PHASE II
ENVIRONMENTAL SITE ASSESSMENT

PROPERTY ACQUISITION ASSESSMENTS
SELECTED PROPERTIES
FITE ROAD REALIGNMENT PROJECT
SHELBY COUNTY, TN
VOAP # 79-865

for

Shelby County Government
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EXECUTIVE SUMMARY

F&A has conducted a Phase II Environmental Site Assessment (ESA) on portions of six tracts of land in preparation for the acquisition of right-of-way (ROW) for the realignment of Fite Road in northern Shelby County, Tennessee. The Phase II ESA was conducted in accordance with a Phase II Work Plan dated May 22, 2009, and approved by the Tennessee Department of Environment and Conservation (TDEC) by letter dated June 1, 2009. The report on the Phase II ESA was reviewed and amended in April 2012. The screening values used to evaluate the analytical results from the sampling were updated with any available more recently published screening values. The report tables were amended to include the more recent screening values shown in "blue" font numerals and any new or revised findings are discussed within the report using "blue" font text. The figures have been amended to reflect the current engineering typical roadway section details.

During the Phase II ESA, fourteen soil borings were advanced using direct push technology (DPT) in select locations based on planned ROW acquisition and construction activities on and adjacent to the former Chromium Mining and Smelting Corporation (Chromasco) site located at 3328 Fite Road. Soil samples from each boring were submitted for laboratory analysis of cyanide, hexavalent chromium, target analyte list (TAL) metals, and PCBs. Ground water samples were also collected from temporary wells installed in each soil boring during the assessment. Additionally, sediment and surface water samples were collected from six locations in a drainage ditch due to potential runoff from the Chromasco site, and historical PCB impacts to the ditch from the neighboring W.R. Grace facility.

Laboratory analytical results detected cyanide [0.41 parts per million (ppm)] in one soil sample collected from 1-foot below ground surface (bgs) and hexavalent chromium (2.3 ppm) in an additional sample location (1-foot bgs) on the former Chromasco property. Both results are below the EPA Regional Screening Levels (RSLs) for Chemical Contaminants at Superfund Sites (May 20, 2008) of 20,000 ppm and 1,400 ppm, respectively. TAL metals analysis indicated several metals concentrations in all soil samples submitted to the laboratory. Each of the positive results was below the RSL, with the exception of arsenic (RSL: 1.6 ppm), which ranged from below detection limits to 15 ppm. Arsenic (As) concentrations detected in the soil samples are within range of naturally occurring background arsenic levels in the Memphis, TN, area. According to a Tennessee Division of Superfund Survey cited in the "Hazardous Trace Elements in Tennessee Soils and Other Regolith" completed by the TDEC Division of Geology in 2001, the naturally occurring background level of arsenic in Tennessee is 10 ppm. Furthermore, a TDEC Solid Waste Management memo, dated July 18, 2006, stated that natural levels up to 20 ppm of As are not uncommon in Tennessee soils. Therefore, the arsenic concentrations observed during the original Phase II ESA do not appear to be a concern to the subject properties.

Sediment sample analysis indicated cyanide (0.26 ppm) in one sampling location in the existing ditch on the City of Memphis property (Tract No. 7), but no screening level for cyanide is found on the EPA Region 4 Ecological Screening Values (ESVs) for sediment that were used for analysis of sediment samples. Several TAL metals constituents were detected in the sediment

samples; however, all concentrations were below the ESV with the exception of nickel. Two concentrations of nickel were above the ESV of 15.9 ppm, however, the concentrations were within the expected background range for the area.

Laboratory analytical results for cyanide and PCBs were below laboratory detection limits in each of the ground water samples submitted to the laboratory for analysis. Hexavalent chromium (0.012 ppm) was detected in the ground water sample collected from CR-2; however, there is no established Maximum Contaminant Level (MCL) for hexavalent chromium for comparison of this result. Several TAL metal constituents exceeded the MCLs in unfiltered ground water samples analyzed by the laboratory; however, when the sediment was filtered from the ground water, the metal constituents were below the MCLs in each sample collected.

Surface water samples collected from the existing ditch exhibited concentrations of aluminum and iron that exceeded the ESVs, however, these concentrations are believed to be background related levels.

A volatile organic compound (VOC) ground water contamination plume from the W.R. Grace site underlies a portion of the ROW to be acquired for the new Fite Rd. alignment. ROW plans indicate 5.6 acres of the W.R. Grace property (Tract No. 5) will be acquired for permanent ROW. Of this total, approximately 17,000 S.F. overlie the estimated contaminant plume. The presence of dissolved VOC concentrations in the ground water could lead to subsurface vapors that could accumulate in excavation areas during the project. If excavations near the contaminant plume are to be entered into by construction workers during the construction phase, VOC vapor monitoring will be conducted to insure safe conditions are present.

Several former monitoring wells and one current monitoring well associated with a ground water monitoring project on the W.R. Grace site are located within the ROW, slope easement, or construction easement for the Fite Rd. realignment project. Monitoring well MW-E5D is the only currently utilized monitoring well and is located within the construction easement on the south side of the proposed Fite Rd. Construction plans will be specific to insure this monitoring well is protected during the construction phase of the project. The construction plans will also specify to inspect the closed monitoring wells when encountered during construction to insure the grout is not compromised thus opening a pathway to the subsurface.

The data suggests that no widespread contamination has been identified in the selected soil, sediment, ground water, and surface water samples. Shelby County Government will seek written concurrence of these findings from the Tennessee Department of Environment and Conservation prior to property acquisitions.

1.0 INTRODUCTION

Fisher & Arnold Environmental (F&A), A Division of Fisher & Arnold Inc., was retained by Shelby County Government to conduct a Phase II Environmental Site Assessment (ESA) on portions of six tracts of land in preparation for the acquisition of right-of-way (ROW) for the realignment of Fite Road in northern Shelby County, Tennessee. The Phase II was conducted based on findings from several reports prepared by F&A for the proposed new ROW, including:

Categorical Exclusion for the Realignment of Fite Road	August 20, 2007
Phase I Environmental Site Assessments (ASTM 1527-05):	
W.R. Grace (Tract No. 5)	June 30, 2009
City of Memphis (Tract No. 7)	October 28, 2009
Timminco Corporation (Tract No. 9)	July 27, 2009
Transaction Screen Process (ASTM 1528-06)	
City of Memphis MLGW Substation (Tract No. 8)	November 3, 2009
Memphis Ready Mix (Tract No. 10)	November 2, 2009
Jimmy R. Grinder (Tract No. 11)	July 23, 2009

The Categorical Exclusion that was approved by FHWA and TDOT for the realignment project identified two former Superfund sites in and adjacent to the project corridor, including the former Chromium Mining and Smelting Corporation (Chromasco) site at 3328 Fite Road and W.R. Grace & Co. site located at 5709 Old Millington Road.

The new Fite Road ROW will require the purchase of approximately 1.05 acre from the former Chromasco property (Tract No. 9, currently owned by Timminco Corp.) located at 3328 Fite Road, and will require the purchase of approximately 5.6 acres from the W.R. Grace & Co. site formerly located at 5790 Old Millington Road. The facility at 5790 Old Millington Road is now Arcadian Fertilizer; however, the parcel to be acquired for the construction of Fite Road remains owned by W.R. Grace & Co. The Fite Road realignment project and new ROW acquisition areas are shown on Figure 1 in Appendix A.

The Phase II ESA was conducted in accordance with the Phase II Work Plan dated May 22, 2009, and approved by the Tennessee Department of Environment and Conservation (TDEC) by letter dated June 1, 2009.

1.1 Limiting Conditions

This assessment presents Fisher & Arnold's professional interpretation and judgment of the existing site conditions based on the information gathered. Professional judgments expressed herein are based on currently available facts within the limits of the mutually agreed upon scope of work, budget, and schedule, which are not intended to be exhaustive in scope. Fisher & Arnold

accepts no liability for hidden or unknown conditions. Fisher & Arnold's work was performed in accordance with generally accepted environmental investigative procedures and in accordance with a May 22, 2009, work plan approved by TDEC. It is Fisher & Arnold's specific intent that the conclusions and recommendations presented herein be used as guidance and not necessarily as a firm course of action, except where explicitly stated as such. We make no warranties, expressed or implied, including without limitation, warranties as to marketability or fitness for a particular purpose. The information provided in this report is not to be construed as legal advice.

The project timeline required development of the Phase II Work Plan prior to completion of the preliminary environmental studies for the properties affected by the realignment of Fite Road. Assessment information noted in the work plan was believed to reflect all Recognized Environmental Conditions (RECs) associated with the properties; however, the Work Plan committed to adjusting the scope of work if preliminary findings warranted. The Phase I ESA completed for the City of Memphis property (No. 7) identified a Historic REC pertaining to evidence in aerial photography of structures on the east side of the property and general land disturbance of the portion of the property north of the creek. Representatives of the Owner did not make themselves available for interview and these conditions could not be adequately evaluated. As such, further evaluation of the subsurface in areas of the property to be acquired for Fite Road ROW were recommended to address the potential for environmental impairment. The limited scope and timeline for completion of the the Phase II ESA did not allow for additional sampling activities, thus, this is considered a limiting condition to the findings of this report.

1.2 User Reliance

Reliance or use of this report by anyone other than Shelby County Government, for whom it was prepared, is prohibited. Reliance or use by any third party of the report does not make said party a beneficiary to Fisher & Arnold's contract with Shelby County Government. Any such unauthorized reliance on or use of this report including any of its information or conclusions will be at the third party's risk. No warranties or representation expressed or implied in this report are made to any third party.

2.0 BACKGROUND

2.1 Chromasco

The Chromasco site is a former State Superfund site that is comprised of 92 acres located along the north ROW of Fite Road. According to the February 2001 Record of Decision (ROD), the Chromasco plant produced ferrochrome alloy, ferrosilicon alloy, and ferromanganese alloy from the 1950s until 1984. The site is home to a 6-acre industrial landfill, former sludge lagoons, and buildings/slabs

used in the production of alloys. Process material and slag reportedly remain on-site in large piles, in the landfill, and as backfill for site grading.

The ROD states the landfill contains ferrosilicate slag, ferrochrome slag, ferromanganese slag, ferrosilicate ladle cleanings, low carbon ferrochrome slag, and ferrochrome air pollution control slag along with various construction debris. The landfill was reportedly covered with 2 feet of low permeability clay.

Arsenic, chromium (total and hexavalent), and lead (total) impacts to ground water, soil, and surface water have been documented at the site. Soil samples collected from the site have exhibited relatively elevated levels of cadmium, chromium, copper, cyanide, manganese, lead, nickel, zinc, and PCBs, and surface water samples collected from a centrally located east-west drainage ditch at the facility exhibited elevated levels of chromium. The east-west drainage ditch discharges to a secondary drainage channel along the western property line that is parallel to the railroad tracks.

A Phase III Site Assessment dated May 1988 showed the process water outflow at the secondary drainage ditch along the western property line. This secondary drainage discharges to a drainage ditch located on the southwestern adjacent City of Memphis property that conveys water to the Loosahatchie River. Information reviewed for the site indicates the facility discharge permit was revoked by the Tennessee Department of Public Health in 1971 due to "inadequately treated sewer discharge." Additional permits were denied due to chromium concentrations [(up to 8.0 parts per million (ppm)] until 1974 when an on-site treatment facility was constructed. Additional permit violations were observed for hexavalent chromium in 1976 (0.25 ppm), 1977 (0.10, 0.33, and 0.08 ppm), 1978 (0.44 ppm), and 1979 (0.54 ppm).

In addition to the process water discharge and potential storm water impacts to the drainage ditch, many of the documented soil impacts were exhibited in shallow soils that have the potential to erode and discharge sediment into the drainage ditches.

2.2 W.R. Grace

The W.R. Grace site is a Voluntary Cleanup Oversight and Assistance Program (VOAP) site under the oversight of the State of Tennessee Department of Environment and Conservation's Division of Remediation located in the proposed Fite Road area. The facility manufactured fertilizers from 1954 through 1988. Prior to the construction of a wastewater treatment system in 1972, the plant's wastewater was held in an earthen bermed holding pond. The earthen pond was converted to a spill retention pond in 1972. An environmental site assessment conducted prior to the sale of the property in 1988 indicated that the holding pond/spill retention pond and a nearby oil/water separator/recovery system were a source area for VOC impacts to soil and ground water. Elevated PCB

concentrations were also found in the pond sludge, soils beneath the pond, and in the pond berms.

The March 2000 ROD indicates remedial action at the site included excavation of impacted sludge and soil from the pond and berm. Excavation activities were halted before complete removal of PCB impact due to concrete surface improvements. PCB impacted soil was left in place on the eastern side of the spill retention pond (110 feet long, up to 17 feet deep) along a concrete run-off ditch and adjacent to a concrete decontamination pad on the southern portion of the spill retention pond (approximately 65 feet long and 7 feet deep).

The ROD indicates confirmed PCB impact to a drainage ditch that conveys storm water from the W.R. Grace site to the Loosahatchie River. This ditch is the same drainage that discharges storm water from the Chromasco site. Models to evaluate sediment loading and PCB impact to the Loosahatchie River were prepared by Woodward-Clyde for W.R. Grace. The February 29, 1996, report titled "Addendum to Evaluation of Potential PCB Impacts to Loosahatchie River" shows discharge from the drainage ditch will not exceed PCB water quality standards.

The ROD also indicates a vacuum recovery system was operated for over three years to recover VOCs in ground water from the source area at the site. After initial reductions of TCE and PCE, it was determined the vacuum system was hindering dechlorination due to oxygenation of the ground water. The vacuum system was decommissioned and institutional controls along with monitored natural attenuation were selected as the most viable remedy for remaining contamination.

F&A reviewed the July 2008 Ground Water Sampling Results and Five-Year Natural Bioattenuation Review report prepared by URS for the W. R. Grace site at the TDEC Memphis Environmental Field Office. The report indicates two solvent plumes located to the west of the Canadian National Railroad are being monitored for natural attenuation of chlorinated solvents.

3.0 AREAS OF CONCERN

Construction plans for the new alignment of Fite Road will include acquisition of ROW from the former Chromasco site as well as five adjacent parcels, dredging and filling an existing ditch to allow for a bridge approach over the Canadian National Railroad, constructing a new ditch alignment on and adjacent to the former Chromasco site, and improvement of a section of sanitary sewer line adjacent to the former Chromasco site. Additionally, the new alignment will require ROW acquisition across the W.R. Grace property where a solvent plume is being monitored for natural attenuation. These tasks and their environmental concerns are further outlined in the following sections:

3.1 Proposed Fite Road Right-of-Way

The proposed ROW will require the acquisition of approximately 1.05 acres from the Chromasco property located along the north ROW of the existing Fite Rd. on the east end of the project corridor as shown on Figures 2 and 3. (All figures have been updated to show the current typical roadway section details) A portion of three adjacent properties located south of the existing Fite Road will also be required for the new ROW. These properties may also have been impacted by surface water runoff from the Chromasco site due to storm water conveyances beneath Fite Road and from ground water migration.

3.2 New Ditch Alignment

The new ditch channel will require excavation of approximately 670 L.F. of soil on the southwest corner of the Chromasco property, on the adjacent City of Memphis property, and on the City of Memphis (MLGW) property south of the existing Fite Rd. (Figure 2). The location of excavation activities is near the portion of the Chromasco property where the landfill is located, and leaching over time could have impacted soil and groundwater in the area to be disturbed for the new ditch alignment. The ground water flow direction at the site is generally to the south, and the ground water impacts observed on the Chromasco property are up-gradient from the new ditch channel, indicating there is a possibility of migration of ground water contaminants to the excavation area.

3.3 Existing Drainage Ditch

The drainage ditch that exhibited historical PCB impact from the W.R. Grace site and was downgradient from the Chromasco process water discharge is also a potential receptor of surface runoff and erosion from surficial contamination from the Chromasco property. Shallow impacted ground water from the Chromasco site may also discharge to the ditch. Construction plans require the relocation of approximately 600 L.F. of the ditch located on the southwest corner of the Chromasco property, the adjacent City of Memphis property, and the City of Memphis property south of the existing Fite Rd as shown on Figure 2 (Appendix A). Backfilling and compaction requirements of the ditch channel may result in removal and potential disposal of ditch bottom soils and sediments.

3.4 Sewer Line Improvement

Construction plans require the improvement of an underground sewer line located in the project corridor near the Canadian National Railroad. The sewer line improvement calls for excavation activities near the Chromasco property. Approximately 360 L.F. of 15-feet deep excavation will allow for an improved sewer line system near the existing lateral aligned on the west side of the rail road, terminating near the existing sanitary sewer alignment north of the proposed Fite Road.

3.5 W.R. Grace Right-of-Way

The new Fite Road alignment will require approximately 5.6 acres of land acquisition from the W.R. Grace & Co. property. A preliminary file review indicated one of two solvent plumes being monitored underlies a portion of the land required for ROW acquisition. The majority of the roadway plans require filling on this portion of the project; however, some shallow (<5-feet) excavation work will be required for roadside drainage improvements.

4.0 SAMPLING METHODOLOGY

4.1 Soil Sampling Methodology

Subsurface soil samples were collected using hydraulic push Geoprobe sampling equipment in accordance with the EPA Region IV Field Sampling Procedures (FSP) for Soil Sampling (SESDPROC-300-R1). Soil samples were collected continuously, utilizing a 48" long Geoprobe sampling barrel lined with a replaceable 2-inch diameter acetate sleeve that was advanced through the subsurface. The acetate sleeve was opened and a representative portion of the soil core placed into laboratory prepared sample containers. Soil from select sampling intervals was placed into plastic bags for screening of volatile organic compounds (VOCs) with a Photo-Ionization Detector (PID).

4.2 Ground Water Sampling Methodology

Temporary wells were installed in Geoprobe boreholes for collection of ground water samples in accordance with the EPA Region IV FSP guidelines for ground water sampling (SESDPROC-301-R1). Samples were collected using low-flow techniques with a peristaltic pump. Both filtered and non-filtered ground water samples were collected for metals analysis. Ground water filtering was conducted in the laboratory to remove suspended sediments that may provide false positive results. Samples for laboratory filtering were submitted to the laboratory, prepared, and held pending results of non-filtered samples.

4.3 Ditch Sediment Sampling Methodology

All ditch bottom sediment sampling activities were conducted in accordance with the EPA Region IV FSP for Sediment Sampling (SESDPROC-200-R1). A variation from the work plan was encountered due to the inability to drive the 48" long Geoprobe sampling barrel through the ditch bottom soils by hand. Very little soft sediment was encountered, and the ditch bottom was primarily comprised of firm, gray, clayey silt. A hand-auger was advanced to approximately 12" bgs for collection of ditch bottom sediment and soil.

The sediment was thoroughly mixed in a bowl in accordance with the SESCPROC-200-R1 guidelines prior to placing a representative portion into laboratory prepared containers.

4.4 Surface Water Sampling Methodology

Surface water sampling activities were conducted in accordance with the U.S Environmental Protection Agency (EPA) Region IV FSP for Surface Water Sampling (SESDPROC-201-R1). Samples were collected from the bank of the ditch utilizing a decanting device. Water sampling activities were conducted prior to sediment sampling activities to prevent disturbance of ditch bottom sediments.

4.5 Laboratory Analysis

All samples were placed into laboratory prepared containers, labeled, and immediately placed on ice. Samples were shipped on ice under chain-of-custody on a daily basis to Environmental Science Corp. (ESC) in Mt. Juliet, TN. Samples were analyzed for PCBs that have been detected on the Chromasco site and in the existing ditch, along with metals, hexavalent chromium, and cyanide that have been detected on the Chromasco property. The sample analysis includes:

- PCB's per laboratory method 8082;
- Target Analyte List (TAL) Metals per laboratory method 6010/6020/7471 (soil) - 6010/6020/7470 (water);
- CR6 (Hexavalent Chromium) per method 3060A/ 7196A (soil) - 7196A (water); and
- Cyanide per method 9012B.

4.6 Sample Label Methodology

Sample locations are identified by the area of concern where the sample is located, including:

CR	Chromasco Right-of-Way Sample
ND	New Ditch Sample
D	Existing Ditch Sample
SL	Sewer Line Sample

The sample numbering system includes information on the area of concern where the sample was collected, the media sampled, and sample location. For example, sample CRW-1 is the water sample collected from sampling location CR-1, and CRS-1 is the soil sample collected from location CR-1.

5.0 FIELD ACTIVITIES

5.1 Preliminary Activities

Prior to sampling activities, F&A, a construction sub-contractor, and F&A surveyors mobilized to the site on Thursday, October 1, 2009, to clear access to overgrown areas for sampling activities. Sampling locations for the proposed Fite Road ROW and the new ditch alignment were surveyed and marked in the field for accurate placement of soil borings. Sampling locations in the existing ditch were identified by existing landmarks and/or measured distances from Fite Road, and the proposed sewer line relocation boring located to the west of the Canadian National Railroad was identified by adjacent surface drainage features.

While surveying sampling locations, a bull-dozer was used to clear excessive growth and vegetation for drilling equipment access. Care was taken not to excessively disturb surface soil or slag identified on the former Chromasco site. No soils or other materials were removed from the site during this project.

5.2 Subsurface Sampling Activities

F&A representatives and a drilling subcontractor (Tri-State Testing Services, Inc.) mobilized a track-mounted hydraulic push probe to the site on October 5, 2009, to advance soil borings and collect soil and ground water samples.

5.2.1 Right-of-Way Sampling

On October 5-7, 2009, eight soil borings (CR-1 through CR-8, Figures 2 and 3) were advanced on the Chromasco property and three adjacent properties south of Fite Road. On October 5, 2009, Chromasco ROW borings CR-1, CR-2, and CR-3 were advanced in the proposed ROW behind the chain-link-fence along the southern border of the Chromasco property. On October 6, 2009, CR-4 was advanced in the proposed ROW on the north side of Fite Rd. on the Chromasco property adjacent to a surface drainage feature that originates on the Chromasco site, and CR-5 was advanced in the proposed ROW adjacent to the drainage feature to the south of Fite Rd. on the Grinder Fabricating property. On October 7, 2009, CR-6 was advanced in the proposed ROW on the Chromasco property south of the chain-link fence, and CR-7 and CR-8 were advanced in the proposed ROW to the south of Fite Rd. on the Grinder Fabricating and Memphis Ready Mix properties, respectively.

Two soil samples were submitted to the laboratory from soil borings conducted on the Chromasco property (CR-1, CR-2, CR-3, CR-4, and CR-6) to determine if subsurface impact is present. Shallow soil samples were collected from approximately 1-foot bgs and deeper samples were collected from the soil at the ground water interface (approximately 10-12 feet bgs). In boring CR-4, saturated

conditions were encountered at approximately 5-feet bgs, therefore; the soil from 4-feet bgs was sampled. Only shallow soil samples (approximately 1-foot bgs) were collected from soil borings conducted south of Fite Road (CR-5, CR-7, and CR-8).

Due to insufficient ground water in CR-1 on October 5, 2009, the temporary well was covered and left in place. F&A returned to the location on October 8, 2009, and collected the ground water samples at this location. Insufficient ground water also prohibited complete sampling of CR-3 on October 5, 2009. The sample for TAL metals was collected on October 5, 2009, and F&A returned to the location on October 8, 2009, to collect samples for cyanide, hexavalent chromium, and PCBs. All other water samples were collected on the date the borings/temporary wells were installed. Sampling locations are shown on Figure 2 and Figure 3.

5.2.2 New Ditch Alignment Sampling

Four soil borings were advanced along the surveyed alignment of the new ditch channel on October 7 and 8, 2009. New ditch sampling location ND-1 was advanced to the south of Fite Rd. on the MLGW property on October 7, 2009. ND-2, ND-3, and ND-4 were advanced along the proposed ditch alignment from east to west across the City of Memphis property and Chromasco property north of Fite Rd. ND-2 was advanced on October 7, 2009, and ND-3 and ND-4 were advanced on October 8, 2009. Two soil samples were submitted from each of the new ditch alignment sampling locations. Shallow soil samples were collected from approximately 1-foot bgs and deeper samples were collected from the soil at the expected excavation depth for the new ditch alignment (11-12 feet bgs).

Ground water was collected from the temporary well installed in boring ND-1 on October 7, 2009. Ground water in ND-2, ND-3, and ND-4 was sampled on October 8, 2009. Insufficient ground water prevented collection of cyanide and TAL metals samples from ND-3, and cyanide and hexavalent chromium were not sampled in ND-4. As with many of the sampling locations throughout the project, the wet silt collapsed into the borehole, preventing the installation of the temporary well to the full depth of the soil boring. Additional unsuccessful attempts were made to prevent the collapse in select locations; however, these attempts yielded the same results.

5.2.3 Sewer Line Relocation Sampling

Two soil borings were advanced along the alignment for the new sewer line on October 8, 2009. Soil boring SL-1 was advanced on the MLGW property north of Fite Rd. where the directional bore terminates and the surface excavation work begins. SL-3 was advanced on the W.R.Grace property to the west of the Canadian National Railroad where the directional bore originates. Proposed SL-2 was to be advanced at a turning point between two directional bore runs, but the location could not be accessed due to surface obstructions and the convergence of

drainage features in the area. Two soil samples were submitted to the laboratory from soil borings conducted for the sewer line relocation. Shallow soil samples were collected from approximately 1-foot bgs and deeper samples were collected from the soil at the ground water interface (7-feet bgs in SL-1, 10-feet bgs in SL-3).

Ground water samples were collected from both sewer line sampling locations (SL-1 and SL-2) on October 8, 2009.

5.2.4 MLGW Containment Outfall Sampling

Based on findings from the environmental study prepared for the MLGW Substation property (Tract No. 8) located south of Fite Rd., F&A used a hand auger to sample sediment located in the drainage pathway for the outfall of a storm water containment basin located on the substation property. This drainage pathway crosses a portion of property to be acquired for the access road to be constructed as part of the realignment project.

5.3 Surface Sampling Activities

Ditch water samples were collected from the existing ditch in six locations on October 12, 2009. D-1 and D-2 were collected south of Fite Rd., and D-3 was collected just north of Fite Rd. D-4 was collected to the west of D-3, and D-6 was collected at the confluence of the ditch and the drainage leading from the Chromasco property. D-5 was collected near the railroad where storm water conveyances along the railroad intersect the ditch. Soil was collected from the base of the existing ditch on October 13, 2009, using a hand-auger in the same six locations as the ditch water samples.

5.4 Deviations from the Approved Work Plan

As previously mentioned in Section 4.3, a hand auger was used to collect soil and sediment samples from the bottom of the existing ditch rather than the Geoprobe sampling barrel. Also, the easternmost sampling location along the proposed right of way shown in the Work Plan was not conducted because no additional ROW is required for construction in this area. As discussed in Section 5.2.3, sewer line sample location SL-2 was not conducted due to a lack of access to the area by surface drainage features. Insufficient ground water prevented collection of cyanide and TAL metals samples from ND-3. Cyanide and hexavalent chromium were not sampled in ND-4.

As mentioned in Section 5.2.4, an additional soil sample was collected from a containment outfall located within the access road ROW on the MLGW property (Tract No. 8).

As shown on the following table, the data completeness objective of 90 percent established in the approved work plan was achieved. The discrepancies included

the inability to access the middle sampling location along the proposed sewer line work, and a sampling location on the east end of the former Chromasco property was not required because no ROW acquisition is required in this area.

Area of Concern		Sediment / Soil		Ground Water / Surface Water		Completeness
		Proposed	Actual	Proposed	Actual	
1	New ROW Sampling	9[1]	8	9[1]	8	89%
2	New Ditch Channel	4	4	4	4	100%
3	Existing Ditch	6	6	6	6	100%
4	Sewer Line Relocation	3	2	3	2	67%
QA/QC Field Duplicates				2	2	100%
Project Data Completeness						91%

[1] – The approved Phase II Work Plan incorrectly listed 10 locations; however, only 9 were shown on the accompanying maps.

6.0 FINDINGS

6.1 Local Geology

In general, findings from soil borings indicate soft brown and/or gray silt from the surface to 4 to 9 feet bgs. Brown silt was present to boring termination depth in CR-5, CR-6, and SL-1. In the remaining borings, the brown silt was primarily underlain by gray and/or brown clayey silt. Gray, brown or dark gray clay was present from depths ranging from 4 to 12 feet bgs in CR-4, ND-1, ND-2, and SL-2. Geologic Boring logs are included in Appendix B.

6.2 Soil Sampling Results

Laboratory analytical results for PCBs in soil were below laboratory detection limits in all soil samples submitted to the laboratory. Cyanide [0.41 parts per million (ppm)] was detected in the sample collected from 1-foot bgs in soil boring CR-4, and hexavalent chromium (2.3 ppm) was detected in the sample collected from 1-foot bgs in CR-3. These results are below the EPA Regional Screening Levels (RSLs) for Industrial Soils, (November 2011) of 20,000 ppm and 5.6 ppm, respectively.

TAL metals analysis indicated several metals concentrations in all soil samples submitted to the laboratory. Each of the positive results was below the RSL, with the exception of arsenic (1.6 ppm). Arsenic concentrations ranged from below detection limits to 15 ppm. The sample containing 15 ppm arsenic was collected from 2 ft. bgs in sample location SL-1. All other arsenic concentrations ranged from below detection limits to 10 ppm. Arsenic concentrations detected in the

soil samples are within range of naturally occurring background arsenic levels in western Tennessee. According to a Tennessee Division of Superfund Survey cited in the "Hazardous Trace Elements in Tennessee Soils and Other Regolith" completed by the TDEC Division of Geology in 2001, the naturally occurring background level of arsenic in Tennessee is 10 ppm. Furthermore, a TDEC Solid Waste Management Memo, dated July 18, 2006, stated that natural levels up to 20 ppm of As are not uncommon in Tennessee soils. Therefore, the arsenic concentrations observed during the Phase II ESA do not appear to be a concern to the subject property.

Due to laboratory dilution, the laboratory detection limit (<5.0 ppm) exceeded the RSL for arsenic (1.6 ppm) in the samples collected from ND-1 (11-feet bgs), ND-2 (11-feet bgs), and ND-3 (11-feet bgs). Since this detection limit is below the naturally occurring background level of up to 20 ppm, the arsenic concentrations (if present) would be below the background concentration. Soil analytical results are summarized on Table 1, included in Appendix C. Laboratory reports (copies) are included in Appendix D.

6.3 Sediment Sampling Results

Laboratory analytical results for PCBs and hexavalent chromium were below laboratory detection limits in all sediment samples submitted to the laboratory. Results for the oil and PCB related constituents in the MLGW outfall were also below detection limits. Cyanide (0.26 ppm) was detected in the sediment sample collected from the existing ditch in location D-4. Laboratory results for sediment were compared with the EPA Region 4 Ecological Screening Values for Sediment (ESVs), dated November 30, 2001. As of April 26, 2012, the Sediment ESVs have not changed, based upon online searches of the EPA Region 4 website. No standard for cyanide was available, and this low concentration is not expected to present a concern to the subject property.

Several TAL metals constituents were detected in the sediment samples submitted to the laboratory; however, all concentrations were below the applicable Sediment ESVs, if established, with the exception of nickel in D-2 (24 ppm) and D-3 (22 ppm). The Sediment ESV for nickel is 15.9 ppm. The nickel concentration in the thirty soil and sediment samples collected from the area during the Phase II ranged from 5.6 ppm to 44 ppm, indicating the nickel concentrations in the D-2 and D-3 sediment samples are within the expected background levels for the area. Additionally, the Soil ESV for nickel is 30 ppm, which the ditch sediment samples are below. Sediment analytical results are summarized on Table 2, included in Appendix C. Laboratory reports (copies) are included in Appendix D.

6.4 Ground Water Sampling Results

Laboratory analytical results for cyanide and PCBs were below laboratory detection limits in each of the ground water samples submitted to the laboratory

for analysis. Hexavalent chromium (0.012 ppm) was detected in the sample collected from CR-2; however, there is no established Maximum Contaminant Level (MCL) for hexavalent chromium for comparison of this result. The groundwater sample concentration is below the MCL for Chromium (Total) of 0.1 ppm.

A number of metals were detected in the ground water samples submitted to the laboratory for analysis, with several of the metals concentrations exceeding the MCLs. These elevated concentrations were detected in unfiltered samples analyzed for total metals by the laboratory. Excessive sediment was present in the ground water samples due to the slow recharge rate of the ground water, the fine particle size of near-surface soils and the nature of temporary wells. Because metals tend to adsorb onto soil particles, excessive sediment could have contributed to the elevated results.

Upon review of the results, the backup samples collected in the field were filtered in the laboratory and analyzed for the select constituents that exceeded MCLs to determine if sediment interference contributed to the results. Dissolved metal concentrations for the select constituents analyzed in the filtered samples were below their respective MCLs. Since the time of the investigation, a MCL for Chromium has been established at 0.1 ppm. The Chromium concentrations in groundwater from most of the unfiltered samples exceed this MCL. Since there was not a MCL listed in the table at the time of the investigation, Chromium was not one of the reanalyzed parameters for the filtered samples, however it was analyzed in three of the filtered samples, (ND-2, duplicate of SL-1, and SL-3) where the filtered sample was analyzed for the full list of TAL metals. All of these samples showed no detectable concentration of Chromium with a detection limit one order of magnitude below the latter established Chromium MCL.

Ground water analytical results are summarized on Table 3, included in Appendix C. Laboratory reports (copies) are included in Appendix D.

6.5 Surface Water Sampling Results

Cyanide, hexavalent chromium, and PCB concentrations were below laboratory detection limits for the surface water samples submitted to the lab. Several TAL Metal concentrations were detected; however, the majority were below the ESV for surface water. Concentrations of aluminum exceeded the ESV of 0.087 ppm in each of the surface water samples collected (0.21 to 0.80 ppm), and iron exceeded the ESV of 1.000 ppm in sample D-2 (1.2 ppm). The aluminum and iron concentrations in the surface water are believed to be background related levels. As of April 26, 2012, the Freshwater Surface Water ESVs have not changed, based upon online searches of the EPA Region 4 website.

Surface water analytical results are summarized on Table 4, included in Appendix C. Laboratory reports (copies) are included in Appendix D.

6.6 W.R. Grace Right-of-Way

Fite Road ROW plans (Figure 4 and 5) show the location of several monitoring wells associated with the W.R. Grace ground water monitoring project located within the ROW, slope easement, and construction easement for the realignment project. The W.R. Grace project manager (Mr. Robert Medler) was contacted regarding these wells, and it was confirmed that all but long term monitoring well MW-E5D are out of use and have been closed. Monitoring well MW-E5D is located within the construction easement on the south side of the proposed Fite Road. Construction plans will be specific to insure this monitoring well is protected during the construction phase of the project. The construction plans will also specify to inspect the closed monitoring wells when encountered during construction to insure the grout is not compromised thus opening a pathway to the subsurface.

Results from the July 2008 Ground Water Sampling Results and Five-Year Natural Bioattenuation Review prepared by URS (December 8, 2008) were reviewed during a file review conducted at the TDEC Memphis Environmental Field Office. In addition to this report, a separate Estimated Plume Delineation, Well Identification Map dated September 8, 2008 (prepared by URS) was provided by the W.R. Grace project manager. These documents show volatile organic compound (VOC) ground water contamination from the W.R. Grace site underlies a portion of the ROW for the new Fite Rd. alignment. ROW plans indicate 5.4 acres (approximately 235,000 S.F.) of the W.R. Grace property (Tract No. 5) will be acquired for permanent ROW. Of this total, approximately 17,000 S.F. overlie the estimated contaminant plume as shown on Figure 6.

The presence of dissolved VOC concentrations in the ground water could lead to subsurface vapors. The presence of these vapors could accumulate in excavations. ROW plans indicate some minor excavation work for roadside drainage along the north side of the new Fite Rd. alignment overlying the dissolved VOC plume. If these excavations are to be entered by construction workers during the construction phase, VOC vapor monitoring will be prescribed.

6.7 Data Quality Objectives

In accordance with the approved work plan, two duplicate ground water samples were collected in the field and analyzed in the laboratory. The sample labeled CRW-11 is a duplicate of CRW-4. Metals were not analyzed in the duplicate because sedimentation issues can prohibit reproducibility of data; however, all other parameters were below detection limits in both the duplicate and the parent sample. NDW-5 is a duplicate of SLW-1. Cyanide, hexavalent chromium, and PCBs were below detection limits in both the duplicate and the parent sample. Metals analyzed from NDW-5 were filtered in the laboratory and metals from SL-

1 were analyzed as unfiltered samples; therefore, comparison of these results was not conducted.

Laboratory Quality Control data is included on the laboratory analytical reports included in Appendix D. Several samples were diluted due to matrix interference, which led to elevated detection limits. These elevated detection limits remained below the MCL for ground water, or the RSL for soil with the exception of antimony and thallium in ground water from CRW-4, selenium in ground water from CRW-7 and CRW-8, and arsenic in soil in NDS-1 (11 ft. bgs), NDS-2 (11 ft. bgs), and NDS-3 (11 ft. bgs). Antimony and thallium in CRW-4 and selenium in CRW-7 and CRW-8 were filtered and re-analyzed by the laboratory by method 6020B to lower the detection limit, and the new detection limit was below the respective MCLs. As stated in Section 6.2, the laboratory detection limit (<5.0 for arsenic in soil in NDS-1 (11 ft. bgs), NDS-2 (11 ft. bgs), and NDS-3 (11 ft. bgs) was above the RSL of 1.6 ppm, however, this detection limit is below the naturally occurring background level of up to 20 ppm for arsenic in Tennessee.

Surrogate recovery limits were exceeded in sample CRS-1 (1 ft. bgs) and CRS-3 (1 ft. bgs and 11 ft. bgs). The results for each of these samples was within the range of other samples collected in the area during the project; therefore, the results are expected to be representative of site conditions.

Sample matrix interference resulted in a low spike value in hexavalent chromium in CRS-5 (1 ft. bgs), NDS-3 (1 ft. bgs), and DBS-2 (0-1 ft. bgs), and cyanide in CRW-4. The associated batch QC was outside the established range for precision for hexavalent chromium in DBS-2 (0-1 ft. bgs). Each of these results was below the laboratory detection limit, which was below the respective MCL or RSL; therefore, these QC failures are not expected to significantly affect the results.

The ground water sample labeled CRW-5 did not pass laboratory QA/QC for analysis of hexavalent chromium on October 7, 2009, and was re-analyzed on October 8, 2009; thus, the sample was reportedly analyzed outside the 24-hour hold time. The sample result was below laboratory detection limits. Total chromium in the TAL metals analysis was also below laboratory detection limits; therefore, hexavalent chromium is not expected to be present above detection limits.

7.0 CONCLUSIONS/RECOMMENDATIONS

Findings from samples collected during the Phase II ESA do not indicate significant environmental impact to the subsurface in the areas of proposed ROW studied. Arsenic concentration in many of the soil samples exceeded the RSL; however, the results are within the range of naturally occurring background arsenic levels in Tennessee (up to 20 ppm). Several metals exceeded the MCL in unfiltered ground water samples collected; however, after filtering out sediment in the laboratory, the results were below the MCLs.

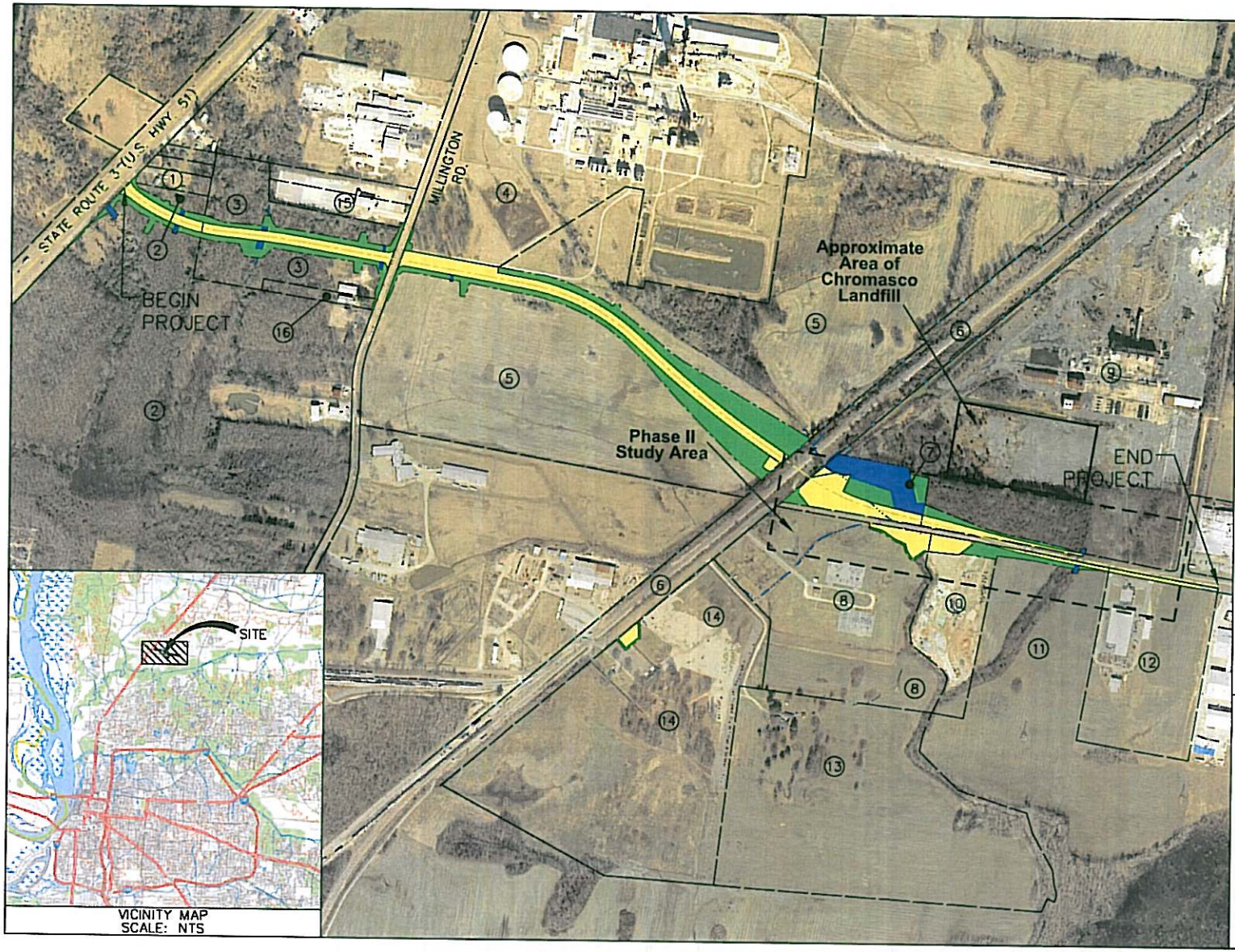
One active ground water monitoring well is located within the proposed construction easement on the W.R. Grace property (Tract No. 5). Construction plans should address the protection of this well, and construction supervisors and equipment operators should be made aware of the location to insure the integrity of the well is not compromised. Other inactive monitoring wells in the ROW that have been plugged with cement/bentonite grout should be inspected to insure the plug material is not damaged, if they are encountered during construction.

The presence of dissolved VOC concentrations in the ground water could lead to subsurface vapors. The presence of these vapors could accumulate in excavations. ROW plans indicate some minor excavation work for roadside drainage along the north side of the new Fite Rd. alignment overlying the dissolved VOC plume. If these excavations are to be entered by construction workers during the construction phase, VOC vapor monitoring will be prescribed.

The data suggests that no widespread contamination has been identified in the project area, based upon the analysis of the selected soil, sediment, and ground water samples. However, the potential exists that very localized soil or sediment samples may exhibit elevated levels of contaminants. Shelby County Government will seek written concurrence of these findings from the Tennessee Department of Environment and Conservation prior to property acquisitions.

APPENDIX A

FIGURES



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Memphis, Tennessee 38125-8538
888-583-9724 Fax: 901-748-3115
Web: www.fisherarnold.com

Figure 1
FITE ROAD PROPERTY ACQUISITION STUDIES

TRANSACTION SCREEN (ASTM 1528-06)
1. LEE M. MOORE
2. E.I. DUPONT DE NeMOURS & Co.
4. ARCADIAN FERTILIZER, L.P.
8. CITY OF MEMPHIS
10. BUZZI UNICEM READY MIX LLC
11. GRINDER FABRICATORS
12. JIMMY R. GRINDER ET. AL.
13. BLAND FARMS LLC
14. ROBERT D. POOLE AND GLORIA POOLE

PHASE I ESA (ASTM 1527-05)
3. B & J MACHINE WORKS
5. W.R. GRACE & CO.
7. CITY OF MEMPHIS
9. TIMMINCO CORPORATION

PHASE II ESA
7. CITY OF MEMPHIS
9. TIMMINCO CORPORATION

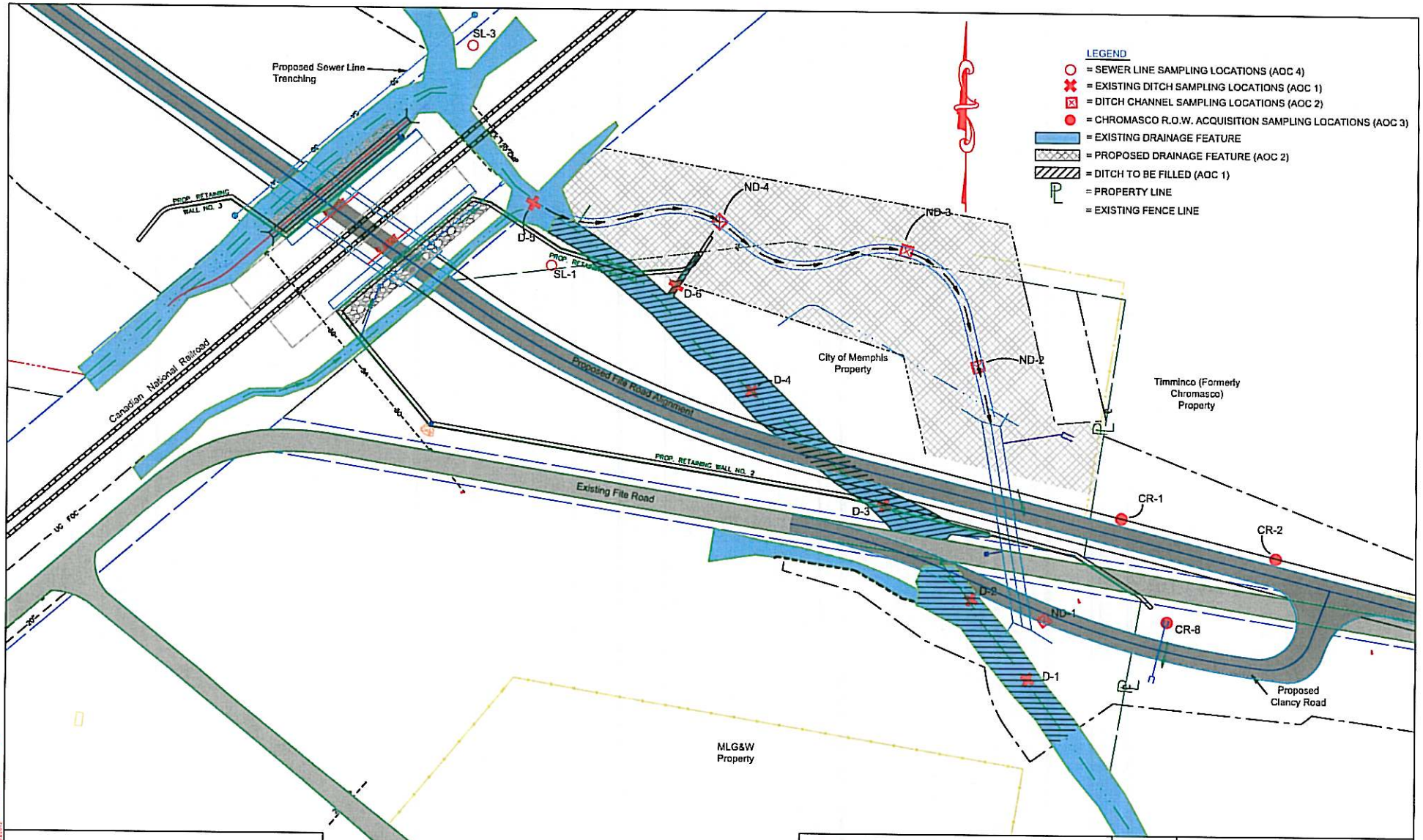
FILE REVIEW OF EXISTING ASSESSMENT & MNA DATA
5. W.R. GRACE & CO.

NO TECHNICAL STUDIES
(CONSTRUCTION EASEMENT ONLY THEREFORE NO PERMANENT ACQUISITION)
15. K&L HOLDINGS LLC
16. JAMES E. LAMB

LEGEND
- DRAINAGE FEATURES
- R.O.W. ACQUISITION
- DRAINAGE EASEMENTS
- SLOPE / CONSTRUCTION EASEMENTS

N

GRAPHIC SCALE (IN FEET)
0 300 600
1 inch = 600 ft.



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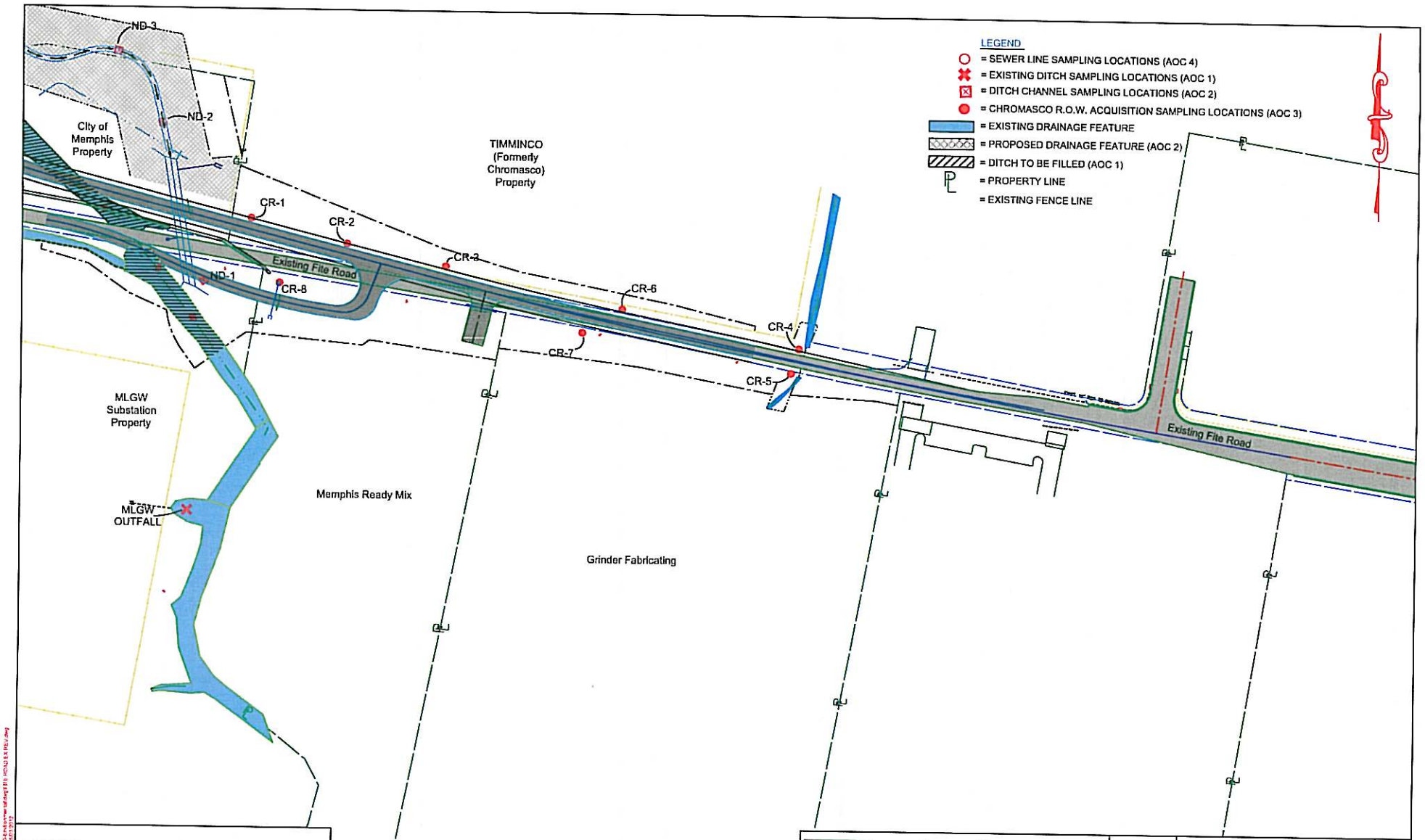
PROJECT NUMBER G7193

DITCH AND SANITARY SEWER RELOCATION SAMPLING LOCATIONS
May, 2012

FIGURE 2



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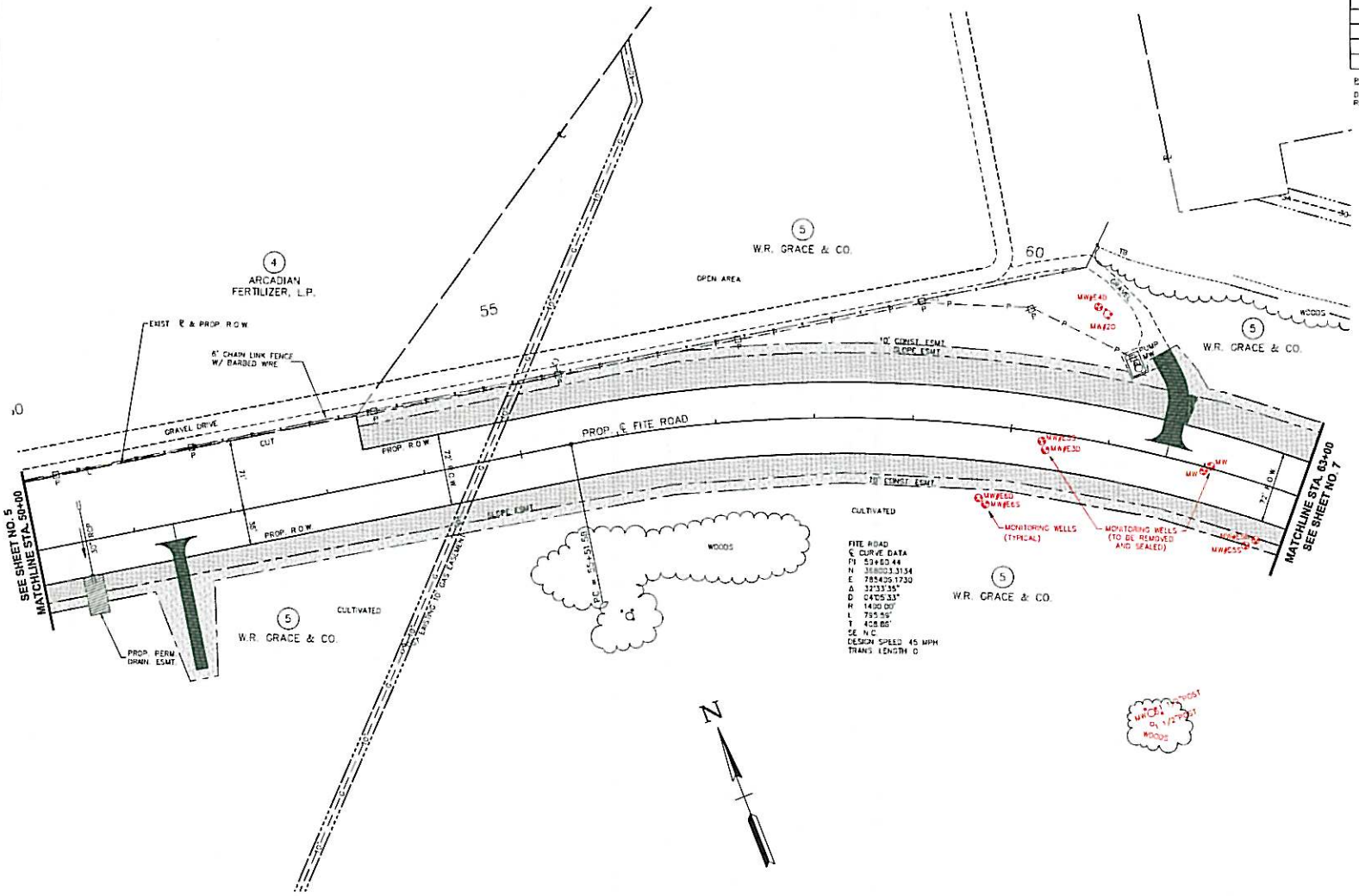
PROJECT NUMBER G7193

R.O.W. SAMPLING LOCATIONS
May, 2012

FIGURE 3

TYPE	YEAR	PROJECT NO.	SHEET NO.
R.O.W.	2012	STP-M-7900(29)	4

REVISIONS:
DATE: 02/24/2012
REVISED FITE ROAD TYPICAL SECTIONS



TENNESSEE D.O.T.
DESIGN DIVISION
FILE NO.

5/03/2012
W:\DD-Environmental\dwg\6513 All Present Layouts rev.dwg

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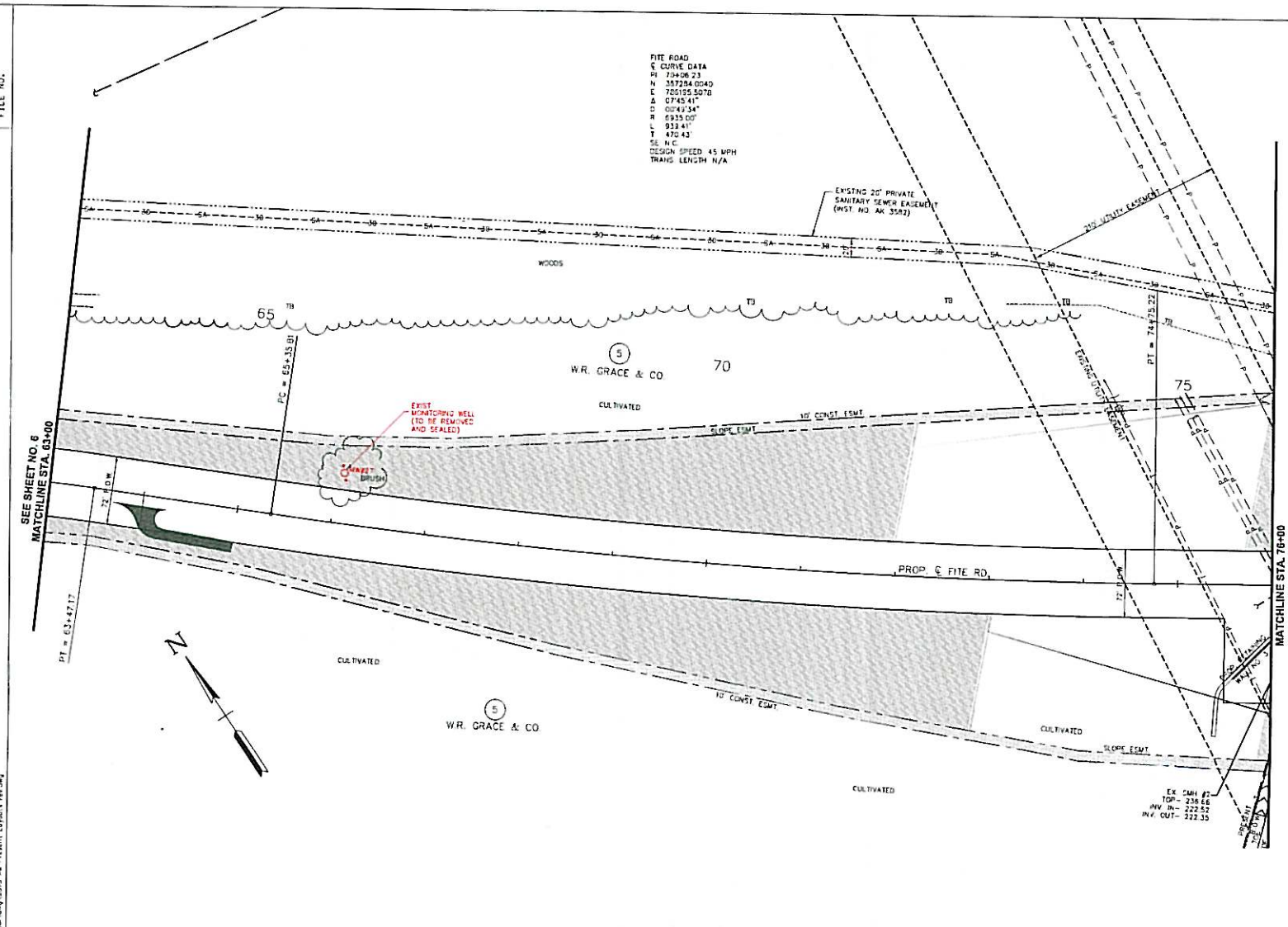
PROJECT NUMBER G7193

W.R. GRACE MONITORING WELL LOCATIONS
FITE ROAD R.O.W. & EASEMENTS
May, 2012

FIGURE 4

TYPE	YEAR	PROJECT NO.	SHEET NO.
R.O.W.	2012	STP-M-7900(29)	5

REVISIONS:
DATE: 02/24/2012
REVISED FITE ROAD TYPICAL SECTIONS



TENNESSEE D.O.T.
DESIGN DIVISION
FILE NO.

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PROJECT NUMBER G7193

W.R. GRACE MONITORING WELL LOCATIONS
FITE ROAD R.O.W. & EASEMENTS
May, 2012

FIGURE 5

5/03/2012
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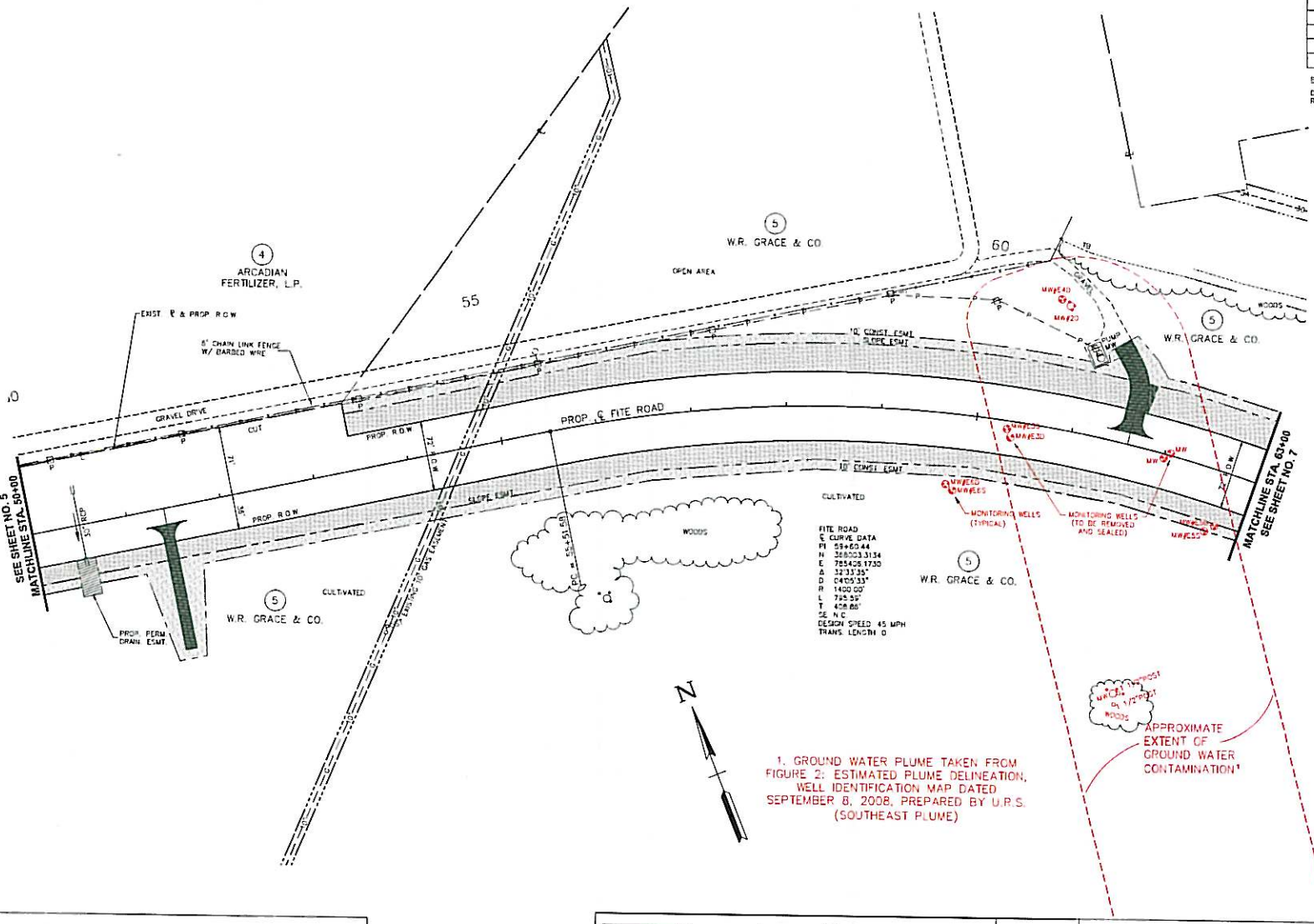
TENNESSEE D.O.T.
DESIGN DIVISION
FILE NO.

TYPE	YEAR	PROJECT NO.	SHEET NO.
R.O.W.	2012	STP-M-7900(29)	6

REVISIONS:
DATE: 02/24/2012
REVISED FITE ROAD TYPICAL SECTIONS.



1. GROUND WATER PLUME TAKEN FROM FIGURE 2: ESTIMATED PLUME DELINEATION, WELL IDENTIFICATION MAP DATED SEPTEMBER 8, 2008, PREPARED BY U.R.S. (SOUTHEAST PLUME)

SHELBY COUNTY GOVERNMENT

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PROJECT NUMBER G7193

APPROXIMATE W.R. GRACE PLUME EXTENT
May, 2012

FIGURE 6

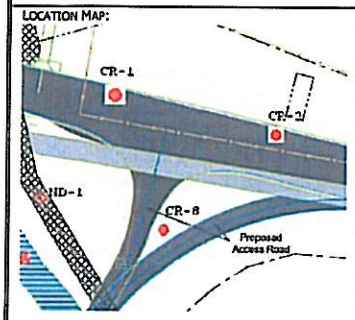
5/03/2012
W:\DD-Environmental\dwg\6513 All Present Layouts rev.dwg

APPENDIX B

GEOLOGIC BORING LOGS

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# B/OR BORING CR-1	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 5, 2009

COMP DATE & TIME: October 5, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

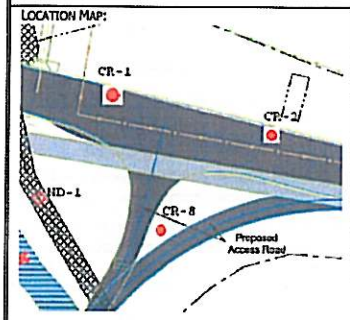
T.D. (MSL):

COMMENTS: temporary well set at 13 ft bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
	[redacted]			5		0.0			X	SILT: some clay, moist, brown
				10						CLAYEY SILT: firm, tan soft, tan and orange light grey & orange, mottled, soft, wet
				15		0.0			X	
				20						mod. firm sl. wet, firm
				25						very wet, tan CLAY: grey, firm, moist, high plasticity
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-2	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 5, 2009

COMP DATE & TIME: October 5, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

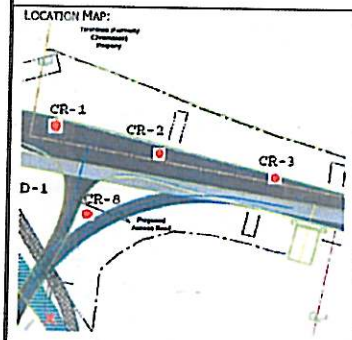
T.D. (MSL):

COMMENTS: temporary well pt. set at 20 ft-bgs
DTW: 8.65'

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Samples & Cores: Int/Rec	Samples & Cores: Anal	Description (Color, Texture, Structure, etc.)
				5		0.0			X	SILT: some clay, soft, dry, brown
				10						CLAYEY SILT: brown, soft, sl. moist
				15					X	lt. grey, brown, mottled, wet, soft SILT: some clay, soft, sl. wet
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-3	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 5, 2009

COMP DATE & TIME: October 5, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

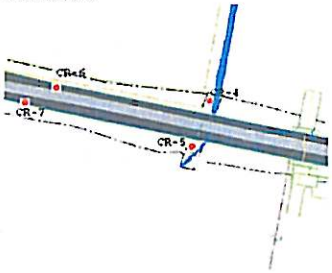
T.D. (MSL):

COMMENTS: temporary well pt. set at 20 ft-bgs
6' of water in hole

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5					X	SILT: brown-gray, soft, dry sl. moist
				10						CLAYEY SILT: gray, sl. moist, soft
				15					X	lt. grey: brown, mottled, wet, soft
				20						brown firm, moist
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-4	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 6, 2009

COMP DATE & TIME: October 6, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

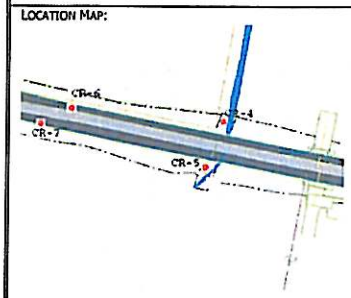
T.D. (MSL):

COMMENTS: temporary well point set at 15 ft-bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5		0.2 0.0			X X	SILT: brown, soft, wet, 3" gravel
				10						CLAY: gray, med. soft, slightly moist firm
				15						SILTY CLAY: light grey, brown, mottled, med. soft, sl moist
				20						firm
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-5	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 6, 2009

COMP DATE & TIME: October 6, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

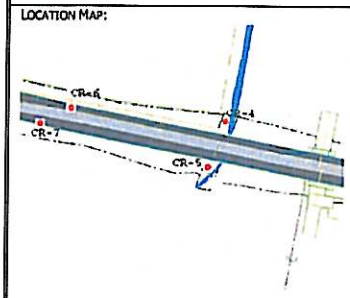
T.D. (MSL):

COMMENTS: temporary well pt. set at 12 ft-bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5		0.0 0.0			X X	SILT: brown, soft, dry
				10						grey, wet
				15						
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-6	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 7, 2009

COMP DATE & TIME: October 7, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

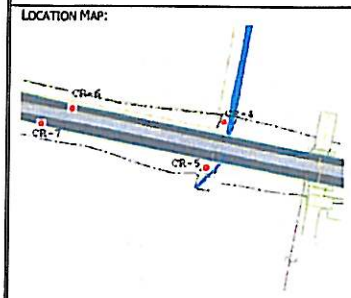
T.D. (MSL):

COMMENTS: temporary well pt. set at 24 ft bgs
DTW 16.8'

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Samples & Cores: Int/Rec	Samples & Cores: Anal	Description (Color, Texture, Structure, etc.)
				5		0.0			X	SILT: brown, soft, moist some clay
				10		0.0			X	wet
				15						some clay, very moist, brown + grey, mottled wet, brown
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# B/OR BORING CR-7	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 7, 2009

COMP DATE & TIME: October 7, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

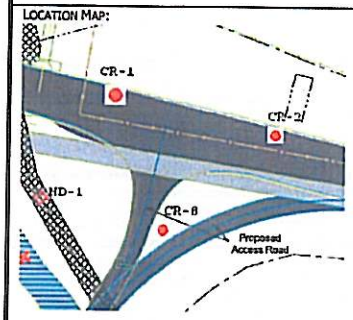
T.D. (MSL):

COMMENTS: temporary well point set at 20 ft bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Samples & Cores: Int/Rec	Samples & Cores: Anal	Description (Color, Texture, Structure, etc.)
				5		0.0			X	SILT: brown, firm, dry
				10						sl. moist CLAYEY SILT: brown, soft, moist
				15						wet
				20						
				25						moist
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING CR-8	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 7, 2009

COMP DATE & TIME: October 7, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

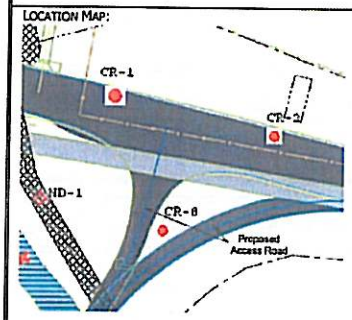
T.D. (MSL):

COMMENTS: temporary well point set at 13 ftbgs
DTW: 6.66'

MSL	Completion Diagram / Borehole Diameter:	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
						0.0			X	SILT: grey, soft, wet dry brown
				5						grey and brown, v. moist some clay
				10						CLAYEY SILT: brown, soft, v. moist - wet
				15						
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING ND-1	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 7, 2009

COMP DATE & TIME: October 7, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

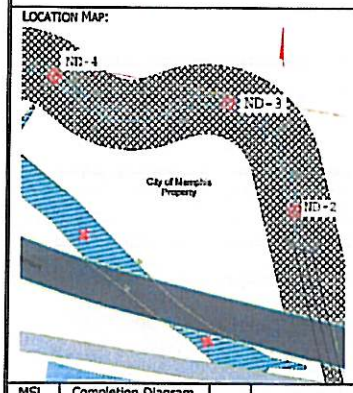
T.D. (MSL):

COMMENTS: temporary well pt. set at 13 ft-bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5		0.0			X	SILT: brown, dry, soft, moist
				10						CLAY: dark grey, sl. moist, hard SILT: grey, soft, moist
				15		0.0			X	
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING ND-2	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 7, 2009

COMP DATE & TIME: October 7, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

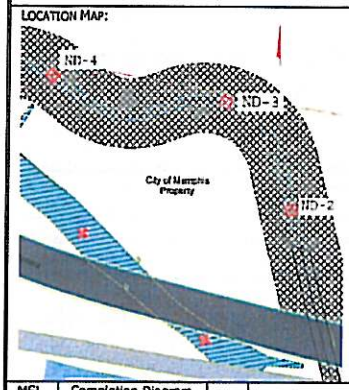
T.D. (MSL):

COMMENTS:

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Samples & Cores: Int/Rec	Samples & Cores: Anal	Description (Color, Texture, Structure, etc.)
				5					X	SILTY CLAY: brown, wet, soft
				10						CLAY: lt. brown, sl. brown, firm, moist SILTY CLAY: mottled, v. moist, soft
				15					X	very wet
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING ND-3	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 8, 2009

COMP DATE & TIME: October 8, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

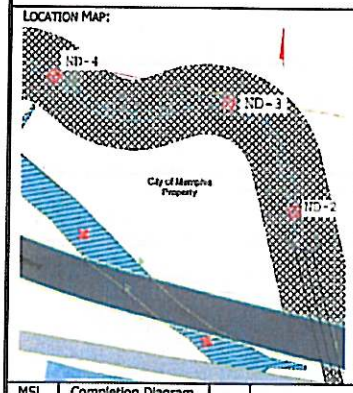
T.D. (MSL):

COMMENTS: temporary well pt. set @ 20 ft-bgs

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5					X	SILT: brown, soft, moist grey sl. moist
				10						CLAYEY SILT: grey, sl. moist
				15					X	SILT: grey, wet, soft very moist
				20						
				25						CLAY: grey, hard, moist
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING ND-4	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 8, 2009

COMP DATE & TIME: October 8, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

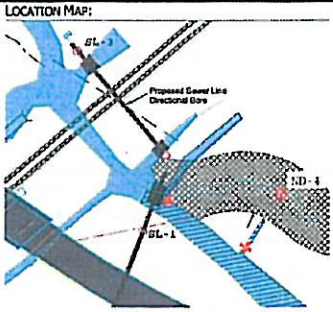
ELEV (MSL): | T.D. (MSL):

COMMENTS:

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5						SILT: brown, wet, soft sl. moist, grey
				10						CLAYEY SILT: brown, med. soft, sl. moist grey very moist
				15						SILT: brown & grey, mottled, wet, soft
				20						
				25						SILTY CLAY: grey, soft, wet SILTY SAND: gray, fine
				30						SILTY CLAY: grey, soft, moist

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING SL-1	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 8, 2009

COMP DATE & TIME: October 8, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

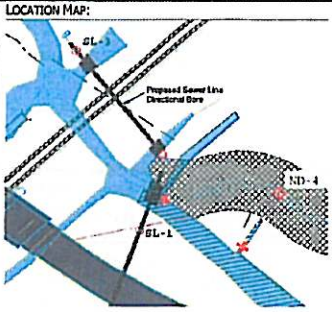
T.D. (MSL):

COMMENTS: temporary well point set

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Int/Rec	Anal	Description (Color, Texture, Structure, etc.)
				5					X	SILT: brown, soft, moist
				10					X	wet
				15						
				20						
				25						
				30						

FACILITY NAME FITE ROAD REALIGNMENT PROJECT, SHELBY COUNTY, TN	FAC ID# VOAP # 79-865	WELL# &/OR BORING SL-3	PG 1 OF 1

LOCATION MAP:



START DATE & TIME: October 8, 2009

COMP DATE & TIME: October 8, 2009

LOGGED BY: K. Yarrow, P.G.

DRILLER: Tri-State Testing Services, Inc.

DRILLING METHOD: Geoprobe

ELEV (MSL):

T.D. (MSL):

COMMENTS: temporary well pt. set

MSL	Completion Diagram / Borehole Diameter: 2"	Water Level	Penetration Rate	Depth	Graphic Lithology	OVD	Samples & Cores: Type	Samples & Cores: Int/Rec	Samples & Cores: Anal	Description (Color, Texture, Structure, etc.)
				5					X	SILT. brown, soft, moist gray, dry brown, sl. moist CLAY: dark gray-brown, hard
				10					X	light brown, some silt SILTY CLAY: light grey, wet, soft mottled lt. brown, orange brown gray
				15						
				20						
				25						
				30						

APPENDIX C

TABLES

Table 1
Soil Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location:		CR-1		CR-2		CR-3		CR-4		CR-5	CR-6		CR-7	CR-8	ND-1	
Sample ID:	RSLs	FITE-CRS-1		FITE-CRS-2		FITE-CRS-3		FITE-CRS-4		FITE-CRS-5	FITE-CRS-6		FITE-CRS-7	FITE-CRS-8	FITE-NDS-1	
Depth:		1ft	12 ft	1 ft	10 ft	1 ft	11 ft	1 ft	4 ft	1 ft	1 ft	10 ft	1 ft	1 ft	1 ft	11 ft
Collect Date:		10/5/2009	10/5/2009	10/5/2009	10/5/2009	10/5/2009	10/5/2009	10/6/2009	10/6/2009	10/6/2009	10/7/2009	10/7/2009	10/7/2009	10/7/2009	10/7/2009	10/7/2009
Cyanide	20,000	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	0.41	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25
Chromium,Hexavalent	5.6	<2.0	<2.0	<2.0	<2.0	2.3	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0
TAL Metals																
Mercury	43	0.025	0.024	0.034	<0.020	0.022	0.023	0.081	0.030	0.033	0.052	<0.020	0.025	0.023	0.035	0.022
Aluminum	990,000	8100	9200	19000	7300	8100	9300	10000	12000	14000	21000	9500	12000	11000	8700	10000
Antimony	410	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<5.0	<10	2.3	<1.0	<1.0	1.1	1.1	<1.0
Arsenic	1.6	3.2	3.2	10	8.7	4.3	7.9	3.8	3.2	4.5	<1.0	<1.0	<1.0	<1.0	2.6	<5.0
Barium	190,000	190	73	99	260	94	180	120	130	170	140	120	130	110	97	140
Beryllium	2,000	0.66	0.53	0.73	0.46	0.47	0.56	0.52	0.62	0.68	0.59	0.42	0.49	0.44	0.35	0.39
Cadmium	800	0.27	<0.25	0.30	0.35	<0.25	0.38	0.36	0.40	0.55	<0.25	<0.25	<0.25	<0.25	<0.25	0.25
Calcium	NSA	760	7700	1600	22000	540	2100	1300	1900	1600	1700	1400	1300	5200	1900	2600
Chromium (III) insol. Salts	1,500,000	7.3	13	16	11	23	12	46	22	26	20	12	13	19	23	15
Cobalt	300	5.8	5.9	8.1	7.2	5	11	6.6	8.3	7.7	8.3	6.6	7	7.1	5.1	6.2
Copper	41,000	4.9	11	17	11	5.5	15	8.3	6.5	8.1	14	13	14	12	10	7.6
Iron	720,000	6600	12000	23000	15000	8000	15000	9700	11000	15000	20000	16000	17000	14000	9900	12000
Lead	800	6.0	6.9	12	5.9	6.5	9.5	17	13	11	12	8	8.9	12	11	6.0
Magnesium	NSA	970	5700	3200	14000	1300	2500	1600	1600	2200	3000	2100	2400	3500	1400	3100
Manganese	23,000	1100	300	370	2000	600	1100	560	600	880	580	460	760	650	600	680
Nickel, soluble salts	20,000	12	13	17	14	8.8	18	12	11	14	17	16	16	16	13	16
Potassium	NSA	660	1100	1800	1100	760	1100	930	1000	1200	2100	1300	1300	1100	1000	1100
Selenium	5,100	<1.0	<1.0	<5.0	<1.0	<1.0	<2.0	<5.0	<5.0	<5.0	6.7	6.1	6.6	4.4	4.0	3.8
Silver	5,100	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	0.52	<0.50	<0.50	<0.50	<0.50	<0.50
Sodium	NSA	28	110	53	230	30	130	40	41	51	44	68	42	88	33	85
Thallium, soluble salts	10	<2.0	<2.0	<5.0	<5.0	<2.0	<2.0	<5.0	<5.0	<10	<5.0	<5.0	<5.0	<5.0	<5.0	<5.0
Vanadium	5,200	15	25	42	24	18	27	22	25	30	41	24	28	26	21	23
Zinc	310,000	23	36	56	26	26	46	41	42	40	49	37	41	35	43	41

Notes:
All results in mg/kg (parts per million)
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 3060A/7196A
RSLs - EPA Regional Screening Levels for Industrial Soils, November 2011
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
NSA - No Screening value Available

Mercury analyzed by method 7470A, all other metals analyzed by method 6010B.
PCB's analyzed by 8082
Benzene, Toluene, Ethylbenzene, Total Xylenes, and Naphthalene analyzed by method 8260B
NA - Not Analyzed
Bold result exceeds RSL

Table 1 - continued
Soil Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location:		ND-2		ND-3		ND-4		SL-1		SL-3	
Sample ID:	RSLs	FITE-NDS-2		FITE-NDS-3		FITE-NDS-4		FITE-SLS-1		FITE-SLS-3	
Depth:		1 ft	11 ft	1 ft	11 ft	1 ft	11 ft	2 ft	7 ft	1 ft	10 ft
Collect Date:		10/7/2009	10/7/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009
Cyanide	20,000	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25	<0.25
Chromium,Hexavalent	5.6	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0	<2.0
TAL Metals											
Mercury	43	<0.020	<0.020	<0.020	<0.020	<0.020	<0.020	<0.020	<0.020	0.023	<0.020
Aluminum	990,000	3400	4900	4900	4600	6800	4400	4800	5500	6600	6600
Antimony	410	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	1.6	1.1	1.0	<1.0
Arsenic	1.6	2.5	<5.0	3.4	<5.0	<1.0	<1.0	15	1.7	<1.0	<1.0
Barium	190,000	82	63	52	210	94	70	540	180	140	120
Beryllium	2,000	0.44	0.27	0.42	0.33	0.41	0.41	1	0.47	0.54	0.51
Cadmium	800	<0.25	<0.25	<0.25	<0.25	<0.25	0.37	1.5	0.77	<0.25	<0.25
Calcium	NSA	580	1900	360	1700	980	5100	1100	2400	3500	3000
Chromium (III), insol.salts	1,500,000	10	12	11	12	13	12	12	14	17	16
Cobalt	300	5.4	2.7	2.4	8.1	6.1	11	48	17	3.3	4.3
Copper	41,000	7.5	7.0	5.9	10	8.1	14	17	14	12	12
Iron	720,000	11000	9900	10000	13000	12000	17000	24000	17000	16000	15000
Lead	800	8.0	4.0	6.6	5.1	8.6	7.2	47	11	6.2	6.9
Magnesium	NSA	1200	2500	920	2600	1700	4000	1500	2800	3300	3200
Manganese	23,000	520	70	240	870	180	490	<0.50	1400	160	170
Nickel	20,000	8.3	8.2	5.6	14	8.1	22	44	36	11	11
Potassium	NSA	640	830	520	820	730	730	670	940	1000	910
Selenium	5,100	5.7	4.5	5.7	6.1	6.4	7.9	17	8.9	7.4	6.8
Silver	5,100	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	0.80	<0.50	<0.50	<0.50
Sodium	NSA	44	140	40	250	33	72	31	69	52	61
Thallium	10	<1.0	1.0	<1.0	<1.0	<1.0	1.9	<1.0	<1.0	<1.0	<1.0
Vanadium	5,200	20	16	21	19	26	26	48	26	23	24
Zinc	310,000	22	36	17	39	26	38	37	41	50	48

Notes:
All results in mg/kg (parts per million)
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 3060A/7196A
RSLs - EPA Regional Screening Levels for Industrial Soils, November 2011
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
NSA - No Screening value Available

Mercury analyzed by method 7470A, all other metals analyzed by method 6010B.
PCB's analyzed by 8082
Benzene, Toluene, Ethylbenzene, Total Xylenes, and Naphthalene analyzed by method 8260B
NA - Not Analyzed
Bold result exceeds RSL

Table 2
Sediment Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location:		D-1	D-2	D-3	D-4	D-5	D-6	MLGW OUTFALL
Sample ID:	**ESV**	FITE-DBS-1	FITE-DBS-2	FITE-DBS-3	FITE-DBS-4	FITE-DBS-5	FITE-DBS-6	OUTFALL
Depth:		0-1 ft.	0-1 ft.	0-1 ft.	0-1 ft.	0-1 ft.	0-1 ft.	0-1 ft.
Collect Date:		10/13/2009	10/13/2009	10/13/2009	10/13/2009	10/13/2009	10/13/2009	10/13/2009
Cyanide	**NSA**	<0.25	<0.25	<0.25	0.26	<0.25	<0.25	NA
TAL Metals								
Mercury	**0.13**	0.023	0.034	0.023	<0.020	0.02	0.026	NA
Aluminum	**NSA**	11000	12000	14000	11000	9100	9400	NA
Arsenic	**7.24**	6.1	4.5	6.6	<1.0	1.7	2.9	NA
Barium	**NSA**	120	200	290	72	88	110	NA
Beryllium	**NSA**	0.58	0.66	0.62	0.48	0.43	0.46	NA
Cadmium	**1**	0.31	0.54	0.6	<0.25	<0.25	<0.25	NA
Calcium	**NSA**	3200	3800	3800	14000	1500	2500	NA
Chromium (total)	**52.3**	16	22	17	14	13	31	NA
Cobalt	**NSA**	6.4	12	11	5.1	4.8	5.6	NA
Copper	**18.7**	12	15	14	10	10	12	NA
Iron	**NSA**	17000	17000	18000	10000	11000	13000	NA
Lead	**30.2**	8.7	15	7.3	6.5	6.1	8.5	NA
Magnesium	**NSA**	2900	3100	3100	9800	2600	2700	NA
Manganese	**NSA**	350	860	1700	200	160	330	NA
Nickel	**15.9**	15	24	22	11	11	14	NA
Potassium	**NSA**	1300	1400	1400	1300	1200	1300	NA
Sodium	**NSA**	71	85	63	110	160	83	NA
Vanadium	**NSA**	32	33	34	27	24	27	NA
Zinc	**124**	52	64	54	36	45	49	NA

Notes:
All results in mg/kg (parts per million)
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 3060A/7196A
ESV - EPA Regional 4 Ecological Screening Values for Sediment (November 30, 2001).
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
NSA - No Screening value Available

Mercury analyzed by method 7470A, all other metals analyzed by method 6010B.
PCB's analyzed by 8082
Benzene, Toluene, Ethylbenzene, Total Xylenes, and Naphthalene analyzed by method 8260B
NA - Not Analyzed
Bold result exceeds RSL

Table 3
Ground Water Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location		CR-1		CR-2		CR-3		CR-4			CR-5		CR-6	
Sample ID	MCL	FITE-CRW-1	FITE-CRW-1 Filtered Analysis	FITE-CRW-2	FITE-CRW-2 Filtered Analysis	FITE-CRW-3	FITE-CRW-3 Filtered Analysis	FITE-CRW-4	FITE-CRW-4 Filtered Analysis	FITE-CRW-11, duplicate of CRW-4	FITE-CRW-5	FITE-CRW-5 Filtered Analysis	FITE-CRW-6	FITE-CRW-6 Filtered Analysis
Collect Date		10/8/2009	10/8/2009	10/5/2009	10/5/2009	10/5/2009 - 10/8/2009[1]	10/5/2009 - 10/8/2009[1]	10/6/2009	10/6/2009	10/6/2009	10/6/2009	10/6/2009	10/7/2009	10/7/2009
Chromium,Hexavalent	NSA	<0.010		0.012		<0.010		<0.010		<0.010	<0.010		<0.010	
TAL Metals														
Antimony	**0.006**	0.0014		<0.0010		0.0020		**<0.010**	<0.0010	NA	<0.0010	<0.0010	0.0011	
Arsenic	**0.010**	**0.20**	<0.0010	**0.019**	<0.0010	**0.032**	<0.0010	**0.18**	<0.0010	NA	0.0023	0.0021	**0.042**	<0.0010
Thallium	**0.002**	**0.0087**	<0.0010	<0.0010		0.0013		**<0.010**	<0.0010	NA	<0.0010	<0.0010	**0.0026**	<0.0010
Aluminum	NSA	640		21		62		780		NA	1.6	<0.10	110	
Barium	2	**6.9**	0.060	0.43		1.0		18	0.28	NA	0.16	0.13	1.3	
Beryllium	**0.004**	**0.030**	<0.0010	<0.0020		0.0036		**0.050**	<0.0010	NA	<0.0020	<0.0020	**0.0081**	<0.0010
Cadmium	**0.005**	**0.028**	<0.00050	<0.0050		**0.0066**	<0.00050	<0.025		NA	<0.0050	<0.0050	**0.011**	<0.00050
Calcium	NSA	520		140		100		370		NA	30	31	60	
Chromium	**0.1**	**1.2**		0.054		**0.13**		**1.4**		NA	<0.010	<0.010	**0.17**	
Cobalt	NSA	0.36		0.021		0.048		0.35		NA	<0.010	<0.010	0.096	
Copper	**1.3**	1.1		0.039		0.12		1.1		NA	<0.020	<0.020	0.13	
Iron	NSA	940		43		150		940		NA	2.1	<0.10	180	
Lead	**0.015**	**0.52**	<0.0010	**0.023**	<0.0010	**0.053**	<0.0010	**0.87**	<0.0010	NA	<0.0050	<0.0050	**0.098**	<0.0010
Magnesium	NSA	400		90		66		180		NA	9.7	9.5	47	
Manganese	NSA	25		1.8		6.4		42		NA	0.38	0.29	9.8	
Nickel	NSA	1.1		0.046		0.13		1.2		NA	<0.020	<0.020	0.19	
Potassium	NSA	48		3		7.7		52		NA	4.3	4.2	13	
Selenium	**0.05**	**0.40**	<0.0010	<0.020		**0.052**	<0.0010	**0.38**	<0.0010	NA	<0.020	<0.020	<0.020	
Sodium	NSA	20		53		42		17		NA	6.3	6.6	18	
Vanadium	NSA	1.4		0.066		0.18		1.1		NA	<0.010	<0.010	0.25	
Zinc	NSA	3.4		0.12		0.4		4.4		NA	0.18	0.12	0.44	

Notes:

All results in mg/l (parts per million)
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 7196A
PCB's analyzed by 8082
Metals samples were unfiltered unless otherwise noted. Filtering was conducted by the laboratory.
Mercury analyzed by method 7470A, other metals analyzed by 6010B or 6020.
NSA - No Screening value Available

NA - Not Analyzed
NS - Not Sampled
1 -CRW-3 Metals collected on 10/5/09; Cyanide, Hexavalent Chromium, and PCBs collected on 10/8/09
2 - Insufficient water for collection of Cyanide and Metals from NDW-3
3 - Insufficient water for collection of Cyanide and Hexavalent Chromium in NDW-4.
MCLs - Maximum Contaminant Level. The highest level of a contaminant that is allowed in drinking water.
Bold results exceed MCL, as of November 2011

Table 3 - continued
Ground Water Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location		CR-7		CR-8		ND-1	ND-2			ND-3	ND-4	SL-1			SL-3	
Sample ID	MCL	FITE-CRW-7	FITE-CRW-7 Filtered Analysis	FITE-CRW-8	FITE-CRW-8 Filtered Analysis	FITE-NDW-1	FITE-NDW-2	FITE-NDW-2 Filtered Analysis	FITE-NDW-2 6020 Analysis	FITE-NDW-3[2]	FITE-NDW-4[3] Filtered Analysis	FITE-SLW-1	FITE-SLW-1 Filtered Analysis	NDW-5, dup. of SLW-1, Filtered Analysis	FITE-SLW-3	FITE-SLW-3 Filtered Analysis
Collect Date		10/7/2009	10/7/2009	10/7/2009	10/7/2009	10/7/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009	10/8/2009
Chromium,Hexavalent	NSA	<0.010		<0.010		<0.010	<0.010			<0.010	NS[3]	<0.010		<0.010	<0.010	
TAL Metals																
Antimony	**0.006**	<0.0050		<0.0050		<0.0010	<0.0010	<0.0010		NS[2]	<0.0010	0.0014		<0.0010	0.0013	<0.0010
Arsenic	**0.010**	**0.31**	<0.0010	**0.22**	<0.0010	0.0017	**0.031**	0.0010		NS[2]	0.0045	**0.015**	<0.0010	<0.0010	**0.029**	<0.0010
Thallium	**0.002**	**0.0075**	<0.0010	**0.0090**	<0.0010	<0.0010	**0.0041**	<0.0010		NS[2]	<0.0010	0.0011		<0.0010	**0.0020**	<0.0010
Aluminum	NSA	290		370		2.8	150	<0.10		NS[2]	6.3	39		<0.10	97	<0.10
Barium	2	**3.2**	0.056	**5.4**	0.19	0.26	1.5	0.044		NS[2]	0.19	0.53		0.090	1.2	0.12
Beryllium	**0.004**	**0.019**	<0.0010	**0.021**	<0.0010	<0.0020	**0.012**	<0.0020		NS[2]	<0.0020	0.0026		<0.0020	**0.0058**	<0.0020
Cadmium	**0.005**	**0.026**	<0.00050	**0.033**	<0.00050	<0.0050	**0.0077**	<0.0050		NS[2]	<0.0050	<0.0050		<0.0050	**0.0051**	<0.0050
Calcium	NSA	95		160		57	220	71		NS[2]	88	100		69	100	68
Chromium	**0.1**	**0.45**		**0.61**		<0.010	**0.23**	<0.010		NS[2]	0.012	0.053		<0.010	**0.12**	<0.010
Cobalt	NSA	0.25		0.23		<0.010	0.11	<0.010		NS[2]	<0.010	0.024		<0.010	0.047	<0.010
Copper	1.3	0.32		0.56		<0.020	0.34	<0.020		NS[2]	<0.020	0.064		<0.020	0.14	<0.020
Iron	NSA	500		650		4.5	310	<0.10		NS[2]	12	72		<0.10	170	<0.10
Lead	**0.015**	**0.27**	<0.0010	**0.28**	<0.0010	0.011	0.13	<0.0050		NS[2]	<0.0050	**0.021**	<0.0010	<0.0050	**0.052**	<0.0050
Magnesium	NSA	92		160		29	160	49		NS[2]	52	62		34	66	30
Manganese	NSA	19		15		0.33	3.6	0.024		NS[2]	1.0	0.82		0.15	1.8	0.034
Nickel	NSA	0.46		0.62		<0.020	0.30	<0.020		NS[2]	<0.020	0.051		<0.020	0.13	<0.020
Potassium	NSA	20		29		2.1	12	0.86		NS[2]	2.2	4.9		0.54	9.1	0.8
Selenium	**0.05**	**<0.10**	<0.0010	**<0.20**	<0.0010	<0.020	<0.020	**0.064**	0.042	NS[2]	<0.020	<0.020		<0.020	<0.020	0.027
Sodium	NSA	13		44		12	130	140		NS[2]	20	7.4		7.1	9.0	8.6
Vanadium	NSA	0.63		0.89		<0.010	0.38	<0.010		NS[2]	0.016	0.094		<0.010	0.21	<0.010
Zinc	NSA	1.1		2.1		0.05	1.3	<0.030		NS[2]	0.041	0.26		<0.030	0.58	<0.030

Notes:

All results in mg/l (parts per million)
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 7196A
PCB's analyzed by 8082
Metals samples were unfiltered unless otherwise noted. Filtering was conducted by the laboratory.
Mercury analyzed by method 7470A, other metals analyzed by 6010B or 6020.
NSA - No Screening value Available

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1 -CRW-3 Metals collected on 10/5/09; Cyanide, Hexavalent Chromium, and PCBs collected on 10/8/09
2 - Insufficient water for collection of Cyanide and Metals from NDW-3
3 - Insufficient water for collection of Cyanide and Hexavalent Chromium in NDW-4.
MCLs - Maximum Contaminant Level. The highest level of a contaminant that is allowed in drinking water.
Bold results exceed MCL, as of November 2011

Table 4
Surface Water Sampling Results Summary
Fite Road Property Acquisitions Studies
Fite Road Realignment Project
Shelby County, TN

Sample Location		D-1	D-2	D-3	D-4	D-5	D-6
Sample ID	**ESV**	FITE-DW-1	FITE-DW-2	FITE-DW-3	FITE-DW-4	FITE-DW-5	FITE-DW-6
Collect Date		10/12/2009	10/12/2009	10/12/2009	10/12/2009	10/12/2009	10/12/2009
TAL Metals							
Arsenic	**0.190**	0.0036	0.0045	0.0048	0.0034	0.0032	0.0029
Aluminum	**0.087**	**0.62**	**0.80**	**0.49**	**0.43**	**0.21**	**0.44**
Barium	NSA	0.11	0.11	0.10	0.11	0.12	0.11
Calcium	NSA	40	40	40	41	48	40
Chromium	**0.11732**	0.012	0.014	0.010	<0.010	<0.010	<0.010
Iron	**1.000**	0.97	**1.2**	0.67	0.59	0.30	0.63
Magnesium	NSA	16	16	16	17	16	18
Manganese	NSA	0.20	0.22	0.12	0.11	0.070	0.13
Potassium	NSA	4.3	4.5	4.3	3.9	3.7	4.0
Sodium	NSA	7	7.3	7.3	7.8	9.2	9.2
Vanadium	NSA	<0.010	<0.010	<0.010	<0.010	<0.010	<0.010
Zinc	**0.05891**	<0.030	<0.030	<0.030	<0.030	0.046	<0.030

Notes:
All results in mg/l (parts per million)
Samples analyzed by Environmental Science Corp. in Mt. Juliet, TN
Cyanide analyzed by method 9012B
Hexavalent Chromium analyzed by method 7196A
PCB's analyzed by 8082
Metals samples were unfiltered unless otherwise noted. Filtering was conducted by the laboratory.
Mercury analyzed by method 7470A, other metals analyzed by 6010B or 6020.
NA - Not Analyzed
NS - Not Sampled
1 -CRW-3 Metals collected on 10/5/09; Cyanide, Hexavalent Chromium, and PCBs collected on 10/8/09
2 - Insufficient water for collection of Cyanide and Metals from NDW-3
3 - Insufficient water for collection of Cyanide and Hexavalent Chromium in NDW-4.
ESV - EPA Regional 4 Ecological Screening Values for Surface Water (November 30, 2001).
Bold results exceed MCL NSA - No Screening value Available

SHELBY COUNTY

Fite Road Extension

Soils and Geology Survey Report

PSI Project No.: 502-35079

Prepared by

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February 16, 2004

Information To Build On
Engineering • Consulting • Testing

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TDOT

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1.0 LOCATION AND DESCRIPTION

The area for the proposed extension is located between an existing section of Fite Road and Highway 51 in Shelby County, Tennessee. Project information regarding the proposed construction was obtained from Fisher & Arnold, Inc. We understand that the proposed construction will consist of extending/widening Fite Road for a total project length of approximately 8,000 feet. The proposed project will span between an existing portion of Fite Road east of the existing railroad track to Highway 51. The extension of Fite Road will be mainly west of the existing railroad track, with most of the project east of the tracks consisting of widening of the existing roadway. In addition, an overpass will also be constructed over the existing railroad track. A typical cross section provided indicated the proposed extension will consist of a 44-foot wide roadway not including curb and gutter, shoulders, and cut/fill slopes. The total maximum right of way width of the project will be approximately 240 feet, which includes the lanes of traffic, curb and gutter, shoulders and fill slopes at the area of the highest fills (fill height of ~40 feet). The highest fills will be required at the ICRR tracks overpass, and these fills are proposed to be constructed at a 2H:1V side slope. A previous geotechnical exploration, for the proposed structures only, was addressed in ETS/PSI Report No. M4250/502-95013 dated June 10, 1999 prepared for Fisher & Arnold, Inc.. It is noted that at that time, ETS/PSI was not asked to perform exploration on the roadway, only on the structures.

Grading information provided indicated that grading along most of the extension will consist mainly of fill, 5 to 7 feet in thickness. However, fills up to about 40 feet in height will be required at the bridge overpass along the railroad right of way. During the construction phase, a portion of the eastern embankment will require a temporary 1H:1V slope with a maximum height of approximately 12 feet.

The proposed bridge over the ICRR tracks will be located from station 76+62.98 to station 78+24.98. Based on the preliminary bridge plans, the proposed bridge will be 162 feet in length and 62 feet in width. Bridge construction will consist of a concrete deck supported by AASHTO Type III beams. Two abutments and two bents will provide support for the proposed bridge. The proposed bents will consist of a reinforced bent cap supported on three 3-foot diameter concrete columns resting on pile caps. Preliminary plans indicate the bridge abutment slopes will be armored with riprap. The stationing and ground surface elevations are summarized in the following table:

Structural Element	Station	Existing Ground Elevation (ft)	Bottom of Pile Cap Elevation (ft)
Abutment #1	76+62.98	232	268.8
Bent #1	77+07.98	240	243.5
Bent #2	77+79.98	238	243.5
Abutment #2	78+24.98	237	268.8

The geotechnical recommendations presented in this report are based on the available project information, bridge and roadway locations, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

2.0 TOPOGRAPHY AND DRAINAGE

The general topography of the project area is comprised of gently rolling hills developed on the loess deposits characteristic of the area. This rolling topography is due to the erosion which occurred in the underlying tertiary deposits of sand, gravel, and clay during the time these soils were exposed. As the overlying loess soils were deposited by wind action these existing topographic features were maintained and amplified by subsequent surface erosion and deposition.

The topography at the existing railroad track area has been somewhat altered to accommodate the existing structure and track. These alterations consist of fills for the construction of the existing bridge and elevated track level.

At the time of exploration, the project area was in several distinct conditions. The area between Highway 51 and Millington Road along the centerline of the proposed road consisted of wooded areas. The trees in this area varied in age from saplings to mature trees. The area between Millington Road and the railroad track consisted of a cultivated field with several isolated patches of wooded areas. The area between the railroad track and the existing section of Fite Road was covered with weeds, and a drainage ditch (about 10 feet deep) crossed this area in a general northwest to southeast orientation. The remaining project area to the east, mostly coincided with the existing Fite Road roadway and as such was mainly an existing pavement structure.

Much of the area along the project length was relatively level with surface drainage flowing towards existing drainage swales or ditches. Based on site observations it appeared that the drainage of the site generally tended toward the east.

3.0 GEOLOGY OF THE AREA

This portion of Shelby County is located in a narrow band which parallels the eastern bluffs of the Mississippi River known as the "Loess Hills". This band lies in a physiographic subdivision known as the East Gulf Section of the Atlantic and Gulf Coastal Plains Province and is included in a portion of the Section called the Mississippi Embayment. The Embayment is a bell-shaped arm of the Gulf of Mexico outlined roughly by Little Rock, Arkansas on the west; Cairo, Illinois on the north; and the Lower Tennessee River on the east.

Structurally, the Mississippi Embayment is a downwarped, partly down-faulted, trough in Paleozoic rocks. The axis of the trough has migrated in past geologic time but now approximates the course of the Mississippi River. The trough has been filled with unconsolidated gravels, sands, and clays ranging in age from upper Cretaceous to Recent.

In Tennessee, the unconsolidated sediments reach their maximum thickness at Shelby County where they are from 2,700 to 3,000 feet thick. Late in the Tertiary Period and early in the Quaternary Period streams carried extensive quantities of sand and gravel into the area and formed wide-spread terraces of these materials mixed with varying amounts of silt and clay on top of the older Tertiary deposits. Aeolian, or wind-blown silts and clays (loess) comprise the presently exposed surface in the Shelby County area. In general, the thickness of the loess is 75 feet or more at the bluffs of the Mississippi River, but thins out to the east and disappears within about 50 miles.

4.0 DRILLING AND SAMPLING TECHNIQUES

4.1 Drilling and Sampling

The site subsurface conditions were explored with 13 soil test borings along the proposed roadway centerline. The borings ranged in depth from 10 to 60 feet. The boring locations and depths were selected by PSI. The borings were located in the field by PSI personnel using the staked roadway centerline as reference. In general, the borings were located along the roadway centerline. Elevations of the ground surface at the boring locations were estimated from the project drawings provided by Fisher & Arnold, Inc. The following table lists the boring station locations, depths, and existing surface elevations:

Table 1. Boring locations, stations, depths, and elevations.

Boring	Station	Depth (feet)	Elevation +/- (feet)
B-1	30+00	10	258
B-2	40+00	10	246
B-3	50+00	10	239
B-4	60+00	10	240
B-5	66+00	10	238
B-6	72+00	40	238
B-7	76+00	60	237
B-8	79+00	60	235
B-9	83+00 (north)	40	237
B-10	83+00 (center)	40	235
B-11	83+00 (south)	40	235
B-12	95+00	10	241
B-13	105+00	10	243

The borings were advanced utilizing hand auger, hollow stem auger, and mud rotary drilling methods, and soil samples were routinely obtained during the drilling process. Borings B-1 and B-2 were drilled using hand auger drilling methods due to the existing conditions (wooded area), and Borings B-12 and B-13 were drilled using hand auger drilling methods due to spatial limitations along the existing shoulder of the roadway. The remaining borings were drilled using either hollow stem auger drilling methods or a combination of hollow stem auger and mud rotary drilling methods. In general, once groundwater was encountered, hollow stem auger drilling methods were ceased and the hole was advanced using rotary wash drilling methods. Drilling techniques were accomplished in accordance with AASHTO procedures. Upon completion of drilling the borings were backfilled

with a cement-bentonite mixture and/or auger cuttings. Borings in excess of 30 feet are required to be backfilled with a cement based material in Shelby County.

The sampling of soils and in-situ testing were accomplished in accordance with AASHTO procedures. Nine thin-walled (Shelby) tube samples were obtained in accordance with AASHTO Designation T-207-87 using factory new tubes. Soils were tested [standard penetration test (SPT)] and sampled using split barrel type sampling techniques at intervals of 2½ feet in the upper 10 feet and at 5-foot intervals thereafter. Penetration test and split barrel sampling were conducted in accordance with AASHTO Designation T-206-87. SPT were performed using both a rope and cat-head safety hammer and a CME automatic hammer, and the N-values listed on the boring logs represent N_{60} values. The SPT values listed on the boring logs for Borings B-1, B-2, B-12, and B-13 were determined based on correlation with Dynamic cone penetrometer readings.

4.2 Subsurface Conditions

Topsoil was encountered in many of the borings. In general, the thickness of topsoil ranged from 2 to 3 inches. The ground surface at the remaining boring locations had been stripped of topsoil. The soils encountered at the 13 borings may be divided into 3 general strata. The upper stratum consisted of fine-grained soils, the second stratum consisted of coarse-grained soils, and the third stratum consisted of fine-grained soils. These soil strata are discussed separately below.

The upper stratum of fine-grained soil extended from the base of the topsoil or ground surface to depths ranging from about 27 to 37 feet or the terminal depths at the boring locations. Based on results of Atterberg limits and visual classification, these soils were classified as A-4, A-6, A-7, or A-7-6 in accordance with the AASHTO classification system or as low plasticity silty/lean clay (CL) and clayey silt (ML) or high plasticity clay (CH) in accordance with the USCS. Standard penetration N-values within these fine-grained soils indicated consistencies of soft to hard, with a majority of the soils having consistencies in the firm range.

The second stratum, encountered in the deeper borings, consisted of sandy soils. This stratum extended to the boring terminal depths, except at Boring B-7 where it extended to a depth of about 57 feet. Based on visual classification and grain size analyses, these soils were classified as A-1, A-1-a, A-1-b, A-2-4, or A-3 in accordance with the AASHTO classification system or as poorly graded sand (SP), or silty sand (SM) in accordance with the USCS. Standard penetration N-values within these granular soils generally indicated the materials to be medium dense to very dense. A sample of loose soil was encountered in Boring B-9 at a depth of 29 to 30.5 feet.

The third stratum of fine-grained soil, encountered only in Boring B-7, extended to the terminal depth at this boring location. Based on visual classification, this soil was classified as A-7 in accordance with the AASHTO classification system or as a high plasticity clay (CH) in accordance

with the USCS. The standard penetration N-value within this fine-grained soil indicated a consistency of stiff.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the appendix should be reviewed for specific information at individual boring locations. These records include soil descriptions, stratifications, penetration resistances (equivalent safety hammer), locations of the samples and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these boring logs. The samples which were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

4.3 Groundwater Conditions

For borings which were drilled solely with hollow stem auger drilling methods, water levels were measured within the hollow stem auger during drilling and after removal of auger. For borings in which mud rotary drilling methods were used, water levels were recorded only at the point in which water was encountered while drilling with hollow stem augers, at which point the boring was advanced with mud rotary drilling methods. Groundwater measurements at the boring locations are summarized in the following table. Some fluctuations of groundwater levels should be anticipated with changing climatic conditions. The groundwater data presented in this report was obtained at the time of the subsurface exploration.

Table 2. Water level readings at boring locations.

Boring	Water level during drilling (ft)	Cave-in depth (ft)	Water level at completion (ft)
B-1	Dry	Open	Dry
B-2	10	Open	10
B-3	Dry	Open	Dry
B-4	1	9	Dry
B-5	9	Open	9
B-6	15	3	Dry
B-7	15	NA	NA
B-8	NE	NA	NA
B-9	15	13	Dry
B-10	7	13	NA
B-11	14	NA	NA
B-12	Dry	Open	Dry
B-13	Dry	Open	Dry

NA-not applicable due to mud rotary drilling. NE-not encountered, began mud rotary drilling above groundwater.

5.0 LABORATORY TESTING

Select soil samples were tested in the laboratory to determine material properties for our evaluation. Laboratory testing was accomplished in accordance with AASHTO procedures. Tests performed included: moisture content, Atterberg limits, sieve analysis for fine and coarse aggregate, pH, standard Proctor, California bearing ratio (CBR), consolidation test, unconsolidated-undrained triaxial test (Q), consolidated-undrained triaxial test with pore pressure measurements (R), and unconfined compression test.

5.1 Moisture Content

Moisture content tests were performed on all soils samples including split barrel and Shelby tube samples in accordance with AASHTO Designation T-265-93. The moisture contents of the fine-grained soils in the upper soil stratum varied from 21 to 32 percent, and the moisture contents in the upper 5 feet ranged from 21 to 30 percent with an average value of 26 percent, which is relatively wet. The moisture contents of the coarse-grained soils in the second stratum varied from 11 to 27 percent. The moisture content of the fine-grained soil in the third stratum was 34 percent. The moisture content for each soil sample is listed on the attached boring logs in Appendix I.

5.2 Atterberg Limits and Sieve Analysis

Atterberg limits and sieve analysis tests were performed on select soil samples to aid in the classification of the soils. The Atterberg limits test was performed in accordance with AASHTO Designations T-89-96 and T-90-96, and the sieves were performed in accordance with AASHTO Designations T-11-91 and T-27-97. The results of these tests are presented on the attached boring logs in Appendix I, and graphic results of the tests are attached in Appendix II.

5.3 pH

Tests to determine the pH of 10 various samples were performed on samples taken from Borings B-8, B-9, B-10, and B-11 at depths varying from 0 to 25 feet. The pH tests were performed in accordance with AASHTO Designation T-289-91. The results of the pH tests indicate a range of 4.3 to 7.8. The results of these tests are tabulated in Appendix II.

5.4 Standard Proctor and California Bearing Ratio

A standard Proctor and a California Bearing Ratio (CBR) test were performed on a bulk soil sample retrieved from the upper 5 feet of the auger cuttings near Boring B-3. The sample was classified as A-6 in accordance with the AASHTO classification system and as silty clay (CL) in accordance with the USCS. The standard Proctor and CBR tests were performed in general accordance with AASHTO Designations T-99-95 Method A and T-193-93, respectively. A minimum of four points were used in obtaining the Proctor results. The maximum dry density and the optimum moisture content of the soil sample were determined to be 103 pcf and 19.5 percent, respectively. The CBR test was performed the bulk sample using a single point compacted in three lifts with the 5.5-pound Proctor hammer. The CBR sample was remolded to a dry density of 97.3 pcf (94.5% compaction) at a moisture content of 21.2 percent. The sample was then soaked for 96 hours with a 10-pound weight as surcharge. After soaking, the sample was tested as specified using a 3 square inch piston. The penetration of the piston on the soil sample indicated CBR values at 0.1-inch and 0.2-inch penetration of 2.4 and 2.2, respectively. The final moisture contents of the sample in the top inch and center of the sample were 25.3 percent and 23.2 percent, respectively. Results of these tests are located in Appendix III.

5.5 Consolidation Test

Consolidation tests were performed on two soil samples extruded for Shelby tubes. The tests were performed on samples from Borings B-6 at 6 to 8 feet and B-8 at 16 to 18 feet in accordance with AASHTO Designation T-216-94. The specific gravity of the samples was estimated to be 2.7. The consolidation tests indicated both samples were overconsolidated. The sample from Boring B-6 had a overconsolidation ratio (OCR) of approximately 2.5, and the sample form Boring B-8 had a OCR of approximately 1.5. Results of the consolidation tests are attached in Appendix VI.

5.6 Triaxial Compression Test

Triaxial compression tests were performed on multiple samples in order to estimate the shear strength parameters of the soils. The triaxial tests performed included three unconsolidated-undrained (Q) compression tests and one consolidated-undrained (R) with pore pressure measurements compression test. These tests were conducted in accordance with AASHTO Designations T-296-94 and T-297-94. The results of the Q-tests indicated undrained shear strengths (ϕ=0 concept) in the range of 1.0 to 2.1 ksf. The results of the R-test indicated total shear stress strength parameters for cohesion of 0.3 ksf and angle of internal friction of 18 degrees and effective stress shear strength parameters for cohesion of 0 ksf and drained angle of internal friction of 31 degrees. The total stress failure envelope for the Q- and R-tests were determined using Mohr's circles at failure, and the effective stress failure envelope for the R-test was determined using both

Mohr's circles at failure and also p-q curves. Results of the triaxial tests are attached in Appendix IV.

5.7 Unconfined Compression Test

Unconfined compression (UC) tests were performed on ten soil samples in accordance with AASHTO Designation T-208-92. These tests were performed with the samples at their natural moisture contents and degree of saturation. The results of the tests indicated undrained shear strengths (ϕ=0 concept) in the range of 0.47 to 1.3 ksf. Results of the unconfined compression tests are attached in Appendix V.

6.0 ENGINEERING ANALYSES

6.1 Slope Stability Analyses

The proposed overpass embankments for the bridge over the railroad tracks will require maximum fill of approximately 40 feet. The final slope is proposed to be constructed with 2H:1V side slopes. In addition, due to space limitations during the construction phases, portions of the embankment on the eastern side of the railroad tracks will be required to be constructed with temporary slopes of 1H:1V. This temporary slope will have a maximum height of approximately 12 feet and taper down to existing grade. Once traffic is diverted to the new overpass the slope will be flattened to the design slope of 2:1.

The slope stability analyses utilized for this project were based on two dimensional limit plastic equilibrium methods. In this method, a trial failure surface is assumed. The mass of soil above the failure surface is divided into vertical strips called slices. The forces acting on each slice are estimated. The forces are separated into those tending to cause failure and those tending to resist failure. The sum of the two sets of forces for a trial failure surface are tabulated and compared by dividing the forces resisting failure by those causing failure. This ratio is termed the factor of safety (FS). When the FS is 1.0, failure of the slope is imminent. When the FS exceeds 1.0, it is assumed the slope is stable. However, there are certain minimum factors of safety which, by experience, are acceptable. In general, these minimum values of FS depend on the consequences of failure as well as the anticipated loading condition. Computer methods are generally employed to allow multiple surfaces to be explored. The computer program GSTABL7 by Gregory Geotechnical Software was used in the estimation of the factors of safety of the various sections.

To determine the forces necessary to perform the analyses, the slope geometry, stratigraphy, soil strength parameters, groundwater levels, and extraneous loads must be determined. The topographic survey is generally used in modeling the slope geometry. Borings and laboratory tests are used in the determination of the stratigraphy of the site and the soil parameters. Soil shear strength, unit weights, compression characteristics, etc. are determined from this data. The groundwater level was taken to be at an elevation of about +225 feet. The proposed embankments were analyzed with slopes of 2:1. It should be noted that the proposed slopes were assumed to be constructed at a 2:1 slope, and the factors of safety are adequate; however, constructing the slopes to a 3:1 slope may provide better maintenance and erosion control.

The shear strength parameters and other properties of the cohesive (fine-grained) soils were based on correlation with SPT N-values, Atterberg limits, unit weight, unconfined compression tests, and triaxial shear tests (Q and R tests). The results of the triaxial tests are located in Appendix IV. For the sandy soils, drained angle of internal friction (ϕ) and unit weights were based on results of in-situ testing (N-values). The shear strength parameters for the embankment fill soils were estimated based on general ranges of fine-grained materials. We recommend that once the embankment soils are

chosen, PSI be provided with samples of the soil for testing purposes. The shear strengths used in the analyses are tabulated on the design plates in Appendix VI. Analyses of the sections were performed using unconsolidated-undrained (UU) and consolidated-undrained (CU) shear strengths to model conditions during and immediately after embankment construction, and analyses of the sections considering long term conditions were performed using consolidated-drained (CD) shear strengths. Analyses were also performed considering a seismic event using the CU shear strengths.

The seismic forces on the slopes were evaluated using a pseudo-static analysis. This analysis approach involves computing a force (seismic), in addition to the standard analysis forces, which is equal to the weight of individual soil wedges times a coefficient of acceleration [horizontal (k_h) and/or vertical (k_v)]. In general, the vertical coefficient of acceleration, k_v, is taken as zero, which was the case in these analyses. The magnitude of the horizontal coefficient of acceleration, k_h, may be estimated using a variety of methods or basic assumptions; however, our determination of k_h was based on an equation as proposed by Hynes-Griffin and Franklin (1984): $k_h = \frac{1}{2}a_{max}/g$ (a_{max} = PGA at the surface). The peak ground acceleration (PGA) used in the analyses was obtained from the United States Geologic Survey (USGS) National Seismic Hazard Mapping Project web page and was determined to be 0.15g. This value is based on a seismic hazard of 10 percent probability of exceedence (PE) in 50 years (475-year return period). The value of k_h used in these analyses was 0.075g based on the source, probability level, and equation discussed above.

Based on the results of the slope stability analyses, it appears that constructing the full 40-foot high embankments in one stage on unmodified in-situ soils will not be possible. The recommended possible construction alternatives include two stage construction with allowance for the in-situ soils to consolidate, single stage construction with rammed aggregate pier reinforced in-situ soils, or semi-stage construction incorporating sand or wick drains to increase the rate of consolidation of the in-situ soils. These alternatives are discussed below.

6.1.1 Staged Construction

Initial evaluation of the 40-foot embankment under UU shear strength parameters (after construction case) indicated a FS of 1.1 which is less than the minimum recommended FS of 1.3. This indicates that staging of the full embankment fill height will be required without other remedial measures. To evaluate the staging process, subsequent analyses were conducted on sections which had smaller fill heights until an adequate FS was obtained. It was estimated that a section with a fill height of 30 feet possessed an adequate factor of safety using the UU shear strength parameters. A subsequent analysis was then performed for the full 40-foot fill height based on CU shear strength corresponding to the initial stage construction assumed to be 30 feet in height. We recommend that the second stage of construction begin once approximately 90 percent of the predicted settlements in the in-situ soils has occurred or settlement plates indicate settlements have subsided. Results of the analyses for this alternative are included in Appendix VI, and the results of specific analyses are listed in Table 3.

Table 3. Results of Staged Construction Slope Stability Analyses.

	Recommended Minimum Factor of Safety	Calculated Factor of Safety
Fill Constructed in One Stage to 40 feet	1.3	1.1
Stage 1- Fill Constructed to Maximum Height of 30 feet	1.3	1.5
Stage 2- Fill Constructed to Final Height of 40 feet after Consolidation of In-situ Soils	1.3	1.3
Earthquake (k_h= 0.075g)	1.1	1.1
Long Term Condition	1.5	1.5

In lieu of stage construction of the embankment, alternative measures could be taken to allow a single stage construction of the proposed embankment. These alternatives include using rammed aggregate piers (Geopiers) for shear strength improvements or using sand or wick drains to increase the rate of consolidation. The use of drains will likely require some delay (staging); however, the time will likely be relatively short.

6.1.2 Rammed Aggregate Piers (Geopiers)

The rammed aggregate pier alternative was evaluated using two reinforcing configurations, for ease of discussion termed Case 1 and Case 2. These are discussed separately below. Both cases were analyzed using composite shear strength parameters in the reinforced zone as recommended in the Geopier Foundation Company Inc. Technical Bulletin No. 5 as outlined in the Federal Highway Administration Publication No. FHWA-SA-98-086. The composite shear strength parameters represent a weighted average of the shear strength components of the aggregate pier and the in-situ soil material. The composite shear strengths of the reinforced soils for both cases were modeled based on a ratio of aggregate pier area to gross footprint area equal to 0.2. The analyses for the rammed aggregate piers should be considered preliminary in nature. The specialty contractor installing the piers should be contacted to perform final design analyses. PSI should review the final design analyses when complete.

6.1.2.1 Case 1

As for the unreinforced section, the initial analysis was performed for the full 40-foot cross-section. The area which was assumed to be reinforced with the aggregate piers extended approximately 70 feet either direction from the centerline of the roadway. This limit corresponds to a cross-sectional fill height of 20 feet. The analysis of the 40-foot high embankment using UU composite shear

strength parameters indicates that a single stage of construction can be used. Results of the analyses for this alternative and case are included in Appendix VI, and the results of specific analyses are listed in the table below.

Table 4. Results of Aggregate Pier Reinforced (Case 1) Slope Stability Analyses.

	Recommended Minimum Factor of Safety	Calculated Factor of Safety
Fill Constructed in One Stage to 40 feet	1.3	1.4
Earthquake (k_h= 0.075g)	1.1	1.2
Long Term Condition	1.5	1.5

6.1.2.2 Case 2

Again, the initial analysis was performed for the full 40-foot cross-section. The aggregate piers for this case were taken as extending to the sandy soils with a length of approximately 30 feet. The area which was assumed to be reinforced with the aggregate piers spanned from the point where the fill height ranged from 20 to 30 feet. Thus, the width of reinforced soils would be approximately 20 feet and would be located on both sides of the embankment as well as across the end slope. Although not modeled, this case would also require aggregate piers between the reinforced bands to control differential settlements. The analysis of the 40-foot high embankment using UU composite shear strength parameters indicate that a single stage of construction can be used. Results of the analyses for this alternative and case are included in Appendix VI, and the results of specific analyses are listed in the table below.

Table 5. Results of Aggregate Pier Reinforced (Case 2) Slope Stability Analyses.

	Recommended Minimum Factor of Safety	Calculated Factor of Safety
Fill Constructed in One Stage to 40 feet	1.3	1.3
Earthquake (k_h= 0.075g)	1.1	1.2
Long Term Condition	1.5	1.5

6.1.3 Sand or Wick Drains

Stability analyses of the sand or wick drain alternative will be identical to those performed for the staged construction alternative. Again, we recommend that the second stage not proceed (waiting period) until approximately 90 percent of the predicted settlements in the in-situ soils have occurred

or settlement plates indicate settlements have ceased. Preliminary estimates for typical spacing of drains (~5 feet) which extend approximately 30 feet below the ground surface to the sandy soils indicate the recommended waiting period can be reduced to approximately one month. Once the stage one settlement criteria have been met, the second stage construction can begin.

6.1.4 Temporary Slope

Finally, analysis of the proposed temporary 1:1 slope east of the existing railroad tracks was analyzed using UU shear strength parameters. This slope was reported to have a maximum height of 12 feet. The analysis indicated a factor of safety in excess of 3 for this condition. The results of this analysis are included in Appendix VI.

6.2 Settlement Considerations

We have performed calculations to estimate the potential settlements which may occur due to the proposed embankments for both unreinforced (in-situ) and reinforced (rammed aggregate piers) soils.

6.2.1 In-Situ Soils

For the unreinforced soils, settlement calculations were performed for fill heights of 20 feet, 30 feet, and 40 feet. These analyses were performed using the results of the consolidation tests performed on samples from B-6 at 6 to 8 feet and B-8 at 16 to 18 feet. Settlements were determined by dividing the subsurface soils into layers, assuming the applied stress was distributed uniformly in the upper soils, using consolidation theory to compute strains for the applied stresses, and summing the computed strains for each layer. Based on the modeled subsurface conditions and the applied stresses, we estimated the following maximum consolidation settlements of the existing unreinforced soils:

Table 6. Results of Settlement Analyses for In-Situ Soil Conditions.

Slope Fill Height (feet)	Estimated Settlements of Unreinforced Soils (inches)
20	4
30	8
40	12

6.2.2 Soils Modified with Rammed Aggregate Piers

For the soils modified with rammed aggregate piers, settlements were calculated for various conditions based on the two cases presented. The analyses were performed using the above mentioned consolidation test results along with recommended settlement analysis procedures (elastic procedures) as outlined in Geopier design literature. The settlements were determined based on upper zone (reinforced zone) settlements using the Geopier analysis procedure and lower zone settlements using consolidation theory. For Case 1, the settlements were estimated for the reinforced zone based on aggregate pier lengths of 20 feet and a lower zone thickness of approximately 10 feet. Settlements for the reinforced zone in Case 2 were estimated based on a reinforced zone with a aggregate pier length of 30 feet and no lower zone thickness.

In addition, several other configurations of rammed aggregate piers were analyzed, as discussed in the next section. These additional aggregate piers were included to control differential settlements. For both Case 1 and 2, rammed aggregate piers were added parallel to the roadway centerline where the fill height varies from 30 feet to 20 feet. These aggregate piers have a length of 10 feet and are spaced on 10-foot centers. Additional aggregate piers were also required between the 20-foot wide reinforced bands proposed in Case 2. These aggregate piers have a length of 15 feet and are spaced on 10-foot centers. The following table lists the various configurations and associated estimated settlements:

Table 7. Results of Settlement Analyses for Rammed Aggregate Pier Soil Conditions.

Case 1	Estimated Settlement of Rammed Aggregate Pier Reinforced Soils (inches)
40' fill height- 20' long aggregate piers on 5' centers	4.5
30'-20' fill height- 10' long aggregate piers on 10' centers	5
Case 2	Estimated Settlement of Rammed Aggregate Pier Reinforced Soils (inches)
20' to 30' fill height (reinforced bands)- 30' long aggregate piers on 5' centers	1.5
40' fill height (between reinforced bands)- 15' long aggregate piers on 10' centers	6
30'-20' fill height- 10' long aggregate piers on 10' centers	5

6.2.3 Fill Soils

In addition to the consolidation of the natural soils, we anticipate the engineered fill of the embankments may also experience some consolidation with time. Studies have shown that settlements can occur within properly compacted engineered fill on the order of about 1 percent of the fill height as the fill mass reaches a state of equilibrium. Therefore, settlements on the order of about 4 to 5 inches may occur within the embankment after construction. We anticipate the time required for these settlements to occur will be less than the time estimated for consolidation of the natural soils, and may occur as the fill is constructed.

6.2.4 Time Rates of Settlements

Estimates of the time rate of consolidation were also calculated for times required for 50 percent and 90 percent consolidation based on double drainage. In addition, we have also estimated the rate of consolidation based on the use of sand or wick drains. The drains were assumed to extend to the sandy soils at a depth of about 30 feet, and a permeable granular layer was also assumed to be placed at the ground surface. An exact design of the potential drain system was not performed, and the estimates provided are based on typical design parameters.

Table 8. Results of Time Rate of Settlement Analyses for In-Situ, Sand or Wick Drains, and Rammed Aggregate Pier Soil Conditions.

Percent Consolidation	Estimated Time Required for Natural In-situ Soils (months)	Estimated Time Required for Soils with Sand or Wick Drains or Rammed Aggregate Piers (months)
50	2	<1
90	6	1

7.0 RECOMMENDATIONS

7.1 Site Preparation

We recommend that topsoil, vegetation, roots, and any soft soils in the construction areas be stripped from the site. The depth of removal should be determined by a representative of the geotechnical engineer at the time of construction. Because of the wooded areas, some clearing and grubbing operations will be required to remove organic matter, roots, stumps and other remnants of the vegetation. Areas disturbed by the tree and stump removal operations should be undercut to suitable undisturbed soils and backfilled with properly compacted structural fill placed and tested in accordance with the recommendations in this report. Utilities should be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations must be backfilled with properly compacted structural fill which is constructed as outlined in Section 7.2 of this report.

After stripping and excavating to the proposed subgrade level, as required, the fill areas should be proof-rolled with a heavily loaded tandem axle dump truck or similar rubber tired vehicle. Soils which are observed to rut or deflect excessively under the moving load, or are otherwise judged to be unsuitable, should be undercut and replaced with properly compacted fill. Many soils encountered at the boring locations indicated either soft or wet soils in the upper 2 to 4 feet. We preliminarily estimate undercut quantities in the range of about 10,000 to 20,000 yd^3 based on the soils encountered at the boring locations; although, we note that it is likely these soils can be processed and compacted in lieu of undercutting. These areas should be carefully observed during construction. In addition, the necessity of these activities will likely be a function of the climatic conditions at the time of construction. The proof-rolling and undercutting activities should be witnessed by a representative of the geotechnical engineer and should be performed during a period of dry weather.

The subgrade soils should be scarified and compacted to at least 95 percent of the standard Proctor maximum dry density as determined by AASHTO T-99-95 Method A for a depth of at least 6 inches below the surface. If fill is required, the structural fill should then be placed on the prepared subgrade.

Particular attention should be directed to the swale/ditch areas during site preparation because wet and/or organic soils are often found in areas such as these. Any such unsuitable materials encountered should be undercut and replaced by properly compacted fill. Past experience indicates that it is not uncommon to find several feet of unsuitable soils within drainage swales/ditches. Any existing steep side slopes shall be sufficiently flattened and any loose materials removed prior to proof-rolling operations. The owner should consider an allowance in the budget for undercut and replacement of such material. If desired, test pits can be excavated in these areas to better define potential undercut requirements.

Relatively wet fine-grained soils were encountered in the borings. Fine-grained soils which have moisture contents more than about 2 to 3 percentage points above the optimum moisture are generally prone to softening when dynamic loads such as those generated by the wheels of construction equipment are imposed upon them even if the soils exhibited substantial strength in an undisturbed state. After disturbance, these fine-grained soils typically rut and deflect significantly and do not provide adequate subgrade support for foundations, pavements, or fill placement.

7.2 Fill Requirements

After subgrade preparation and observation have been completed, fill placement may begin. The first layer of fill material should be placed in a relatively uniform horizontal lift on the prepared subgrade. Fill materials should be free of organic or other deleterious materials, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. Most of the on-site soils are suitable for use as structural fill with the exception of the existing topsoil; however, the moisture content will most likely have to be adjusted to coincide with the moisture range required for structural fill. If a fine-grained silt or clay soil is used for fill, close moisture content control will be required to achieve the recommended degree of compaction.

Structural fill should be compacted to at least 95 percent of standard Proctor maximum dry density as determined by AASHTO Designation T 99-95 Method A. Fill should be placed in maximum lifts of 8 inches of loose material and should be compacted within the range of 3 percentage points below to 2 percentage points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be tested by a representative of the geotechnical engineer prior to placement of subsequent lifts.

After fill placement is complete for the embankments, we recommend the fill be allowed to achieve a state of equilibrium prior to installing the pile. We anticipate a majority of the consolidation settlements in the natural soils will occur as the fill is being constructed. We anticipate the estimated time rate of consolidation for the natural soils will provide ample time for the fill soils to also reach a state of equilibrium.

7.3 Construction Considerations

The upper fine-grained soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils which become wet may be slow to dry and thus significantly

retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork, pavement and foundation construction activities during dry weather.

Also, some of the existing in-situ soils which will be present near the top of exposed subgrades or will be cut and used for fill must be processed to a suitable moisture content (3 percentage points below to 2 percentage points above the standard Proctor optimum moisture). Dry weather with favorable drying conditions (sunshine, low relative humidity, and/or wind) is best suited for these operations. This is especially important at this site considering the relatively wet soils which were encountered in the subsurface exploration.

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for pavements during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. The site grading plan should be developed to provide rapid drainage of surface water away from pavement areas and to inhibit infiltration of surface water beneath the pavements. The grades should be sloped away from the pavement areas.

While limited areas of apparent perched groundwater were encountered in the upper soils at the time the field exploration was conducted, we do not anticipate groundwater issue will be a construction concern. Any water accumulation should be removed from excavations by pumping. Should excessive and uncontrolled amounts of seepage occur, the Geotechnical Engineer should be consulted.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

7.4 California Bearing Ratio

Bulk soil samples were retrieved from the upper 5 feet of the auger cuttings near Boring B-3. Visual classification and Atterberg limits indicated these soils were brown silty clay (CL). Standard Proctor and California Bearing Ratio (CBR) tests were performed on the sample in accordance with AASHTO specifications. The CBR test performed on the soil sample indicated CBR values at 0.1-inch and 0.2-inch penetration of 2.4 and 2.2. Based on the results of these tests we recommend a CBR value of 2.3 be used for the existing silty clay soils.

7.5 Pile Foundations

Once the site has been properly prepared as discussed above, the proposed bridge can be supported on pile foundations. A variety of deep foundations (augered, cast-in-place piles; drilled piers; driven steel H-piles; or driven steel or concrete displacement piles) are available which may produce a satisfactory deep foundation system; however, we understand TDOT prefers to use driven square prestressed concrete displacement piles. Considering this, TDOT has developed pile design tables intended for use with these driven piles. The design tables correlate field N-values with both adhesion (f_s) and end bearing (q_b) values based on soil type (clay, silt, or sand). The attached pile capacity diagrams (Appendix VII) were developed using these tables and can be used to estimate individual pile capacities for driven displacement piles. The individual pile capacities (in tons) estimated using these diagrams represent an ultimate capacity, and as such, an appropriate factor of safety should be applied to these values. We recommend a minimum factor of safety of 2.0 for compression loading be used in the design of the piles. In addition, we recommend the piles be driven a minimum of 10 feet into the sandy soil stratum. We recommend a minimum center to center pile spacing of 2.5 pile widths.

Each pile should have an elevation recorded on the pile head upon completion of driving. Upon completion of driving of the pile group, these elevations should be checked to verify heave of the soil during driving of adjacent piles has not resulted in a pile becoming unseated. If this is found to have occurred, the pile should be re-driven to the original tip elevation. We recommend the estimated pile compression capacities be verified with an in-situ load test for each size and configuration of pile selected for use in accordance with ASTM Designation D-1143, "Standard Method of Test for Piles Under Static Axial Compressive Loads" (quick load test method for individual piles).

It is important to note that if the piles are installed prior to completion of consolidation of the in-situ or embankment soils, negative skin friction forces can be developed on the piles. For piles embedded in the sandy soils, as recommended above, this negative skin friction force could exceed the capacity of the pile. Therefore, we recommend the piles either be installed after settlements have ceased (based on either settlement plate data or estimated time rates stated in this report) or the upper portions of the piles be installed in predrilled holes and be coated with a bituminous material. We note that only small settlements are required to mobilize the negative skin friction forces on the piles.

The pile capacity diagrams are based on the information obtained in the subsurface exploration at the existing grade. However, at the abutment locations, the piles will be driven through approximately 32 feet of fill. As a result, additional capacity could be realized from this fill material provided settlements due to fill loading are complete prior to pile installation. Assuming the fill material is a fine-grained soil (assumed to be a CL/ML soil) compacted to the recommended dry density, adhesion and end bearing values can be determined using the TDOT pile design tables for a N-value of about 8. However, based on the fill height and the potential for even small settlements to occur in the naturally deposited or fill soils after pile installation, we recommend pile capacities be calculated neglecting the potential capacity from the fill soils.

In driving the piles a proper size hammer should be chosen such that the piles are driven at a reasonable rate and such that the piles are not damaged during driving. Once the required pile capacity is identified, PSI can provide information as to the acceptability of a hammer for use on this project.

7.6 Slope Stability

Based on the results of the slope stability analyses, it appears that constructing the full 40-foot high embankments in one stage on unmodified in-situ soils will not be possible. The recommended possible construction alternatives include two stage construction with allowance for the in-situ soils to consolidate, single stage construction with rammed aggregate pier reinforced in-situ soils, or semi-stage construction incorporating sand or wick drains to increase the rate of consolidation of the in-situ soils. These alternatives are discussed below. For all these alternatives, we recommend installing multiple settlement plates and piezometers (or pore pressure transducers) in the embankments. The settlement plates should be installed near the existing ground surface at various fill heights along the centerline of the roadway. We recommend installing the piezometers in the embankment areas along the roadway centerline where the fill height will be equal to or greater than 30 feet. The piezometers should be installed to a depth of about 15 feet below existing grade.

7.6.1 Staged Construction

In regards to the two stage construction alternative, we recommend the embankment initially be filled to a maximum height of 30 feet. The settlement plates should be monitored during embankment construction and after completion of the first stage. Once either the estimated time of consolidation has passed (estimated to be about 6 months) or the settlement plate data indicate movements have subsided, the second phase of construction may commence.

The two alternatives involving rammed aggregate piers (Geopiers) may be constructed in a single, continuous stage of construction. In the analyses conducted, the aggregate piers in the reinforced soil zones were assumed to cover 20% of the reinforced area, and this equates to a center-to-center spacing of approximately 5 feet. Areas where the aggregate piers were installed for settlement control were assumed to be spaced at 10 feet on center. Specific recommendations for the two alternatives are discussed below. For either case and regardless of function, the rammed aggregate piers are not necessary where the fill height is less than 20 feet.

7.6.2 Case 1 – Rammed Aggregate Piers

The reinforced area for this case was taken as an area spanning away from the centerline of the roadway to a fill height of 20 feet in the 40-foot high cross-section, and the aggregate piers were assumed to be 20 feet long. The total width of this cross-section is approximately 230 feet, with the reinforced section covering the middle approximately 140 feet. We recommend this reinforced zone extend along the centerline of the roadway to a point where the maximum fill height is 30 feet. This would equate to a reinforced zone length of approximately 350 feet for each embankment.

Then, parallel to the centerline of the roadway from fill heights ranging from 30 feet to 20 feet, we recommend aggregate piers be 10 feet in length and be installed on 10-foot centers for the purpose of controlling differential movements. As with the reinforced zone, this zone of aggregate piers should span between fill heights of 20 feet based on cross-section fill heights.

For cross-sections with less than 20 feet of fill height, piers do not appear to be necessary.

Based on the above, we have estimated a total of 4,920 rammed aggregate piers will be required for this alternative. The total lineal feet of aggregate piers will be approximately 87,000 feet. Preliminary estimates indicate the cost of Geopiers for both embankments for Case 1 would be in the range of $1,750,000 to $2,000,000.

7.6.3 Case 2 – Rammed Aggregate Piers

The reinforced area for this case was taken as an area spanning between fill heights of 20 to 30 feet in the 40-foot high cross-section, and the aggregate piers were assumed to extend to the sandy soils

with an average length of 30 feet. The total width of this reinforced band would be 20 feet wide. We recommend this reinforced zone extend parallel to the centerline of the roadway to a point where the maximum fill height is 30 feet. This would equate to a reinforced band length of approximately 350 feet for each embankment. This narrow band of reinforced soil would be constructed on either side of the centerline of the proposed roadway. Additionally, for this case, aggregate piers will also be required between the reinforced bands in order to control differential settlements. These aggregate piers should be 15 feet in length and on 10-foot centers.

Again, as in Case 1, we recommend aggregate piers be installed which are 10 feet in length on 10-foot centers in an area which is parallel to the centerline of the roadway for fill heights ranging from 30 feet to 20 feet. This will control differential settlements. As with the reinforced zone, this zone of aggregate piers should span between fill heights of 20 feet based on cross-section fill heights.

For cross-sections with less than 20 feet of fill height, piers do not appear to be necessary.

Based on the above, we have estimated a total of 3,666 rammed aggregate piers will be required for this alternative. The total lineal feet of aggregate piers will be approximately 68,370 feet. Preliminary estimates indicate the cost of Geopiers for both embankments for Case 1 would be in the range of $1,250,000 to $1,500,000.

7.6.4 Sand or Wick Drains

For the alternative involving sand or wick drains the embankments will have to be constructed similarly to the unmodified alternative with the exception that the time required to achieve 90 percent consolidation will be much shorter. The drains should be installed to a minimum depth of 35 feet to ensure the drains penetrate into the sandy soil stratum. The initial lift for the embankments should consist of a granular material such that water transported up the drains may exit the drains. We recommend the drains be installed in the same plan areas (fill height ranges) as described for the aggregate piers. The ratio of the diameter of the zone draining to a drain to the diameter of the drain should be in the range of 40 to 80. Again, we recommend the embankment initially be filled to a maximum height of 30 feet. The settlement plates should be monitored during embankment construction and after completion of the first stage. Once either the estimated time of consolidation has passed (estimated to be about 1 month) or the settlement plate data indicate movements have subsided, the second phase of construction may commence.

7.6.5 General Slope Recommendations

The following are some general issues which should be noted concerning the proposed slopes of the embankments as follows:

- A slope of 2:1 can generally be maintained provided the slope face is protected from

erosion and provided any shallow slope failures (if they occur) are repaired immediately. Since fill soils are typically highly susceptible to erosion and sloughing, the contractor will have to take immediate precautions to protect the constructed slopes from surface water and to provide vegetation or other slope protection measures (e.g., erosion mats, rip rap, etc.). Surface water at the crest of the slope should be collected and removed without allowing the water to flow over the face of the slope. The slope should be periodically inspected for any shallow, surficial slips and repaired. Failure to repair shallow slips in a timely manner may lead to progressive shallow failures, 'sympathetic slips', which may progress into a serious slope situation. The use of a slope of 3:1 would help to mitigate some of the problems discussed above and would be preferable from a maintenance standpoint.

- Water should not be allowed to stand or pond at the crest or at the toe (base) of the slope. Increases in moisture content will tend to soften the existing fine-grained soils and reduce the shear strengths of the soil.

7.7 Earth Retaining Structures

Any below grade walls must be designed to withstand the soil pressures which will be exerted upon them. The magnitude of the earth pressure is dependent upon several factors including the type of loading and the soil conditions. It is recommended that the backfill adjacent to the below grade retaining walls be consistent with backfill material as indicated in drawing STD-10-1 of the Tennessee Department of Transportation 'Standard Structure Drawings'. A positive drainage system should be employed to prevent the accumulation of water (from any source) in the backfill behind the wall.

For drained loading conditions, lateral earth pressure may be estimated using the standard loading diagrams as presented on STD-13-1. The earth forces determined from STD-13-1 assume the wall yields sufficiently such that active earth pressures are developed; however, if earth retaining structures are designed as non-yielding structures these earth forces should be increased by 67 percent to reflect an at-rest earth pressure condition. The actual earth pressures on the walls will vary according to the building material types and the way backfill material is compacted. Heavy compaction equipment and/or over-compaction of fill next to the walls may induce higher pressures than estimated herein. Build-up of hydrostatic pressures behind the earth retaining structure should be avoided.

In regards to sliding, the ultimate horizontal friction resistance may be computed by multiplying the vertical wall load by a coefficient of friction which is dependent on the type of soil in contact with the base of the foundation elements (assumed to be in a horizontal plane). We recommend the

coefficient of friction as stated on STD-13-1 be used for design. Factors of safety recommended on STD-13-1 should be used in design of the earth retaining structures.

Seismic Force

In regards to lateral earth pressures on retaining walls due to seismic forces, estimates of the magnitude of these forces may be made in accordance with methods outlined in Technical Engineering and Design Guides as adapted from the U.S. Army Corps of Engineers, No. 4, Retaining and Flood Walls, Mononobe-Okabe (M-O) Analysis, published by the American Society of Civil Engineers.

In using the M-O analysis procedure we have assumed that the backfill surface beyond the below grade walls will be horizontal with no surcharge loading, the face of the walls will be in a vertical plane, and the walls will be above the groundwater level (or that a drain system will lower the groundwater level). The retaining wall height (or depth) below the highest ground surface is denoted as H with units of feet. Based on the recommended free draining granular backfill discussed above, a value of 30 degrees was selected for the angle of internal friction for the granular backfill and a unit weight of 120 pounds per cubic foot (pcf) was used. The wall friction angle was assumed to be zero.

In order to perform the analyses, it is necessary to make assumptions regarding the seismic event causing the lateral forces on the wall; obviously, a larger event will impose larger forces. For the case at hand, we used a seismic event based on a 10 percent probability of exceedence in 50 years as the design seismic event. The pseudostatic horizontal seismic coefficient of acceleration was calculated to be 0.075 based on the Peak Ground Acceleration (PGA) of this seismic event. The pseudostatic vertical seismic coefficient of acceleration was assumed to be zero.

Analyses utilizing the M-O equations and the above data results in a predicted Lateral Dynamic Component Force (LDCF) for active earth pressure conditions with units of pounds per linear (horizontal) foot of wall of $3H^2$. If the earth retaining structure is designed for an at-rest earth pressure condition in which the structure will be non-yielding the predicted LDCF will be $9H^2$. This dynamic force may be assumed to act at the upper one-third point as relates to the portion of the wall between the top of the backfill and the base. The minimum required deflection for use of an active earth pressure condition is estimated to be about 0.3 percent of the wall height in the granular backfill.

7.8 Seismicity

Shelby County, Tennessee is in the general proximity of New Madrid, Missouri, the epicenter for some of the largest earthquakes in the North American continent. Four earthquakes of about magnitude 8 occurred within this seismic zone during the winter of 1811-1812. Presently, the New

Madrid area contains the highest level of seismicity in the central and eastern parts of the United States.

According to Figure 1607.1.5A of the Standard Building Code, 1999 Edition, the project site is located near the 0.2 effective peak velocity related acceleration coefficient (A_v) contour. Based on Figure 1607.1.5B, the project site is located near the 0.2 effective peak acceleration coefficient (A_a) contour. Upon evaluation of the available data, the site coefficient, S, may be assumed to have a value of 1.2 since the soils encountered in the subsurface exploration most closely approximate standard soil profile type "S2" as defined in Table 1607.3.1.

7.9 Liquefaction

In order for liquefaction to occur, site conditions must include a layer of saturated soil which is susceptible to liquefaction (a loose sand, gravel, or non-plastic silt). In addition, this layer of loose, saturated soil must either be bounded by layers which are relatively impermeable with respect to the liquefiable layer or have a low enough permeability such that the time required for drainage to occur is too long to avoid buildup of excess pore pressures.

For ease of illustration assume we have a liquefiable layer which is bounded by relatively impermeable layers. This means the liquefiable layer has a specific height, H. Looking at a column of saturated soil in this layer with a height of H, all the volume is taken up by either soil particles or water (both of which are taken as incompressible) and the height of the soil matrix is equal to the height of the layer. Assuming the stress conditions in the soil profile have been constant for a long time (i.e. all consolidation has occurred), the soil layers will be at a state of equilibrium (i.e. no excess pore pressure). As a result, the overburden stresses supported by the liquefiable layer (soil and any other loads) will be taken by the grain to grain contact of the soil particles. As the soil column is subjected to shear stresses (earthquake motions), the loose soil particles tend to move closer together forming a denser soil matrix (i.e. the height of the soil matrix is no longer equal to the height of the layer). As this occurs the overburden stress are transferred to the water (i.e. the pore pressure increases) due to the fact that the materials are incompressible and the water is trapped and can not flow out of the column. As the soil matrix densifies (soil matrix height reduces) and the overburden pressures are transferred to the pore water, the available shear strength of the column of soil is reduced and the soil is said to liquefy.

Because water has no shear strength and the grain to grain shear strength of the soil particles is reduced, any shear strength demand placed on the liquefiable layer to resist a bearing capacity failure (shallow foundation, deep foundation, slope stability) or provide adhesion (skin friction) will be drastically reduced until the pore pressure dissipates. Also, the settlements associated with liquefaction result from the densification of the soil matrix and the subsequent dissipation of the pore pressure (outflow of trapped groundwater).

7.9.1 Liquefaction Analysis

This evaluation of liquefaction potential for this study was performed using a software program developed by PSI based on the simplified procedure as proposed by Seed and Idriss with associated modifications derived from a 1996 National Center for Earthquake Engineering Research (NCEER) review of the procedure. The simplified procedure for evaluating liquefaction potential is considered the standard of practice. This procedure was developed using a data base of information collected from sites throughout the world where liquefaction did and did not occur. Empirical correlation between the cyclic stress ratio and either corrected N-values, corrected shear wave velocities, or corrected CPT tip resistance were developed which indicate soil conditions in which liquefaction is either likely or unlikely to occur. The liquefaction potential for this site was evaluated based on corrected N-values. The N-values were corrected for overburden pressure, energy transfer, sampler style, borehole diameter, rod length, and percent fines. The N-values corrected in this manner are termed $(N_1)_{60cs}$.

The moment magnitude, M_w, and PGA used in the analyses were determined to be 6.5 and 0.15g. The value of PGA was determined based on results obtained from the United States Geologic Survey (USGS) National Seismic Hazard Mapping Project web page. The values presented above are consistent with a level of seismic hazard of 10 percent probability of exceedance in 50 years (475-year return period). The groundwater level was assumed to be at a depth of 10 feet below the existing ground surface. Analyses were performed for conditions with minimal fill depths and for the maximum fill depth of 40 feet. The subsurface conditions analyzed consisted of an average soil profile, as defined from all the deeper borings, and a profile consisting of the conditions encountered at Boring B-9.

The results of the liquefaction analyses are presented in Appendix VIII. These plots indicate the calculated cyclic stress ratio (CSR), the cyclic resistance ratio (CRR), and the factor of safety (CRR/CSR) at 5-foot intervals within the sandy soils. The CSR represents the applied load or demand placed on the soil by the seismic event. The CRR represents the capacity available in the soil to resist liquefaction due to the applied load. Therefore, liquefaction potential would be indicated at depths where the CRR is less than the CSR (i.e. factor of safety less than 1). Generally it is recommended the CRR should be a minimum of 1.2 times the CSR (factor of safety $\geq$ 1.2). Based on the soil conditions, assumed parameters, and seismic hazard level; the analyses indicate that liquefaction of the sands at the site are unlikely. However, it should be noted that liquefaction is a function of the seismic event; therefore, liquefaction may be possible if a stronger seismic event (larger magnitude or PGA) occurs than what was determined for use in these analyses.

REPORT LIMITATIONS

The recommendations submitted in this report are based on the available subsurface information obtained by PSI and design details furnished by Fisher & Arnold, Inc. for the proposed project as delineated in this report. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the recommendations are required. If PSI is not retained to review these changes, PSI will not be responsible for the impact of those conditions on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Fisher & Arnold, Inc. for the specific application to the proposed Fite Road Extension in Shelby County, Tennessee.