fall of 1999 and early Winter of 2000, geophysical surveys were conducted along a 200 foot test section of I-70 . The following methods were tested:

- 1) surface ground penetrating radar (GPR);
- 2) cross-hole GPR;
- 3) high resolution surface shear wave measurements;
- 4) spectral analysis of surface waves (SASW);
- 5) cross- hole seismic shear wave velocity logging (CSL); and,
- 6) cross-hole seismic tomography measurements (CST).

In addition, conductivity and gamma ray measurements were made in the eighteen holes drilled for the cross-hole work. The results of these measurements are given in appendices of this report.

#### **2.2.3.1 Conclusions**

The following conclusions are based primarily on the data quality achieved at this site, and an analysis of the data under the severe conditions imposed by site. Those conditions included severe winter weather, continuous traffic noise, and pre-existing site disturbance caused by earlier drilling and grout in-filling of subsurface voids and slumps. In spite of these hostile conditions, the investigators are confident that several of the methods tested can be combined to provide a good indication of the presence and location of voids and disturbed zones beneath the highway.

- There is no single technique that will unambiguously detect voids over a wide range of depths.
- The subsurface and surface complications at the site are not an overwhelming obstacle to making good geophysical measurements along the highway.
- Surface GPR using both co-pole and cross-pole orientation of antennas can provide an indication of disturbances in the very near surface (less than five feet) on the I-70 site, and can be used to guide drilling searches for voids. The frequency range of these antennas should be 50-100 Mhz (centerband), and the spacing between lines should not be more than 1 ft.
- Surface seismic shear wave reflection measurements can be used to investigate disturbed zones that cannot be detected with surface GPR. A slump region that appears to be associated with a channel sand that cuts into the shale bedrock is clearly seen on the surface seismic shear wave reflection data.
- Seismic shear wave and GPR cross-hole techniques provide detailed information on the subsurface conditions that are indicated as anomalous by the surface GPR and seismic explorations.

### **2.2.3.2 Recommendations**

Based on these conclusions, we recommend the following sequence of surveys to analyze the entire 2200 feet of subsurface underneath the roadbed:

- 1) Seismic shear wave reflection lines should be run as a reconnaissance tool for locating slump zones in the subsurface that might be associated with mine collapse that has propagated to the overburden/bedrock interface. The seismic lines should be run along both sides of eastbound and westbound lanes.

The seismic shear wave cross sections clearly show the slump region in the record sections, including details on the size and orientation of the slumps. The surface seismic shear wave reflection data can also be used to detect interruptions in the stratigraphy that is caused by channel sands, which might also be associated with the mine collapse features. In addition, the low seismic velocities indicate the “slowness”, or “softness” of the slump region. It is further recommended that the seismic data be measured in a manner so that SASW computations can be made from the multiple-geophone arrays used for the seismic shear wave reflection data. These SASW computations will only require that a p-wave source be used in addition to the shear wave source. The SASW measurements are a means of determining shear-wave velocity profiles in the near-surface and will complement the seismic shear wave reflection data, with a minimal increase in cost. Seismic energy sources required to collect

the SASW data in conjunction with the seismic surface shear-wave reflection (SSSWR) data are readily available.

- 2) Surface GPR should be run on the pavement along closely-spaced lines to detect voids in the very near surface. This data can be collected along lines spaced 1 foot apart and run the length of the Project Area.

The GPR data indicates that shallow data can be collected over the pavement without interference by the reinforcing bars. The data in this report are difficult to interpret because the anomalies occur on individual lines, and shapes of objects cannot be determined from individual profiles. Three dimensional data collection (data along very closely spaced lines and displaced in 3D plus color) will enable the anomaly shapes to be determined. Collecting 3D data on the roadway will require closing one lane at a time. It is estimated that the measurements for a single lane will take approximately 6 hours, and a survey of all four lanes will take approximately four days.

- 3) Cross hole geophysical measurements should be run in holes that are drilled to investigate anomalies that are located by the surface seismic reflection and GPR measurements.

For locations, zones or points where techniques used in recommendations 1 and 2 identify potential soft, weak, or void zones in the sub-base, soil drilling and rock coring should be performed to confirm these indications. The borings required for this exploration should be incorporated into cross hole seismic and GPR surveys to further confirm the existence of, quantify weakness of, and identify size and shape of the weak, soft, or void regions.

- 4) Geophysical well logs should be run in each hole drilled. A minimum logging suite of gamma ray, conductivity, and density should be run. A complete suite that includes video, acoustic television-viewer, neutron- thermal-neutron, and seismic velocity would also be useful to further define the hydrogeology in each drill hole.

Geophysical well logs augment conventional engineering sampling methods in many ways. The geophysical well logs provide accurate depth information on geologic and hydro geologic boundaries. The gamma ray log will define the lithologic boundaries at the site in a manner that is different than conventional visual descriptive sampling. The conductivity (resistivity) log will provide a determination of the top and bottom of any voids that are encountered. The density log can give information about the strength of the material that can be used with the cross-hole seismic data to assist the engineers with determining strength parameters for the sub-grade. These strength parameters are also directly related to locating slump features. Geophysical well logs augment conventional sampling methods in many ways, with a minimal increase in operating costs.

#### **2.2.4 Integration of Investigations**

Phase I of the investigation was primarily an evaluation of methods, and as such, each method was generally completed independently of each other and the results compared and integrated at the completion of work. Because Phase II will be focused primarily on site conditions, integration of the investigative methods will be completed as the work progresses so that decisions and evaluations of conditions which will determine potential problem areas can be made based on all appropriate data.

It is the integration of the data which determined the sequencing of the work described in the Executive Summary. The installation of the monitoring wells will generally be completed concurrently with the completion of the surface seismic work. Subsurface information from past drilling at the site and information obtained during the installation of the wells will provide lithologic information needed to fully evaluate the findings of the surface geophysical investigations. The findings of the surface geophysical investigation will focus and direct both the geotechnical evaluation and subsurface geophysical investigations. During the installation of the geophysical wells, both geotechnical and groundwater data will be obtained. This data will be used to help in the interpretation of the subsurface geophysical data. The ultimate evaluation of the site conditions related to potential problems with the roadway will be based on the findings of the groundwater, geophysical, and geotechnical investigations.

**PART 3:**  
**INVESTIGATIVE METHODS:**  
**PROCEDURES, FINDINGS, AND EVALUATION**

**3.1 GROUNDWATER INVESTIGATION**

**3.1.1 Purpose**

The purpose of the groundwater investigation in the test area was to:

- determine the hydrogeologic conditions in and near the test area;
- evaluate conditions unique to the test and project areas;
- evaluate the methods used to characterize the hydrogeologic conditions;
- determine which conditions and characteristics of the hydrogeologic setting are important to potential impacts to the highway; and
- provide recommendations for the groundwater investigation of the entire site.

The conditions investigated included determining:

- horizontal and vertical extent of water-bearing units;
- characteristics of water-bearing units such as permeability and groundwater quality;
- interconnectivity between water-bearing units;
- vertical and horizontal components of flow including gradient, velocity, quantity, and travel times; and,
- constituent loadings for the aquifers encountered.

### **3.1.2 Regional Hydrogeology**

The project area lies east of the City of Cambridge, Ohio within Mud Run Valley in Guernsey County. This area of the state is unglaciated and is part of the Appalachian Plateau. In upland areas, the unconsolidated materials overlying the bedrock generally consists of thin residual soils. In valley bottoms, alluvial material is present and in some areas, lacustrine deposits as a result of glacial ice dams are found. The majority of water supply wells in the area derive groundwater from bedrock aquifers. Regional flow within the bedrock aquifers can be expected to generally follow the dip of the local bedrock, which in the vicinity of the project is to the south-southeast at approximately 30 to 35 feet per mile. The bedrock aquifers, however, are dissected by ravines and valleys which often act as recharge and discharge points. It can be expected that groundwater flow near recharge and discharge points would be toward or away from these features independent of the local bedrock structure. Other conditions which could potentially influence the groundwater flow direction included bedrock facies changes, fracture zones, and variable permeability of the material through which the groundwater flows.

The uppermost bedrock aquifer beneath the interstate highway in the vicinity of the site is the No. 7 Upper Freeport Coal which is the uppermost member of the Pennsylvanian System Allegheny Formation. None of the valleys in the immediate vicinity of the site appear to cut to a low enough elevation into the bedrock to permit discharge to the ground surface from this aquifer. However, the Mud Run Valley may have been deep enough (southwest of the site), prior to being filled with sediments, to permit subsurface discharge from the coal zone. Major recharge sources to the coal zone aquifer may include Salt Fork Lake, located 4.5 miles north of the site and Senecaville Lake, located 6 miles southeast of the site. Based on the local hydrogeology at the test area, Mud Run Valley appears to be a local recharge source for the coal zone aquifer. There are an inadequate number of household water wells completed into the coal zone to confirm the localized direction of groundwater flow. A Location Map for household water wells, logs of household water wells, and a Summary of Household Water Wells are included in Appendix D.

The No. 7 Coal has been extensively mined in the vicinity of the site including areas beneath the interstate highway. Abandoned mine mapping indicates that the mine passageways which lie beneath the valley could be connected with a mine complex which encompasses approximately 2.0 square miles up-dip (northwest) of the roadway. Groundwater flow within the in-place coal can be expected to flow in a laminar manner. Groundwater flow in the abandoned mines or ineffectively grouted portions of the mines which have interconnected void spaces, however, can be expected to generally flow in a manner similar to flow in karst bedrock regions; flow in these areas may exceed velocities normally associated with laminar flow through a porous medium. Groundwater flow through the effectively grouted portions of the mines, can be expected to flow in laminar manner.

According to the Hydrologic Atlas for Ohio, 1991, the area of Ohio where the site is located receives approximately 40 inches of annual rainfall. Of the 40 inches, approximately 15 inches flows overland to streams and the remaining 25 inches either evaporates or infiltrates. Assuming a 10%/90% relationship between evaporation and infiltration and assuming 50% of all infiltration becomes recharge to the mine workings, approximately 305,500 gallons per acre per year reaches the mines annually as groundwater recharge. Using an up-dip mine complex area of 2.0 square miles, this volume equates to 1.07 million gallons per day of recharge.

Historic measurements of groundwater flow within the coal zone indicate that, between June of 1995 and March of 1998, local groundwater flow in the coal zone aquifer may have been from northwest to southeast across the grout curtain (down-dip). The test area investigation found the flow within the 200 foot wide test area to be in the opposite direction. Inadequate data is available to determine the direction of flow prior to June of 1995. Based on data collected by ODOT, the reversal in flow appears to have occurred between March and June of 1998. All data collected between June of 1998 and August of 2000 indicates a local direction of flow within the grouted portion of the mine to be from southeast to northwest. Graphical plots of the groundwater levels from historic Wells P-001A and P-002A are included in Appendix G. The graphs indicate a generally falling static level in Well P-002A (northwest of the grout curtain) and a generally rising level in Well P-001A (southeast of the grout curtain). A comparison of the averages for each well for data collected before and after the Spring of 1998, indicates that the static level in Well P-001A has risen 1.9 feet and the static level in Well P-002A has fallen 1.6 feet.

The historic (pre-grouting) local flow direction, the cause of flow to the southeast between June of 1995 and March of 1998, and the cause of the flow direction reversal, are at this time unknown. It is possible that the natural direction of flow is to the southeast and the reversal during the Spring of 1998 was caused by the formation of either a discharge point northwest of the roadway or a recharge source southeast of the roadway. However, if only one of these events were to occur, it would be expected that the levels on both sides of the grout curtain would change in the same direction, with the level on the same side of the roadway as the occurrence changing at a faster rate than on the side away from the occurrence. Because the levels are changing in different directions (falling northwest and rising southeast) and changing at similar rates, either both a new recharge source and a new discharge source have occurred, or the reversal is due to some other cause. It can be expected that placement of 18,000 cubic yards of grout in a 6-foot-thick aquifer would impact the groundwater flow patterns. This has been confirmed by the variation of permeabilities and gradients of flow found in the test area. Over time, it can also be expected that groundwater flow would return to an equilibrium. Whether the new equilibrium patterns are similar or dissimilar to the historic flow patterns is dependent upon the new flow pathways that are established.

It is possible the data collected prior to the Spring of 1998 represents the natural local flow direction and the post Spring 1998 data represents a delayed impact on the flow direction as result of grout placement. It is also possible the flow direction prior to the Spring of 1998 was already impacted by the newly placed grout and the post Spring 1998 flow direction represents a return of flow to the natural local direction. In the geologic setting in which the site is located, it is unlikely that a natural 180 degree reversal of groundwater flow would occur. It is believed, therefore, that either the June 1995 to March 1998 or the June 1998 to August 2000 flow directions are anomalous and not representative of the post-grouting natural local groundwater flow direction. It is believed that there are two alternatives for determining which is the case:

- 1) Complete long-term monitoring of the static levels in the wells installed along the roadway. If the levels measured between June 1998 and August 2000 are the anomalous measurements, it would be expected that the direction of flow would at some time re-reverse; or,
- 2) Complete a regional groundwater study of the No. 7 Freeport Coal. It is noted that currently there are inadequate household wells in the vicinity of the project which are completed into the coal to complete such a study without the installation of additional wells beyond the project area.

Because this research project is focused on potential impacts to the roadway, it is recommended that funds be used to further investigate conditions in the immediate vicinity of the roadway, and long term monitoring should be completed to determine which direction of flow is anomalous. It is also noted that whichever direction of flow is the historic local direction, comparisons of data from up-gradient to down-gradient must be made based on the direction of flow at the time and location from which the data is collected. A field review could be conducted after substantial Phase II data is collected to attempt to reconcile data with any local field observations.

### **3.1.3 Site Stratigraphy**

The general stratigraphy in and near the test area is depicted graphically on the Schematic Subsurface Sections included in Appendix A. Full descriptions of the stratigraphy encountered at each boring location are included on the logs of the explorations. The borings drilled at the site encountered generally 35 to 40 feet of soil overlying bedrock. For reference in this report, the stratigraphy has been grouped into the following categories:

- 1) Roadway Fill;
- 2) Upper Silty Clay;
- 3) Upper Sand;
- 4) Lower Silty Clay;
- 5) Lower Sand;
- 6) Miscellaneous Sands;
- 7) Uppermost Bedrock; and,
- 8) Coal Zone.

Roadway Fill: The roadway fill is generally comprised of very-stiff to hard silty clay and can be considered to be in good condition. The fill is generally 6 to 8 feet thick, but in some locations is as thick as 12 feet. The base of the fill is typically near elevation 820 msl.

Upper Silty Clay: The upper silty clay extends from near Elevation 820 msl to near Elevation 805 msl. The material generally consists of silty clay (CL) but also contains pockets of clayey silt (ML). The consistency of the material varies both laterally and vertically from very-soft to very-stiff. The stratum generally becomes weaker with depth and is typically medium-stiff to stiff. The stratum has a low granular content (generally less than 15%) and contains thin lenses of silt and fine sand. Based on consolidation testing, the permeability of this material is believed to be near  $5.4 \times 10^{-8}$  cm/sec. Based upon extensive lab data of Ohio silty clays, this permeability value ( $10^{-7}$  to  $10^{-8}$  cm/sec) is a typical value.

Upper Sand: The upper sand deposit is not consistently present within the test area. The approximate lateral extent is shown graphically on the Potentiometric Maps for the Upper Sand included in Appendix A. Where present, the deposit is, on average, 4.6 feet thick and extends from near Elevation 805 msl to 800 msl. The material is generally medium-dense and is typically comprised of 35% sand, 30 % aggregate, and 35 % fines. The material is fully saturated and would be considered a confined water bearing unit. Based on field test results, the permeability of this of the material is believed to be near  $2.2 \times 10^{-2}$  cm/sec.

Lower Silty Clay: The lower silty clay extends from near Elevation 800 msl to near Elevation 795 msl. The material is generally comprised of soft to stiff silty clay (CL). The stratum has a very low granular content (generally less than 5%) and at some locations contains thin seams of silt and fine sand. Based on consolidation testing, the permeability of this material is believed to be near  $1.1 \times 10^{-8}$  cm/sec. Based upon extensive lab data of Ohio silty clays, this permeability value ( $10^{-7}$  to  $10^{-8}$  cm/sec) is a typical value.

Lower Sand: The approximate lateral extent of the lower sand is depicted graphically on the Potentiometric Maps for the Lower Sand included in Appendix A.. The deposit appears to be linear and is generally present beneath most of the test area. The deposit does however “pinch” out beneath the roadway north of the test area as the bedrock surface rises in elevation. The average thickness of the deposit is 7.9 feet and the upper surface of the deposit is generally near Elevation 795 msl and the deposit extends to the bedrock surface near Elevation 785 msl. The deposit is typically medium-dense to dense. The gradation of the material is less consistent than the upper sand, and the material varies from a sand to a gravel; the material, however, typically is comprised of 40% sand, 40% aggregate, and 20% fines. Pockets of cohesive material are present within the deposit, most commonly just above the bedrock surface. The material is fully saturated and would be considered a confined water bearing unit. Based on field test results, the permeability of the material is believed to be near  $2.9 \times 10^{-2}$  cm/sec. The material is most likely a pre-glacial stream channel deposit.

Miscellaneous Sands: The Miscellaneous sand refers to saturated materials which are believed to be limited in lateral extent, not present beneath the pavement areas, or not hydraulically connected to the more extensive sands. Four wells were installed into these types of materials. The largest of these deposits is believed to be a recent alluvial deposit associated with Mud Run south of the test area, two wells were installed into this deposit. The sand, while present within the test area, is not believed to extend beneath the travel lanes (see section D-D’). The material is generally medium-dense and typically consists of 45% sand, 35% gravel, and 20% fines. When groundwater levels are high, the deposit is fully saturated and would be considered a confined water bearing unit; when groundwater levels are low, the deposit becomes unconfined. Based on field test results, the permeability of the material is believed to be near  $4.1 \times 10^{-3}$  cm/sec. The other two wells installed in miscellaneous sands are located north of the westbound lanes. At both of these locations, no granular deposits were encountered; however, saturated cohesive material was present near the soil bedrock contact.

Uppermost Bedrock: The bedrock surface within the test area is generally present near Elevation 785 msl. However, the bedrock surface rises both northwest and southeast of the test area. At the location of Boring P-224 (90 feet southeast of the test area), the bedrock surface is near Elevation 790 msl and at the location of Boring P-225 (320 feet northwest of test area), the bedrock surface is near Elevation 820 msl. A depression is present in the bedrock surface near Station 483+30, 20' Right. The length of the depression parallel to the roadway is approximately 20 feet. The width of the depression perpendicular to the roadway is unknown. The depression is believed to be approximately 10 feet deep (bottom near Elevation 775 msl).

The uppermost bedrock consists of the shales belonging to the lower members of the Glenshaw Formation which is part of the Pennsylvanian Age Connemaugh Group. The bedrock is generally comprised of medium-hard shale which, at some locations, is silty or sandy. The RQD of the bedrock is generally lower than would be expected for the consistency. The low RQD is most likely the result secondary fracturing caused by collapse of the material into the mine present at the site. The Glenshaw formation extends to approximate Elevation 760 msl. The permeability of this type of material can be expected to be  $1.0 \times 10^{-7}$  cm/sec. However, the secondary fracturing could cause the permeability to be higher.

Coal Zone: The Upper Freeport Coal (No.7) was mined at the site. The coal is the uppermost member of the Pennsylvanian Age Allegheny Group. The thickness of the coal varies but is generally 6 to 6.5 feet thick and is underlain by soft shales. The coal is generally present between Elevations 760 and 755 msl. Groundwater is present within the coal zone. The water is confined and the static level is near elevation 809 msl. The coal zone is found generally in two conditions, grouted and ungrouted. Based on field test results, the permeability of the grouted portion is believed to be near  $6.6 \times 10^{-3}$  cm/sec and the permeability of the ungrouted portion is believed to be near  $8.1 \times 10^{-1}$  cm/sec.

Voids were encountered within the coal zone and within the overlying bedrock. The voids were generally small (typically less than 1 foot thick), however, voids as thick as 2 feet were encountered within the grouted portion of project. The voids were most commonly encountered west of Station 484.

### **3.1.4 Monitoring Well Locations**

The primary water bearing zone evaluated was the coal/mine/grout zone. This unit is present in generally two conditions, ungrouted and grouted. The ungrouted portion is divided into two parts, one northwest of the grout curtain, and the other southeast of the grout curtain; resulting in three areas and two contact margins to be evaluated. Well locations were selected in order to determine the characteristics and flow both within (minimum of 3 wells per zone) and between each zone (wells in close proximity inside and outside of the grout curtain on both sides of the grout curtain). Additionally, because saturated granular strata are present in the soils at the site, cluster wells were installed at each location where the granular strata were encountered so that vertical components of flow could also be determined. At least 3 wells were installed in each saturated granular deposit encountered beneath the paved portion of the test area. These wells were installed so that the characteristics of the granular strata could also be determined and evaluated. A map depicting the locations of the wells is presented in Appendix A.

### **3.1.5 Field Work**

#### **3.1.5.1 Water Level Measurements**

The static water levels in the monitoring wells were measured periodically during the investigation. The groundwater levels from which groundwater flow was calculated are summarized in Appendix A. Full sets of water levels (all wells measured during the same day) were obtained during each groundwater sampling event and on two additional dates. The water levels in the wells were measured using an electric tape with markings at each one-hundredth of a foot. The measurements were referenced to the top of the PVC casing which had been surveyed to the nearest one-hundredth of a foot. The static levels, referenced to msl, were determined by subtracting the depth to the water surface from the elevation of the top of the PVC casings. The cumulative potential error due to surveying and water level measurements allows the water level elevations to be accurate to within +/- 0.02 feet. It is believed that these methods were adequate for the data collection for Phase I and it is recommended that the same methods be used in Phase II. Other methods for measuring water levels included wetted tapes, chalk lines, and pressure transducers. Some of these methods may be incorporated into the Phase II work.

#### **3.1.5.2 Aquifer Permeability Testing**

Three methods were used in the field to collect data in an attempt to estimate the hydraulic conductivity (permeability) of the water bearing zones at the site:

- 1) Specific Capacity
- 2) Modified Slug Tests
- 3) Pump Tests

##### **Specific Capacity**

The monitoring wells at the site were developed in-part by hand bailing; hand bailing was also used to purge the wells prior to sampling. Data for the calculation of specific capacity was collected based on the change in the water level during the bailing of the wells. A pumping (bailing) rate was calculated by measuring the quantity of water removed and the time required to remove the water. Generally, between 10 and 30 gallons of water were removed during each bailing event. The induced drawdown was determined by lowering the bailer until the bottom touched the water surface prior to bailing. The rope was then marked. When the last bail of water was raised above the water level, the bailer was again lowered until the bottom touched the water surface and the rope again marked. The distance between the two marks was measured and recorded as the drawdown. Marks on the ropes were used in lieu of electronically measured water levels because of the time required to remove the last bail and insertion of the water level

measuring device (30 to 40 seconds). Recharge to the well during that time lag would cause the measured drawdown to be less than the actual drawdown if electronic devices were used to measure the water levels. It is believed that these methods are adequate for type of tests being completed and it is recommended that they be continued in Phase II.

### Modified Slug Tests

A static water level was measured in the monitoring wells prior to removal of water by hand bailing. Multiple water levels were again measured after bailing had been terminated as the water level returned to a static condition. The post-bailing measured levels were recorded with the amount of time that had passed after bailing was stopped. The difference between the measured levels and the initial static level, prior to bailing, is the residual drawdown. The water levels were made to the nearest one-hundredth of a foot using an electronic tape measure, and the times were measured to the nearest second using a stop watch. Generally, the residual water levels were measured for 10 to 30 minutes after bailing was completed. In several of the wells, the water levels had returned to a static condition prior to the first measurement of residual drawdown (30 to 40 seconds). In several other wells, little or no recovery of the static level was observed during the test period. The methods used are believed to be adequate for the granular deposits and for the grouted portion of the coal zone and it is recommended that they be continued during Phase II for these conditions. The methods were less effective for extensive void area (beyond grout curtain). The methods used may be modified to include displacement (in lieu of bailing) and measurement of water levels using pressure transducers for Phase II.

### Pump Test

Well PW-1 is a 6-inch diameter well which was installed during the construction grouting program. It is believed that the well is completed in the coal zone and that it was used as a water supply source for the construction grouting. During this investigation, a submersible pump was installed in the well, and the well was used as a water supply source for the drilling operations. Two "mini" pump tests were completed in the well in an attempt to provide another means to calculate permeability and in an attempt to determine if interconnections were present between the coal zone and the water bearing sand zones. Static water levels were measured in the pumping well and in nearby monitoring wells completed in both the coal zone and the sand zones. The pump was then run at approximately 10 gallons per minute for several hours. During the pumping, water levels were periodically measured in both the pumping well and the other nearby wells. Approximately 0.02 feet of drawdown was induced in the pumping well and no measurable change was evident in any of the nearby monitoring wells. The data collected was inadequate for the determination of permeability using text book methods; however, permeability could be estimated based on the specific capacity. Because of the small amount of induced drawdown, inter-connections between water-bearing units could not be determined. It is believed that PW-1 is completed in a mine void. Larger capacity pumps would be necessary to complete pump tests in void area. Because the installation of additional large diameter wells and the

completion of pump tests is beyond the scope of the work, it is not recommended that additional pump tests be completed.

### **3.1.5.3 Groundwater Sample Collection and Analysis**

Four groundwater sampling events were completed as part of the test method investigation. The sampling events were completed at approximately 3 week intervals. During the first sampling event, all wells at the site were sampled (18 new wells and 5 existing wells). This set of data provides a background data set for all wells currently at the site. Select representative wells were sampled during the 2<sup>nd</sup>, 3<sup>rd</sup>, and 4<sup>th</sup> sampling events (13 wells sampled per event). The sampled wells included 9 wells completed in the coal zone, 2 wells completed in the lower sand, and 2 wells completed in the upper sand. Wells selected for the 2<sup>nd</sup>, 3<sup>rd</sup>, and 4<sup>th</sup> sampling events were chosen in order to determine the general water quality for each zone. Three wells were sampled for each of the coal zones and two wells were sampled from each of the two primary sand zones (upper and lower). The selection criterion also provided for a vertical comparison of water quality at several cluster well locations. It is believed that the selection of wells for sampling was adequate for the Phase I goals and it is recommended that a similar program be completed for Phase II (sampling all wells one time and sampling select wells during subsequent sampling events).

#### Sample Collection

The groundwater samples were collected using hand bailing methods. New disposable bailers and ropes were used to purge and sample each well and were properly disposed of after use. The personnel purging and sampling the wells wore disposable Nitrile gloves which were changed prior to purging and sampling each well. Prior to collecting samples from the wells, the wells were purged by removing three times the volume of water in the wells by hand bailing. After purging the wells, the samples were collected and placed into the sample containers. All sample containers were supplied by the testing laboratory with the proper preservative already in the containers. It is believed that these procedures are adequate for the completion of Phase II.

#### Field Analysis

The temperature, pH, specific conductance, and total dissolved solids of each water sample was measured in the field using a portable Corning Checkmate M-90 meter. The equipment was thoroughly rinsed with distilled water prior to each measurement. The instrument was calibrated in accordance with the manufacturer's specifications prior to use. The measurement of total dissolved solids in the field is a relative measurement and is based on the measurement of specific conductance. Because of the heavy reliance of total dissolved solids for the grout dissolution calculations, it is recommended that total dissolved solids testing be completed in the laboratory as a direct measurement rather than in the field as a relative measurement.

## Laboratory Analysis

Analysis of parameters which could not be efficiently analyzed in the field, was completed by independent chemical laboratories (Advanced Analytics of Columbus, Ohio and Zande Environmental of Columbus, Ohio). Samples were properly preserved, tracked, and the analysis completed within the appropriate holding times. It is noted that the samples were unfiltered and the metals analysis was, therefore, for total metals rather than dissolved metals. During Phase II of the investigation it is recommended the samples be filtered and the analysis and comparisons be based on the dissolved metals content. Use of dissolved metals content should allow the comparison to be completed independent of well turbidity and possible secondary reactions as a result of the turbidity. Filtering would be completed in the field using a 0.45 micron disposable filter and a peristolic pump.

### **3.1.6 Grout Characterization**

#### **3.1.6.1 Geochemical Analysis**

Two samples of grout were submitted for geochemical analysis by means of X-Ray Diffraction. One of the sample is what appears to be barrier grout (contains sand and aggregate) which was collected from Boring P-221A. The other is a sample of what appears to be production grout (contains sand but does not contain aggregate ) which was collected from Boring P-228A. The analysis was completed in order to attempt to identify the percentages of minerals which comprise the grout. The results of the analysis are included in Appendix B.

#### **3.1.6.2 Dissolution Testing**

Dissolution analysis is currently in-progress for a sample of what appears to be barrier grout (contains aggregate) collected from Boring P-221 A and a sample of what appears to be production grout (contains sand but does not contain aggregate ) collected from Boring P-228A. The analysis is attempting to determine the dissolution rate of the grout samples when immersed in a sample of the up-gradient groundwater (collected from Well P-224A). Because of the slow dissolution rates the test is a long-term analysis and may take 12 to 18 months to complete. Preliminary test results are included in Appendix B.

The testing was completed by collecting and analyzing a water sample (15 gallons) from Well B-224A (up-gradient coal zone well). The water sample was then split into 3 containers. A sample of barrier grout was placed in one container, production grout was placed in a second container, and third container was left containing only the water from Well B-224A. Periodically water samples are collected and analyzed from each container. Any changes in the water chemistry of the container containing only water are subtracted from changes in the water chemistry of the containing samples of the grout so that an estimate of the chemistry changes as a result of the grout can be made. Testing was completed at a 1 week and 1 month after grout immersion (results reported in this report). Sampling and testing is scheduled again at 6 months and 1 year after immersion.

### **3.1.6.3 Manufacturer Data**

The fly ash for the grout used to grout the mine beneath the roadway was provided by the American Electric Power (AEP) Conesville Plant. No records have been located relating to the chemical composition of either the fly ash or grout specifically placed during the grouting of the GUE-70 mine workings. AEP, however, completed fly ash/grout chemical and strength testing as part of a concurrent mine grouting program. The results of the AEP testing are included in Appendix B. The results contained in Appendix B include: leachate characterization for various fly ash grout mixtures; strength vs. time relationships for various cement/flyash/bentonite mixtures (table and graph form); recommended grout mixes based on depth and condition of mine to be grouted; and, a material safety data sheet for coal ash (fly ash, bottom ash, and boiler slag). This data provided by AEP, should be considered generic rather than specific to the materials placed for GUE-70.

### **3.1.7 Interpretation and Analysis**

#### **3.1.7.1 Permeability Determination**

The following average permeabilities were determined based on the field testing completed at the site:

Coal Zone Inside Grout Curtain	$6.6 \times 10^{-3}$ cm/sec
Coal Zone Outside Grout Curtain	$8.1 \times 10^{-1}$ cm/sec
Lower Sand	$2.9 \times 10^{-2}$ cm/sec
Upper Sand	$2.2 \times 10^{-2}$ cm/sec

These values were used for the horizontal groundwater flow calculations. The permeability calculations and summaries of the results of the permeability testing are included in Appendix C.

With the exception of the coal zone outside the grout curtain, the variance of permeabilities measured within each zone was generally within 2 orders of magnitude. Variation for the coal zone outside the grout curtain was in excess of 4 orders of magnitude. The large variation is believed to be the result of the substantially different conditions of the coal zone where it is ungrouted, specifically in-place coal, open passageways, and collapsed roof material. Furthermore, because of limitations in the upper limit field testing can determine, the average permeability of the coal zone outside the grout curtain is most likely higher than the value used for the calculations of groundwater flow. Because of the large variation (lack of homogeneity) of the coal zone outside the grout curtain, it is believed that traditional calculations of groundwater flow using Darcy's equations may not be accurate. It is noted, however, that assuming a constant quantity of flow, the relationship between permeability and gradient should be linear. If a difference of two orders of magnitudes is present between the permeability of the grouted coal zone and the ungrouted coal zone (as indicated by the averages), it is expected that two orders of magnitude of difference would be present between the gradient of the two zones. This is approximately the relationship that was found at the site. On this basis, it is believed that the permeabilities values used for the materials are generally representative of actual site conditions.

It is believed that grouting of the coal zone in the test area has reduced the variation in permeability of the coal zone, in affect homogenizing the material (with regards to permeability). Because of the more consistent permeabilities, it is believed that Darcy's equation can be used to determine groundwater flow in the coal zone within the grout curtain and for flow within the granular deposits. Additionally, the permeabilities of the granular deposits were relatively consistent between the wells and determining flow using Darcy equations is believed to be adequate.

The methods used in the field to determine permeability are described in the following paragraphs.

### Specific Capacity

The permeability of the water-bearing units was estimated from the specific capacity by using the general relationship between Specific Capacity and Transmissivity and the relationship between Transmissivity and Permeability. Specific Capacity can generally be described as the quantity of water which will flow into a well under an induced gradient (drawdown); Transmissivity is the quantity of water which will flow through a unit width of an aquifer under a unit gradient. These values are similar when the efficiency of the wells is taken into account. It should be noted that the values are not equal as the Transmissivity of an aquifer is generally constant at any given point

withing the aquifer and the Specific Capacity will vary over time at the same point at a constant pumping rate. However, the values are sufficiently similar in that the permeability calculated from the estimated Transmissivity can be relied upon to within an order of magnitude. However, because the value of permeability is an estimate, the values were used only when more traditional methods failed to obtain permeability results. It is recommended that specific capacity be used only in this manner during Phase II of the investigation.

One problem associated with the specific capacity test is the generally small drawdowns which can be induced either as a result of the low bailing rates and/or high Transmissivities. However, because of the simplicity of the test, both in the field and the calculation of permeability, it is believed the test is useful as an estimate or for comparison to other data.

### Modified Slug Tests

Permeability was calculated in accordance with the equations detailed by Bauer and Rice (Groundwater, Vol. 27, No. 3, May-June 1989). The test is considered modified because rather than plotting residual drawdown vs. time based water level changes from injecting or removing a “slug”, residual drawdown vs. time is based on water level recoveries from hand bailing or air lifting. Slugs are not used because there is no practical way to remove a slug of water from a 2-inch well and injecting a slug of water (or other fluid) into a 2-inch well which is to be sampled for groundwater quality is not recommended. We have, in the past, attempted to displace a “slug” by lowering and removing a solid object into 2-inch well. This has not been successful, the object tends to cause a “plunger” effect in the well causing water levels to drop rather than rise when the object is lowered and causing water levels to rise rather than fall when the object is removed. During Phase II of the work use of solid “slug” may be incorporated into the work as an alternate method of determining permeability. If solid “slug” are used it may be necessary to record water levels by use of pressure transducers.

The use of data collected from hand bailing or air lifting yields results in the range expected for the materials being tested, and the data tends to plot as predicted by Bauer and Rice (see plots in Appendix D). Problems with the test include small drawdowns and rapid recoveries which limit the range of permeability which can be measured. The test is also time-intensive, both in the field and in the office. However, it is believed that the test does yield valid measurements of permeability where the site conditions permit collection of data.

Both of the tests (specific capacity and slug) described above measure the net permeability of the screened portions of the wells. Both of the tests can be affected by the effects of partial penetration. Partial penetration of the well screen causes water flowing into the screen from above or below the screen to follow a curved path. The effect of the longer curved flow path can increase the recovery time in the well which can cause the calculation of permeability to be slightly lower than the actual permeability. Because the aquifers at the site are relatively thin, most of the wells are nearly fully penetrating, and a single permeability value for each zone based on average test results for each zone was used in the calculation of flows, it is believed that the effects of partial penetration are not significant.

### **3.1.7.2 Groundwater Flow**

#### Horizontal

- Methods

Horizontal groundwater flow (direction, gradient, velocity, quantity, and travel times) was calculated from measured groundwater elevations and calculated permeability values for 3 distinct settings for the coal zone: southeast of the grout curtain; inside the grout curtain; and, northwest of the grout curtain. Additionally, the flows across the margins of the grout curtain were also estimated. At least 3 wells (triangulation) with spacings of approximately 200 feet were used to determine the flow within each area. The gradients of flow across the margins were determined based on the static levels in wells completed just inside of and just outside (approximately 60 feet apart) of both the northwest and southeast margins of the grout curtain. Based on the gradients, permeability, and cross-sectional area, the quantity of flow was determined using Darcy equations.

The flow within the grouted portion of the mine complex was well defined by the methods used, and the gradients of flow across both of the margins were also well defined. The quantity of flow through the grouted portion of the mine was also determined by use of flow net calculations. The primary difference between the method described above and the use of a flow net is the data used to determine the gradient. When using Darcy equations, the gradient is determined by data from within the zone of interest and data from outside that zone is ignored. When using a flow net, the gradient is determined by data collected from outside the zone of interest and the data from inside that zone is ignored. This is believed to be valid for this site because of the extremely flat gradients outside of the grouted portion of the coal zone which indicates that grout curtain is acting as a groundwater dam causing flow through the curtain to be perpendicular to the grout curtain. One advantage of the use of flow nets is that it permits calculation of flow based on 2 wells rather than 3.

The grouting program appears to have reduced the variability of the permeability inside of the grout curtain by somewhat homogenizing the material by filling the passageways and voids within the collapsed material. Some portion of the flow, however, is through likely through ungrouted void spaces, but based on behavior of the static levels in wells completed inside the grout curtain during development and testing (ability to induce drawdown), it is believed that flow through these relatively small void spaces is laminar and use of Darcy equations or flow net equations for calculation of flow is appropriate.

The flow outside of the grout curtain, however, was not well defined. The gradients in both areas outside of the grout curtain were very “flat”. Because of the gradient and the spacing of the wells, the static levels from which the gradient was determined (and therefore direction of flow, velocity, quantity, and travel times) had to be based on static levels with differences of 2 to 3 hundredths of a foot between the wells, which are very near the precision of the measurements. Static levels with this magnitude of difference are not believed to be reliable for accurate estimates of flow. A solution to this problem would be to substantially increase the spacings between the wells so that the difference between the static elevations at the well locations is not as close to the precision of the measurements as it was in Phase I. Depending upon the prioritized objectives for Phase II this may (or may not) be attempted. Another problem for estimating the flow outside of the grout curtain is the wide range of permeabilities of the mine complex. Four or more orders of magnitude of difference can easily be present between in-place coal, areas of roof collapse, and open mine passageways.

For the coal zone outside the grout curtain, flow was also estimated assuming flow through a single open passageway with dimensions of 15 feet by 6 feet. The estimates found that given the gradients evident outside the grout curtain (approximately 1 foot per mile) the quantities were 4 orders of magnitude higher and the velocities were 5 orders of magnitude higher for flow through a single open passageway than flow through the full test area width using Darcy’s equations. Because these values do not correspond with the findings from inside the grout curtain where the flows are well defined, it is believed that open passageway calculations (pipe flow) are not valid for this site.

Other methods for determining of flow include the use of heat pulse flow meters and aqua vision. The biggest advantage to these methods are that flow can be determined from a single well. However, both methods have limitations and depending upon other objectives from the wells and costs may or may not be used as part of the Phase II work.

Potentiometric maps for the Coal Zone, Upper Sand, and Lower Sand were prepared and are present in Appendix A. Two types of potentiometric maps are presented: computer generated and manual interpretation. On the manually generated maps for the coal zone the potentiometric contours have not been connected across the grout curtain. The contours were not connected so that different contour intervals could be used which was necessary due to the significantly different gradients (resulting from different permeabilities) inside and outside the grout curtain (60 to 70 feet/mile inside and 1 to 2 feet per mile outside). It is believed that the manually interpreted maps are a better representation of the actual flow conditions than these computer generated maps. These computer generated maps generally assume homogeneous conditions throughout the aquifer (similar permeability and thickness) and, therefore, tend to smooth the potentiometric contours to as uniform as possible. At the GUE-70 site there are 2 orders of magnitude difference between the permeability of the coal zone inside the grout curtain and outside the grout curtain which is reflected by the data collected but is generally disregarded by the computer generated contours. During Phase II, potentiometric maps will be hand drawn by a hydrogeologist based interpretation of the data.

- Findings

Using Darcy equations, the gradient of flow inside the grout curtain was determined based on the potentiometric surface generated using the static levels in Wells P-001A, P-002A, P-221A, P-223A, and P-228A. When a flow net calculation was completed, the gradient was determined using the static levels in Wells P-227A and Wells P-222A. Both of the methods yielded similar quantities of flow which are detailed in the following table in gallons per day (gpd):

<u>Date</u>	<u>Darcy Calculation</u>	<u>Flow Net Calculation</u>	<u>Percent Difference</u>
1/5/00	1,678	1,886	11%
1/27/00	2,731	2,925	6.6%
2/14/00	2,131	2,347	9.2%
3/6/00	1,944	1,796	-8.2%
3/18/00	1,491	1,437	-3.8%
8/25/00	1,654	1,668	0.8%
<b>Averages</b>	<b>1,939 gpd</b>	<b>2,010 gpd</b>	<b>3.5%</b>

Because the values are similar, it is believed that either method is valid for determining flows within the grouted portion of the mine, and it confirms that flows are well defined for this zone.

Static water levels measured in the coal zone during the test area investigation consistently indicated groundwater levels 1.5 to 2 feet higher southeast of the grout curtain than northwest of the grout curtain. Additionally, flow within the grout curtain was consistently from southeast to northwest. Based on these findings, it is believed, that during the period of the test area investigation, the general direction of flow within the grout curtain in the vicinity of the test area was from southeast to northwest.

Because of the precision at which the static elevations can be determined (+/- 0.02') relative to the static level elevation differences between the wells outside the grout curtain (0.02' to 0.03'), the indicated directions of flow are variable both up-gradient and down-gradient of the grout curtain. For the same reasons, the gradients could be in error, perhaps as much as 50 to 100 percent. This magnitude of potential error would be carried over into the calculation of velocity, travel time, quantity of flow, and loading calculations. The data does conclusively indicate that, on both sides of the grout curtain the gradients are very flat. The problems associated with the similar static elevations in the wells might be alleviated if the spacing between the wells outside the grout curtain were increased from 200 feet to near 1,000 feet. Calculations of horizontal flow are included in Appendix D and the average gradients and velocities are summarized in the following table:

	Gradient (feet/mile)	Direction	Velocity (feet/day)
<u>Coal Zone</u>			
Southeast of Grout	1.4	variable	0.6
Across Southeast Margin	6.2	W-NW	1.4
Inside Grout	56	NW	0.2
Across Northwest Margin	14	W-NW	3.0
Northwest of Grout	0.7	variable	0.3
<u>Lower Sand</u>	18	S-SE	0.3
<u>Upper Sand</u>	212	S-SE	2.5

Groundwater flows within the sand zones encountered were determined using Darcy equations and the same procedures used for the coal zone and were generally successful. Three granular zones were identified near the test area. Flow, however, was determined in only two of the zones, upper sand and low sands. Only two wells were installed into the third zone, miscellaneous sand. The third zone appears to be a recent, near surface, alluvial deposit associated with Mud Creek; the deposit does not appear to extend beneath the pavement. Based on the static levels in the wells installed in the miscellaneous sand, the direction of flow appears to be toward Mud Creek with a gradient of over 135 feet per mile. It is believed that the miscellaneous sand is hydraulically connected to Mud Creek, and the sand acts as a recharge source for Mud Creek.

## Vertical

- Methods

Vertical flow was determined by comparing the static water levels at each cluster well location. The vertical gradient was calculated by dividing the difference between the static levels by the separation distance between the water-bearing units. The gradients are believed to be reliable at all cluster well locations and the flow between the granular zones is also believed to be valid. However, the velocity, quantity, and travel time between the coal zone and the granular zones are not believed to be reliable. The cause of uncertainty is the vertical permeability of the fractured bedrock. Vertical permeability cannot be determined in the field, and laboratory measurements of vertical permeability cannot be relied upon when fracturing is present. Therefore, the calculated vertical flow through the fractured bedrock cannot be relied upon. Vertical gradients however, are important to potential vertical flow and material movement and can be determined by the methods used at the site. It is recommended that they continue to be used during Phase II of the investigation.

UngROUTED boreholes were not detected during the investigation, and it is not known if any exist in the area studied. If ungrouted boreholes are present under the roadway and they are not specifically detected, the methods used will not be effective in determining the hazards association with vertical flow and potential piping at the specific locations of the ungrouted boreholes. Vertical velocities at the location of ungrouted boreholes could be high enough to cause piping and transport of soil material.

- Findings

Calculations of vertical flow were completed within the portion of the test area where pavement is present (200 feet long by 38 feet wide). The calculations are included in Appendix D. The average results are summarized in the following table:

	Gradient (feet/foot)	Velocity (feet/day)	Quantity (gal/day)	Time (years)
Upper Sand to Lower Sand	0.7	3.7	1.5	1,350
Lower Sand to Coal	0.0005	0.018	0.01	63,700

The vertical gradient between the upper sand and lower sand is high which indicates a potential for piping. However, because of the low permeability of the soil between the sand zones, the velocity of flow is low, and therefore, the potential for active transport of material is also low.

Based on the similar static levels and resulting low vertical gradients between the lower sand and the coal zone, it is believed that the two zones are most likely hydraulically connected. The connection is most likely the result of fractures within the bedrock separating the two zones. It should be noted that the groundwater quality data indicates that the zones are not actively mixing; this is most likely due to the lack of a driving head and the physical distance between the two zones. However, this could change if the coal zone were dewatered and a driving head induced.

### 3.1.7.3 Groundwater Quality

Groundwater samples collected for the Test Area Investigation were analyzed for the following parameters:

<u>Field Analysis</u>	<u>Laboratory Analysis</u>	
Temperature	Calcium, total	Hardness
pH	Sulfates, total	Alkalinity
Specific Conductance	Iron, total	
Total Dissolved Solids		

The results of the analysis are summarized in Appendix B.

Spatial variability (particularly in the coal zone) of the water quality is evident. This is to be expected as a result of the variety of chemical compositions of the material the groundwater comes in contact with as it flow through the grout curtain (coal, shale, clay, cement, flyash, and possibly others). With the exception of iron, however, the variability is not large enough to cause the data itself to be unuseable (data was normally distributed). It is noted that the data set is extremely limited and valid statistical comparisons cannot readily be made. This condition (limited data set size per sampling point) will not change during Phase II of the investigation. Because of this limitation detailed statistical analysis was not performed during Phase I and is not recommended for Phase II. The comparisons which were made for Phase I and which are recommended for Phase to are comparisons based on simple averages and standard deviations of the data.

Temporal variability was not evident in the groundwater quality data. However, the sampling covered only a 3-month period, and this duration is generally inadequate to determine temporal variability. Temporal variability can be more fully evaluated during Phase II of the work when sampling event will be completed during each season. However, conclusive temporal variability cannot be determined due to the limited data sets which will be available from each season.

Based on the groundwater quality data collected in and near the study area, it is believed the chemical composition of the two sand zones present in the test area are similar. However, because of the different static water elevations and the different directions of flow, it is believed that the zones are not hydraulically connected. The quality similarity is most likely the result of the water recharging the zones coming in contact with similar types of materials.

Most of the parameters have similar concentrations in the coal zone and the sand zones. However, calcium, iron, and hardness are sufficiently different to conclude that significant mixing of the water is not occurring.

Based on the overall direction of groundwater flow within coal zone (southeast to northwest), a general comparison was made between the water quality of the coal zone southeast and northwest of the grout curtain. The comparison indicates that the following parameters generally increase as water flows through the grout curtain:

- Total Dissolved Solids;
- Specific Conductance;
- Sulphate; and,
- Alkalinity.

It is believed that the increases are the result of water coming in contact with the grout as it flows from southeast to northwest. It is noted that while pH does not show a substantial increase (or decrease) when compared from up-gradient to down-gradient, high values were present in the samples collected from inside the grout curtain. It is believed that these high levels are the result of the water being in immediate contact with the grout materials.

For all constituents, the concentrations were unexceptional. The ranges of concentrations were generally within the current or proposed EPA Drinking Water Standards. While the water in the coal zone, both up-gradient and down-gradient of the grout curtain, would generally be considered “poor”, the aquifer could be developed for household use. The water is not typical of acid mine drainage which is generally characterized by a low pH and high acidity. It is believed that the water does not exhibit these characteristics because of the anaerobic conditions of the coal zone. If the mine were to become dewatered and the water exposed to air, the metals in the water could precipitate causing an increase in acidity and a decrease in the pH more typical of acid mine drainage.

### 3.1.7.4 Loadings

- Methods

Loadings are determined by multiplying the concentration of a groundwater quality parameter by the quantity of flow. The result is the pounds per day of a constituent which is being carried within the groundwater. The results of the calculations completed for Phase I are included in Appendix D. The primary purpose of determining loading was to attempt to determine if the dissolution of the grout curtain is taking place. This was determined by comparing the loadings in the coal zone up-gradient of the grout curtain to the loadings in the coal zone down-gradient of the grout curtain.

Loadings were calculated using two methods: zone flow and balanced flow. Plots for each sampling event for both methods are included in Appendix D. As shown on the graphs, there is higher variability for both the percent increase and pounds increase for the zone flow calculation. It is believed that the variability is the result of inaccurate flows caused by non-precise gradients outside the grout curtain. In some cases, the variability is such that during some sampling events there is a decrease in loadings, while for other events there is an increase in loading for the same parameter when the direction of flow is the same for both events. It would be expected that the actual changes would be similar (at least consistently positive or negative) when the direction of flow is consistent. The balanced flow calculation is much more consistent both in terms of percent change and pounds of change and it is believed that this method yields a better estimation of actual loading changes at the site. It is noted that because the balanced flow calculation tends to yield loading changes which are consistently positive or negative for each parameter and because the zone flow calculation has both positive and negative numbers for each parameter, the average dissolution rate for each parameter is higher (more conservative) for balanced flow compared to zone flow. It is therefore, recommended that the balanced for calculation be used for Phase II of the investigation.

There are several methods which can be used to compare the calculated loadings from up-gradient to down-gradient. These include:

- pooled loads up-gradient compared to pooled loads down-gradient;
- pooled loads up-gradient compared to loads at individual well locations down-gradient; and
- loads at individual well locations up-gradient compared to loads at individual well locations down-gradient.

Because of the relative small size of the test area (200 feet long) it was decided that for the Test Area, a comparison of pooled loads up-gradient to pooled loads down-gradient was the most appropriate comparison to be made. During Phase II of the investigation where the spacing between the wells will be greater, it is anticipated that the comparison will be made from individual well locations up-gradient to individual down-gradient well locations down-gradient.

Loadings calculations were also completed for the upper and lower sand zones and are presented in Appendix D. A comparison of loads from up-gradient to down-gradient, however, was not made because the sand zone water does not flow through substantial quantities or the critical location of grout (coal zone). Because the load within the sand zone is not particularly useful for determining grout dissolution and the loads are not useful for evaluation of piping potential it is believed that determination loadings for the sand zones is not necessary for Phase II of the work.

- Findings

The following table summarizes the average loading increase from up-gradient (southeast) to down-gradient (northwest) for the coal zone using the balanced flow calculation and pooled loading comparison:

	<u>Pounds per Day</u>	<u>Percent</u>
TDS	3.4	69%
Calcium	0.2	23%
Sulfates	2.4	463%
Iron	0.06	103%
Hardness	-0.3	-11%
Alkalinity	1.4	33%

It is believed that the majority of the loadings changes are the result of water coming in contact with the grout, in particular the cement portion of the grout. TDS is believed to best represent the net dissolution rate at the site. Grout records indicate that between 72 feet left and 72 feet right between Station 483+00 and 485+00, approximately 18,000 cubic yards (estimated 10,905,9840 pounds) of grout were placed. Using the loading increase for TDS as the net linear dissolution rate, this quantity of material would require nearly 9,000 years to fully dissolve. It is noted, however, that the dissolution rate is not expected to be constant either spatially or over time.

## **3.2 GEOTECHNICAL INVESTIGATION**

### **3.2.1 Background Data Review**

Various documents were obtained and reviewed for the site evaluation. These documents included well logs, boring logs, maps, previous investigation reports, grout takes, grout and flyash compositions, geologic data, and groundwater data. The information was obtained from ODOT's Central Office, ODOT District 5 Office, ODNR, AEP, and other sources. Site-specific information pertinent to the analysis of the site conditions was converted to electronic data files. These files generally include four fields ( X, Y, Z, and attribute) for information such as grout takes, SPT results, stratigraphic changes, and others. This information was (and will continue to be) inserted into an Arc View GIS 3-D spatial model. This model will be used to aid in understanding site conditions and to direct and focus further investigation.

### **3.2.2 Field Testing**

Four types of geotechnical field tests were completed during the drilling of the borings in the test area investigation:

- 1) Visual Identification (soils and bedrock);
- 2) Standard Penetration Tests (soils)
- 3) Approximation of Unconfined Compressive Strength (soils)
- 4) Determination of Rock Quality Designation (bedrock)

The results of these tests and measurements are shown on the boring logs included in Appendix E.

Standard Penetration Tests are completed by counting the blows per six inch increment as a 140 pound hammer is used to drive a standard split-barrel sampler into the soil. The number of blows per foot is referred to as the N values and is used to approximate the consistency of the soil. The N values have been shown on the Cross-Sections included in Appendix A.

Hand-penetrometer measurements are taken on samples exhibiting cohesion. The resulting "H" value is the approximate unconfined compressive strength of the cohesive portion of the sample.

Rock Quality Designation (RQD) is a measure of the general competency of the bedrock as related to horizontal bedding and fracturing. The RQD is expressed as a percentage and is determined by summing the length of all core pieces with a length equal to or greater the 2 times the diameter of the core and dividing the sum by the total length of the core run. ASTM uses RQD to designate bedrock into categories as follows:

<b>BEDROCK QUALITY</b>	<b>RQD</b>
VERY POOR	0-25
POOR	25-50
FAIR	50-75
GOOD	75-90
EXCELLENT	90-100

It is recommended that all of the field tests described above be continued during Phase II of the investigation.

### **3.2.3 Laboratory Testing**

The soil samples were visually identified in the field and then further analyzed in the laboratory. In the laboratory classification testing including 133 visually identifications, 18 natural moisture contents, 16 Atterberg limit determinations, and 27 gradation analyses were performed on select representative samples. The classification testing was used to confirm the visual identifications completed in the field. The boring logs were modified (if necessary) based on the laboratory testing. Triaxial and consolidation testing were also performed on select representative undisturbed samples. Results of these and other tests permit an evaluation of the strength and compressibility characteristics of the soils present at this site. Soil permeability was determined in accordance with Lambe, Soil Testing for Engineers, 1951, from the results of the consolidation tests.

Two triaxial compression test series were performed to determine the strength and stress-strain relationships of a cylindrical specimen from undisturbed samples of the upper and lower silty clays. Samples are isotopically consolidated and sheared in compression without drainage at a constant rate of axial deformation (strain controlled). Pore pressures are measured to calculate effective stresses. Consolidation testing is used to determine the magnitude and rate of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled stress loading. Consolidation testing was performed on a total of four samples of the cohesive soils.

The results of the laboratory testing are shown on the boring logs and/or are included in Appendix C.

It is believed that the results of the laboratory testing are reliable and that additional classification testing should be completed as part of the Phase II. It is believed that the soils have been adequately characterized for strength and consolidation. Additional triaxial and consolidation testing may be completed for Phase II of the investigation if substantially different soil types are encountered and/or during the investigation of anomalous areas. The permeability of the cohesive soils was determined based on calculations using the consolidation test results. During Phase II it is recommended that laboratory permeability testing be completed on undisturbed samples in order to confirm the permeability values.

### **3.2.4 Stratigraphy Characterization**

#### **3.2.4.1 Soils**

Soils are classified in the field and in the laboratory into different categories based on several characteristics. The characterization is based on consistency, color, material type, and the percentage of minor components. Soils described in this report have been classified generally in accordance with the Unified Soil Classification System; this system has been augmented by the use of special adjectives to designate the approximate percentages of minor soil components. The soil descriptions coupled with the field testing are used to distinguish stratigraphic units. The methods used for the test area are well documented and believed to have adequately categorized the soils at the site and it is recommended that they be continued during Phase II of the project.

#### **3.2.4.2 Bedrock**

Bedrock at the site was identified based on the consistency, color, and rock type. In addition, the descriptions include discussion of bedding, fracturing, and minor inclusions or other modifications. Where cores were obtained, measurements of percent recovery and RQD were also made. It is believed that these parameters are adequate for the classification of bedrock at the site.

In a number of the borings, rock core was not obtained. In these cases, the rock was identified based on the drill cutting returned to the ground surface in the drilling fluid and based on the drill stem behavior as the boring was advanced using the tricone bit. When these procedures were used, less bedrock information could be obtained, the descriptions are briefer, and the stratigraphic changes are determined less accurately. Additionally, distinguishing between the grout (medium-hard gray fines) and shale (medium-hard gray fines) proved difficult. The use of the tricone bit in lieu of rock core does provide adequate information for determining the presence of voids and also expedites the drilling program. In lieu of rock core or cuttings identifications down-hole camera work could be completed during Phase II, specifically acoustic televiewer. While this method may be better able to identify fracture zones, it is less appropriate for determining other characteristics minor components and inclusions, RQD, and other engineering properties than is rock core. A limited amount of acoustic televiewer work may be proposed for Phase II.

### **3.2.5 Interpretation and Analysis**

#### **3.2.5.1 Pressure Ratios**

The ratio between the buoyant pressure and the overlying soil confining pressure was determined at the contact between granular and cohesive soils. The ratio is an indication as to the potential for movement of the soils as a result of uplift pressures. If the pressure ratio meets or exceeds a value of 1, “quick” conditions can result, which facilitate the mixing and migration of materials. The analysis results are believed to be useful for the determination of potential soil migration and formation of soil voids.

Because the migration of materials as a result of excessive ratios is an upward movement, the analysis was completed at the contact between the upper surfaces of water-bearing zones and the overlying confining materials. The highest individual value ratio calculated was 0.43 at the contact between the upper surface of the coal zone and the shale at the location of Well B-222A. This value indicates a low potential for upward migration of materials. The calculation of pressure ratios are included in Appendix D, and the average pressure ratio for each zone is summarized in the following table:

Coal Zone	0.38
Lower Sand	0.28
Upper Sand	0.27
Miscellaneous Sands	0.15

### 3.2.5.2 Piping Potential

The potential movement from cohesive material into granular strata (piping) was completed using filtering capabilities of the materials and the flow velocities. The filtering capabilities were determined by calculating the appropriate filter criteria ratios from gradation curves of the soils. The calculations are included in Appendix D. It is believed that adequate laboratory testing was completed so that the analysis results are valid.

Based on filter criterion, there is a potential for movement of cohesive material into both the upper and lower sand zones. However, because of the current horizontal velocities in the sand zones (upper sand 3.9 feet/day and lower sand 0.35 feet/day), it is believed that substantial soil movement is not likely occurring. If the flow velocities were to substantially increase (measurable in ft/min), piping of the soils could occur.

### 3.2.5.3 Consolidation Testing

Consolidation testing was completed on the two samples of the upper silty clay and on two samples of the lower silty clay. The laboratory test results are included in Appendix C and are summarized in the following table:

	GC-218 <u>8.5' to 9.0'</u>	GC-215 <u>15.0' to 16.9'</u>
<u>Upper Silty Clay</u>		
Calculated Max Past Pressure	5,600 psf	3,200 psf
Existing Overburden Pressure	990 psf	1,870 psf
Over-Consolidation Ratio (OCR)	5.65	1.17
Cc (within overburden load range)	0.04	0.10
	GC-207 <u>33.5' to 34.0'</u>	GC-207 <u>34.0' to 34.4'</u>
<u>Lower Silty Clay</u>		
Calculated Max. Past Pressure	3,600 psf	4,600 psf
Existing Overburden Pressure	3,740 psf	3,740 psf
Over-Consolidation Ratio (OCR)	none	1.23
Cc (within overburden load range)	0.135	0.09

These results indicate that the upper silty clay has been over-consolidated, most likely as a result of material having been deposited and subsequently eroded. The forces which over-consolidated the upper material are believed not to have over-consolidated the lower silty clay because the forces would have had less effect due to the depth to the lower stratum.

### 3.2.5.4 Triaxial Testing

Triaxial strength testing was completed on one sample of the upper silty clay and one sample of the lower silty clay. The laboratory test results are included in Appendix C and are summarized in the following table:

	<u>Upper Silty Clay</u>	<u>Lower Silty Clay</u>
	GC-216	GC-216
	<u>10.0' to 12.0'</u>	<u>33.0' to 32.0'</u>
Phi Angle, Total	37 degrees	21 degrees
Phi Angle, Effective	31 degrees	29 degrees

The results indicate that the upper silty clay has a higher shear strength and lower compressibility than the lower silty clay. These results are consistent with the conclusion that the upper silty clay is over-consolidated.

### 3.2.5.5 Effectiveness of Grouting Program

The test area is defined as the area between Station 483+00 to 485+00 from the centerline south to the right-of-way to the south right-of-way fence (approximately 150 right). Within this area, grout was placed from Station 483+00 to 485+00 from the centerline to 72 feet right. Grout records indicate that within this area 2,340 cubic yards of grout were placed. Assuming a thickness of 6.6 feet for the coal and an extraction rate of 50%, 3,520 cubic yards of void should have been originally present within the area where grout was placed (66% effectiveness). It is noted that, as roof material collapses, it expands at a rate of approximately 10% (fallen material is not as tightly packed as original in-place material). While the total void space does not change, the voids become small somewhat discontinuous pockets rather than generally-continuous open mine rooms. This condition restricts grout from filling all voids but also reduces the need for all voids to be filled due to support gained by the rubble fill. Piping of soils through the resulting bedrock fractures may have served to increase the void space in the soil strata. However, because the soils were most likely redeposited in the void spaces of the collapsed material, the net increase of void space as a result of piping can be expected to be minimal. It is also noted that grout placed at the southern limit of grouting would not have been confined, and it is expected that

grout would have flowed beyond the 72-foot limit of placement. This grout flow would have decreased the effectiveness of the grouting program.

Although confirmation drilling to determine the actual outside limits of the grout placement is not within the scope of this research project, such a confirmation would aid in both the determination of the grouting effectiveness and assessing the potential impact to the roadway as a result of mine collapses beyond the grout limits due to the angle of draw.

The roof of the former mine is near Elevation 760 msl and the ground surface of the eastbound lanes in the test area are near Elevation 827 msl. Angle of draw is a function of the phi angle of the soil and rock material, and could conceivably vary for about 25 to 45 degrees. Assuming a net angle of draw of 35 degrees off the vertical for the soil and bedrock, collapse of the mine roof at 72 feet right could affect the ground surface northward a distance of 47 feet (to 25 feet right of the right of way centerline). The eastbound pavement extends from 26 feet right to 64 feet right, and the travel lanes are present from 29 feet right to 54 feet right. There is, therefore, a potential that mine collapse beyond the limits of grouting could impact the pavement.

### **3.2.6 3D Stratigraphic Computer Model**

Information obtained from the data review, previous drilling, the current drilling, monitoring wells, testing, and some geophysical data has been entered into a three-dimensional computer model of the site. Some of the data in the model includes the following:

- ground surface topography;
- grout takes;
- surfaces contours for upper and lower surfaces of sand layers and the coal zone;
- bedrock surface topography;
- potentiometric surfaces;
- N and RQD values; and,
- Gamma Logs.

The model is a visual representation of some of the characteristics present at the site. It is useful for the identification of conflicting data and potential problem areas. The model is a “work-in-progress” and will continue to be developed in Phase II.

### 3.3 GEOPHYSICAL INVESTIGATION

The overall goal of this research was twofold:

- 1) conduct a detailed geophysical study of a specific 2200 foot long reach of I-70; and,
- 2) perform an evaluation of a variety of geophysical techniques for their appropriateness for predicting potential mine slumps and/or other subsurface weakening processes.

The specific reach of I-70 had been subject to surface depressions and subsequent remediation in the 1995-1997 period and it was the goal of this research to determine whether or not there was a likelihood that further remediation would be necessary. The more general goal was the investigation of state-of-the-art Geophysical Methods for determining the presence of mine caused voids and predicting the potentially detrimental effects of ongoing soil and water movement within and around mines at other highway locations in Ohio. This project offers a unique opportunity for verifying the efficiency of certain field methods for potential use in a predictive mode.

During the Fall of 1999 and early Winter of 2000, geophysical surveys were conducted along a 200 foot test section of I-70. The following methods were tested:

- 1) surface ground penetrating radar (GPR);
- 2) cross-hole GPR;
- 3) high resolution surface shear-wave reflection seismology;
- 4) spectral analysis of surface waves (SASW);
- 5) cross-hole seismic shear wave velocity logging (CSL);
- 6) cross-hole seismic tomography (CST); and,
- 7) down-hole geophysical logging including conductivity and gamma ray measurements.

Full descriptions and results of the investigative methods are presented in Appendices I through O of this report. Summary descriptions of the methods and associated field work are presented in the following sections.

### **3.3.1 Borehole Geophysical**

Borehole geophysical well logs were run in drill holes GC-201 through GC-218, and two other holes that were outside of the test area of immediate interest. The purpose of these geophysical well logs was to provide a continuous record of physical properties in the drill holes, and to provide a basis for establishing the lateral continuity of the geologic units. The two geophysical logs that were run, included the conductivity (resistivity) and the natural gamma-ray logs. Details and findings of the borehole geophysical work are included in Appendix I.

### **3.3.2 Surface GPR**

Surface GPR measurements were made along the berm of the I-70 test section, across the road, along the eastbound lane of the road, and along the northern side of the southern right-of-way boundary. Variations of these surveys included the following:

- 1) the use of two different systems (GSSI and Sensors and Software);
- 2) the use of different center-band frequency antennas (50, 80, 100 Mhz); and,
- 3) the measurement of different polarization components (co-pole and cross-pole).

Measurement on the road consisted of making the measurements with 100 Mhz antennas (centerband, time-domain system) along lines that were perpendicular to the direction at 40 ft. intervals. In addition, three lines 520 ft. long were run along the road surface, with one line located along the center line of the eastbound lanes of traffic, and the other lines along the north and south sides of the eastbound lanes. The line spacing here was too wide to provide a detailed view of the sub-highway conditions, but they did demonstrate the effectiveness of GPR for seeing beneath the highway. The wide line spacing was necessary because of the high volume of traffic on the road, which made it impossible to safely control line location down to the 1 ft (or less) spacing that is necessary for a high resolution 3D survey. The lines in the test area off of the road served to demonstrate the usefulness of 3D GPR. Details and findings of the Surface GPR work are included in Appendix J.

A test area was established off the road surface on the south side of the highway right-of-way to test the seismic and GPR systems under background conditions. Approximately 40 lines of GPR data were measured with 80 Mhz and 50 Mhz antennas over this grid. Three dimensional images of the 50 Mhz data were generated to demonstrate the potential of three dimensional imaging on the road surface.

### **3.3.3 Side Looking Underground Radar**

The side looking underground radar method (SLUR) utilizes conventional GPR equipment run in trenches cut into the ground. The “look-angle” provided by the trenches allows the energy of the antennas to be directed at an angle underground. SLUR measurements are the only way to obtain a directed remote view underneath the highway from surface measurements. However, experiments conducted in this test area indicate that high resolution reflection shear wave surveys may be an alternate method for investigating underneath the highway from the surface. For this project, SLUR was considered for its potential as a supplemental techniques to vertical and slant-hole drilling, and the other surface and borehole techniques. Details and findings of the SLUR methods are included in Appendix K.

SLUR measurements were made in six trenches that were spaced 16, 20, 22, 25, 30, and 32 feet from the edge of the road. Each trench had a side facing the road that sloped at an angle of approximately 45 degrees. The trenches were cut with a road grader and a small bulldozer. SLUR measurements were made with 100 Mhz antennas (co-pole and cross-pole polarization), and 50 Mhz antennas (dual trench and endfire mode). The different frequencies, polarizations and modes were run to test a wide range of possible operational procedures. Since it was necessary to cover the trenches at night, it was only possible to keep two trenches open at the same time. The field operations for making SLUR measurements proved to be labor intensive.

### **3.3.4 Cross-Hole GPR**

Cross hole GPR consists of placing a transmit antenna in one drill hole (TX), and the receiver antenna (RC) in another drill hole, as illustrated by the photograph of field operations that is shown in Figure 1 if Appendix L. The signal received is modified by the material between the drill holes and off-to-the side of the drill holes. The principles are illustrated in Figure L3. The velocity varies with the material between the holes. For example, a void (air space) has a higher velocity than solid rock, or soil, and the arrival times of the transmitted waves would be earlier for waves that passed through a void. In addition, a void would produce refractions and diffractions that would show up later in time on the GPR record than the initial reflection from the void. A void that is located off to the side of a plane between the transmit and receive drill holes would cause the radar wave energy to reflect and will give an anomalous reading that is indicated by coherent reflections on adjacent traces. The vertical location (depth) of the anomalous time, and/or reflection, anomalies is a function of the transmitter and receiver depths with respect to the anomalous object (void, fracture, mine, etc.). Transmitters and receivers are placed at different depths in the drill holes to help locate objects at different depths in the drill holes. The tests on I-70 utilized both fixed and moving transmitter and receiver orientations. Details and findings of the cross-hole GPR work are included in Appendix L.

### **3.3.5 Spectral Analysis of Surface Waves (SASW)**

The SASW method consists of generating surface (Rayleigh) waves at a source point (S) and recording those waves at two receiver points (R1 and R2). A vertical component (P-wave) vibroseis truck was rented as a source, and low frequency geophones connected to a dual channel FFT analyzer was employed to record and analyze the propagating Rayleigh waves. Details and findings of the SASW investigation are included in Appendix M.

### **3.3.6 Cross-Hole Seismic Tests**

Cross-hole seismic tests (CSL) consist of measuring seismic wave travel time between two boreholes of known spacing with the source and receivers at the same elevation. By testing at various depths, we obtain a velocity profile (see Figures N4 to N6 for example). An elastic wave is generated in the source borehole using a mechanical hammer and the time it takes for this wave to arrive at a receiver borehole, as indicated by the voltage output of a geophone, is determined. Velocities of two types of body waves are measured in this way, primary waves (P-waves, compressional waves) and secondary waves (S-waves, shear waves). The secondary waves may be polarized by resolving velocities in the vertical (SV) and horizontal (SH) planes. Different sources and receiver orientations are necessary to separate the two polarities of the shear waves. In an isotropic medium, the shear-wave velocities in both orientations will be the same, while in an anisotropic medium the velocities will be different. This fact can be used to help define the soil and rock for refined site characterization.

Voids will not propagate seismic body waves unless filled with water in which case they will propagate only the P-wave. Loose or soft in-fill material caused by collapse or slumping of materials underground will have very slow seismic wave velocities which are easily distinguished from the velocities of the undisturbed surrounding materials. In the cross-hole test, the arrival of seismic waves across a void or very loose/soft zone cannot be identified while at locations above and below the void, clear seismic wave arrivals will be evident. There are two reasons for the lack of clear seismic wave arrivals across voids or soft areas, i.e., materials with high porosity: 1) the stiffness or moduli of those zones are very low; and, 2) the internal damping is very high. Both of these conditions lead to either no identifiable arrival at the receiver or a very low amplitude (weak signal) at the receiver. In some instances, the weak signal is caused by the seismic energy diffracting around the void or soft zone, but this is also an indication of an anomalous region in the ground. At the lower reaches of the cross-hole tests at the I-70 site, no clear seismic wave arrivals could be detected suggesting void or loose materials, see Fig. N5 where no arrivals were detected below 30 feet while in Fig. N4 clear arrivals were present to a depth of 47 feet.

For seismic wave propagation, the key physical properties of the medium are moduli and density while for GPR the key physical properties are permittivity and electrical conductivity. Consequently, these two geophysical methods measure different yet complementary properties of the ground. Contrasts in type of ground (sand, clay, rock, coal, etc.), in degree of consolidation (porosity, void ratio, specific gravity, water content), and in anomalous geo-features (voids, slumps, collapse, stoping, etc.) Cause changes in the key physical properties that can be identified by one or the other or both cross-hole seismic or GPR. Details and findings of the Cross-Hole Seismic work are included in Appendix H.

Cross-hole seismic tomography (CST) is a collection of methods for determining the seismic velocity and reflection distribution within a volume using multiple combinations of sources and receivers located around the volume. CST uses two or more boreholes, and sometimes the ground surface, for placement of sources and receivers. For CST, the spacing of the holes depends on a number of factors. One primary factor in drill hole spacing is the source strength, or determining the distance that we can receive signals with a high signal-to-noise ratio. Spacing also depends on the target, the availability of boreholes, the drilling budget, and in the case of highway work, the width of the highway.

Measurements are made at each receiver depth position for each source position. This provides multiple “look angles” across the two drill holes, which can allow image reconstruction through a process called tomographic imaging. The goal is to provide one or more two-dimensional sections of seismic properties within the rock-mass volume. CST extends CSL; CSL is equivalent to the zero-offset result (source and receiver depths are the same, hence zero offset) from a CST survey. At the I-070 site, we used a down-hole orbital vibrator (DHOV) source (Cole, 1989, 1997; Hardage, 1992; Dong, 1994) and an array of eight three-component geophones to acquire the data. We recorded signals to nearly 250 Hz across the highway at distances of about 45 to 73 feet. We collected four low-resolution (source and receiver spacing = 1 meter) crossroad surveys and one high-resolution (source and receiver spacing = 1/3 meter) crossroad survey.

### **3.3.7 High Resolution Surface Seismic**

High resolution seismic reflection was also run at the site along two primary lines that were 200 ft. long. The seismic reflection method is well documented in the literature. It has been used successfully for many years in petroleum exploration, and has recently been adapted to shallow (near-surface) investigations. However, the site conditions presented a particular challenge to using the seismic method along a highway. In fact, high resolution seismic reflection data has never been successfully collected along a busy highway for the purpose of investigating very near surface features (structural, geologic, or hydrogeologic). Traffic noise was anticipated as being the primary problem along the highway, so it was decided prior to the surveys that we should use a shear-wave seismic source, which inputs a transverse (polarized) seismic shear wave (vibrator). Also, several geophone orientations were used to take advantage of the polarization characteristics

of the seismic shear waves (vertical, and two horizontal components), which resulted in three component surveys. In addition, shear wave source orientations transverse and parallel to the recording line were used along the profile line that was located adjacent to the road. Multi-fold shooting was conducted, with the source being placed at each geophone location, resulting in 48 fold coverage. A 1 foot geophone spacing was used, which is considered to be a very high resolution spacing. Details and findings of the High Resolution Surface Seismic Work are presented in Appendix O.

## **3.4 DRILLING, SAMPLING, AND INSTRUMENTATION INSTALLATION**

### **3.4.1 Drilling and Sampling Methods**

Maps depicting the locations of the borings completed on and near the site are shown in Appendix A. Logs of explorations drilled during and after the completion of grouting are included in Appendix E. Logs of historic borings (drilled prior to construction grouting in April 1995) are included in Appendix G. In addition to the stratigraphic descriptions, the boring logs contain notes regarding the occurrence of groundwater, cobbles, sampler type, sample recovery, and some test results (limits, gradation, Hand-Penetrometer measurements, Standard Penetration Test (SPT), and Rock Quality Designation (RQD)).

Drilling at the site was completed using truck and all-terrain vehicle mounted drill rigs using auger and fluid rotary drilling methods to advance the borings. Disturbed soil samples were attempted at approximately 5-foot intervals using a 2-inch O.D. split-barrel sampler driven by blows from a 140-pound hammer freely falling 30 inches (Standard Penetration Test). Undisturbed samples were obtained by hydraulically pressing thin-walled tube samplers (Shelby Tubes) into the soil at a constant rate of penetration. Split-barrel samples were identified in the field and preserved in air-tight containers. Undisturbed samples were preserved in the sampling tube by removing cuttings from the ends of the tube and sealing the tube with wax. The undisturbed samples were extruded and identified in the laboratory. Bedrock was cored in selected borings and at select depths using a double-tube NXM core barrel with a diamond rock bit using water as the circulating fluid. Recovered rock cores were identified in the field by geologists from our office and were preserved in compartmented boxes.

In general, the borings were advanced through the soils using hollow-stem augers and advanced into the bedrock using fluid rotary drilling methods. The hollow stem-augers were generally advanced to the bedrock surface. Prior to advancing the borings into the bedrock, the hollow-stem augers were removed and replaced with 6-inch steel casing. The borings were advanced into the bedrock using rotary methods and either water or drilling mud as a circulating fluid. Where borings were cored, the stratigraphy was identified based on the core recovered. At the locations and depths where cores of the bedrock were not obtained, the borings were advanced using tricone bits, and the stratigraphy was identified based on the cuttings returned to the surface in the circulating fluid.

### **3.4.2 Geophysical Drilling and Instrumentation Installation**

The borings which were drilled primarily for geophysical use have been designated as “GC” borings. Geophysical borings were drilled at 18 locations, Borings GC-201 through GC-209 are located on the north side of the eastbound lane and Borings GC-211 through GC-219 are located on the south side of the eastbound lanes. The borings were drilled using 3.25-inch ID hollow-stem augers in soils and 5-7/8-inch tricone bits in the bedrock. An attempt was made to advance the borings to a depth of approximately 10 feet beneath the base of the coal zone. Borings B-202, B-203, B-212, B-213, B-214, however, were terminated short of this depth as a result of fluid circulation loss into voids in the bedrock stratigraphy.

The borings were converted for geophysical use at completion by installing 4-inch diameter, flush-joint, unslotted, PVC casings. The casings were grouted into place using either a neat-cement grout or a 50-50 bentonite-cement grout. The casings were installed and grouted generally by completing the following:

1. Connecting an expansion seal (shale trap) to the casing near the bottom or at an elevation slightly above any voids encountered in the borings into which fluid loss was evident. The expansion seal served to resist upward buoyant forces as the grout solidified and to prevent grout loss into voids;
2. Prior to installing the casings into the borings, approximately 20 gallons of grout was placed into the bottom of the boring using tremie methods;
3. Setting the casing into the boring and displacing the grout in the bottom of the hole;
4. Filling the installed casing with water to compensate for buoyant forces as the grout solidified;
5. Using tremie methods to fill the remainder of the annular space between the casings and the boring wall with grout; and,
6. Cutting off the portion of the casing which extended above the ground surface to near the ground surface and installing a flush man-hole type cover in concrete over the casing.

A summary of completion details and a completion diagram for each boring are included in Appendix F.

### **3.4.3 Groundwater Monitoring Well Drilling and Installation**

The borings which were drilled primarily for groundwater monitoring have been designated as “P” borings. Clusters of wells were installed at 8 locations (Borings P-221 through P-228). At locations where cluster wells were installed, the well completed into the coal zone was designated as the “A” well and subsequent shallower wells were designated as “B” and “C” wells. A total of 18 groundwater monitoring wells were installed. The borings were advanced using 3.25-inch hollow-stem augers in soils and NXM core barrels in the bedrock. The bedrock portion of the wells were reamed using a 5-7/8-inch tricone bit prior to installing the wells. At locations where voids were encountered at the coal zone, the borings were reamed to a depth a few feet above the void using the 5-7/8-inch tricone bit and a 3-1/8-inch tricone bit was used to ream the boring to the final depth. The borings were advanced to a depth of approximately 2 to 3 feet beneath the coal zone.

The shallow off-set wells (“B” and “C” Wells) were installed at locations where saturated sands were encountered above the bedrock surface. The off-set wells were drilled using 4.25-inch I.D. hollow-stem augers, and the wells were installed through the augers.

The wells consist of 2-inch diameter, flush-joint, PVC casings and screens. The well screens are nominal 5-foot lengths with 10 or 20 slot openings, the mid-point of the screen was generally set near the mid-point of the water bearing zone being monitored. The wells were installed by completing the following:

1. Setting the well screen and casing to the appropriate depth;
2. Installing a filter pack of washed quartz sand around the wells screen. The filter packs were installed by pouring the sand from the surface and measuring the level of the sand in the boring using a weighted tape. Borings P-221A and P-225A are completed into mine voids and no filter pack was installed, the grout for these borings was placed above expansion seals;
3. Placing a bentonite seal approximately 2 feet thick above the filter pack, the seal was comprised of 3/8-inch bentonite chips which were poured into the boring from the ground surface; the level of the bentonite was measured using a weighted tape.
4. Filling the remainder of the annular space with bentonite slurry grout (8 mesh Benseal) which was placed using tremmie methods; and,
5. Cutting off the portion of the casing which extended above the ground surface to near the ground surface and installing a flush man-hole type cover in concrete over the casing.

A summary of completion details and a completion diagram for each boring are included in Appendix F.

The man-hole covers for all “A” wells have been painted black, the covers for all of the “B” wells have been painted red, and the covers for all the “C” wells have been painted white. Additionally, brass tags indicating the well designation have been attached to the expansion caps for each well.

The monitoring wells were developed using a combination of air-lifting and hand-bailing. The development was completed in order to remove sediment from the screens and casings and in order to sort the filter packs to optimize flow into the wells. Any well which was air-lifted as part of development was also hand bailed afterward and was not sampled for a period of weeks after the development.

#### **3.4.4 Locations and Elevations**

The boring locations and elevations shown on the borings logs and on the summary sheets are based on the station and offset used during the grouting program completed at the site. The station and off-set have in some cases been converted to a grid system for simplification of computer modeling. This was completed by converting the stationing to an “easting” and the offsets to a “northing”. For example, Station 483+50, 25' left has been converted to East 48,350 North 25 and Station 485+00, 60' right has been converted to East 48,500 North -60.

The PK nail grid installed by ODOT in the test area was used as a reference for the site surveying. The PK nail near Station 483+00, 59' Right was used as the primary site benchmark and presumed to be located at East 48,300.000 North -59.000 and to have an elevation (as provided by ODOT) of 826.720 feet above msl. Additionally, the row of PK nails installed at approximately 59 feet right were assumed to be located at North -59.000. A registered professional surveyor was contracted to determine the locations and elevations of the borings. The field work for the surveying was completed using an all-station instrument, to an accuracy of +/- 0.01 feet.

**Part IV:**  
**COMPARATIVE ANALYSIS**

Groundwater, geotechnical, and geophysical investigations provide complementary pieces of information. Since Phase I was primarily a methods investigation, the objective was to determine how effective the individual methods were in gathering pieces of data. Lessons learned in Phase I for application in Phase II suggest the sequence of investigation needs to be modified so that the individual methods provide more complementary information prior to completion of the field work and to permit better integration of the data collected. The sequence which is believed to be most effective is as follows:

- 1) Review existing stratigraphic and geologic data so that the subsurface conditions can be anticipated and to identify the areas most critical to the investigation.
- 2) Complete surface geophysical investigation of the entire site. This work is intended to screen the site for anomalous subsurface conditions which warrant more detailed investigation and to provide a general characterization of the current stratigraphy. The majority of the work would be completed using seismic and radar methods.
- 3) Concurrently with the seismic screening, install monitoring wells into the coal zone at locations which are believed, based on the review of existing data, to be critical areas. Such areas included areas of high grout takes and/or areas which are believed to be major haulage-ways. At locations where granular zones are encountered, shallow wells would also be installed. The stratigraphic data collected would be used as current condition ground-truth data which will aid in the evaluation of the surface geophysical data collected.
- 4) Evaluate the geophysical data and prioritize any anomalous areas identified.
- 5) Complete down-hole and possibly additional surface geophysical investigations of the prioritized anomalous areas. During the drilling phase of the work, additional granular zone monitoring wells could be installed (if necessary) to define flow in these zones.
- 6) Evaluate the geophysical data an attempt to more fully characterize the nature of the anomalies and attempt to define the three-dimensional extent of the anomalous areas investigated.
- 7) Complete geotechnical investigations of features which are believed to be critical to pavement performance.

It is believed that the sequence described above, including partial evaluation of data prior to the completion of field work, will maximize the complementary nature of the different methods and focus investigative efforts in the most critical areas.

During Phase II of the investigation an attempt will be made to maximize the use of the data collected by applying multiple evaluation techniques to many of the pieces of information collected. Examples of this which were completed in Phase I and will continue to be used in Phase II include the following:

- use of groundwater flow velocities to evaluate piping potential;
- use of static water levels to determine pressure ratios;
- use of consolidation data to determine permeability;
- use of RQD to evaluate potential bedrock flow zones;
- use of geophysical data to determine locate the vertical and horizontal extent of water bearing zones for the groundwater investigation;
- use of geophysical data to identify stratigraphic units; and,
- use of geophysical data to identify areas of geotechnical concern.

Some example geophysical data showing this multiple use are included on the following pages.





