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April 4, 1997

Robert Lemmer P.G.
Geologist Supervisor
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Arkansas Department of Pollution Control & Ecology
8017 I-30, P.O. Box 8913
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SUBJECT: Submittal of Revised Sections of Volume 3 Sunray Class I Permit Modification, Hydrogeological and Geotechnical Report, Sunray, Tontitown, Arkansas

Dear Bob:

Genesis Environmental Consulting, Inc. (GEC) is submitting the following revisions to the Hydrogeological and Geotechnical Report submitted to ADPC&E on January 21, 1997 associated with the Sunray Class I Permit Modification. These revisions reflect the results of additional field work and the outcome of the dye test. The enclosed submittal contains:

- A complete revised text portion of the report. All associated Figures, Tables and Appendices that were previously submitted will need to be combined with this text;
- Additional boring logs to be inserted into Appendix D;
- Additional test pit logs to be inserted into Appendix F;
- An additional Appendix containing Dr. Ogden's Report
- A complete set of revised drawings.

GEC and Sunray looks forward to written comments, if any concerning this report. If you have any questions, please contact Mark Witherspoon or myself.

Sincerely,

GENESIS ENVIRONMENTAL CONSULTING, INC.

Dan McCullough

Dan McCullough,
Senior Hydrogeologist

David Hopkins for
Mark Witherspoon P.G.
President

cc: Kevin Hodges Sunray\USA Waste
enclosure: revised sections of report and Drawings

REVISED TEXT PORTION

HYDROGEOLOGICAL AND GEOTECHNICAL REPORT

SUNRAY SERVICES, INC.
USA WASTE SERVICES, INC.

TONTITOWN, ARKANSAS

PREPARED BY:

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January 21,1997
Revised April 1, 1997

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6.0 HYDROGEOLOGICAL AND GEOTECHNICAL REPORT
REGULATION 22 (CHAPTER 11)

This hydrogeological and geotechnical report (Volume 3 of 3 of the Permit Modification) summarizes the hydrogeologic investigation conducted in association with the Class I Permit Modification Application for the Sunray Services Inc. (Sunray), a subsidiary of USA Waste Services, Inc., Tontitown facility. The investigation was designed to meet the requirements of Chapter Eleven *Geotechnical and Hydrogeological Investigations* and Section 22.425 *Landfills in Boone and St. Joe Formation* of Arkansas Department of Pollution Control & Ecology (ADPC&E) Regulation 22 (April, 1995), and the Four-County Solid Waste District rules (4-County Rule) (December, 1993).

Section 6.1 contains a brief summary of the background information concerning the status of the existing Class I Landfill and generally describes the proposed modification. Section 6.2 describes the general regional hydrogeologic characteristics of the area. Section 6.3 describes the details the details of the on-site hydrogeologic and geotechnical investigations conducted to specifically characterize the size for the proposed modification. Section 6.4 described the site specific hydrogeologic model based upon the regional and site investigation presented in the previous sections. Sections 6.5 and 7.0 present the groundwater monitoring program proposed to address the permit modification and hydrogeologic model.

Subsections include specific Regulation 22 references in order to assist the reviewer in determining administrative and technical adequacy.

- Section 22.1101 states the general purpose for conducting a geotechnical and hydrogeologic investigation.

- Section 22.1102 (a) establishes the applicability of Chapter 11 of Regulation 22 to the proposed Class I Landfill modification. Since the applicant will receive household solid waste, commercial, and industrial wastes, the data and information required in Section 22.1102 must be compiled.

- Section 22.1102 (b) of Regulation 22 requires a registered professional geologist to determine the local and site specific geology and structure of the study area. Section 6.2 of this document provides the regional characteristics of the study area, while Section 6.3 of this document concentrates on the more site specific information.
- The conceptual hydrogeologic model required by Regulation 22 Section 22.1102 (b)(1) is specifically addressed in Section 6.4 of this document.
- The geotechnical characterization required by Regulation 22 Section 22.1102 (b)(2) is addressed in Section 6.3.7 of this document.
- The monitoring well location and design required by Regulation 22 Section 22.1102 (b)(3) is addressed in Section 6.5 of this document.

6.1 INTRODUCTION

The data and information presented in the following report is intended to characterize all aspects of the Sunray property which may directly or indirectly affect the design, construction, operation or monitoring of the solid waste containment structure as authorized in the above specified regulations. Genesis Environmental Consulting, Inc. (GEC) is compiling this hydrogeologic characterization on behalf of Sunray in association with a permit modification to the existing solid waste permits.

Sunray currently operates the Class I landfill at the Tontitown Arkansas facility under Solid Waste Permit Numbers 123-SR-2 and 162-SR-2. The permit modification proposed in this application involves the expansion of the Class I Landfill into the area between the current Site 3 and Site 4 (see DRAWING 1 of 14). Since there has been a permitted Class I Landfill on this site for over 15 years, this hydrogeologic characterization is partially based on characterization work performed by other investigations and is limited by the extent of the existing waste mass. The intent of this investigation was to verify and build upon the results of previous investigations.

6.1.1 BACKGROUND

The Sunray Tontitown facility occupies a large portion of the northwestern quadrant of Section 23, Township 17 North, Range 31 West about three miles south-southwest of Tontitown in Washington County, Arkansas (see DRAWING 2 of 14). The total acreage owned by Sunray at this location is 360 acres. Actual acreage utilized by Class I and Class IV waste disposal is approximately 75 acres.

The current Class I Landfill consisting of Site 3 (Permit # 123) and Site 4 (Permit # 162) was originally constructed in two parallel valleys north of Clear Creek and west of Little Wildcat Creek. Site 3 is located in the western valley and Site 4 is located in the eastern valley. The landfills were permitted by previous owners prior to 1979. Sunray began operations at the site in the early 1980s. It is believed that an unpermitted dump was operated north of the current Site 4 area prior to purchase of the landfills by Sunray. This area is visible on the aerial photographs located in APPENDIX C.

Sunray has conducted at least two previous hydrogeological characterizations of the site. These reports are entitled *"Hydrogeological Characterization of Sunray Landfills"* prepared by Geraghty & Miller, Inc. and Environmental Management, Inc. (1987), and *"Hydrogeologic Investigation Report Sunray Services Sanitary Landfill Tontitown, Arkansas"* prepared by SCS Engineers (1992). The data from these studies were utilized to supplement the hydrogeologic information presented in this report. As mentioned previously, this characterization is limited by the fact that waste has been in place for many years preventing detailed characterization of acreage underlain by in-place waste.

The current groundwater monitoring system which was approved by ADPC&E in April, 1993, consists of ten (10) monitoring wells surrounding the Class I landfill area. Groundwater quality data has been collected at the Tontitown facility since 1987. See Section 6.5 of this Volume for a detailed discussion of the existing and proposed groundwater monitoring system. FIGURE 6.1 represents a time line of significant occurrences related to groundwater monitoring at the Sunray Tontitown landfills.

6.1.2 PROPOSED MODIFICATION AREA

The proposed Class I permit modification covers approximately 37 acres. However, only 9.9 acres of this area are not currently included as part of the Site 3 or Site 4 permitted areas. As discussed in the Engineering Design (Volume 1), a double composite liner which includes two separate

60 mil High Density Polyethylene (HDPE) and 24" thick clay bottom liners is proposed across the 9.9 acres that is currently not part of Site 3 and Site 4. A single composite 60 mil HDPE and 24" thick clay bottom liner is proposed over the portion of the modification that will be constructed over the current Site 3 and Site 4 areas. A single 40 mil LDPE cap is proposed to be placed over the entire modification area including the current Site 3 and Site 4 areas upon closure.

A passive gas recovery system is also proposed to be installed in association with the permit modification. A detailed description of the proposed gas system is included in Volume 1.

Sunray feels that the proposed modifications incorporates advanced engineering design that will significantly reduce the possibility of future environmental problems associated with the landfill facility. These improvements are unquestionably a significant improvement to the existing facility that was originally constructed under less stringent regulatory standards.

6.2 REGIONAL CHARACTERIZATION

This section of the report presents the regional conditions in the vicinity of the Sunray Tontitown site. This information was collected from published information found in previous studies conducted at the site, and from various State and Federal government documents. The intent of this investigation was to verify and build upon the results of previous investigations. References are provided following the text.

6.2.1 REGIONAL CLIMATE

The climate of Northwest Arkansas is characterized as humid, subtropical. The summers are classified warm with abundant rainfall characteristic for the summer months. Winters are cool with frequent continental polar air mass invasions. The mean average precipitation at Fayetteville is 44.3 inches and the mean average temperature is 58.4° F (14.7° C) (Reference 1). The temperature in Northwest Arkansas is consistently 4 to 6 degrees cooler than Arkansas counties to the southeast (Reference 2). FIGURE 6.2 presents the monthly precipitation during the period of the hydrogeologic investigation conducted by GEC during August to November 1996. The daily precipitation measured at the Tontitown Landfill over the last two years is presented in APPENDIX A.

As indicated on FIGURE 6.2, over 12 inches of rainfall has been recorded at the Sunray Tontitown site between September 26, 1996 and November 25, 1996. As will be discussed in Section 6.4 of this Volume, groundwater flow conditions should have been optimized by the amount of rainfall at the site during this period. Therefore tests to determine the hydrogeologic characteristics beneath the site should be representative of flow capabilities of the aquifer.

6.2.2 REGIONAL HYDROLOGY

The Sunray landfills are located in the watershed of Little Wildcat Creek. Surface runoff from the site is controlled and collected in three on-site sedimentation ponds. One pond is located on the south end of Site 3, one pond is located on the south end of Site 4, and one pond is located on the east side of Site 4. A detailed discussion of site run-on/run-off control and proposed improvements is provided in Volume 1.

Drainage from the on-site ponds discharge off site in ephemeral tributaries approximately 1/2 mile to Little Wildcat Creek. Little Wildcat Creek then flows in a south/southwesterly direction for 1/2 mile until its convergence with Clear Creek which also flows in a westerly direction. Clear Creek flows to the Illinois River.

6.2.3 REGIONAL GEOLOGY/HYDROGEOLOGY

The following four sections briefly discusses the geology/hydrogeology of the region surrounding the Sunray Tontitown facility. The information contained in these sections was compiled from published literature and previous studies conducted at the site. References are provided following the text.

6.2.3.1 STRUCTURAL GEOLOGY

Northwest Arkansas lies in the Ozark Plateau physiographic province. The rocks of the Ozark Plateau were deposited in an essentially horizontal position, and have undergone no great deformation (folding or faulting) since their deposition. The Arkansas Ozarks are the southern and southwestern part of the Ozark structural dome formed by the uplift of the St. Francis Mountains in southwestern Missouri. Consequently, the prevailing dip of the rock strata in northern Arkansas is toward the south and southwest primarily in response to past regional stresses associated with the Ouachita orogeny to the south. The inclination of the beds is slight, commonly between one to two degrees, particularly near the northern border of the State. Minor faults are common. Vertical displacements of 50 feet to 300 feet have occurred along faults or breaks. Generally the faults are normal and downthrown south of their traces.

In the vicinity of the Sunray property, Little Wildcat Creek (east and south of the site) lies on a major northeast trending lineament oriented approximately N35°E (FIGURE 6.3). This feature has not been mapped as a fault (Reference 3) and no offset has been documented, but it has similar characteristics of other lineaments that are documented faults. These characteristics include linearity at a scale of several miles, springs and sinking streams, steep-walled linear valleys, occurrence of second and third magnitude springs and "dry valleys" that are incised by tens of feet into the surface with no apparent channel or evidence of surface erosion (Reference 3).

Clear Creek basin (south of the site) is another feature that has recently been documented as a fault, in an area where no faulting had been mapped previously (Reference 4). Defining the northern margin of Clear Creek Basin, the fault scarp marks a zone across which the southern fault block is displaced more than 100 feet below the northern block. The displacement is based on gamma logs from 8 wells that lie along a 1-mile line section normal to the fault. The stratigraphic contacts correlated are the top and bottom of the Chattanooga Formation (Reference 3). The Chattanooga Formation was utilized as the stratigraphic marker since a continuous marker bed was not present in the portion of the Boone Formation which underlies the region surrounding the site.

Other linear features have been identified as possible fractures based on low-altitude (less than 1000 feet above land surface) aerial reconnaissance, interpretation of aerial photography, surface hydrogeologic reconnaissance, dye tracing, and groundwater geochemistry. Linear features in the area have a preferred orientation of N50°-60°W and N30°E. One fault has been mapped in the Springdale 7.5-minute quadrangle, but it is several miles from the study area to the east (FIGURE 6.4). Sections 6.3.2 and 6.3.3 of this Volume provide a more site specific discussion of lineaments and faults.

6.2.3.2 REGIONAL STRATIGRAPHY

The rocks of the Ozark Plateau include beds of limestone, shale, sandstone, dolomite, chert, and conglomerate, and range in age from Ordovician to Pennsylvanian. Most of the beds, particularly those of Carboniferous age are fossiliferous. The general stratigraphy of the area surrounding Tontitown consists of a weathered residuum of the Boone Formation, overlying the cherty limestone of the Boone Formation (Mississippian age). The Boone Formation rests conformably upon the St. Joe Member (Mississippian) and together they comprise one hydrostratigraphic unit known as the Boone-St. Joe Aquifer. The Boone-St. Joe rests unconformably on the Devonian-aged Chattanooga Shale. This shale unit acts as the upper confining layer for the underlying Sylamore, Clifty, and Everton Aquifers. FIGURE 6.5 presents a stratigraphic column for northwest Arkansas.

The Boone residuum occupies the tops and flanks of hills and ridges. It is characterized by red (iron rich) clay, weathered limestone and chert. The chert content in the clay varies widely across the area. Weathered "pinnacles" of Boone Formation limestone commonly extend into the overlying residuum (see Section 6.4). The thickness of the residuum varies from 30 to 50 feet on ridge tops.

Large quantities of chert are left behind as the semi-insoluble surface residue after the limestone of the Boone Formation has been dissolved by surface or groundwater. The chert is fairly light weight and very porous due to the removal of calcium carbonate and some silica. (Reference 5).

The Boone Formation is a gray, predominately crinoidal limestone abundantly interbedded with gray, black, and blue chert. This formation is a massive, well cemented limestone. The Boone Formation is approximately 280 feet thick in Northwestern Arkansas. All the limestone of the Boone Formation above its lower limestone member is comprised of a nearly pure calcium carbonate which is very soluble, such that groundwater has formed a network of underground drainage channels. Sinkholes, caves, and fissures are common occurrences, and are often partially filled with boulders of chert, and red clay (Reference 6).

Chert is found at nearly all horizons of the Boone Formation above the St. Joe member. At some places the chert occurs as widely separated nodules; at others as connected nodules; and still at others it occurs in layers interbedded with limestone. More rarely, distinct beds consist entirely of chert.

The St. Joe Member is typically a light-gray, mud supported Crinozoan-Bryozoan crystalline limestone (Reference 5). The St. Joe Member is easily recognized by its absence of chert. The thickness of the St. Joe Formation in Northern Arkansas ranges from 6 to 84 feet with an average thickness of 45 feet (Reference 7).

The Devonian aged Chattanooga Shale is exposed in Benton, Washington, Carroll, and Madison counties of Arkansas. The Chattanooga Shale thickens to the west and can reach up to seventy (70) feet in thickness. In the vicinity of the site, it acts as a barrier to vertical groundwater flow. Lithologically, the Chattanooga Shale is a black, fissile and carbonaceous rock with abundant pyrite.

6.2.3.3 REGIONAL GEOMORPHOLOGY

Northwest Arkansas is underlain by the Boone Limestone Formation as discussed in Section 6.2.3.2 of this Volume. The dissolution of this limestone is responsible for the formation of distinctive surface topography known as karst terrain.

The most direct evidence of karst terrain are the landforms that are unique to karst regions. These landforms are the direct result of the dissolution of soluble carbonate bedrock and typically have vertical and horizontal underground drainage. Karst features include sinkholes; karst windows; springs; caves; and losing, gaining, sinking, and underground streams (Reference 16).

The typical karst terrain of Northwest Arkansas is characterized by solution valleys. These valleys are uniform in width are most generally linear, and have steep slopes. Valley heads are somewhat cirque-like and simulate the appearance of sinkholes cut in half along a vertical plane. These valleys commonly join each other at an angle of less than 90 degrees, and are numerous across the region.

Few surface expressions showing dissolution of carbonate rocks occur within and near the vicinity of the site. This does not mean the features are not present, but that the thick regolith that develops from the Boone Formation masks karst features at the regolith-bedrock interface. Those features that were identified in the region by Brahanna (Reference 3), included a sinkhole, and a sinking stream both located to the east of the landfill site in the Little Wildcat drainage basin (see FIGURE 6.3). In addition, several springs are identified on DRAWING 2 of 14. A detailed discussion on the groundwater flow characteristics associated with karst terrain is presented in Section 6.2.3.4. A discussion on site specific structure is provided in Section 6.3.3.

6.2.3.4 REGIONAL HYDROGEOLOGY

As discussed earlier, Northwestern Arkansas is generally characterized as a karst region. Groundwater recharge in the region occurs as the infiltration through the unconsolidated material overlying the Boone Limestone or as direct inflow from sinking streams. The recharge water moves vertically until it encounters a horizontal conduit which allows lateral movement. These solution enlarged conduits range from microscopic to feet in diameter. Springs located at the base of the regional flow system are the discharge points for these conduits. These springs are located directly above the contact where insoluble rocks are encountered or at structural barriers such as faults which impede the development of conduits.

As indicated on DRAWING 2 of 14, the location springs in the region are believed to be controlled by the contact between the Boone-St. Joe Formation and the underlying Chattanooga Shale Formation. The elevation of this contact has been structurally altered by the faults as discussed in Section 6.2.3.1. The regional base of the Boone-St. Joe Aquifer appears to be flat lying as indicated by the level contact of Chattanooga Shale Formation and the overlying Boone Formation determined from boring logs in the area recorded at the Arkansas Geologic Commission (Reference 3).

The movement of groundwater in Boone-St. Joe Aquifer occurs in a combination of two ways: concentrated flow through subsurface conduits flowing to springs, and diffuse flow through the aquifer which also discharges at springs.

Areas of the Boone-St. Joe Aquifer characterized by concentrated flow represent a mature karst system. Mature systems are generally defined as having well developed sink holes and losing streams that drain surface water directly to the subsurface conduits. The groundwater flow velocities are generally high typically in the feet per hour range. The conduits are well developed and connected to solution fractures that discharge to springs. The discharge from the springs generally responds rapidly to rainfall and is "flashy" (Reference 16).

Areas of the Boone-St. Joe Aquifer characterized by diffuse flow represent a less mature karst system. The groundwater flow in this system is through small bedrock openings that have undergone only limited solutional enlargement. Groundwater flow velocities are low and groundwater may require months to travel a few feet through the aquifer (Reference 16). The discharge from the diffuse flow is generally uniform and slow to respond to storm events.

As mentioned previously, groundwater movement is characterized by either concentrated or diffuse flow. The degree of concentrated versus diffuse flow depends upon the degree of solutional development (Reference 16). Based on the findings of previous and recent hydrogeologic investigations conducted in the vicinity and at the Sunray Tontitown site, (Reference 3), it is believed that the karst system underlying the site is immature. Further discussion of the hydrogeologic model will be presented in Section 6.4 of this Volume.

Groundwater within the Boone-St. Joe Aquifer is generally found under semi-confined conditions. In most cases, groundwater is found somewhere within the fractured porosity of the limestone bedrock. This groundwater is not exposed to the atmosphere and is confined by the limestone and clay regolith. Once groundwater is penetrated, it will seek its potentiometric surface. The amount of confining pressure (hydrostatic head) is dependent upon the location and elevation of recharge and the degree of fracturing with the limestone. Unconfined conditions may exist within the Boone-St. Joe Aquifer near the recharge area (see Section 6.3.6 for discussion on site specific groundwater condition).

The Boone-St. Joe Aquifer is underlain by the Chattanooga Shale Formation which acts as a lower confining unit for the aquifer. The local groundwater flow generally follows topography to the bottom of valleys. The hydraulic gradient generally ranges from 4.7 feet per mile (0.09 percent) to 121 feet per mile (2.29 percent) with an average of 53 feet per mile (1.01 percent). (Reference 10) See Section 6.3.6.3 of this Volume for site specific information. Also, the Chattanooga Shale can act as an upper confining layer for the underlying Sylamore, Clifty, and Everton Aquifers.

The base flow of almost all streams and rivers in the area is derived from the Boone-St. Joe Aquifer. Large springs are commonly found at the head of major creeks. In addition, water wells completed in the Boone-St. Joe Aquifer are a significant portion of the discharge from the aquifer (Reference 3). DRAWING 2 of 14 provides a map of the area with the location of water wells according to records obtained from the Arkansas Geologic Commission. Copies of the water well logs are included in APPENDIX B.

For a discussion of the site specific hydrogeology, see Section 6.3.6 of this Volume. Also, see Section 6.4 for discussion of the conceptual hydrogeologic model and Section 6.5 for groundwater monitoring discussion.

6.2.4 REGIONAL SOILS

According to the Soil Survey of Washington County, Arkansas published by the U.S. Soil Conservation Service (Reference 1), the soils in the vicinity of the Tontitown Landfill can be classified as six distinct soil units:

(1) Captina;
(2) Nixa;
(3) Clarksville;
(4) Razort;
(5) Elsah; and
(6) Johnsburg.

FIGURE 6.6 displays the locations of the soil units in relation to the landfill property boundary. Note the majority of soils on site consist of the Captina, Nixa, and Clarksville soil units, with the Razort, Elsah, and Johnsburg soils only occurring in relatively small tracts. A portion of these soils have been excavated and utilized in the landfilling operations.

The Captina soil unit is commonly found in areas with a slope grade of 1 to 3 percent, on mountain tops in the Boston Mountains, on the Springfield Plateau, and on stream terraces. The surface layer is brown and is 6 to 10 inches thick. The upper part of the subsoil is yellowish-brown or strong-brown silt loam or silty clay loam, and is 10 to 22 inches thick. The lower part is a firm, brittle fragipan of mottled yellowish-brown, grayish-brown, and red silt loam or clay loam. The depth to cherty limestone bedrock is 36 to 60 inches. This soil is strongly acidic, with a moderate natural fertility. The available water capacity is moderate. Roots and moisture easily penetrate the subsoil as far down as the fragipan, which retards further penetration. Runoff is medium, and the erosion hazard is moderate.

The Nixa soil unit is commonly associated in areas with 3 to 8 percent slopes, occurring as long, narrow, winding ridgetops. The surface layer is dark grayish brown or brown and is 7 to 11 inches thick. The subsoil is cherty silt loam. The uppermost 7 to 16 inches generally is brown, yellowish brown, or strong brown. The lowermost part is a mottled grayish-brown, yellowish-brown, and strong-brown, compact, very slowly permeable fragipan. The depth to the pan is generally 14 to 24 inches. This soil has a medium acidity with a low fertility. The available water capacity is low. Roots and moisture penetrate easily as far down as the fragipan, but very slowly through the pan. Runoff is medium, and the erosion hazard is moderate.

The Clarksville soil unit is found on short, steep hillsides of V-shaped valleys with 12 to 60 percent slopes. The surface layer is grayish brown or brown and is 6 to 12 inches thick. The subsoil is yellowish-brown, strong-brown or pale-brown cherty silt loam. The chert content is 50 to 90 percent in all horizons. The depth to the chert bed ranges from 24 to 54 inches. This soil is medium acid or strongly acid with a low natural fertility. The available water capacity is low because of the high chert content. The root zone is 24 inches or more thick. Runoff is medium, and the erosion hazard is moderate.

The Razort soil unit occurs as long narrow areas parallel to stream channels, occasionally situated in flooded areas. The surface layer is dark brown, dark yellowish brown, or very dark brown and is 7 to 15 inches thick. The subsoil is dark-brown, brown, or dark yellowish-brown gravelly silt loam 2 to 4 feet thick. In places it has gray mottles below a depth of 36 inches. The underlying material commonly has layers of chert gravel 6 to 18 inches thick. Chert gravel makes up 15 to 40 percent of the soil mass. The depth to bedrock ranges from 5 to more than 12 feet below land surface (bls). The soil is slightly acidic with a medium natural fertility. The available water capacity is moderate. The root zone is 5 feet or more in thickness, with roots and moisture easily penetrable. Runoff is slow, and the overflow hazard is moderate.

The Elsah soil unit occurs as long, narrow areas along small streams, normally as 0 to 3 percent slopes. The surface layer is very dark grayish-brown, dark-brown, or very dark brown gravelly silt loam or gravelly loam. The underlying material is brown or yellowish-brown gravelly silt loam or gravelly loam. The gravel content is 75 to 90 percent throughout the profile. The depth to bedrock is 5 feet to more than 8 feet below land surface. This soil is medium acid, and medium in organic matter content. The available water capacity is low because of the high gravel content. The root zone is 3 feet or more thick. Runoff is slow, and the overflow hazard is severe.

The Johnsburg soil unit occurs as 0 to 2 percent slopes in nearly level or depressional areas. The surface layer is dark grayish brown or brown and is 6 to 10 inches thick. The upper part of the subsoil is mottled grayish-brown, yellowish-brown, and strong-brown silt loam or silty clay loam that is 10 to 20 inches thick. The lower part is a firm, brittle fragipan 30 to 50 inches thick. The pan is silt loam or silty clay loam and is mottle with gray, dark brown, yellowish brown, and red. This soil is medium acid, and has a low natural fertility. The organic-matter content is low, and the available water capacity is moderate. Water and roots move readily in the upper part of the soil but are limited in the lower part by the fragipan, which slows root penetration and percolation of water. Runoff is slow, and there exists a slight hazard of erosion on more sloping areas.

Soil unit abbreviations as they appear on FIGURE 6.6:

CaB (large)	Captina silt loam
NaC (large)	Nixa Cherty silt loam
CIG (large)	Clarksville cherty silt loam
Rg (trace)	Razort Gravelly silt loam
Eg (trace)	Elsah gravelly soils
Jo (trace)	Johnsburg silt loam

6.2.5 REGIONAL WATER QUALITY

Groundwater is utilized for domestic supplies without treatment throughout most of the region. The majority of wells within the region are completed within the Boone formation. The quality of groundwater at shallow depths within the Boone Formation is generally suitable for most uses, but water that is high in sulfide and sulfate occurs in the black shale (Chattanooga Shale) and phosphatic sandstone (Sylamore Sandstone Member). Some wells are known to tap these formations where they occur at shallow depths in parts of Washington County. The following table (TABLE 6.1) provides a summary of regional water quality in the region according to Reference 8.

TABLE 6.1

SUMMARY OF REGIONAL GROUNDWATER QUALITY (mg/l)

Parameter	**Number of samples**	**Median Concentration**	**Maximum Concentration**	**Minimum Concentration**
SiO2	19	7.5	11	5.1
Fe	28	0.02	0.45	.00
Mn	28	.00	0.01	.00
Ca	28	52	105	33
Mg	28	2.5	38	0.5
Na	26	2.4	13	0.9
K	26	0.8	16	0.2
HCO3	28	200	426	104
SO4	28	3.0	21	0.0
Cl	28	4.6	17	1.5
F	28	0.1	0.8	0.0
NO3	20	3.8	13	0.0
Diss. solids	28	196	353	118
Hardness CaCo3	28	162	368	88
Spec. Cond. (umhos/cm)	19	364	622	192
pH (S.U)	28	7.5	8.7	7.0

Note: Taken from Reference 9

6.3 SITE HYDROGEOLOGIC INVESTIGATION
(Chapter 11 of Regulation 22)
(22.1101 and 22.1102)

The site specific Hydrogeologic Characterization for the proposed modification is presented in the following sections to comply with Chapter Eleven of Regulation No. 22. Subsections include specific Regulation 22 references in order to assist the reviewer in determining administrative and technical adequacy. Section 22.1101 states the general purpose for conducting a geotechnical and hydrogeologic investigation. The regulatory purpose is almost identical to the purpose stated in Section 6.1 of this document.

Section 22.1102 (a) establishes the applicability of Chapter 11 of Regulation 22 to the proposed Class I Landfill modification. Since the applicant will receive household solid waste, commercial, and industrial wastes, the data and information required in Section 22.1102 must be compiled.

Section 22.1102 (b) of Regulation 22 requires a registered professional geologist to determine the local and site specific geology and structure of the study area. Section 6.2 of this document provides the regional characteristics of the study area, while Section 6.3 of this document concentrates on the more site specific information.

- The conceptual hydrogeologic model required by Regulation 22 Section 22.1102 (b)(1) is specifically addressed in Section 6.4 of this document.

- The geotechnical characterization required by Regulation 22 Section 22.1102 (b)(2) is addressed in Section 6.3.7 of this document.

- The monitoring well location and design required by Regulation 22 Section 22.1102 (b)(3) is addressed in Section 6.5 of this document.

6.3.1 SITE CHARACTERIZATION (22.1102 (c))

Under section 22.1102 (c) of Regulation 22, a site characterization workplan outlining the initial investigations to be performed must be submitted to the ADPC&E for approval prior to beginning work. In July, 1996, GEC submitted a workplan for a hydrogeologic investigation associated with the Class I permit modification at the Sunray Tontitown facility (see APPENDIX M). The purpose of the workplan and the hydrogeological site investigation was to characterize aspects of the Class I expansion area (between Site 3 and Site 4) and the surrounding area which directly effect the design, construction, operation, and monitoring of the landfill facility.

The workplan was prepared in accordance with Chapter Eleven (Geotechnical and Hydrogeological investigations) and Section 22.425 (Landfills in Boone and St. Joe Formation) of ADPC&E Regulation 22 (April, 1995), and the Four-County Solid Waste District rules (4-County Rule). The scope of work that follows provides the step by step approach that GEC implemented in the execution of this Hydrogeologic Investigation.

6.3.2 AERIAL PHOTOGRAPH ANALYSIS (22.1102 (c)(1))

In accordance with Section 22.1102 (c)(1) of Regulation 22 and Section 13.01 (a)(1) of the 4-County Rule, aerial photographs of the study area were reviewed to determine:

1. Fracture traces and fracture orientation if any;
2. Topographic features including sedimentary and depositional features;
3. Drainage patterns; and
4. Land use characteristics.

In limestone aquifer systems, lineaments identified on aerial photographs may represent zones of secondary porosity related to fractures, faults, joints, or other structural features. The aerial photographs utilized for this analysis were dated 1969, 1976, 1982, and 1993. Copies of these photographs are included in APPENDIX C. The 1969 photograph was taken prior to landfilling activities at the site.

In addition, the results of previous lineament analyses conducted in the study area were reviewed and compared to available aerial photography. One of these studies is the work of John Van Brahana of the USGS and the University of Arkansas (Reference 3). A discussion of this analysis was included in Section 6.2.3.1. As previously discussed, linear features in the area have a preferred orientation of N50° to 60°W and N30°E. FIGURE 6.3 presents the lineaments identified by Brahana in the vicinity of the study area.

Based on the 1969 aerial photograph, Site 3 and Site 4 were located in valleys that probably constitute an extension of select lineaments. In addition, several of the current landfill monitoring wells are located along these lineaments. According to the boring logs from these wells prepared by Geraghty & Miller (Reference 1), no indication of solution enlargement was observed during drilling (APPENDIX D). **Similar regional features are sometimes referred to as "dry valleys".** Dry valleys typically contain surface flow only following heavy rains.(Reference 10). **Based on previous and current site investigations, evidence of solution features beneath these valleys was not observed therefore these on-site valleys cannot accurately be classified as "dry valleys".** See Section 6.4 for a discussion on the structure of these valleys related to the site conceptual hydrogeologic model.

Surface expression of faults were not observed on the aerial photographs reviewed. Expression of deep structural features such as faults are sometimes not visible on aerial photographs because of the thick overburden that occurs in the area. See Section 6.3.3 for additional discussion concerning faulting on the site. In addition, the only sedimentary features observed were small alluvial deposits along Little Wildcat Creek and Clear Creek. The surface and subsurface characteristics are dominated by the insitu weathering and erosion of the Boone Formation.

6.3.3 GEOLOGIC MAPPING AND STRUCTURE (22.1102 (c)(2))

In accordance with Section 22.1102 (c)(2) of Regulation 22 and Section 13.01 (a)(2) of the 4-County Rule, GEC conducted mapping of the surface geology within a one mile radius of the property boundary to identify the following:

1. Surface stratigraphy;
2. Structural features;
3. Springs and seeps;
4. Karst features (sinkholes, caves, dolens, etc.): and
5. Domestic, agricultural, and municipal water wells

These features are located and identified on an area map presented as DRAWING 2 of 14.

The surface stratigraphy immediately surrounding the site is the Boone Limestone of Mississippian age. The observed outcrops were highly fractured cherty limestone which is characteristic of the Boone Formation See Section 6.2.3.2. As indicated on DRAWING 2 of 14, an outcrop of the Chattanooga Shale Formation was mapped approximately 2.5 miles southwest of the site. This formation was observed as the characteristic black, dense fissile shale characteristic of the Chattanooga. This contact between the Chattanooga and the overlying Boone-St. Joe was at approximately 1080 feet mean sea level (fmsl).

Structural control for evaluation of faulting was established across the site by utilizing the top of the Chattanooga Shale as a marker bed. The contact between the Boone\St. Joe and the Chattanooga was identified on the geophysical logs from two domestic wells located north and south of the site. The well to the north of the site (Gina Marie Well) is located in the SW, NW, NE portion of Section 14, Township 17 N, Range 31 W, approximately one mile north of the site. The well to the south (Sutton Well) is located along the southern property boundary of the site. The locations of these wells are identified on DRAWING 2 of 14. The borehole geophysical log for the Gina Marie well was conducted by the USGS. GEC conducted the natural gamma log of the Sutton Well. These geophysical logs are included in APPENDIX H.

The elevation of the top of the Chattanooga Shale was approximately 998 fmsl to the north (Gina Marie Well) and 994 (Sutton Well) fmsl to the south. Considering the regional dip of approximately 1 to 2 percent and the distance between these wells, it was determined that the contact is basically level and structural offset (i.e displacement due to faulting) does not appear to have occurred across the site.

However, an approximate 80 foot difference occurs between the top of the Chattanooga beneath the site and the outcrop to the southwest (see DRAWING 2 of 14). **As discussed in Section 6.2.3.1, Clear Creek basin lies to the south of the site and has been documented as a fault. The landfill is located on the upthrown side of the fault block. However, an outcrop of the Chattanooga Shale is present approximately 2.5**

miles to the southwest of the facility (DRAWING 2 of 14). The Chattanooga outcrop also appears along the upthrown side of the fault block. Based upon calculations performed using a constant dip of one degree to the southwest, the Chattanooga at the outcrop is 391 feet higher than expected. Based upon the limited information available, it is possible that a normal fault with a slight rotation to one of the fault blocks is present, or that wrenching associated with faulting along Clear Creek has occurred. This could explain the southeasterly groundwater flow direction as opposed to the regional southwesterly direction.

As discussed in Section 6.2.3.1, the structural lineaments in the area seem to trend to the northeast at approximately N35°E. If this is the case, if a lineament such as a fault bisected the site, structural displacement would have been observed.

Several faults have been documented or suspected within a five-mile radius of the site. However, review of area geophysical logs confirm the fact that these faults do not bisect the site. For further discussion on the presence and influences of faults, see Sections 6.2.3.1, 6.3.2, and 6.4.

Few surface expressions showing dissolution of carbonate rocks occur within and near the vicinity of the site. This is the result of the thick regolith that develops from the Boone Formation which masks karst features at the regolith-bedrock interface. Only three types of surface expressions were identified within 5 miles of the site. These include a sinkhole, a sinking stream, and springs. Of those only two are input forms, include a sinkhole, and a sinking stream; output forms are springs (see FIGURE 6.3 and DRAWING 2 of 14)(Reference 3).

Also presented on DRAWING 2 of 14, are the domestic wells within a one mile radius of the site according to records obtained from the Arkansas Geologic Commission. Further discussion of these wells is presented in Section 6.2.3.4.

6.3.4 SURFACE GEOPHYSICAL INVESTIGATION (22.1102 (c)(3))

According to Section 22.1102 (c)(3) of Regulation 22, a surface geophysical study must be conducted across the proposed Class I expansion area utilizing one of the methods identified in the regulations.

However, Section 13.01 (a)(3)(B) of the 4-County Rule specifies that two surface geophysical methods must be utilized. The two geophysical methods utilized were surface conductivity and resistivity. These studies are discussed in the following sections.

6.3.4.1 SURFACE GEOPHYSICAL STUDY

GEC conducted a surface conductivity geophysical survey over the approximate 9.9 acres located between Site 3 and Site 4. The conductivity geophysical survey was not conducted over areas in which waste was present (Site 3 and Site 4). In addition, GEC collected two lines of resistivity data within the proposed expansion area.

Prior to beginning the surveys the entire area was divided into a grid on 100 foot centers. This grid spacing provides a total of approximately 70 locations for conductivity readings. The conductivity readings were conducted on the 100 foot centers. This spacing provided sufficient resolution for mapping as required in Regulation 22. All data points including both geophysical and geotechnical data points were specifically located on the established grid. This provided more exact feature location information that can be accurately applied to the landfill design. DRAWINGS 1, 3, 4, 5, 6, and 7 of 14 presents the location of these points.

The conductivity survey was conducted utilizing electromagnetic (EM) surface geophysical methods. Electromagnetic surface geophysical techniques provide information about the terrain conductivity of the subsurface (Reference 11). High-terrain conductivity values are the result of subsurface materials, such as clay or shale, or saturated sediments. When combined with additional geohydrologic data, such as data obtained from test borings or test pits, this method provides a possible means for defining vertical and horizontal extent of clay layers and the overburden bedrock interface. This method also provides a correlation between different soil and rock types and their associated conductivity values.

The resistivity survey works on the same principle as the conductivity survey with the exception that the equipment is measuring the resistance between two points (electrodes) as opposed to the conductivity. The electrodes can be arranged in several different arrays consisting of two current electrodes and two potential electrodes (Reference 12). The array chosen for the survey at the site was the “Dipole - Dipole Array”.

The following field procedures were used in conducting the surface geophysical surveys.

Since the existing borings indicate a depth to bedrock as deep as 60 feet below land surface (bls), a surface conductivity survey was conducted utilizing both a Geonics EM-34 and an EM-31 transmitting and receiving system. A conductivity measurement was taken in both the horizontal dipole position (HD) and the vertical dipole position (VD) at each of the grid points utilizing both Geonics instruments. Depending on the depth of interest, and utilizing both instruments, data was collected at intercoil spacings of 3.5, 10, and 20 meters (M) in both the vertical and horizontal dipole configurations. The effective depths of these intercoil spacings are:

	3.5 M	10M	20M
HD	8.5 ft.	25 ft	50 ft
VD	17 ft.	50 ft	100 ft

The Geonics instruments are most responsive to the near-surface conditions in the HD configuration and to conditions at one half the coil separation in the VD configuration.

The data obtained from the survey was placed in a computer contouring program (Surfer^{TM} from Golden Systems, Inc.) and terrain conductivity maps were generated for each coil spacing.

Two lines of surface resistivity data were collected within the expansion area. The location of these lines in indicated on DRAWING 1 of 14. Utilizing the dipole - dipole array, electrodes were spaced at 10-foot intervals along each line. Each line was approximately 250 feet in length. Multiple readings were collected at each measurement configuration and the average recorded. The total depth of measurement along the profile was approximately 30 feet below land surface (bls).

6.3.4.2 SURFACE GEOPHYSICAL RESULTS

Terrain conductivity values were contoured using the Surfer^{TM} program to provide a graphical presentation of the aerial extent of certain subterranean features. Resistivity data was used to develop a profile cross section utilizing interpretative software. The contour maps associated with the surface conductivity survey are presented on DRAWING 3 - DRAWING 7 of 14. The recorded data points at each measurement point for each of the different intercoil spacings, are presented in APPENDIX E. TABLE 6.2 presents a table of the terrain conductivity results. From these contour maps, profiles and other hydrological data (i.e., test borings and geologic field mapping), the following interpretations can be implied:

- As indicated on the conductivity drawings, the conductivity values did not vary widely across the site. This could be interpreted to indicate that the materials encountered in each individual coverage (i.e. 10M, 20M, etc...) are somewhat uniform.

- A difference is evident between the 10 M and 20 M inter-coil spacing readings. The 20 M readings were usually half the 10 M readings. This difference may be attributed to the situation where the majority of the 10 M readings are within clay, while the 20 M readings were within limestone (see Section 6.3.4.1).

- The readings taken utilizing the 10 M inter-coil spacing (effective depths 25 to 50 feet (bls)) confirmed that the clay regolith across the site is extensive and to a depth of approximately 50 feet (bls) in the investigation area.

- The readings taken utilizing the 20 M HD configuration (effective depth 50 feet (bls)) indicate the highest variation of conductivity readings of any of the spacings. This could have been caused by the proximity of the regolith\limestone contact. This contact was found to be irregular in places and interpreted to be caused by the differential weathering of the underlying limestone. This irregular surface often is expressed as pinnacles that extend into the overlying regolith as is typical of the Boone Formation (see Sections 6.2.3.2 and 6.3.5.5 for further discussion).

- The readings utilizing the EM-31 indicate little variation, with the exception of areas where buried waste is suspected. The areas with buried waste were easily distinguished by the conductivity readings beyond the scale of the meter. Actual readings were not recorded at these points, however the areas of suspected waste are presented on DRAWING 1 of 14.

The results of the resistivity survey were compiled into two profiles labeled A and B and are presented in APPENDIX E. The A profile bisected the proposed expansion area north to south, while the B profile bisected the proposed expansion area east to west. The location of these profiles is presented on DRAWING 1 of 14.

Cross Section A-A'

A set of twenty-eight electrodes established an array for these readings. At a depth of 2 meters (6.5 feet) a layer of clay was detected as a lens. This clay lens extended from the initial data point to 48.8 meters (160 feet). At a depth of 1.6 meters (5.2 feet) a subsurface anomaly was detected and interpreted as cherty material. This material starts at 61 meters (200 feet) from the initial point and extends beyond the array. Several anomalies were detected at shallow depths in close association to PZ-1. Possible interpretation suggests that the well was being purged at the time the data was being collected and the anomalies are saturated clay soils.

Cross Section B-B'

A second set of twenty-eight electrodes were utilized to establish this array. At a depth of 2.6 meters (8.5 feet) a possible chert bed was detected to approximately 40 meters (130 feet) in length and approximately 1 meter (3 feet) in height. Directly underlain by the chert bed, possible cherty limestone "float" material was detected beginning at a depth of 5.1 meters (16.5 feet).

Specific areas on both profiles are indicated with increasing resistance. These areas are believed to be cherty limestone residual material which were not receptive to weathering. This conclusion was later verified during the drilling of piezometer PZ-1. Cherty limestone was first encountered in this boring at a shallower depth than other borings in the area. Further discussion concerning this boring is presented in later sections of this report (see Section 6.3.5.5).

6.3.5 SUBSURFACE EXPLORATION PROGRAM 22.1102 (c) (4)

As part of the Hydrogeologic Investigation, GEC conducted a subsurface exploration program utilizing test pits and borings as required by Regulation 22 Section 22.1102 (c) (4). The locations of these borings and test pits were selected based on the results of the surface geophysics program and were intended to provide extensive coverage across the site. The information gathered from the subsurface program included:

1. Thickness and aerial extent of each distinct soil texture unit or petrologic unit; (see cross sections DRAWINGS 10 and 11 of 14)
2. Depth to bedrock and bedrock topography; (see DRAWINGS 8, 10, and 11 of 14)
3. Rock Quality Designation; (see Table 6.3 and APPENDIX D)
4. Fracture density and fracture orientation; (see APPENDIX D)
5. Bit drop and loses or gains in drilling fluid; (see Section 6.3.5.5 and APPENDIX D)
6. Borehole geophysical logs (Section 6.3.5.4 and APPENDIX H).

As presented on DRAWING 1 of 14, extensive geologic and hydrogeologic data is available from studies previously and currently conducted on site. GEC utilized previously collected data in the development of this investigation in order to compile the hydrogeologic model to be discussed later in this report (see Section 6.4).

Geraghty & Miller, Inc. and Environmental Management, Inc. (GM&EM), SCS Engineers (SCS), and GEC have conducted separate site sub-surface investigations in order to identify changes in lithology while collecting hydrogeologic information. These investigations included **34** boreholes, 8 of which were cored (2 by SCS June,1992, 6 by GEC September, 1996). Soil samples were collected in 10 borings (4 by SCS November, 1991, 6 by GEC September, 1996) and monitoring wells or piezometers were installed in 20 of the **34** borings (9 by GM & EM, 5 by SCS June, 1992 and March, 1993, and 6 by GEC September,1996). Boring logs and construction diagrams for the drilling events are included in APPENDIX D.

A total of **65** test pits have been excavated during various investigations across the site.

- SCS excavated 37 pits in November, 1991 (Reference 13). and

- ADPC&E excavated 6 pits in December, 1995 during the presite for the proposed Class IV Landfill.

- GEC excavated 11 pits in July, 1996 between Site 3 and Site 4 in association with this Hydrogeologic Investigation.

- ADPC&E also excavated 4 additional pits in this area during the presite investigation for the Class I modification in November, 1996.

- **GEC excavated 7 pits in March, 1997 in the northern portion of the proposed Class IV Landfill to address concerns expressed by ADPC&E.**

Test pit logs are included in APPENDIX F. The majority of the subsurface investigation locations are indicated on DRAWING 1 of 14.

6.3.5.1 TEST PITS

Test pits were utilized to more accurately define the depth of excavatable material at various locations throughout the entire property. SCS conducted a total of 37 test pits with each pit excavated until "backhoe denial" or until the backhoe had reached its extension limits, whichever came first. The locations of the test pits are shown on DRAWING 1 of 14. The walls and base of each test pit were inspected and mapped to a depth of four (4) feet below ground surface. Below four (4) feet, the soils were inspected at the surface. Test pits were backfilled upon completion. Detailed logs were maintained by SCS personnel and are included in APPENDIX F.

A total of nineteen (19) samples were obtained from the test pits to test for suitability of soils for landfill cover and liner materials. Samples were tested for sieve and hydrometer grain size analyses, atterberg limits, moisture content, density, soil classification, standard proctor density and hydraulic conductivity. Laboratory geotechnical results from this event can be found in APPENDIX G. It should be noted, that the majority of the soils from the SCS test pits have been subsequently been excavated.

In addition to the backhoe pits dug by SCS, ADPC&E staff geologists (Pennington and Thompson) and a geotechnical engineer (Young), strategically isolated areas where additional sub-surface investigations were thought to be necessary as part of the ADPC&E presite investigation for the proposed Class IV Landfill. Six (6) backhoe test pits were excavated at these locations and logged from the surface. The locations of these test pits are shown on DRAWING 1 of 14. In general, a red, silty clay with chert and limestone cobbles was the principal soil encountered.

Depths of the 6 pits range from 2 feet to 7 feet (bls) in depth. Boring logs associated with these test pits are included in APPENDIX F.

On July 25, 1996, a GEC Arkansas Registered Professional Geologist logged an additional eleven (11) test pits for the proposed Tontitown landfill expansion, as required Regulation 22 and the Four-County Rule, Section 13.01 (a) 3 (A). Similar to other test pits performed on the Tontitown Landfill property, a silty, orangish-brown clay with chert and limestone cobbles was encountered. Test pits at the site were excavated on one (1) acre spacings to depths ranging from 13.9 to 17 feet (bls) in depth. The upper 6 to 18 inches consisted of a silty gray topsoil with the presence of limestone and chert cobbles.

On March 13, 1997, a GEC Geologist logged an additional seven (7) test pits for the proposed Class IV Landfill. Similar to other test pits performed on the Tontitown Landfill property, a silty, orangish-brown clay with chert and limestone cobbles was encountered in each of the excavations. Test pits at the site were excavated until "backhoe refusal" or the backhoe had reached its extension limits, whichever came first. (see APPENDIX F for logs of these holes)

The Boone residuum consist of an orangish-brown plastic clay with chert and limestone cobbles. The residuum is an approximate mix of 50% chert\limestone and 50% clay. No bedrock was noted in any of the excavations. Geotechnical samples were collected from test pits and submitted for geotechnical testing as required by Regulation 22, Chapter Eleven, Section 22.1102 (c) (6). Test results are found in APPENDIX G and are discussed in later sections of this report (See Section 6.3.7). Based upon field observations of the test pits, the required ten (10) foot separation distance between the bottom of the liner and the top of bedrock can be maintained as required in Regulation 22, Chapter Four, Section 22.425 (b) (2) (See also the Engineering cross section Volumes 1 & 2). The test pit locations can be found on DRAWING 1 of 14. Logs corresponding with these test pits are included in APPENDIX F.

6.3.5.2 BOREHOLES

Boreholes were drilled in various locations throughout the site in order to better define the thickness of the silty clay residuum and the extent of the cherty limestone unit, and to install monitoring wells/piezometers. A total of

25 borings have been drilled at the site during various studies. The locations of these boreholes are shown on DRAWING 1 of 14.

The initial objectives of the soil boring programs were to obtain clay samples for physical laboratory testing and to determine the thickness of the overlying silty clay unit (from the ground surface to the consolidated limestone). Soil samples of the overburden were collected in 16 of the 25 borings drilled.

The subsurface investigation conducted by GM & EM (July, 1987) (Reference 1) consisted of the drilling of nine borings which were later converted to monitoring wells MW-1 through MW-9. Soil sampling was conducted with split spoons and augered cuttings from B-1 and B-3, while B-6, B-7, and B-9 utilized a hard rock core barrel and cuttings for sampling. Soils samples were collected from depths ranging from 10 to 15 feet (bls). These borings were not within the proposed expansion area. The conclusions from this investigation were:

- the site is underlain by residuum and limestone bedrock (Boone Formation);
- the residuum varies in thickness from 28 to 79 feet and has hydraulic conductivities ranging from 3.2×10^{-4} to 8.8×10^{-6} cm/sec (horizontal) and 1.6×10^{-4} to 6.3×10^{-7} cm/sec (vertical);
- the Boone Limestone was encountered beneath the residuum and was found to be a massive, well cemented, competent limestone that allowed open-hole monitoring well completions. The limestone has an average horizontal hydraulic conductivity of 3.5×10^{-5} cm/sec.

SCS conducted a soil boring investigation in November, 1991 which consisted of four soil borings drilled to the top of competent limestone for the determination of geotechnical properties. These borings were designated B100, B200, B300, and B400. SCS practiced continuous sampling on the borings, with shelby tube samples obtained at select horizons. The geotechnical results from these borings are presented in APPENDIX G and discussed in Section 6.3.7 The depths of these borings ranged from 15 to 79.5 feet (bls). The location of these borings were not within the proposed Class I expansion area. Boring logs for the SCS investigation are included in APPENDIX D.

GEC conducted a subsurface investigation in September, 1996. During the subsurface investigation, GEC collected split spoon samples at five foot intervals from the surface to bedrock at six boring locations (PZ-1, PZ-2, PZ-3, MW-1R, MW-2R, MW-3R). All samples and cuttings were logged in the field. Locations for all soil borings are displayed on DRAWING 1 of 14. Geotechnical laboratory results for the related soil sampling can be found in APPENDIX G. Boring logs are included in APPENDIX D.

The overburden material encountered during the drilling at the six GEC locations, consisted of redish, silty, cherty clay typical of the area (see Section 6.2.3.2). Split-spoon samples were collected at five foot intervals, through hollow-stem augers utilized during the drilling of the overburden.

In response to correspondence from ADPC&E, dated January 17, 1997, concerning the technical review of the Class IV permit application, Sunray performed additional characterization of the proposed Class IV Landfill footprint. The primary purpose of the January 17, 1997 drilling program was to evaluate the depth to bedrock in order to document liner/bedrock separation requirements. Therefore, 9 of the borings terminated at bedrock or auger refusal. The additional investigation occurred January 20 - 28, 1997 and included drilling activities, surveying, borrow area soils characterization, and permeability test of the uppermost aquifer.

During the January 20-28, 1997 investigation a total of 10 borings were drilled within the proposed Class IV Landfill footprint. One boring was utilized to perform an insitu hydraulic conductivity test. An Arkansas Registered Professional Geologist logged each of the borings and was present during drilling activities and permeability testing. (see APPENDIX D for borehole logs). Geologic cross sections have been revised to incorporate information from the most recent drilling.

6.3.5.3 COREHOLES

In order to obtain information associated with the underlying bedrock in the study area, a total of 8 boreholes were cored utilizing standard air rotary drilling procedures. Coring activities took place during two separate drilling events. The coreholes varied in depth from 50 to 146 feet (bls), with the thickness of the cored material ranging from 16 to 72 feet (bls). Corehole depths were adjusted in order to gain relative information associated with the local stratigraphy and to set the monitoring wells/piezometers in the uppermost aquifer. Field personnel representing SCS (in 1992), and GEC (in 1996) were on hand during all drilling activities. The field representative collected core samples and recorded relative information on a standard boring log form. An Arkansas Registered Professional Geologist supervised all GEC field activities.

The first rock coring event was conducted by SCS in June, 1992 to install monitoring wells MW-10 and MW-11. Each of the initial borings were cored utilizing an NX-core barrel. The rock cores reported indicate that bedrock was composed of gray, cherty limestone with chert interbeds. Numerous horizontal fractures were also observed. The fractures ranged in thickness from 1 to 10 millimeters. The fracture density of the bedrock core collected from MW-11 was 1.8 fractures per foot. The fracture density of the bedrock core collected from MW-10 was 1.3 fractures per foot. Packer tests conducted in the bedrock portion of these borings indicated a range of hydraulic conductivities from 1.50×10^{-5} to 3.04×10^{-4} cm/sec.

Also during the SCS June, 1992 investigation, a downhole camera was lowered into existing monitoring wells to determine the extent and size of fractures in the bedrock aquifer. Bedrock was observed in monitoring wells MW-1, MW-2, MW-4, and MW-8. The camera indicated horizontal fractures similar to those observed in the rock cores. The downhole video camera also indicated improper well construction in monitoring wells MW-3, MW-4, MW-7 and MW-9.

The second rock coring event directed by GEC geologists in 1996, consisted of the coring and installation of three monitoring wells (MW-1R, MW-2R, and MW-5R) and three piezometers (PZ-1, PZ-2, and PZ-3). All drilling activities were supervised by an Arkansas Registered Geologist. DRAWING 1 of 14 displays the locations of each of the monitoring wells and piezometers. Boring and coring logs for each location are included in APPENDIX D. Where applicable, the field representative logged the recovery and calculated the rock quality designation (RQD) of the material. The actual cores are available for inspection at the Tontitown site.

Borehole geophysical logs (natural gamma and down-hole camera) were conducted in each of the GEC (1996) borings. These borehole geophysical logs are included in APPENDIX H. As indicated on the boring logs, some horizontal fracture solution features were recorded in some of the coreholes. A downhole camera survey was utilized by both SCS and GEC upon coring completion to further characterize the clay/limestone contact and to note any presence of fractures and potential voids. The actual tapes from the downhole camera survey are available for inspection at the GEC office in Little rock AR. See Section 6.3.5.4 for further discussion of the borehole geophysical logging results.

The following presents a summary of the data obtained from each of the 1996 coreholes supervised by GEC:

Boring PZ-1

Hard limestone was first encountered at a depth of 43.5 ft in Boring PZ-1. Coring through the hollow-stem augers began at 44 feet (bls). A apparent 1 foot void was encountered at a depth of 60-61 ft as noted by bit drop during drilling. From 62-71 feet (bls) a apparent clay filled void was encountered where drill cutting return was lost. It appeared that these "voids" were not saturated and are interpreted as the residual material surrounding bedrock pinnacles of residual Boone Formation (See cross-sections presented on DRAWINGS 10 and 11 of 14). The presences of this pinnacle was also confirmed during the resistivity survey (see Section 6.3.4.2).

Bedrock pinnacles occur when the highly soluble calcium carbonate limestone beds are dissolutioned leaving the areas of bedrock with greater chert content. The chert beds remain in place while the limestone beds above and below are removed by weathering. These pinnacles are sometimes attached to the competent bedrock. The voids observed in PZ-1, MW-1R and MW-5R were believed to be formed by this process.

At 71 feet (bls) the use of air coring was abandoned and wash rotary methods were utilized. During the use of wash rotary, water usage was recorded (refer to boring logs in APPENDIX D for volume of water lost). Another clay filled void with chert gravel was encountered at a depth of 71-76 feet (bls) From 76-81 feet (bls) a fractured zone was encountered. Horizontal fractures were noted every 6 to 12 in. with iron staining. RQD values were recorded on each of the seven (7) core runs beginning at 44 feet (bls) and ending at the total depth of the borehole 89.5 feet (bls). Runs

1 through 6 had RQD values less than 89%. The RQD value recorded for Run 7 was 100%. TABLE 6.3 presents a summary of the cored intervals, core recovery percentages, and RQD values.

Water lost was observed during coring of PZ-1. It is believed that the majority of this water was lost in the void encountered from 71 to 76 feet (bls). The boring was later reamed and a four inch piezometer was installed to a depth of 90.5 feet (bls). It was determined that the zone from 76 to 80 feet (bls) was producing the majority of the groundwater based on the voids, fractures, iron staining, and water blown from the borehole. The piezometer was installed according to the procedures found in APPENDIX I. The screened interval for PZ-1 was 70.5 to 90.5 feet (bls).

Boring PZ-2

Limestone was first encountered in PZ-2 at 63.5 feet (bls) Since this boring was located only 145 feet from PZ-1, (see DRAWING 1 of 14) soils samples were not collected. However based on auger cuttings, the material was typical of the silty, cherty clay found across the site. Coring began at 64 feet (bls) utilizing air rotary and a conventional core barrel through the hollow-stem augers. Small horizontal fractures were noted in Run 2 from 70 to 80 feet (bls) and in Run 3 from 80 to 90 feet (bls).

Water production was observed coming from the fractures, as indicated by the iron staining along the fracture planes and the presence of water blown to the surface during air coring. RQD values were recorded on each of the three (3) core runs and ranged between 55.7% to 90%. TABLE 6.3 presents a summary of the cored intervals, core recovery percentages, and RQD values. No voids were encountered while coring this borehole. The boring was later reamed and a two inch piezometer was installed in the boring according to procedures found in APPENDIX I. The screened interval was from 70 to 90 feet (bls).

Boring PZ-3

Competent limestone was first encountered in PZ-3 at 75.5 feet (bls) Coring began at 76 ft utilizing air rotary and a conventional core barrel through the hollow-stem augers. Small horizontal and vertical fractures were noted in all four core runs. In addition, several hard brecciated chert and stylolitc zones were indicated in the cores. Water production was noted coming from the fractures as indicated by the iron staining along the fracture planes and the presence of water blown to the surface during air coring. RQD values recorded from the four core runs ranged between 56% to 100%. No voids were encountered while coring this borehole. TABLE 6.3 presents a summary of the cored intervals, core recovery percentages, and RQD values.

The boring was later reamed and a four inch piezometer was installed in the boring according to procedures found in APPENDIX I. The screened interval was 80 to 120 feet (bls). The boring was advanced beyond the extent of coring in order to insure sufficient groundwater for the well.

Boring MW-1R

Competent limestone was first believed to be encountered in MW-1R at 40 feet (bls). Coring began at 40 ft with air rotary and a conventional core barrel through the hollow-stem augers. During Run 1 (40-50 feet (bls)) apparent "voids" were encountered at 43.5 to 44 feet (bls) and 48 to 50 feet (bls). The RQD for the run was 21%. The recovered core consisted of weathered limestone and clay with a hard chert band. Runs 2 and 3 had RQDs of 0%. The cores consisted of weathered limestone and clay that was highly fractured. The zone from 40 to 70 feet (bls) is interpreted to be a weathered bedrock pinnacle. The apparent "voids" are therefore limited areas where the Boone residuum is encountered both above and below the bedrock pinnacle (see DRAWING 11 of 14).

Run 4 (70-77.5 feet (bls)) had a RQD of 36% with small horizontal and vertical fractures. Water production was noted during this run coming from the fractures as indicated by the iron staining along the fracture planes and the presence of water blown to the surface during air coring. Run 5 (77.5-87.5) and Run 6 (87.5-93) had an RQD of 83% and 80% respectively. Horizontal fractures were indicated every 6 to 12 inches and appeared to be producing water. TABLE 6.3 presents a summary of RQD values.

The boring was later reamed and a four inch piezometer was installed in the boring according to procedures found in APPENDIX I. The screened interval was 70 to 110 feet (bls). The boring was advanced beyond the extent of coring in order to insure sufficient groundwater for the well.

Boring MW-2R

Competent limestone was first encountered in MW-2R at 72 feet (bls) Coring began at 72 feet (bls) with air and a conventional core barrel through the hollow-stem augers. Small horizontal and vertical fractures were noted in all six core runs. Water production was noted coming from the fractures as indicated by the iron staining along the fracture planes and the presence of water blown to the surface during air coring at approximately 105 feet (bls). RQD values were recorded on each of the six core runs. RQD values ranged from 0% to 100%. No voids were encountered while coring this hole. TABLE 6.3 presents a summary of the cored intervals, core recovery percentages, and RQD values.

The boring was later reamed and a four inch monitoring well was installed in the boring according to procedures found in APPENDIX I. The screened interval was 84 to 124 feet (bls).

Boring MW-5R

The first limestone encountered in MW-5R at auger refusal was highly weathered. A chert gravel filled "void" was observed from 46 to 54 feet (bls). Competent limestone was not encountered until 60 feet (bls). The zone from 46 to 60 feet (bls) is believed to represent a weathered bedrock pinnacle.

Coring began at 60 feet (bls) utilizing air and a conventional core barrel through the hollow-stem augers. Small horizontal and vertical fractures were noted in all four core runs. Water production was noted coming from the fractures as indicated by the iron staining along the fracture planes and the presence of water blown to the surface during air coring. RQD values were recorded on each of the four core runs. The RQD values ranged from 81% to 92%. TABLE 6.3 presents a summary of the cored intervals, core recovery percentages, and RQD values. No voids were encountered while coring this hole with the exception of the voids found prior to beginning coring.

The boring was later reamed and a four inch monitoring well installed in the boring according to procedures found in APPENDIX I. The screened interval was 70 to 100 feet (bls). The boring was advanced beyond the extent of coring in order to insure sufficient groundwater for the well.

Based on the results of the boring program, it was observed that competent limestone was found in each of the borings at approximately 70 feet (bls). During the drilling of PZ-1, MW-1R, and MW-5R, limestone was found at a much shallower depth (approximately 40 feet (bls)), however, this limestone was found to be incompetent and to contain clay and chert gravel filled voids. These areas are believed to represent bedrock pinnacles.

Bedrock pinnacles occur when the highly soluble calcium carbonate limestone beds are dissolutioned leaving the areas of bedrock with greater chert content. The chert beds remain in place while the limestone beds above and below are removed by weathering. These pinnacles are sometimes attached to the competent bedrock. The voids observed in PZ-1, MW-1R, and MW-5R were believed to be formed by this process.

6.3.5.4 BOREHOLE GEOPHYSICAL LOGS

As required in Regulation 22 and the 4-County District Rule, borehole geophysical logs were conducted on each of the boreholes. The logs utilized were natural gamma and down-hole camera. GEC originally anticipated conducting natural gamma logs within all existing monitoring wells at the landfill, but was unable to gain access due to the dedicated pumps in each of these wells. It should be noted that down-hole camera logs were conducted in several of the existing monitoring wells across the site by SCS in 1992. GEC utilized all these data in the overall evaluation of the borehole geophysical logging program.

In addition to logging the newly installed monitoring wells and piezometers, GEC also conducted a natural gamma log within an on-site domestic well (Sutton Well) to a depth of 550 feet (bls). Copies of all borehole geophysical logs are included in APPENDIX H.

Natural gamma logs are records of the amount of natural gamma radiation that is emitted from all soils and rocks. The main use of this method is for the identification of lithology and stratigraphic correlation in open or cased, liquid or air-filled holes. The main gamma emitting isotopes normally found in sediments and rocks is potassium-40. Potassium, which contains about 0.012 percent potassium-40 is abundant in feldspars and micas which decompose readily to clay. Probably the most important application in groundwater hydrology is in the identification of clay or shale bearing sediments (Reference 14). A good example of a shale occurrence on a natural gamma log was observed in the Sutton well log. At approximately 195 feet (bls), the Chattanooga Shale Formation was encountered as an increased gamma response. Further discussion of this log is presented in Section 6.3.3.

The down-hole video logs consisted of lowering a specially designed video camera with a light source down each borehole. The camera was designed to fit within boreholes three inches or greater in diameter. Visibility was good within the unsaturated zone, however it became poor beneath the water table in most cases. Since the down-hole video logs were recorded within an open hole, the possibility existed for the collapse of the 40 to 70 feet (bls) of cherty clay overburden above the bedrock. As viewed on the logs, material from the side walls was falling into the hole throughout the logging process. For this reason, GEC was unable to allow the formation water to clear, allowing increased visibility during logging. The only video logs that clearly recorded bedrock were within boreholes where the groundwater surface was below the bedrock overburden contact. A copy of the video logs is included in APPENDIX H.

6.3.5.5 RESULTS OF SUBSURFACE EXPLORATION PROGRAM

The hydrogeology of the Tontitown Landfill site has been characterized by numerous borings and test pits over the years. Very little variation has been observed in the extent or character of the clay residuum of the Boone Formation. GM&EM, SCS, and GEC have conducted separate site sub-surface investigations in order to identify changes in lithology while collecting hydrogeologic information. These investigations included **34** boreholes, 8 of which were cored (2 by SCS 6/92, 6 by GEC 9/96). Soil samples were collected in 10 borings (4 by SCS 11/91, 6 by GEC 9/96) and monitoring wells or piezometers were installed in 20 of the **34** borings (9 by GM & EM, 5 by SCS 6/92 and 3/93, 6 by GEC 9/96) and **65** test pits.

The Boone residuum consist of an orangish-brown plastic clay with chert and limestone cobbles. The residuum can consist of a mix of as much as 50% chert\limestone and 50% clay. However, limited areas have been found across the site with a much greater clay content.

The overburden thickness ranges between 20 to 70 feet. Numerous geotechnical samples have been collected and tested for physical properties. A discussion of these results is presented in Section 6.3.7. Based upon the test pits, and borings installed within the modification footprint, the ten (10) foot separation distance between the bottom of the liner and the top of bedrock can be maintained as required in Regulation 22, Chapter Four, Section 22.425 (b) (2).

The subsurface investigation conducted by GM & EM (July, 1987) (Reference 1) consisted of the drilling of nine borings which were later converted to monitoring wells MW-1 through MW-9. The conclusions from this investigation were:

- The site is underlain by residuum and limestone bedrock (Boone Formation).
- The residuum varies in thickness from 28 to 79 feet and has a hydraulic conductivities ranging from 3.2×10^{-4} to 8.8×10^{-6} cm/sec (horizontal) and 1.6×10^{-4} to 6.3×10^{-7} cm/sec (vertical).
- The Boone Limestone was encountered beneath the residuum and was found to be massive, well cemented, competent limestone that allowed open-hole monitoring well completions. The limestone has an average horizontal hydraulic conductivity of 3.5×10^{-5} cm/sec.

Boring logs and well construction diagrams for the GM & EM investigation are included in APPENDIX D.

The subsurface investigations conducted by SCS (1992) consisted of test pits, borings, and coreholes. results of these investigations were as follows:

- The residuum thickness of the Boone residuum ranged from 15 to 79.5 feet.
- Numerous horizontal fractures between 1 to 10 millimeters thick were observed in the coreholes. The fracture density of the bedrock cores were approximately 1.8 fractures per foot.
- Packer test conducted in the bedrock portion of these borings indicate a range of conductivities from 1.50×10^{-5} to 3.04×10^{-4} cm/sec.

- The downhole video camera indicated improper well construction in monitoring wells MW-3, MW-4, MW-7 and MW-9. Monitoring wells MW-3, MW-4, and MW-7 were abandoned and replaced. Monitoring well MW-9 was abandoned.

GEC conducted the current subsurface drilling program in September, 1996. A total of six borings were drilled within, or in the vicinity of the proposed Class I modification area. These borings were later converted to monitoring wells and piezometers. Borehole geophysical logs (natural gamma and down-hole camera) were conducted in each of these borings.

Based on the results of the boring program, it was observed that competent limestone was found in each of the borings at approximately 70 feet (bls). During the drilling of PZ-1, MW-1R, and MW-5R, limestone was found at a much shallower depth (approximately 40 feet (bls)), however this limestone was found to be incompetent and to contain clay and chert gravel filled voids. These areas represent bedrock pinnacles.

All zones characterized as voids were zones where highly soluble calcium carbonate limestone beds are dissolutioned leaving the areas of bedrock with greater chert content. The chert beds remain in place while beds above and below are removed. The voids observed in PZ-1, MW-1R and MW-5R were believed to be formed by the process described above.

The "void" zones are not solution cavities within the limestone bedrock. Drilling extended from bedrock into a clay void and returned to competent bedrock. Since the surrounding surface geophysics and borings did not indicate competent bedrock, the "voids" are actually competent bedrock pinnacles that are limited in extent. As explained in Section 6.2.3.2, these pinnacles are typical of the Boone Formation.

DRAWING 8 of 14 presents a bedrock contour map prepared from the boring information collected to date. Upon comparison of the surface topography to the bedrock topography it is evident that the patterns are similar. In addition, this confirms the conclusion that the ten foot separation between the bottom of the landfill liner and bedrock will be maintained as required by Regulation 22.

DRAWING 9 of 14 presents a location map for the geologic cross sections which were developed from the recent borings conducted by GEC and historical boring installed by SCS. DRAWINGS 10, and 11 of 14 present the geologic cross sections. As indicated on these cross-sections, the required separation of 10 feet required by Regulation 22 for bedrock and groundwater, should easily be maintained.

The following items are included on the cross sections provided on DRAWINGS 10 and 11 of 14, as required by Section 22.1102 (d)(7):

- Stratigraphy
- Maximum depth of excavation
- Physical Properties
- Geophysical logs
- Screen Intervals
- Aquifer Testing Results

6.3.6 HYDROGEOLOGIC INVESTIGATION (22.1102 (c)(5))

In accordance with Section 22.1102 (c)(5) of Regulation 22 and Section 13.01 (a)(3)(C) of the 4-County Rule, the groundwater flow characteristics must be determined via installation of piezometers. The current groundwater monitoring system consists of 10 monitoring wells surrounding the Tontitown site. In addition to these wells, 3 piezometers and three monitoring wells were installed during this investigation.

These formal control points in addition to regional groundwater information provide a base of groundwater information adequate to characterize groundwater movement in the study area. Since many of these data points have been in place for several years, the seasonal groundwater flow characteristics can also be accurately evaluated.

It should be noted that the direction of groundwater flow across the site has been a source of discussion for several years. The Solid Waste Division of ADPC&E has expressed concern that the groundwater flow direction across the site cannot be expected to always be in a single direction due to karst groundwater flow characteristics. Therefore the ADPC&E has required an intra well statistical comparison of groundwater quality data, as opposed to the upgradient to downgradient comparisons generally utilized. Because a long period of groundwater elevation information is available from numerous data points, a final resolution of this issue should be possible from the data presented in this document. This should lead to a revision of the statistical comparisons currently utilized and a revision of the placement of the actual groundwater monitoring system (see Section 6.5 of this document).

According to Section 22.1102 (c)(5), the following information must be obtained for a proposed expansion of a Class I Landfill:

- Potentiometric surface and groundwater flow direction;
- Hydraulic conductivities of units;
- Hydraulic gradient (horizontal and vertical);
- Hydraulic communication between units and monitoring points (based on pump tests);
- Evaluation of groundwater flow characteristics determined by dye traces;
- Groundwater flow velocity; and
- Ambient groundwater chemistry

See also Section 6.4 for the conceptual hydrogeologic model of the site.

6.3.6.1 MONITORING WELLS AND PIEZOMETERS

Monitoring wells or piezometers were installed in 20 of the 24 borings at one time or another, nine by GM & EM (1987), five by SCS (June, 1992 and March, 1993), and six by GEC during the recent hydrogeologic investigation.

The current groundwater monitoring system was approved in April, 1993 and consists of 10 monitoring wells. Originally the groundwater monitoring system included eleven monitoring wells (Monitoring wells MW-1 through MW-11), however monitoring well MW-9 was plugged and abandoned in 1993 to accommodate expansion of the landfill. The monitoring wells were installed by GM&EM and SCS during three separate events. Well construction diagrams for these wells are included in APPENDIX D. Additional discussions of the groundwater monitoring system are found in Sections 6.3.6.2, 6.3.6.3, 6.3.6.4, 6.3.6.5, and 6.5.

During the investigation in association with the current permit modification application, three piezometers and three monitoring wells were installed (see Drawing 1 of 14). A 2-inch piezometer (PZ-2) was installed within one of the borings between Site 3 and Site 4, and was utilized as a monitoring point during pump and dye test studies (PZ-2). A 4-inch piezometer (PZ-1) was installed in the second boring between Site 3 and Site 4, and was utilized as a pumping well during the pump test and an injection point during the dye tests (see DRAWING 1 of 14). In addition, a piezometer (PZ-3) was installed within the boring drilled at the southwest corner of the property (See DRAWING 1 of 14). These piezometers were installed to a depth of approximately 125 feet (bls). The piezometers were installed according to the procedures presented in APPENDIX I. See Section 6.3.6 for additional discussion of the piezometers.

Three new monitoring wells were installed during field activities. The first monitoring well was installed to the north of existing monitoring well MW-1 and was designated MW-1R (see DRAWING 1 of 14). The second monitoring well was located to the west of existing monitoring well MW-5 and was designated MW-5R. The third monitoring well was installed to the west of MW-2 and was designated MW-2R. These monitoring wells were utilized as monitoring points during the dye test study and as data points for potentiometric surface determination (See Section 6.3.6.2). The monitoring wells were constructed in accordance with the procedures presented in APPENDIX I.

The three newly installed wells were installed as new monitoring points to replace existing monitoring wells MW-1, MW-2, and MW-5. Due to proposed landfill expansions at the Tontitown facility and poor placement, the existing monitoring wells will be plugged and abandoned and monitoring discontinued (See Section 6.5 for discussion of proposed monitoring system).

The current monitoring well MW-1 is located within the proposed waste disposal area under the proposed permit modification and monitoring well MW-2 is located within the proposed Class IV permit area. Monitoring well MW-5 is located near an area of waste disposal prior to Sunray operations. The extent of this waste was recently confirmed during the surface geophysical investigation. A complete discussion of the revised groundwater monitoring system is presented in Section 6.5 of this document.

All new monitoring wells and piezometers were developed by the drilling contractor utilizing a combination of surging and pumping with a stainless steel submersible pump. The typical pumping rate was 4 gallons per minute. The pump was decontaminated between each well location by pumping a solution of Alconox phosphate free soap and water through the pump and hose.

All new wells/piezometers were pumped at a constant rate until extracted groundwater visually cleared or the well pumped dry after stabilization of field parameters. However, several cycles of pumping were required at each well. Turbidity, pH and conductivity were also measured periodically during the development. Once these parameters had stabilized and the water had cleared, the wells were considered adequately developed. Well development records are included in APPENDIX D. The monitoring system is further discussed in Sections 6.4 and 6.5.

6.3.6.2 GROUNDWATER FLOW DIRECTION

Groundwater levels have been measured on a quarterly basis in the existing monitoring wells at the landfill from February, 1990 to the present. The piezometers and monitoring wells installed during this investigation were added to the existing system and a current comprehensive piezometric surface map was generated (DRAWING 12 of 14).

A determination of flow direction and seasonal fluctuation in the groundwater surface was also made based on the amount of currently available data. TABLE 6.4 presents the historical groundwater elevations. DRAWING 12 of 14 presents the potentiometric surface map constructed from groundwater elevations collected on October 30, 1996. As indicated on DRAWING 12 of 14, the general groundwater flow direction is to the southeast. Upon comparison to the bedrock contour map (DRAWING 8 of 14), it is evident that the potentiometric surface closely approximates the top of the unweathered bedrock. Additional discussions on groundwater flow direction are found in Sections 6.2.3.4 and 6.4.

Regional groundwater flow is to the southwest. As stated above, flow across the site appears to be to the southeast. The difference is believed to be attributed to the rotation of the fault block as discussed in Section 6.3.3.

Eventhough the potentiometric surface maps have consistently indicated a definite southeast flow direction across the site over a

long period and under a steep gradient, interpretations of the dye test performed in conjunction with this application, suggest flow in the opposite direction (see Section 6.3.6.5.2). This flow component may be attributed to localized gradients within fractures. The monitoring system and statistical analysis method has been revised to account for flow in all directions.

The hydraulic gradient was calculated based on the latest water level data utilizing the equation:

$$i = \frac{h1-h2}{L}$$

where:

i = hydraulic gradient
$h1-h2$ = difference in water level elevation between two wells
L = the distance between the two wells

The hydraulic gradient was calculated to be 0.019 between MW-1R and MW-10. Since these wells are located at opposite ends of the site, this gradient value is considered representative of the overall site conditions.

6.3.6.3 AQUIFER TESTING

As required in Regulation 22 and the 4-County District Rule, aquifer testing was conducted in the piezometers or wells to determine hydraulic conductivities and velocities of the uppermost (limestone) aquifer. Slug tests were conducted on three monitoring wells (MW-1R, MW-2R, MW-5R) in addition to the slug tests conducted by previous researchers on all other wells. One multi-well pump test was performed with PZ-1 acting as the pumping well and drawdown measured in PZ-2. GEC utilized a data logger with pressure transducers to record data points in the calculations.

Slug tests are an insitu rising-head permeability test that were performed in order to measure the insitu hydraulic conductivity at specific monitoring wells. The procedure involves lowering a cleaned slug or sealed tube of known volume into the water in the well and allowing the water level to recover to the pre-insertion level. The slug then is removed quickly and the recovery of the water level back to static level is monitored, timed, and recorded. Slug test results were evaluated in accordance with the methods presented by Bouwer and Rice (REFERENCE 15). More specific methodologies for the slug tests are provided in APPENDIX M.

A pump test was conducted in order to more accurately evaluate the insitu hydraulic conductivity of the uppermost (limestone) aquifer. Since all previous studies have indicated a very slow groundwater movement rate, the pump test was designed utilizing a single pump well and a single observation well spaced closely within the proposed modification footprint. A step draw-down test was conducted on the 4 inch piezometer (PZ-1) in order to determine the appropriate pumping rate for the required pump test. The test results and additional details of the methodology are presented in the following section.

The pump test basically involved the pumping of the 4 inch piezometer at a constant rate until the drawdown stabilized. Drawdown was recorded in the pumping well PZ-1, and the 2 inch piezometer PZ-2 during the test. Once equilibrium of the drawdown was reached, the pump was shut off and the recovery was measured in each of the wells until static conditions had been reached. See APPENDIX M for a detailed description of pump test procedures. The results of the slug tests and the pump test are included in APPENDIX J and TABLE 6.5.

6.3.6.4 RESULTS OF AQUIFER TESTING

According to the GM&EM investigation (1987), the limestone has an average horizontal hydraulic conductivity of 3.5 x 10^{-5} cm/sec. This value was determined by GM&EM from slug tests conducted in monitoring wells MW-1 through MW-9.

SCS conducted packer tests within the bedrock at monitoring wells MW-10 and MW-11. The range of hydraulic conductivities was from 1.50 x 10^{-5} to 3.04 x 10^{-4} cm/sec.

The results of the slug tests conducted by GEC in the three newly installed monitoring wells are presented in TABLE 6.5. As indicated, the hydraulic conductivities of the bedrock aquifer range between 1.64 x 10^{-5} to 2.89 x 10^{-6} ft/min. Based on these hydraulic conductivities, a groundwater flow velocity of the bedrock aquifer in the vicinity of the tested monitoring wells was calculated by a derivation of Darcy's Law:

$$V = \frac{K\,(dh/dl)}{n}$$

where:

V = average velocity
K = hydraulic conductivity
dh/dl = hydraulic gradient
n= effective porosity

Utilizing 0.05 for effective porosity, 0.019 for hydraulic gradient, and the highest hydraulic conductivity of 1.64 x 10^{-5}, the average velocity of the bedrock groundwater was determined to be 6.23 x 10^{-6} ft/min or 3.27 ft/yr (See APPENDIX J for calculations).

It must be noted that the slug type insitu hydraulic conductivity test is conducted solely in individual wells, and provides an accurate evaluation of groundwater flow velocity in the vicinity of the tested well. This type of test does not provide an indication of interconnection between or within hydraulic units. However, the very slow flow velocity calculated during these slug test are similar to the very slow groundwater flow velocities indicated by insitu permeability tests conducted by previous researchers and indicated in the published literature.

A pump test is another type of insitu permeability test that takes into consideration the actual flow characteristics between two or more wells and is generally considered the more representative of true groundwater flow characteristics. A constant rate pump test was conducted utilizing PZ-1 as the pumping well and PZ-2 as the observation well. PZ-2 is located 145 feet downgradient from PZ-1. Prior to beginning the test, a step draw-down test was conducted to determine the optimum pumping rate for the test. The step-drawdown test was started at 1 gpm and the pumping rate increased by 1 gpm every hour until the optimum pumping rate was determined. The well was allowed to fully recover between steps of the test. It was determined that a pumping rate of 4 gpm was optimum (see APPENDIX M for details).

The pump test was conducted by pumping PZ-1 at a constant rate of 4 gpm for 10 hours. Drawdown was measured in the pumping well utilizing a pressure transducer. Drawdown was also measured in PZ-2 utilizing an electronic water level indicator. The test was stopped after 10 hours when the drawdown in the pumping well had stabilized. At that point the recovery in each of the piezometers was recorded (see APPENDIX M).

Utilizing the Theis Method, a transmissivity of 1.185 ft^2/min was determined from the constant rate pump test (see APPENDIX J). A transmissivity of 0.979 ft^2/min was determined during the recovery phase. The calculated transmissivity was utilized to determine the hydraulic conductivity by solving for K in the equation

$$T=Kb$$

where:

T= Transmissivity (1.185 ft^2)
K= hydraulic conductivity
b= aquifer thickness (121 feet)

The above calculation provides a hydraulic conductivity of 9.79 x 10^{-3} ft/min This value is utilized in the derivation of Darcy's Law described earlier in this section to determine average velocity. The average flow velocity calculated by the above method is 3.72 x 10-3 ft/min (5.3 ft/day or 1956 ft/yr). Actual calculations were performed via computer modeling software (see APPENDIX J for details).

As discussed previously, this velocity calculation should be more representative of actual aquifer characteristics since it was determined utilizing a pump test of the aquifer. Even though the results of the dye test study should help to confirm this calculated velocity, actual groundwater velocities in carbonate aquifers can be highly variable. Past tests conducted by past researchers also indicate similar very slow groundwater velocity of the bedrock aquifer.

6.3.6.5 DYE TEST STUDY (22.1102 (e))

Section 22.1102(e) of Regulation 22 states that a dye trace study designed to test the accuracy of the conceptual hydrogeologic model developed for new and expansion to new landfills is required for sites located on the Boone formation. Since the proposed site is located on the Boone formation, Sunray has conducted the required study.

The original hydrogeologic model for the site was prepared in conjunction with past permit work at the existing landfills. The original model presented basic information concerning groundwater flow direction and rate within the identified stratigraphy. This document (Volume 3) is intended to supplement the available hydrogeologic information to meet Regulation 22 and District specifications and to update the site hydrogeologic model. A formal discussion on the site hydrogeologic model is presented in Section 6.4 of this document.

Since the existing landfill operation and the facility groundwater monitoring system has been permitted for several years, the purpose of the dye trace analysis is to test the existing and the revised conceptual hydrogeologic model presented in this report, and predict contaminant movement by:

1) providing an indication of the interconnection of subsurface structure;
2) providing an indication of general groundwater flow direction in the vicinity of the facility; and
3) providing an indication of general groundwater flow velocity within the bedrock aquifer.

The essential elements of the existing and the revised groundwater conceptual model for the site are as follows (See Section 6.4 for more detailed information):

- The groundwater velocity of the bedrock aquifer is very slow;
- Flow occurs within the bedrock as diffuse flow through joints and fissures some of which may be solutionally enlarged;
- Groundwater follows a definite flow direction across the site in a basic southerly direction;
- The uppermost aquifer and the first avenue of escape from the landfill is the bedrock aquifer; and
- The residual Boone overlying the bedrock aquifer is a low permeability clay that partially confines the underlying uppermost aquifer.

GEC was assisted on the dye test operations by Dr. Albert Ogden, an Arkansas Registered Professional Geologist, and an Assistant Professor of Hydrogeology at Middle Tennessee State University. Dr. Ogden is one of the few nationally recognized experts in karst hydrogeology and dye test methodology. Dr. Ogden was associate professor at the University of Arkansas in Fayetteville for 8 years and has published extensively concerning groundwater flow characteristics in Northwest Arkansas. Dr. Ogden supervised dye selection and quantity, and performed actual injection procedures, as well as, assisted with all aspects of the dye study and interpretation.

A specific work plan for the dye study was submitted to the Solid Waste Division of the ADPC&E on October 4, 1996. GEC received comments from the Solid Waste Division by letter dated November 4, 1996. All of the ADPC&E concerns and considerations were discussed on site with the Solid Waste Division Staff, GEC Staff, and Dr. Ogden on November 8, 1996. The resolution to the items identified in the November 4, 1996 letter from ADPC&E were summarized in a letter dated December 2, 1996 from GEC. GEC received authorization for the dye test from the Water Division on October 22, 1996. All of the correspondence concerning the dye test is provided in APPENDIX N.

6.3.6.5.1 DYE TEST METHODOLOGY

In general, the dye test study was conducted to approximate leakage from the proposed expansion and to help confirm the hydrogeologic model. As such, the test involved injecting dyes into piezometer PZ-1 which was drilled within the footprint of the proposed expansion (between the existing fills) and testing for the recovery of the dyes in the existing monitoring wells, piezometers and other select locations in the vicinity of the property. A positive detection of the injected dyes should indicate definite groundwater interconnection between the injection footprint and the trap location.

It was anticipated that dye injected within the bedrock underlying the proposed footprint would be detected initially in the nearest downgradient piezometer (PZ-2) then in one or more of the existing monitoring wells and then possibly detected within one of the offsite traps. The following sections provide the details of dye injection, recovery, and analysis.

Dyes and Procedures Utilized for Injection

Much consideration was given to the type and amount of dyes to be injected in this study. The Solid Waste Division of the ADPC&E expressed concern that a sufficient amount of dyes be utilized to conduct an adequate study and yet not endanger drinking water supplies (letter from ADPC&E dated November 4, 1996 presented in Appendix N).

The actual amount of dye to be injected was discussed in detail with all parties on several occasions including an onsite conference on November

8, 1996 prior to any injection. The ADPC&E referenced guidance that specifies a methodology for calculating the amount of dye to be injected (APPENDIX N). After discussions with Dr. Ogden, GEC staff, and ADPC&E, it was recognized that this formula requires the utilization of input variables that must be estimated, causing an unacceptable risk. Rather than estimating these variables, the amount of dye used was based on the experience of Dr. Ogden, and the amounts utilized in similar dye tests conducted in the area. Extensive dye testing experience by Dr. Ogden and comparisons with other studies performed in the vicinity, indicated that greater concentrations (up to two pounds of concentrated dye) had produced positive dye traces without apparent health hazards. See APPENDIX N for correspondence concerning the type and amount of dyes.

The dye types were selected based on consideration of material safety data sheets (MSDS), consultation with individuals currently conducting dye studies in the area, the experience of Dr. Ogden with the selected dyes in the study area, and consultation with the Technical Services Division at the Arkansas Department of Pollution and Ecology. Three different dyes were selected in order to provide the best possibility of a positive trace between the injection point and the receiving traps since various dyes have different mobility rates in groundwater and different attenuation characteristics through various media.

The three tracing agents injected were eosine, rhodamine WT, and **fluorescein**. A different dye was injected at each of the three injection locations (MW-1, MW-5, and PZ-1). Approximately one pound of eosine (MW-1), one pound of rhodamine WT (MW-5), and two pounds of fluorescein (PZ-1) were used for the test. Each dye was dissolved **at a ratio of one pound per** five gallons of water and then the five gallon **mixtures were** added at each injection point.

Upon injection of each dye, additional water (up to a 1,000 gallons) was introduced at each location to accelerate movement of the dye from the well into the aquifer. In addition, excessive rainfall conditions before and after the dye injection have created optimum conditions for maximum dye migration. Since all injection locations are well casings, the dyes were manually poured from the mixing container into a hose which extended to the groundwater within the well casings. Additional procedures and methodologies are provided in APPENDIX N.

Type of Traps for Dye Detection

As discussed previously and in Section 6.4 of this document, the site hydrogeologic model predicts that dyes will move very slowly based upon the tested groundwater velocity. In addition, some attenuation can be expected in the bedrock aquifer. Therefore, specially designed samplers referred to as "traps" or "bugs" must be strategically placed at expected receiving locations. These traps must have the ability to absorb dyes from the surrounding water since the receiving concentration will be below visual levels. The traps must be processed through a laboratory specifically experienced with dye trace methodology. The samplers ("traps") consisted of nylon screen mesh packets approximately one and one-half inches by four inches (1.5" x 4"). Each packet contained approximately 10 grams of 6-14 mesh activated carbon.

The samplers placed in wells were suspended with nylon cord to below the static water level in each well. Samplers that were placed in springs and flow pathways were secured in a matter that maximizes the flow through the sampler. Packets of glass marbles constructed similar to the carbon filled traps were utilized as weights to secure the samplers in place.

In addition to the activated carbon filled traps, a water sample was collected from each trap location. The water sample was collected in a specifically designed glass bottle filled to capacity with no head space. A separate bottle was collected for each trap location, each time the trap was collected.

Establishment of Background Conditions

The establishment of adequate background conditions was another subject of controversy during the development of the dye test procedures. The ADPC&E expressed concern that several other dye trace studies had recently been conducted in the area, and that the other studies could either cause concentrations of dye in the regional groundwater that would threaten public health or that the other studies would mask the effect of the current dye study (see November 4, 1996 letter in APPENDIX N).

In response to the concerns of ADPC&E, GEC instigated communications with Dr. Van Brahana of the University of Arkansas, who was responsible for the regional dye trace studies in progress. After numerous discussions between GEC, Dr. Ogden, and Dr. Brahana, it was decided that the Sunray dye test would be delayed several weeks to insure that interference with past tests would not cause problems in the proposed dye study. The injection of all three dyes was further delayed at the request of ADPC&E until the results of the background samples (described below) were analyzed and shown to exhibit no remaining effect from other dye tests.

Sample packets were placed at each of the monitoring locations on October 31, 1996, nine days prior to injection. The samplers were retrieved on November 6 and 7, 1996. The samples were analyzed for the dye tracing agents to be injected during the test. The results of the background analysis indicated possible background levels of some of the injection dyes only in the parts per billion and parts per trillion range. The results of the background sampling are presented in APPENDIX K.

These almost negligible ambient concentrations were considered within the range of expected values for ambient conditions, and not attributed to possible interference from other regional dye tests. Results obtained after the test had started were compared to these background levels and further indicated that the observed trace background concentrations were ambient conditions.

Method of Analysis of Traps

As mentioned previously, the amount of dye expected at trap locations cannot be expected to be in the visual concentration range. It is also necessary to quantify the amount of dye received at trap locations in order to infer certain flow characteristics. For these reasons, it is necessary to utilize quantitative laboratory analysis methods for the interpretation of dyes potentially received at trap locations.

It is also necessary to utilize only laboratories experienced with the analysis of dyes as utilized in dye test studies. Laboratory analysis was conducted by Crawford and Associates, Inc. located in Bowling Green Kentucky. This laboratory also prepared all of the actual traps utilized in the study to ensure that cross contamination did not occur. The ADPC&E expressed concerns with some aspects of the laboratory procedures. Therefore, the laboratory procedures were modified as specified by ADPC&E (see APPENDIX N for correspondence). Detailed laboratory procedures are presented in the Dye Test Workplan dated October, 1996 and are in included in APPENDIX N.

Dye injection locations and procedures

It is important to select the proper location for the injection of dyes in order to meet the purposes of the dye study. As mentioned previously, the injection points were chosen to simulate the possible fate of contaminants if they leak from the proposed landfill modification. Also as noted previously, the dyes are injected directly into the bedrock aquifer without simulating the engineering modifications that will be in place to collect and control possible contaminant migration (liners, leachate collection system, and the overburden clays of the residual Boone Formation).

The three (3) injection sites for the dye trace study were selected due to the fact that they are located on topographic lineaments that indicate structural control associated with the valleys that originally were in place prior to any waste disposal on site. It is anticipated that these lineaments are the preferred pathway for migration of groundwater under the site due to secondary porosity associated with joints or fissures that could be solutionally enlarged (See Section 6.3.2 for discussion).

The injection sites (MW-1, MW-5, and PZ-1) are within existing monitoring well system and within one of the piezometers installed during the hydrogeologic investigation (see DRAWING 1 of 14 for location of injection points). Monitoring wells MW-1 and MW-5 were selected based on the fact that they are hydraulically upgradient of Landfill Site 3 and Site 4. Piezometer PZ-1 was selected based on the fact it is located within the proposed modification expansion footprint. By injecting at these locations, flow transport of any contaminates from the waste units into groundwater flow beneath the site should be simulated. As mentioned previously, this simulation is "worse case" due to the fact that dyes are injected directly into the receiving uppermost aquifer without consideration of the liner, leachate collection system, and the natural clay residuum that will serve to restrict movement into the bedrock aquifer.

Monitoring locations and trap placement procedures

The receiving samplers must be strategically located in the optimum positions in order to:

1) Insure that background conditions are established and the influence of ambient dye concentrations is continually monitored so that false positives are not utilized in the interpretation of the results.

2) Provide the best possibility of positive traces taking into account the known hydrogeologic conditions.

3) Confirm or re-evaluate the site hydrogeologic model.

The location of monitoring points was also the source of much consideration in the development of the dye study. The ADPC&E expressed concern that adequate background locations were utilized to detect possible interference with past dye studies in the vicinity, and that sufficient investigation was done to identify springs that could be utilized as receiving points for dye. A complete spring inventory was conducted during the field investigation covering an area bound by Little Wildcat to the east and Clear Creek to the south.

In response to the comments from ADPC&E, each spring was revisited prior to injection and several new monitoring points were added at the suggestion of Dr. Ogden. In addition, an upgradient monitoring point on Clear Creek was added to confirm background conditions throughout the study. It should be noted that the monitoring program not only includes wells and springs, but also includes stream locations in order to monitor the receiving area for undetected springs.

The monitoring locations for trap placement are presented on DRAWING 2 of 14 and are as follows:

- *Existing Monitoring Wells- MW-2, MW-3, MW-4, MW-6, MW-7, MW-8, MW-10, and MW-11;*
- *New Monitoring Wells and Piezometers- MW-1R, MW-2R, MW-5R, PZ-2 and PZ-3;*
- *Spring/Stream Locations- SP-1, SP-2, SP-3, SP-4, SP-5, SP-6, Glass Spring, and Up Clear Creek (Greathouse Spring); and*
- *Domestic Wells - DOM-1 (Stutts), DOM-2 (Sutton).*

All of the monitoring wells at the facility were included in the sampling program. All springs located immediately downgradient of the site on lineaments were monitored. However, it is anticipated that the base flow of Clear Creek and Little Wildcat Creek is from springs that are in the creek bottom and cannot be visually identified. Therefore, various stream locations were sampled to anticipate this possibility. As shown on DRAWING 2 of 14 upgradient spring and stream samples were monitored throughout the study in order to continually evaluate ambient conditions.

The actual placement of the traps for monitoring was accomplished systematically in order to insure that the dyes were not inadvertently introduced into the samples. As mentioned previously, the traps were prepared and sealed directly from the laboratory and handled only during actual placement. Cross contamination of sampling points was eliminated by not placing any trap in contact with anything prior to actual placement.

All materials placed in a sampling location were new and/or cleaned properly. New plastic gloves were used and replaced when handling any material utilized during trap placement. It should be noted that the sampler (trap) placement was before any dye was handled. Individuals responsible for dye injection did not handle traps at any time. The dyes were pre-mixed with five gallons of water before being transported to the site and injected. Eosine and fluorescein were mixed from a powder form. Rhodamine WT was mixed from a liquid form.

The samplers placed in wells were suspended with nylon cord to below the static water level. Samplers that were placed in springs and flow pathways were secured in a matter that maximizes the flow through the sampler. Dedicated disposable bailers were utilized to obtain the water sample from each well during each trap collection event. A similar packet of glass marbles was utilized as weights to secure the samplers in place.

Each sampling point had a separate plastic bag prepared in advance by the laboratory. The bag contained the described carbon trap, the marble weight, and the sample bottle. Each bag was identified and marked according to the sample location prior to actual placement. The individual plastic bags were systematically filed in a sealed, light proof, portable file box prior to placement. During actual placement activities the sealed plastic bag was removed from the box, and the exposed sample trap was placed in the bag that contained the new trap to replace that monitoring point. Thereby, contact and cross contamination was minimized. An entry was made in the chain of custody and the field book for all trap placement and retrieval activities.

After the determination of background fluorescence levels as previously described, sample traps were placed at each sample location on November 8, 1996. The dyes were injected at the three locations on November 9, 1996. The date and time of each injection was recorded in the test log book.

Sampler removal procedures

As described above, the sampler (traps) were removed from individual locations in a manner necessary to insure that dye was not introduced from outside sources. As described, samplers were individually packaged in resealable plastic bags for transport to the lab. Sample number, time, date of collection, and the name of the collector were recorded on the sampling bag and on the chain of custody at the time of collection. Disposable gloves were worn at all times during sample collection. A new pair of gloves were utilized at each trap location to avoid cross contamination of samples. Samples were stored in a cool dark place until analysis was performed. See APPENDIX N for typical additional handling procedures.

Frequency and Initiation of Monitoring

The sampling frequency must be designed according to the site hydrogeologic model at intervals necessary to evaluate the positive arrival at a sampling location. The sampling frequency was also a source of discussion in the design of the dye trace study. The ADPC&E expressed concern that the sample retrieval begin soon enough to detect early positive traces and that samples be retrieved at frequent enough intervals to evaluate flow velocity. ADPC&E also required continued monitoring at each sample location even if a positive detection has been documented in the case that more dye flushed through in response to storm events (see November 4, 1996 letter in APPENDIX N).

As mentioned previously and in Section 6.3.5.4, all available site information including tests conducted from previous researchers and tests conducted during this hydrogeologic investigation indicated that flow in the bedrock aquifer is through secondary porosity at a very slow velocity. Based upon this assumption, the dye would move very slowly to the receiving point, making it necessary to retrieve the samples at less frequent intervals until the predicted arrival time of the dye at the closest sampling point. The sampling initiation and frequency was increased at the request of ADPC&E to cover the possibility of non uniform flow velocities and departure from the predicted arrival times.

Background samplers were placed at each monitoring location 9 days prior to injection as described and retrieved 6 to 7 days later. Another set of samplers was placed in each sample location on November 8, 1996, one day prior to dye injection. The dye was injected on November 9, 1996.

The first round of sample collection was approximately 36 hours following the injection as agreed by ADPC&E. The following schedule was agreed upon for the remainder of the study:

- The second sampling event was at 72 hours following injection (11/13/96).
- The third sampling event was at 120 hours following injection (11/15/96).
- The fourth sampling was at 96 hours (4 days) following the third round (11/19/96).
- The fifth sampling was on November 26, 1996, 168 hours (7 days) following the fourth sampling
- The sixth sampling was on December 3, 1996 at 168 hours (7 days) from the fifth sampling.
- The remaining sampling events until the conclusion of the test will be collected at **approximate** 336 hour intervals (14 days) from the sixth round.

The test **was terminated** when it **was** determined that positive dye trace detections **had** been found at **a sufficient number of sampling locations to determine the flow characteristics of the aquifer. Since an upgradient flow direction was suggested by the dye test and a downgradient flow direction is documented by the historical potentiometric data, the dye test study served the purpose of evaluating flow direction. The monitoring system was therefore modified to account for radial flow from the site. The dye test was concluded on March 13, 1997.**

6.3.6.5.2 DYE TEST RESULTS

One of the purposes of the dye test is to indicate groundwater flow characteristics and to evaluate the accuracy of the site conceptual hydrogeologic model. As stated previously and described in more detail in Section 6.4 of this document the essential elements of the existing and the revised groundwater conceptual model for the site are as follows:

- The groundwater velocity of the bedrock aquifer is very slow **according to aquifer tests conducted at the site.**
- Flow occurs within the bedrock as diffuse flow through joints and fissures some of which may be solutionally enlarged.
- Groundwater follows a definite flow direction across the site in a basic southerly direction **based upon five years of potentiometric surface contouring. However, the dye test also suggested an upgradient flow direction from a portion of the site. The monitoring system was therefore modified to account for radial flow from the site.**
- The uppermost aquifer and the first avenue of escape from the landfill is the bedrock aquifer.
- The residual Boone overlying the bedrock aquifer is a low permeability clay that partially confines the underlying uppermost aquifer.

The most critical element of evaluating the site conceptual hydrogeologic model in terms of the dye test is the predicted groundwater flow **direction**. As detailed in Section 6.4 of this document, the flow **direction** for the bedrock aquifer according to the hydrogeologic model prior to the current study was **to the south to southeast according to historical potentiometric surface maps constructed from groundwater elevations collected from the site monitoring wells.**

The dye test results are presented in APPENDIX K. TABLE 6.5.1 provides a summary of possible significant detections based on the specified ADPC&E assumption that a detection is considered significant if the dye concentration is measured one order of magnitude over background. As discussed previously, the reported concentrations are in the part per trillion range. Therefore an increase of 10 parts per trillion would be considered significant under this criteria.

It should be noted that data collected from the spring locations were not included in this evaluation. Upon discussion with Dr. Albert Ogden, it was determined that the springs were probably influenced by dye levels in the creeks based on the levels found in the background springs SP-6 and upper Clear Creek. Further discussion of these data is presented in Dr. Ogden's report found in Appendix O. The following provides a discussion of each individual monitoring well presented on TABLE 6.5.1 as significant:

- **<u>Monitoring Well MW-4</u>- Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported to be significant for eight of the twelve events. Monitoring well MW-4 is located to the south of Site 3. All reported concentrations were less than 1 part per billion. Dr. Albert Ogden's report does not list this well with any positive detections. (see APPENDIX O)**

- **<u>Monitoring Well MW-1R</u>- Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported as significant one out of twelve events and eosine was reported as significant two out of twelve events. It should be noted that MW-1R is upgradient from the injection points according to historical potentiometric flow maps. It appears that a possible upgradient flow components may exist. The fact that eosine had not been detected until the last two events and appeared 2 orders of magnitude greater than background, appears to indicate a positive trace to the north from MW-1. Dr. Ogden believes that a positive detection for eosine occurred on 2/19/97, however he did not list a positive detection for fluorescein (see APPENDIX O)**

- **<u>Monitoring Well MW-5R</u>- Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein and eosine were reported as significant. Fluorescein was reported to be significant 3 of the 12 sampling events with approximately 41 days between the first two significant detections. Only one of the eleven events was reported as significant for eosine. It should be noted that MW-5R is upgradient from the injection point according to historical potentiometric flow maps. Although possible upgradient flow**

TABLE 6.5.1

Sunray/USA Waste - Tontitown Landfill
Dye Test Detection Summary*

*Fluorescein Positive Detection**	*Date Injected*	*Date Detected**
MW-4	11/9/96	11/11/96
MW-1R	11/9/96	11/26/96
MW-5R	11/9/96	11/11/96
MW-7	11/9/96	11/11/96
MW-10	11/9/96	11/11/96
PZ-2	11/9/96	11/11/96
DOM-1	11/9/96	11/19/96

*Eosine Positive Detection**	*Date Injected*	*Date Detected**
MW-1R	11/9/96	2/19/97
MW-2	11/9/96	1/23/97
MW-2R	11/9/96	12/31/96
MW-5R	11/9/96	11/11/96

*Rhodamine WT Positive Detection**	*Date Injected*	*Date Detected**
MW-2R	11/9/96	2/19/97
MW-3	11/9/96	11/26/96

* Detection considered positive if dye concentration is measured one order of magnitude over background concentration.

** See Section 6.3.6.5.2 for discussion of individual positive detections.

components may exist, the lack of repeatable significant detection leads to the conclusion that these two detections for fluorescein and eosine are either false positives or an upgradient flow component (to the north) exists at various times. It should be noted that all detections were less then 1 part per billion. Dr. Ogden did not specify any positive detections within this well (see APPENDIX O).

- Monitoring Well MW-7- Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported as significant for only the first event. Monitoring well MW-7 is located south (downgradient) from the injection points. Based on the lack of repeatability of significant detections the single event is believed to be a false positive or attributable to sporadic groundwater flow conditions. All reported concentrations were less then 1 part per billion. Dr. Ogden did not specify any positive detections within this well (see APPENDIX O).

- Monitoring Well MW-10 - Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported as significant for the first event. Based on the lack of repeatability of significant detections as was seen in MW-4, the single event is believed to be a false positive. There is no reason to believe that if significant detection were reported in both MW-4 and MW-10 two days after injection, that significant detections would not continue in MW-10 as they did in MW-4 unless sporadic groundwater flow condition exist. Dr. Ogden did not specify any positive detections within this well (see APPENDIX O).

- Piezometer PZ-2 - Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported to be significant for six of the twelve events. Based on the repeatability of significant detections, a positive trace between PZ-1 and PZ-2 was probably indicated. Piezometer PZ-2 is located 145 feet south (downgradient) of PZ-1, the injection point for fluorescein. Dr. Ogden believed that a positive detection occurred on 12/31/96 for fluorescein.

- **DOM-1 -** **Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, fluorescein was reported to be significant for six of the ten events after injection. It should be noted that DOM-1 is upgradient from PZ-1. This may indicate a possible upgradient flow component or potential contamination introduced into the uncased well as discussed in this report. Dr. Ogden believes a positive detection occurred for fluorescein on 11/11/96, two days after injection.**

- **Monitoring Well MW-2 - Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, eosine was reported to be significant for one of the eleven events. Monitoring well MW-2 is located south (downgradient from the injection points. The lack of repeatable significant detection leads to the conclusion that the detection for fluorescein is a false positive. Dr. Ogden did not specify any positive detections within this well (see APPENDIX O).**

- **Monitoring Well MW-2R - Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, eosine was reported to be significant for three events and rhodamine was reported to be significant for one of twelve events. MW-2R is located upgradient and to the west of the injection points for eosine (MW-1) and rhodamine (MW-5R). Based on the location of MW-2R in relation to the injection points and the lack of repeatable significant detections, the events reported as significant on TABLE 6.5.1 are believed to be false positives. Dr. Ogden did not specify any positive detections within this well, however he believed that eosine may have reached MW-2R on 2/19/97 (see APPENDIX O).**

- **Monitoring Well MW-3 - Based on the assumption presented above that one order of magnitude above background constitutes a significant detection, rhodamine was reported as significant only one out of twelve events after injection. Based on the lack of repeatability of significant detections the single event is believed to be a false positive or attributable to sporadic groundwater flow**

conditions. Dr. Ogden did not specify any positive detections within this well (see APPENDIX O).

As discussed above, concentrations **found to be one order of magnitude above background were indicated in several of the upgradient wells. For example** at sampling point DOM-1 (Stutts Well) **several events were calculated as significant**. As indicated in FIGURE 6.7, the Stutts well is not hydraulically connected to PZ-1 (the injection well). The elevation of the bottom of Stutts well is approximately 1,235 fmsl and the elevation of the groundwater surface at PZ-1 is 1,213 fmsl. The concentrations in the Stutts well are believed to be attributable to the pre-Sunray buried waste around the well and the lack of a septic system for the nearby house. The well is uncased with hand laid rock sides. The **two** mechanisms that would allow groundwater to flow upgradient would be under pressure through a small conduit or fracture **or a possible groundwater divide**.

As indicated on FIGURE 6.7, it does not appear to be possible for a conduit to connect PZ-1 and DOM-1 that would allow flow upgradient. **Therefore a groundwater divide is possible. This conclusion is further supported by possible positive eosine traces in MW-1R.** The validly of any samples collected from the Stutts well are questionable based on the **observed** groundwater gradients. **However, groundwater has been noted to roughly follow surface topography and a surface topographic divide is near the northern portion of the site. Although not documented, if extreme groundwater mounding occurred at the northern portion of the landfill and a groundwater divide did exist at times, it may be possible for temporary upgradient flow conditions to occur. During measurement of groundwater elevations historically, this condition has never been documented. In fact, as discussed in Section 6.4 and presented on TABLE 6.4, groundwater elevations have varied very little historically.**

Three scenarios are possible for the final analysis of the dye trace analysis:

1) *Existing Hydrogeologic Model is Confirmed*. The dye was detected in the closest downgradient well near the calculated arrival time of 28 days according to GEC pump test. This positive trace will confirm the site model and demonstrate that groundwater flow is at a slow velocity,

in the predicted downgradient direction, with very little retardation. Since the pump tests have indicated a positive hydraulic connection between the injection point and the closest sampling point, the longer that the dye takes to move the short distance to the closest trap, the more contaminant retardation (i.e. attenuation) is indicated.

2) *Assumed Flow Direction is Incorrect.* In this scenario, the dye was detected in a trap that is not downgradient according to the apparent flow direction depicted in the site model. In this case, the conceptual model must be re-evaluated to indicate flow in multiple directions off the site. **The positive dye trace interpreted in the upgradient traps indicates an upgradient flow component not previously detected. The monitoring system and statistical evaluation is therefore modified to account for radial flow.**

3) *Existing Hydrogeologic Model Needs to be Revised.* In this scenario, the dye will not be detected in any traps. Since a larger concentration of dye was utilized than required, extra water was flushed through the injection wells, the site itself has received an unusually large amount of precipitation during the period following the injection, and the pump tests indicate a hydraulic connection between the injection and receiving wells, the dye should be detected *unless the retardation properties of the aquifer are significantly effecting the dye.* In this case, it can be concluded that possible contaminants from the modification footprint will not only flow very slowly, but will also be subject to significant retardation.

A final analysis of the dye test results was prepared by Dr. Ogden and is presented in APPENDIX O. Briefly the conclusions of Dr. Ogden are as follows:

- **A positive "hit" must be at least an order of magnitude over background.**

- **Utilizing this criteria, the results calculated as positive "hits" are hard to believe with the exception of fluorescein in DOM-1 and PZ-2, eosine in MW-1R and possibly monitoring well MW-2R. If all these were assumed to be positives, it would indicate groundwater movement in all directions from the injection point, and all points receiving very small amounts of dye. Two pounds of fluorescein**

were injected. If groundwater movement were in the order of feet per hour, dye concentrations in the closest monitoring points would be very high.

- **Looking at the data, particularly at distant points from the injections, it is apparent that background fluctuates significantly. This is a common occurrence in a karst terrane, particularly in a landfill situation where there are undoubtedly sources of the dyes.**
- **Using the laboratory's 400 to 1 estimate of the ability of a charcoal trap to concentration dye suggests that in the other 21 sampling sites, the concentration of dye in the water probably did not exceed 5 to 10 parts per trillion. Dye concentrations from the charcoal traps fluctuated considerably in the part per trillion range at the 21 sites. This is certainly to be expected in a landfill environment where sources for the dyes likely occur with fluctuations in concentrations being related to recharge events. Therefore, it is extremely difficult to pinpoint a low concentration positive when background levels vary so radically.**
- **The positive indication of fluorescein in PZ-2 on 12/31/97, and eosine in MW-1R on 2/19/97 suggest that groundwater is moving very slowly (3 to 4 feet per day) in a west/northwestern direction contrary to the gradient indicated by the potentiometric surface map.**
- **The Glass Spring shows an increase in fluorescein soon after injection, but concentrations are low and nearly mimic MW-4, MW-7, and MW-10. It is not until 12/17/96 that Glass Spring shows a repeated increase over background that could possibly be attributed to the fluorescein injection. However, based on the fluorescein levels indicated in the background spring upper clear creek, these results are not conclusive.**
- **The results of the dye traces demonstrate that the aquifer is very anisotropic and heterogeneous, thus producing differing flow velocities. For a carbonate aquifer, the flow rates are quite low.**

- **The dye trace results further indicate that a groundwater divide exists on the landfill property that has not been depicted by the potentiometric surface map.**

Based on the results of the dye test and the different interpretations of these results, a groundwater monitoring system is proposed in Section 6.5 that takes into consideration possible multiple flow directions.

6.3.6.6 GROUNDWATER CHEMISTRY

Groundwater quality has been collected quarterly at the landfill since February, 1990. Sunray currently prepares statistical evaluations of the groundwater data on a quarterly basis. The results of these evaluations are briefly discussed in the "1996 FIRST HALF 1996 STATISTICAL EVALUATION REPORT" dated December, 1996.

Sunray is currently operating an approved groundwater monitoring system under the "Assessment Monitoring Program" provisions of 40 CFR 258.55. Sunray began their Assessment Monitoring Program based on the results of the Third Quarter 1994 Groundwater Report dated January 11, 1995. A February 13, 1995 letter from GEC on behalf of Sunray to the ADPC&E, and a response letter from ADPC&E to Sunray dated February 27, 1995 outline the details of the program. According to the approved plan of action, the approved contingency plan will be exercised by September, 1997 to the point of either: 1) Returning to Detection, 2) Remaining in Assessment, or 3) Going to Corrective Action.

As presented on the summary tables and charts in APPENDIX L, based on a comparison to the first four sampling events taken from the approved groundwater monitoring system (April, 1993 - November, 1993), statistically significant increases, were calculated for two volatile parameters in Monitoring Well MW-1. These parameters were 1,1 Dichloroethane, and 1,4 Dichlorobenzene. This monitoring well is hydraulically upgradient of the Landfill but located near the Landfill limits. In addition, several possible upgradient sources have been identified during the recent investigation (see DRAWING 1 of 14 for buried waste areas). As discussed in Section 6.5, the elevated concentration that initiated the assessment monitoring are believed to be attributable to monitoring well locations and construction or alternative sources such as pre-Sunray buried waste.

A discussion of spring water quality was presented in the SCS Hydrogeologic Report (Reference 7). Two springs located downgradient of the landfill are discussed in this report. These springs are known as Greathouse Spring and Glass Spring. Greathouse Spring is located at the intersection of State Highway Route 112 and Greathouse Springs Road (SE1/4, NE1/4, NW1/4, SEC. 20, T17N, R30 W). The Glass Spring is located just outside the town of Wheeler on the road between Wheeler and Tontitown (SE1/4, NE1/4, NW1/4, SEC 26, T17N, R31W). Both of these locations were monitored during the dye test study. The location of these springs in also presented on DRAWING 2 of 14.

The springs were sampled from March 1991 to March 1992. SCS compared the water quality results of these springs to the results obtained from monitoring wells MW-10 (downgradient) and MW-11 (upgradient). The comparison of values is presented in the following table.

TABLE 6.6
GROUNDWATER & SPRING WATER QUALITY

Parameter	Glass Spring	Greathouse Spring	MW-10	MW-11
Spec. Cond.	275 - 319	328 - 369	220	210
Bicarbonate	110 - 200	148 - 210	85.4	92.7
pH	6.7 - 7.3	6.6 - 7.3	7.5	7.0
Ammonia Nitrogen	1.43 - 4.13	1.55 - 3.45	0.02	1.66
Nitrate	0.01 - 0.26	0.01 - 0.09	0.117	0.177
Sulfate	4.2 - 13.2	4.2 - 12.5	8.83	12.67
Chloride	7.6 - 11.05	8.8 - 12.30	11.99	9.99
Calcium	45 - 56	62 - 70	54.3	39.5
Magnesium	1.36 - 1.65	1.18 - 1.75	7.48	0.68
Sodium	4.3 - 5.7	4.3 - 6.6	9.38	2.04
Potassium	1.00 - 1.87	0.76 - 2.07	2.28	1.14

Note: **(Reference 8)**

These results indicate that the water quality in both in MW-10 (downgradient) and MW-11 (upgradient) were comparable to the spring water quality.

6.3.7 GEOTECHNICAL TESTING

In order to characterize the site in terms of geotechnical properties, detailed geotechnical investigations were conducted by SCS and GEC. The investigations involved taking material samples at various locations and at various depths throughout the site and analyzing the material in the laboratory to gain information on the engineering properties of the samples. The first investigation, conducted by SCS Engineers during 1991, involved the collection of samples from soil borings and test pit excavations. The second investigation was conducted by GEC during 1996, and consisted of soil sampling during test pit excavations.

The thirteen sections that follow provide a brief summary of the investigations and the results of the associated laboratory analyses. Sections 6.3.7.1 and 6.3.7.2 summarize each field event and Sections 6.3.7.3 through 6.3.7.11 provide summaries to the various soil characteristic parameters that were analyzed in the study.

The geotechnical lab tests outlined below were conducted for characterization purposes, and compliance with ADPC&E Regulation No. 22, Section 22.1102(c)(6):

- Atterberg Limits (ASTM D4318)
- Standard Penetration Test (ASTM D1586-84)
- Sieve Analysis (ASTM D1140 & D422)
- Dry Density, Hydraulic Conductivity/Molding Water Content (%) Relationship
- Remolded Hydraulic Conductivity
- Unconsolidated, Unconfined Shear Strength of Soils (ASTM D2850)
- Standard Proctor Density (ASTM D698)
- Moisture-Density Relations of Soils and Aggregates (ASTM D1557-78)
- One Dimensional Consolidation Properties of Soils (ASTM D4546)
- Moisture Content of Soils (ASTM D2216-80)

APPENDIX G contains all geotechnical laboratory results associated with the following sampling and testing activities from the two investigations.

The subsurface investigations consisted of soil drilling, test pit excavation, and soil sampling. Samples were obtained at various locations and at different depths throughout the site during subsurface investigation activities. The sample locations were chosen in order to properly characterize soil properties in association with future Landfill uses. Cuttings were collected in some instances where it was impossible to obtain a sample using a shelby tube or split spoon sample due to the nature of the material. Shelby tubes were utilized in situations where it was desirable to obtain an undisturbed soil sample.

6.3.7.1 SCS SUBSURFACE INVESTIGATION

The SCS investigation was conducted to determine a possible final cover borrow source. The investigation was performed in the southwestern area of the Tontitown Site. According to the SCS document entitled "Final Closure Modifications Sites 3 and 4 (February 19, 1992), the SCS subsurface investigation consisted of geotechnical laboratory testing from 26 representative soil samples obtained from four soil borings and 19 test pits. Geotechnical laboratory tests included sieve and hydrometer grain size analyses, Atterberg Limits, moisture content, density, and soil identification in accordance with the Unified Soil Classification System (USCS). Additionally, 8 samples were tested for standard proctor densities and saturated hydraulic conductivity. The results from the lab testing can be found in APPENDIX G, and are summarized below.

The SCS investigation reported the soils encountered in the test pits and soil borings to be classified as silty gravels, clayey gravels with sand, and clayey gravels. Hydraulic conductivity tests were conducted on remolded samples compacted to 95-percent of the maximum dry density only using material passing a 3/8-inch sieve. Hydraulic conductivity values for these materials ranged from 1.71×10^{-6} to 7.38×10^{-7} cm/sec.

It should be noted that this material has since been utilized for Landfill cover material and is no longer available for use. It also should be noted that the Tontitown Landfill has access to a screening operation that can screen out unwanted chert fragments to decrease hydraulic conductivity values of materials utilized for clay liners or covers.

The percent of materials passing the No. 200 sieve for all samples tested ranged from 17.3 to 93.6 percent. The samples collected from the soil borings had a range of 39.0 to 93.6 percentage of material passing the No. 200 sieve while the material from the test pits had a range of 17.3 to 53 percent. The difference can be attributed to the fact that the continuous sampler utilized in the soil borings did not collect as much rock as in the test pits. DRAWING 1 of 14 indicates areas A, B, and C that contained material greater than 30 percent of the material passing the No. 200 sieve. The apparent volume of material in the outlined areas of A, B, and C was approximately 295,463 cubic yards.

6.3.7.2 GEC SUBSURFACE INVESTIGATION

GEC conducted a subsurface study in 1996 to characterize soil materials available for potential on-site borrow areas. A total of 3 test pits were excavated on the Landfill property. DRAWING 1 of 14 displays locations of the GEC test pits.

Geotechnical laboratory results pertaining to the test pit samples are located in APPENDIX G. The material sampled consisted of a reddish brown clayey chert composition with a water content range from 26.8 to 36.6 percent. Average Liquid Limit, Plastic Limit, and Plastic Indices were determined to be 50, 33, and 17, respectively. The percent of material passing the No. 200 sieve ranged from 35 to 42 percent. Hydraulic conductivity testing was performed on each of the three samples, utilizing three different water contents. The average permeability was calculated as $8.5\text{x}10^{-6}$ cm/sec, with a range of $4.7\text{x}10^{-7}$ to $3.2\text{x}10^{-5}$ cm/sec. Blow count datum were recorded by GEC geologists during the installation of monitoring wells and piezometers (as described in Section 6.3.6.1 of this report) and are noted on boring logs presented in APPENDIX D. If needed, "corrected" standard penetration values can be determined from the blow count data reported.

The following sections combine the SCS and GEC geotechnical results to portray a more accurate overall site geotechnical summary. All results utilized for the following summaries are also available in APPENDIX G of this document.

6.3.7.3 SIEVE ANALYSIS SUMMARY

Particle size analyses were conducted by GEC and SCS on various soils at various locations and depths for the purpose of analyzing grain size distribution and classification associated with soils native to the area.

In the sieve analysis, a series of sieves (screens) having different-sized openings are stacked with the larger sizes over the smaller. The soil sample being tested is dried, clumps are broken, and the sample is passed through the series of sieves by shaking. Larger particles are caught on the upper sieves, and the smaller particles filter through to be caught on one of the smaller underlying sieves. The weight of material retained on each sieve is conventionally presented as a grain or particle size distribution curve plotted on semilog coordinates.

The appearance of the particle size distribution plot depends on the range and amounts of various sizes of particles in the soil sample. These in turn have been affected by the soil's origin or the method of deposition. Well graded soils (a distribution of particles over a relatively large range of sizes) produce a curve. A uniform soil plots showing most of the particles of approximately similar size. The grain-size plots can provide an indication of a soils history. APPENDIX G contains the sieve analysis results for the GEC and SCS samples.

As indicated in TABLE 6.7, samples were analyzed in the laboratory for grain size distribution in accordance with ASTM D422. A total of 27 samples from boring locations and test pits were obtained for the purpose of characterizing the grain size distribution of local soils.

TABLE 6.7
GEOTECHNICAL RESULTS SUMMARY

TEST DESCRIPTION	AVERAGE RESULTS	RANGE OF RESULTS
% Passing # 200 Sieve	35.6	17.3 - 93.6
Natural Moisture (%)	28.4	13.5 - 50.3
Dry Unit Weight (pcf)	84.3	82.7 - 86.2
Liquid Limit (%)	70	28 - 100
Plastic Limit (%)	30	16 - 48
Plasticity Index (%)	40	15 - 59
Optimum Moisture (%)	31.1	22 - 38
Max. Dry Density (pcf)	86.9	78.5 - 103.5
Hydraulic Cond. (cm/sec)	5.6×10^{-6}	4.96×10^{-8} - 3.2×10^{-5}
U-U Triaxial Shear Strength: Cohesion (TSF)	0.99	0.43 - 1.4

6.3.7.4 ATTERBERG LIMITS SUMMARY

In the remolded state, the consistency of clay soil varies in proportion to the water content. At a higher water content, the soil-water mixture possesses the properties of a liquid. At lesser water contents a soil-water mixture possesses properties that resemble a plastic. At still lesser water contents, soil-water mixtures approach a solid or semi-solid state. The water content indicating the division between the liquid and plastic state has been designated the Liquid Limit. The division between the plastic and semi-solid state is referred to as the Plastic Limit. The numerical difference between the Liquid Limit and the Plastic Limit is identified as the Plasticity Index. These values are often referred to as Atterberg Limits. Atterberg Limits are used widely in soil applications and is a good measure of a soils workability for use in landfill liner systems. TABLE 6.7 summarizes Atterberg Limits for soil samples collected by GEC and SCS in the vicinity of the Tontitown Class 1 Modification Area.

In general, on-site clays determined to have Plasticity Indices greater than 10 (all samples) can be considered for used in the construction of any clay liner system. However, it should be noted that both subsurface studies determined that the on-site clays contain varying amounts of chert. This material must be screened in order to be utilized in a compacted clay liner system. As such, Sunray has developed an on-site screening process for such construction activities. The Atterburg Limits results are presented in APPENDIX G.

6.3.7.5 COMPACTION SUMMARY

Both SCS and GEC obtained samples from soils within the study area to determine their suitability in the construction of the clay liner system. A minimum of one composite sample per soil type was obtained and analyzed in the laboratory for determining the moisture-density relationship as defined in ASTM D698 and D1557. Based on Standard Proctor analyses taken from composite samples, it is anticipated that the optimum moisture content will be approximately 31% with a maximum dry density of approximately 86 pcf. Soil samples containing less percentages of chert will undoubtedly yield higher optimum moisture contents for compaction purposes. Standard proctor results are presented in APPENDIX G and TABLE 6.7.

6.3.7.6 HYDRAULIC CONDUCTIVITY SUMMARY

Soil samples were obtained from various locations by SCS and GEC for the purpose of characterizing the permeability characteristics of area clays. Results of these samples are presented in APPENDIX G. TABLE 6.7 summarizes the results of remolded hydraulic conductivity analysis for local soils.

Based on the above laboratory hydraulic conductivity results, it is conceivable that suitable clay material should be available on-site for use in the compacted clay liner system. Some of the clay material to be utilized in the construction of the clay liner may need to be screened to remove chert fragments. However, the clay material shall be capable of achieving a minimum hydraulic conductivity of 1x10-7 cm/s at compactions greater than 95% standard proctor density (0-4% wet of the optimum moisture content) as outlined in the Engineering Report (Volume 1).

6.3.7.7 SHEAR STRENGTH EVALUATION SUMMARY

Shear strength analysis was conducted on soil samples for the purpose of defining the relative stability of area soils in natural and engineering applications. More specifically, an unconsolidated-undrained (UU) triaxial test was performed on select samples collected by GEC from Samples #1, #2, and #3. The apparent cohesion was found to average 0.99 tons per square foot (tsf) at a 10 psi confining pressure. This information was utilized in slope stability calculations associated with natural and engineered slopes (RE: Permit Modification Application-Volume 1).

6.3.7.8 SOIL CLASSIFICATION SUMMARY

The Unified Soil Classification System (USCS) is commonly used in engineering and construction applications. Classifications are on the basis of coarse and fine grained soils and are categorized based on laboratory tests including the grain size distribution analysis and Atterberg Limits. In general, the following soil classifications were identified on site by SCS:

CH: Inorganic clays of high plasticity (fat clays);
SC: Clayey sands, sandy clay mixtures;
SM: Silty fine sands;
GC: Gravely fine clays
GM: Gravelly fine silts
MH: Silts with high plasticity

6.3.7.9 ONE-DIMENSIONAL SWELL POTENTIAL SUMMARY

According to the USDA Soil Conservation Service Soil Survey of Washington County, soils in the vicinity of the Landfill are not associated with significant shrink/swell properties which can negatively impact a clays use in construction applications. For this reason, it is not anticipated that the shrinkage/swelling characteristics of the native clay material will negatively impact the stability of the Landfill structure, therefore, one-dimensional swell potential tests were not performed on the soil material on-site.

6.3.7.10 STANDARD PROCTOR DENSITY SUMMARY

Standard Proctor density tests were performed on samples taken during both the GEC and SCS investigations in order to better classify the engineering properties of the soils on-site. Results indicate the average natural moisture content in the samples to be approximately 28.4 percent, with a range from 13.5 to 50.3 percent. The average dry unit weight was determined to be 84.3 pcf, with a range from 82.7 to 86.2 pcf. TABLE 6.7 and APPENDIX G contain all results for the standard proctor densities associated with SCS and GEC sampling events.

6.3.7.11 STANDARD PENETRATION SUMMARY

Standard penetration tests (SPT) were conducted on overburden soils in five borings during the GEC investigation. This test, when properly evaluated provides an indication of the soil strength and compressibility. This information is of particular interest associated with any seismic/liquefaction analysis that may be performed in conjunction with future investigations. The boring logs completed by GEC, located in APPENDIX G, note the "field" blow counts associated with the standard penetration test analysis.

6.3.7.12 SUITABILITY FOR LANDFILL USES

In general, soils in the vicinity of the Tontitown Class 1 Modification Area possess engineering properties which are conducive to applications pertaining to landfilling. When selecting clay for use in the construction of any compacted clay liner system, care will be given to segregate any clay materials which contain large percentages of chert.

Based upon the results of the SCS and GEC studies, the hydraulic conductivity characteristics of on-site clays are favorable to landfill applications as they provide good barriers to infiltration of water. When utilized in the construction of the compacted clay liner system, the moisture content should be maintained between 0 to 4% wet of the optimum moisture content in order to insure that the maximum hydraulic conductivity standard of $1x10^{-7}$ cm/s is met. The moisture content of clay liner materials shall also be closely monitored during construction to prevent any cracking or desiccation of the clay due to significant changes in moisture content.

Again, it should be stressed, that the soil material with high chert content will be screened prior to use as compacted clay liner material.

6.3.7.13 SOIL BUDGET

An approximate soil budget was determined for estimating the required soil for the Tontitown Class 1 Modification Area and the available soil in the vicinity of the Tontitown Class 1 Modification Area. A soil budget summary is provided in Volume 1 of the Permit Modification Application. In general, it is estimated that approximately 794,240 cubic yards (cy) of earthen material will be required associated with the construction and operation of the Tontitown Class 1 Modification Area. Of the estimated 794,240 cy needed, 346,540 will be utilized for clay liner and cap material, with 447,700 cy allocated for cover soil requirements. It is estimated that approximately 156,000 cy of earthen material will be available for use from the excavation of the Site 3 and Site 4 areas (within the boundary of the Tontitown Class 1 Modification Area). The remaining 638,240 cy is to be obtained from on-site borrow sources that are available.

6.4 HYDROGEOLOGIC MODEL (22.1102(d))

Arkansas Regulation 22, Section 22.1102(d) requires a summary report to compile the individual elements of the overall hydrogeologic and geotechnical information. More specifically, Section 22.1102(d) requires a comprehensive hydrogeologic model certified by a qualified groundwater scientist. Section 22.1101(b) further describes the conceptual hydrogeologic model of the site as an integration of all characterization studies that accurately describes and explains the site hydrogeology. The model must be of sufficient detail to base the design and be used as a predictive tool for potential contaminant migration, and serve as a basis for corrective actions.

Since the currently proposed action is a modification of an existing Class I operation that has been in place for many years, the purpose of the current hydrogeologic study is to review and if necessary revise the existing hydrogeologic model.

The existing conceptual hydrogeologic model has been revised over time. However, prior to the preparation of this document, the most comprehensive site hydrogeologic model was prepared as a joint effort by Geraghty & Miller, Inc. and Environmental Management, Inc. in a report titled *Hydrogeological Characterization of the Sunray Landfills* dated September 2, 1987 (hereafter referred to as the GM/EM Report).

The conclusions of the GM/EM report were as follows:

1. *The field program confirmed that the Sunray Landfill sites are underlain by residuum and limestone bedrock.*

2. *The residuum varies in thickness from 28 to 79 feet and has measured hydraulic conductivities ranging from* 3.2×10^{-4} *to* 8.8×10^{-6} *cm/sec (horizontal) to* 6.3×10^{-7} *cm/sec (vertical).*

3. *The Boone Limestone was encountered beneath the residuum and was found to be a massive, well cemented competent limestone that allowed open-hole monitoring well completion. The limestone has an average horizontal hydraulic conductivity of* 3.5×10^{-5} *cm/sec.*

4. *Groundwater in the limestone flows from northwest to southeast beneath the sites at an average velocity of 12 ft/yr.*

In an effort to test and revise the existing GM/EM conceptual model GEC has provided the following additional information throughout this document:

1. Additional fracture trace analysis via mapping and photography has been provided (Section 6.3.2).

2. Additional surface geologic investigations have been performed (Section 6.3.3)

3. Surface geophysical studies have been performed (Section 6.3.4).

4. Additional subsurface explorations have been conducted including additional borings and down hole geophysics (Section 6.3.5).

5. The hydrogeology of the site has been further investigated via the installation of additional piezometers and additional aquifer testing techniques (Section 6.3.6).

The final product of the current hydrogeologic investigation is the revised comprehensive conceptual hydrogeologic model presented in DRAWING 13 of 14, supplemented with the actual cross sections presented in DRAWINGS 9, 10, and 11 of 14, and the bedrock and potentiometric contours presented in DRAWINGS 8 and 12 of 14 respectively. A combination of the existing and the new hydrogeologic information provides a comprehensive evaluation of the site stratigraphy, structure, groundwater flow direction, and groundwater velocity. From this comprehensive evaluation of the site, a final groundwater monitoring system can be developed as presented in Section 6.5 of this document. The details of the final revised conceptual hydrogeologic model are as follows:

Stratigraphy

As mentioned above, the stratigraphy specified in the existing site hydrogeologic model is a relatively thick (28 to 79 feet) cherty clay residuum, underlain by Boone Limestone, which is hydraulically confined by the underlying Chattanooga Shale. The Boone Limestone was characterized as massive, well cemented competent limestone that allowed open-hole monitoring well completion (See GM/EM report dated 1987).

The data from the current investigation confirmed the cherty clay overburden above the bedrock as extensive both horizontally and vertically. The thickness of the overburden indicated in the current study ranges from 45 to 70 feet (bls). Based on the borings, test pits, and surface geophysics, it is believed that a clay thickness of at least 30 feet, and as much as 70 feet underlies the entire Class I landfill. The required ten foot separation distance from the bottom of the liner should be maintained (See Engineering Cross-Sections Volumes 1 and 2).

The current investigation also confirmed that the Boone Limestone underlies the residuum and that the Boone formation is generally massive and competent at depth. However, clay filled "voids" were indicated in several borings at the bedrock-overburden contact. It is well documented and observed on-site that the Boone Formation commonly weathers into bedrock pinnacles. These pinnacles occur in areas of limestone with high chert content which is more resistant to weathering.

An example of a bedrock pinnacle was encountered in PZ-1 (see Section 6.3.5.5). This boring first encountered limestone, then encountered more residuum before finally reaching the total depth in the competent bedrock limestone. This was confirmed with the surface resistivity survey and drilling. It appears that the clay filled "voids" encountered during drilling correspond to the areas where pinnacles also occur. Basically the limestone is weathered away leaving the chert beds and clay filled voids. In most cases these voids were not found to be saturated.

All zones characterized as voids were zones where highly soluble calcium carbonate limestone beds are dissolutioned leaving the areas of bedrock with greater chert content. The chert beds remain in place while beds above and below are removed. The voids observed in PZ-1, MW-1R and MW-5R were believed to be formed by the process described above.

The "void" zones are not solution cavities within the limestone bedrock. Drilling extended from bedrock into a clay void and returned to competent bedrock. Since the surrounding surface geophysics and borings did not indicate competent bedrock, the "voids" are actually competent bedrock pinnacles that are limited in extent. As explained in Section 6.2.3.2, these pinnacles are typical of the Boone Formation.

The nature of the competent Boone Limestone can be readily interpreted from the boring logs, the downhole geophysics, and examination of the actual cores of the borings. In general, the Boone Formation consists of competent limestone with fractures and joints common throughout. RQD values of greater than 80% were noted in the GEC borings (see TABLE 6.3). Groundwater was generally encountered in iron stained fractures. The Chattanooga Shale which underlies the Boone Formation was shown in the geophysical log of the deep on site well (See APPENDIX H).

Geologic Structure

The previous site conceptual hydrogeologic model does not address the geologic structure of the site in detail. The GM/EM report does state that the landfills were originally located in a valley that was stripped of the upper soils and a clay liner placed prior to initial waste disposal.

The current investigation addressed structure in terms of lineament analysis, aerial photography, geologic mapping, and literature research. Site 3 and Site 4 appear to have filled two separate "valleys" according to aerial photographs taken prior to landfill operations. These valleys are possibly surface expressions of a bedrock lineament. Although solution enlargement of fractures was not evident in the borings or the geophysics, it is possible that the original valleys are structurally controlled and represent preferred groundwater movement. Dye injection and monitoring well placement is therefore in the same lineaments. Based on the bedrock contour map, surface topography closely approximates the underlying bedrock topography (see Bedrock Contour Map DRAWING 8 of 14).

The current study also investigated the occurrence of faults in the vicinity of the landfill site. As described in Section 6.2.3.1, the site is surrounded by regional faults on three sides. These faults structurally control both Wildcat Creek on the east and Clear Creek to the South. The identification of the regional faulting is from apparent displacement of the Chattanooga Shale underlying the Boone limestone.

Two existing wells were identified as part of the site investigation that penetrate the Chattanooga Shale. These wells are located immediately northeast (Gina Marie Well) and southwest (Sutton Well) of the landfill site (See DRAWING 2 of 14). As explained in Section 6.3.3, there is no apparent displacement in the Chattanooga Shale at these locations, indicating that a fault does not extend across the existing landfill operations.

Additional structural information that can be added to the existing site conceptual model is the presence of bedrock "pinnacles". This weathering feature of the Boone formation is well documented and is described conceptually in the previous sections of this document.

Few surface expressions showing dissolution of carbonate rocks occur within and near the vicinity of the site, because the thick regolith that develops from the Boone Formation masks karst features at the regolith-bedrock interface. Only three types of surface expressions were identified within 5 miles of the site. These include a sinkhole, a sinking stream, and numerous springs. Of those only two are input forms, include a sinkhole, and a sinking stream; output forms are springs (see FIGURE 6.3 and DRAWING 2 of 14)(Reference 3).

Groundwater flow characteristics

The final element of the existing conceptual hydrogeologic model concerns the direction and rate of groundwater flow. As mentioned previously, the existing model states that the groundwater in the limestone flows from northwest to southeast beneath the sites at an average velocity of 12 feet/yr. The hydraulic conductivity of the overlying residuum was also indicated at a very slow rate (10^{-4} to 10^{-7} cm/sec range).

The GEC investigation confirmed the slow groundwater movement potential with measured hydraulic conductivities of the residuum in the 10^{-5} to 10^{-7} cm/sec range (See Section 6.3.7.6). Hydraulic conductivities of the bedrock were measured in the range of 6.4 $\times 10^{-5}$ to 2.89 x 10^{-6} (see Section 6.3.6.3). Even though hydraulic conductivities are known to vary widely in carbonate aquifers, the reported conductivity values do not.

The current investigation also performed a pump test between two wells that provided a more representative average flow velocity of 5.3 ft/day. Even though this test confirmed the very slow velocity of groundwater in the uppermost aquifer, it is faster than previously assumed. Based on the groundwater flow velocities determined from the slug tests, pump test, and the dye test, groundwater in the uppermost aquifer does not appear to move via conduit flow as might be expected in a carbonate aquifer. Groundwater is moving in the less than 5 feet per day range instead of feet per hour range expected in open void spaces. Therefore it appears that groundwater flow can be characterized as diffuse or fissure (concentrated flow).

Areas of the aquifer characterized by diffuse flow represent a less mature karst system. The groundwater flow in such a system is through small bedrock openings that have undergone only limited solutional enlargement. Groundwater flow velocities are low and groundwater may require months to travel a few feet through the aquifer (Reference 16). The discharge at the streams is generally uniform and slow to respond to storm events. Groundwater movement is characterized by either concentrated or diffuse flow. The degree of concentrated versus diffuse flow depends upon the degree of solutional development (Reference 16). Based on the findings of previous and recent hydrogeologic investigations conducted in the vicinity and at the Sunray Tontitown site (Reference 3), it is believed that the karst system underlying the site is immature.

Groundwater flow direction in the uppermost limestone aquifer is previously indicated in a northwest to southeast direction. This flow direction continues to be evident by the water level measured over several years in the on-site monitoring system. Extremes of seasonal precipitation have now been observed, however groundwater elevations do not vary widely (see TABLE 6.4). Therefore it appears recharge to the aquifer does not occur rapidly via conduits.

As determined by the dye test, a possible groundwater divide exist along the northern portion of the site. Positive traces were believed to have been detected at monitoring points MW-1R and DOM-1 indicating a north to northwest flow direction.

Dye Test Analysis and the Conceptual Hydrogeologic Model

As described in Section 6.3.6.5, one of the purposes of the dye test study is to evaluate the existing and modified hydrogeologic model. **The dye test has confirmed the following elements of the hydrogeologic model**:

1) **The flow velocity of the underlying limestone aquifer is very slow. Therefore, the flow of potential contaminants across the site is very slow. This characteristic is particularly evident because the flow rate was not influenced by the excessive precipitation experienced during the dye test.**

2) **Since dye may have been traced in a direction opposite of the observed groundwater flow direction, the dye test may disqualify the assumed flow direction measured by the existing monitoring wells. However, if the flow velocities are accurate as discussed previously, it could take months for the dye to reach any of the sampling points.**

In summary the essential elements of the conceptual hydrogeologic model as modified by the current study are as follows:

- The groundwater velocity of the bedrock aquifer is very slow;
- Flow occurs within the bedrock as diffuse flow through joints and fissures some of which may be solutionally enlarged;
- **Groundwater follows a flow direction across the site in a basic southerly direction according to historical groundwater elevation measurements; however the dye tests indicated a northern flow component attributed to a possible groundwater divide.**
- The uppermost aquifer and the first avenue of escape from the landfill is the bedrock aquifer;
- The residual Boone overlying the bedrock aquifer is a low permeability clay that partially confines the underlying uppermost aquifer;

DRAWING 13 of 14 presented an idealized three dimension look at the hydrogeologic model.

6.5 PROPOSED GROUNDWATER MONITORING SYSTEM

As detailed in Section 6.3.5.6 and 6.3.6.1 of this document, Sunray currently operates an approved groundwater monitoring system consisting of 10 monitoring wells. This system has been modified over time as explained in Section 6.3.5.5. During the course of the current hydrogeologic characterization and the related redesign of the facility, several potential deficiencies were identified with the existing system. These deficiencies are significant enough to require the replacement of the existing groundwater monitoring system. This section of Volume 3 describes the deficiencies with the existing monitoring system and proposes a groundwater monitoring system.

The deficiencies with the existing groundwater system have been discussed throughout the report and are summarized as follows:

1) Physical observations and a review of the past history of the existing monitoring wells indicate potential problems that could threaten the ability of the system to provide samples representative of groundwater quality. Physical observations of the existing wells indicated unvented wells retrofitted with dedicated submersible pumps. The wiring of the pumps and the condition of the wells themselves may be adversely affecting the groundwater quality.

   In addition to the observable condition of the wells, it should be noted that a downhole camera survey conducted by SCS Engineers, Inc. in 1993 revealed serious well construction problems that required the replacement of MW-3, MW-4, MW-7, and MW-9 (See SCS report dated August 13, 1993). It is possible that the remaining wells could have equally serious well construction problems that were not detected previously. Since the wells were completed as open hole construction, the seal along the annual space between the casing and the residuum is very important to avoid surface contamination.

2) MW-3, 4, 6, and 8 are within 50 feet or less of an existing surface water control pond. The surface water control ponds on the site have been enlarged to comply with applicable stromwater Run-off design regulations since the establishment of the monitoring wells. A borrow

area is also located immediately adjacent to MW-3 that has possibly been excavated below the screened interval of MW-3. Surface water influences are a possibility in these wells.

3) Several of the existing monitoring wells including MW-1, 4, 5, and 6 are located very close to the waste mass. It is possible that MW-5 is almost within an old waste disposal area filled prior to Sunray's acquiring purchase of the site.

4) Another major problem with the existing groundwater monitoring system is the fact that the proposed design of the modification will require the movement of most of the wells. More specifically, the proposed modification specifies a design that provides for long term stability of the waste mass. The slopes must be extended over the existing wells in many cases in order to provide the most stable configuration. In addition, the proposed design incorporates a composite liner in the cap that requires anchoring and leachate/gas collection provisions that extend beyond the existing waste boundaries.

5) MW-2 is located in the center of the new Class IV landfill. The size of the existing ponds must also be enlarged to comply with the 100 year containment requirements of the 4-County District Rule. See Volumes 1 and 2 of this permit modification application for additional details.

Proposed replacement wells for three of these wells:, MW-1, MW-2, and MW-5, were installed during the recent hydrogeologic investigation. These wells were proposed to be moved based on the proximity of the existing wells to waste and to accommodate the currently proposed expansion plans submitted by Sunray.

Based on the final proposed engineering design submitted with Volume 1, monitoring wells MW-3, MW-4, MW-6, MW-7 will also need to be moved to accommodate the new designed landfill contours and the 100 year runoff control ponds. Additional land has been purchased since the monitoring system was installed allowing the wells to be moved to accommodate the proposed modification.

This permit modification therefore formally proposes to properly abandon all of the existing monitoring wells with the exception of MW-8, MW-10 and MW-11. These wells were installed more recently by SCS Engineers, Inc. MW-11 is located definitely upgradient of all of the site. All of the wells are to be replaced in the location indicated on DRAWING 14 of 14. Sunray also proposes to start monitoring the entire system under the provisions of a detection monitoring program, sampling on a quarterly basis, and utilizing an upgradient to downgradient statistical analysis (see Section 6.3.5.6 of this document for the statistical analysis discussion).

7.0 GROUNDWATER MONITORING AND CORRECTIVE ACTIONS

(Ch 12 of Regulation 22)

As described in the previous section of this document (Section 6.3.5.5), Sunray proposes to modify the current groundwater monitoring system by replacing most of the existing monitoring wells to accommodate the new point of compliance for the proposed engineering design. Therefore, according to Section 22.1201 of Regulation 22, the provisions of Chapter 12 of Regulation 22 concerning groundwater monitoring and corrective action are applicable to the proposed modification. Sunray is currently in compliance with the requirements of this sub-part in accordance with the schedules outlined in Section 22.1201(b) of Regulation 22. However, the proposed groundwater monitoring program must maintain compliance with all of the requirements of Chapter 12.

- Section 22.1201(c) provides for alternative compliance schedules that are not applicable to the current facility.

- Section 22.1201(d) establishes that the groundwater monitoring shall be conducted throughout the active life and the post-closure period of the landfill. Sunray does not intent to petition for a waiver as provided under Section 22.1201(e).

- As required by Section 22.1201(f), the proposed groundwater monitoring system will be certified by an Arkansas Registered Professional Geologist experienced with landfill groundwater monitoring system at Arkansas landfills.

- The Alternative compliance schedules specified in Section 22.1201(g) are not applicable to the proposed system.

7.1 GROUNDWATER MONITORING SYSTEM (Section 22.1202)

As provided in Section 22.1201(a), a groundwater monitoring system must be installed that consists of a sufficient number of wells, installed at appropriate locations and depths, to yield groundwater samples from the uppermost aquifer that: 1) represent the quality of background groundwater that has not been affected by leakage from a unit and 2) represent the quality of groundwater passing the relevant point of compliance.

As explained in Section 6.3.5.6, ADPC&E has specified an intra-well statistical analysis of groundwater monitoring data due to the potential of conduit flow in alternate directions due to seasonal influences. Since the historical piezometric surface has consistently indicated a definite northwest to southeast direction and the dye test has not disproved this assumption, Sunray feels that the new groundwater monitoring system will be in compliance with Section 22.1202 with the designated upgradient wells. The downgradient locations indicated on DRAWING 14 of 14 are intended to comply with the point of compliance requirements specified in Section 22.1202(a)(2).

The new monitoring wells will be cased in a manner that maintains the integrity of the monitoring well bore hole in compliance with the requirement in Section 22.1202(c). The Director will be notified by the qualified groundwater scientist that the documentation of the design, installation, development, and the decommission of the existing wells has been properly executed and placed in the facility operating record as required in 22.1202(c)(1) and 22.1201(c)(2)

Section 22.1202(d) states that the number, spacing, and depths of monitoring systems shall be determined based upon site-specific technical information developed in accordance with the requirements of Chapter Eleven. Section 6.4 of this document provides the details of the hydrogeologic characterization and the hydrogeologic model for which the groundwater monitoring system is designed. The uppermost aquifer is the Boone Limestone. Water occurs in this formation within secondary porosity associated with fractures and joints in the more massive bedrock. The uppermost aquifer is partially confined by the overlying cherty clay of the Boone residuum. As explained previously, the uppermost aquifer is confined at the lower boundary with the Chattanooga Shale Formation.

The proposed monitoring wells are located in the lineament traces associated with the site in an effort to intersect the greatest amount of secondary porosity associated with structural control. The monitoring wells are all extended into the Boone Formation to a depth necessary to produce enough groundwater to provide the required sampling parameters. All of the wells will be completed to ASTM and TEGD standards.

The groundwater monitoring system will be certified by an Arkansas Registered Professional Geologist experienced with the installation of groundwater monitoring systems at Arkansas landfills. The certification will be placed in the facility permanent operating record and notification provided to the Director of the ADPC&E as specified in Section 22.1202(e).

REFERENCES

1. Geraghty & Miller, Inc. and Environmental Management, Inc. 1987 "Hydrogeological Characterization of Sunray Landfills"

2. United States Department of Agriculture, Soil Conservation Service, March 1969 "Soil Survey, Washington County, Arkansas"

3. Brahana, John Van, 1996 "Draft Methodology For Delineating Groundwater Recharge Area of Springs in Carbonate-Rock Terrain of Northwest Arkansas" U.S. Geological Survey

4. Staton, G.P. and Brahana, J.V. 1996, Structural Control on Hydrogeology of a Mantled Karst Aquifer in Northwestern Arkansas: Geological Scoiety of America, Abstracts with Programs, V. 28, No. 6 (in press)

5. Branner George C. 1968, Benton, Carroll, Madison, Washington, Counties, Mineral Report, Arkansas Geological Survey County Report 2.

6. Croneis Carey, 1930, Geology of the Arkansas Paleozoic Area, Arkansas Geological Survey Bulletin 3.

7. Manger, W.L. and Shanks, J.C., 1977, Lower Mississippian Lithostratigraphy, Northern Arkansas: Arkansas Academy of Sciences Proc., 1976 V. 30., pp.78-80.

8. SCS Engineers, 1992 "Hydrogeologic Investigation Report Sunray Services Sanitary Landfill Tontitown, Arkansas".

9. Bahar, R.C., 1955 " Arkansas' Ground-Water Resoures" State of Arkansas Geological and Conservation Commission, Water Resources Circular No. 1

10. Ogden, A.E., 1979 "Hydrogeologic and Geochemical Investigation of the Boone - St. Joe Limestone Aquifer in Benton County, Arkansas" Arkansas Water Resources Research Center.

11. Keys W. Scott and MacCary L.M., "Application of Borehole Geophysics to Water-Resources Investigations". Book 2 Chapter E1 U.S. Geological Survey.

12. Zohody, A.A.R. 1974, "Applications of Surface Geophysics To Ground-Water Investigations". Book 2 Chapter D1 U.S. Geological Survey

13. SCS Engineers 1992, Final Closure Modification Site 3 and Site 4 Engineering Report

14. McNeill, J.D., 1980. Electromagnetic Terrian Conductivity Measurements at Low Induction Numbers: Technical Note TN-6 Geonics Ltd., Mississauga, Canada

15. Bouwer H. and R.C. Rice. 1976. A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells. Water Resources Research. Vol. 12, No 3 pp. 423-428

16. U.S. EPA 1988 "Application of Dye-Tracing Techniques For Determining Solute-Transport Characteristics of Ground Water in Karst Terranes" EPA 904/6-88-001 October 1988

Boring #: C4B-1
Date: 1/20/97
Elevation:1223.02
Job No.: 9541

GEC GENESIS ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		Clay, gray to orangish brown with chert fragments	—△— —△—			Auger refusal @ 4.5ft. Total depth 4.5ft.
	5'					
	10'					
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-1-(2)
Date: 1/20/97
Elevation: 1224.95
ob No.: 9541

GEC
GENESIS
ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		Clay, gray to orangish brown with chert fragments				Auger refusal @ 4.5ft. Total depth 4.5ft.
	5'					
	10'					
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-2	GEC GENESIS ENVIRONMENTAL CONSULTING, INC. 11400 West Baseline Road Little Rock, AR 72209	Location: Sunray / Tontitown / Class IV
Date: 1/21/97		Drilling Method: Augers
Elevation: 1220.38		Driller: GG&H / Aron Todd
ob No.: 9541		Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
	5'	Clay, gray to orangish brown, silty with chert fragments	—Δ—			Auger refusal @ 6.5ft. Total depth 6.5ft.
	10' 15' 20' 25' 30' 35' 40'					

Boring #: C4B-3	**GEC** GENESIS ENVIRONMENTAL CONSULTING, INC. 11400 West Baseline Road Little Rock, AR 72209	Location: Sunray / Tontitown / Class IV
Date: 1/20/97		Drilling Method: Augers
Elevation: 1258.18		Driller: GG&H / Aron Todd
ob No.: 9541		Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		0–11ft. Clay, orangish brown silty with chert fragments				Did not encounter auger refusal or bedrock Total depth of boring 20ft.
	5'					
	10'					
		11–20ft. Clay, gray, silty, with chert fragments				
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-4
Date: 1/21/97
Elevation: 1261.98
ob No.: 9541

GEC
GENESIS
ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		Clay, orangish brown, silty with chert fragments				Boring T.D. @ 20ft. Did not encounter bedrock or auger refusal
	5'					
	10'					
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-5
Date: 1/21/97
Elevation: 1230.34
ob No.: 9541

GEC

GENESIS
ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		0–8.5ft. – Clay, orangish brown silty with chert fragments				Did not encounter Auger refusal or bedrock. Boring was T.D. @ 20ft.
	5'					
	10'	8.5–20ft. – Clay, gray, silty with chert fragments				
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-6
Date: 1/21/97
Elevation: 1232.03
ob No.: 9541

GEC
GENESIS ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
	5' 10' 15' 20'	Clay, orangish brown, silty with chert fragments				Boring was terminated @ 20ft. Did not encounter bedrock or auger refusal
	25' 30' 35' 40'					

Boring #: C4B-7
Date: 1/21/97
Elevation: 1222.75
ob No.: 9541

GEC GENESIS ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

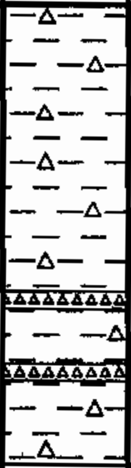
Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
		Clay, gray, silty with chert fragments & thin chert layers	—△— — △— — — —△— — — — —△— — — —△			Boring was terminated @ 4ft. due to auger refusal
	5'					
	10'					
	15'					
	20'					
	25'					
	30'					
	35'					
	40'					

Boring #: C4B-8
Date: 1/21/97
Elevation: 1220.21
Job No.: 9541

GEC
GENESIS
ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

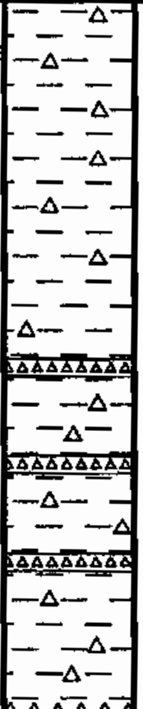
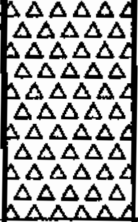
Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
	5' 10'	*0–7ft. Clay, gray, silty with chert fragments* *7–12.5ft. Clay, orangish brown silty chert fragments and occasional chert layers*				*Boring was terminated @ 12.5 feet due to auger refusal. From 7 feet to 12.5 feet occasional thin chert layers were encountered*
	15' 20' 25' 30' 35' 40'					

Boring #: C4B-9
Date: 1/21/97
Elevation: 1210.61
Job No.: 9541

GEC
GENESIS
ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Sunray / Tontitown / Class IV
Drilling Method: Augers
Driller: GG&H / Aron Todd
Logged By: Quin Baber

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Blow Count
	5' 10' 15'	0–19ft. Clay, orangish brown to gray with chert fragments & occasional thin chert seams				Augered to 19 feet & set steel casing to prepare for coring
	20'	At 19ft.–Chert, white, chalky, iron staining, disolutioning features, small horizontal & vertical fractures		97%	Run 1 19–24	Small horizontal & vertical fractures with iron staining
	25'	Chert, white, with limestone clast brecciated, horizontal fractures every 4–6 inches		▼ 81%	Run 2 24–28	Horizontal fracturing every 4 to 6 inches Fracture probably producing water
	30'	Chert, white with brecciated limestone fragments, fossiliferous & horizontal fractures		100%	Run 3 28–33	Horizontal fractures probably making water
	35'	Chert, white, with brecciated limestone fragments, fossiliferous, horizontal fractures, solution features iron staining along fractures		100%	Run 4 33–38	Horizontal fracture probably making water. Stabalized water level recorded at 26ft.
	40'					Boring T.D. @ 38ft.

TO BE INSERTED INTO APPENDIX F

Test Pit #: (see below)
Date: 3/12/97
Elevation: N/A
Job No.: 9541-3

GEC GENESIS ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Tontitown LF
Drilling Method: Excavated with Backhoe
Backhoe Operator: Jim (Sunray Employee)
Logged By: Steve Jett

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Remarks
TEST PIT #1		Brown Silty Clay	▨			Topsoil
	5'	Red, Silty Clay with Considerable Amounts of Chert Cobbles Present	▩			No Samples Collected, All Visual Logging. Verified Chert/Limestone Material w/ HCl Acid.
	10'					TD=10'
TEST PIT #2						
	0'	Brown Silty Clay	▨			Topsoil
	5'	Red Silty Clay with Chert Cobbles and Chert Layers Present Throughout	▩			
	10'					TD=10'
TEST PIT #3						
	0'	Brown Silty Clay	▨			Topsoil
		Tannish Brown Silty Clay	▨			
	5'	White Chert Layer	△			
		Red Silty Clay with Very Stiff Chert Cobbles and Chert Layers Present	▩			TD=8'
	10'					

NOTE: BACKHOE'S MAXIMUM EXCAVATION POTENTIAL IS 10' DEPTH

Test Pit # (see below)
Date: 3/12/97
Elevation: N/A
Job No.: 9541–3

GEC GENESIS ENVIRONMENTAL CONSULTING, INC.
11400 West Baseline Road
Little Rock, AR 72209

Location: Tontitown LF
Drilling Method: Excavated with Backhoe
Backhoe Operator: Jim (Sunray Employee)
Logged By: Steve Jett

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Remarks
TEST PIT #4		Brown Silty Clay	▨			Topsoil
	5'	Red, Silty Clay with Considerable Amounts of Chert Cobbles Present	▨			No Samples Collected, All Visual Logging. Verified Chert/Limestone Material w/ HCl Acid.
	10'					TD=10'
TEST PIT #5	0'					
		Brown Silty Clay	▨			Topsoil
	5'	Red Silty Clay with Chert Cobbles and Chert Layers Present Throughout	▨			
	10'					TD=9.5'
TEST PIT #6	0'					Topsoil
		Brown Silty Clay	▨			Soils highly saturated Water seeping in test pit at bottom, making logging visually difficult so stopped at 8'
		Red Silty Clay with Chert Cobbles	▨			
	5'	White Chert Layer	△			
		Red Silty Clay with Chert Cobbles	▨			TD=8'
	10'					

Test Pit #: (see below)	GEC // **GENESIS** ENVIRONMENTAL CONSULTING, INC. 11400 West Baseline Road Little Rock, AR 72209	Location: Tontitown LF
Date: 3/12/97		Drilling Method: Excavated with Backhoe
Elevation: N/A		Backhoe Operator: Jim (Sunray Employee)
Job No.: 9541-3		Logged By: Steve Jett

Elev.	Depth	Classification	Litho. Symbol	% Core Recovery	Sample or Box No.	Remarks
TEST PIT #7		Brown Silty Clay	▨			Topsoil
	5'	Red, Silty Clay with Chert Cobbles and Thin Layers Present Throughout	▨			No Samples Collected. All Logging Visual. Verified Chert/Limestone material w/HCl Acid. Soils highly saturated Water Seeping in hole at 3' depth. Filled up bottom of test pit and still running a couple of hours later when checked.
	10'					TD=10'

NEW APPENDIX O

TO BE INSERTED IN APPENDIX K
REPLACE EXISTING INFORMATION

DYE TEST RESULTS

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	MW-1R	ND	ND	0.017
11/11/96	MW-1R	0.032	ND	0.014
11/13/96	MW-1R	0.024	ND	0.014
11/15/96	MW-1R	0.017	ND	0.021
11/19/96	MW-1R	0.018	ND	0.01
11/26/96	MW-1R	0.078	ND	0.051
12/4/96	MW-1R	0.008	ND	0.008
12/17/96	MW-1R	0.018	ND	0.014
12/31/96	MW-1R	0.012	ND	0.013
1/23/97	MW-1R	0.025	ND	0.013
2/7/97	MW-1R	0.01	ND	0.014
2/19/97	MW-1R	ND	0.128	0.074
3/13/97	MW-1R	ND	0.116	ND
11/11/96	MW-2	0.02	ND	0.015
11/13/96	MW-2	0.02	ND	0.009
11/15/96	MW-2	0.007	ND	0.008
11/19/96	MW-2	0.008	ND	0.008
11/26/96	MW-2	0.022	ND	0.029
12/4/96	MW-2	0.006	ND	0.01
12/17/96	MW-2	0.006	ND	0.01
12/31/96	MW-2	0.006	ND	0.012
1/23/97	MW-2	ND	0.057	ND

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	MW-2R	0.008	ND	0.024
11/11/96	MW-2R	0.045	ND	0.021
11/13/96	MW-2R	0.026	ND	0.008
11/15/96	MW-2R	0.012	ND	0.009
11/19/96	MW-2R	0.008	ND	ND
11/26/96	MW-2R	0.023	ND	0.031
12/4/96	MW-2R	0.006	ND	0.007
12/17/96	MW-2R	ND	ND	0.006
12/31/96	MW-2R	ND	0.184	ND
1/23/97	MW-2R	0.035	ND	0.011
2/7/97	MW-2R	0.006	ND	0.01
2/19/97	MW-2R	ND	0.583	0.335
3/13/97	MW-2R	ND	0.379	ND

11/7/96	MW-3	0.028	ND	0.011
11/11/96	MW-3	0.012	ND	0.014
11/13/96	MW-3	0.05	ND	0.016
11/15/96	MW-3	0.01	ND	0.016
11/19/96	MW-3	0.023	ND	ND
11/26/96	MW-3	0.08	ND	0.111
12/4/96	MW-3	0.02	ND	ND
12/17/96	MW-3	0.035	ND	0.012
12/31/96	MW-3	0.015	ND	0.015
1/23/97	MW-3	0.01	ND	0.019
2/7/97	MW-3	ND	ND	0.013
2/19/97	MW-3	ND	0.096	0.032
3/12/97	MW-3	0.038	0.005	0.02

DYE TEST RESULTS
(CONT)

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/7/96	MW-4	0.007	ND	0.013
11/11/96	MW-4	0.262	ND	ND
11/13/96	MW-4	0.05	ND	0.014
11/15/96	MW-4	0.033	ND	0.01
11/19/96	MW-4	0.084	ND	ND
11/26/96	MW-4	0.386	ND	0.088
12/4/96	MW-4	0.151	ND	ND
12/17/96	MW-4	0.427	ND	ND
12/31/96	MW-4	0.273	ND	ND
1/23/97	MW-4	0.085	ND	ND
2/7/97	MW-4	0.086	ND	ND
2/19/97	MW-4	0.06	ND	0.008
3/12/97	MW-4	0.218	ND	ND
11/7/96	MW-5R	0.007	ND	0.014
11/6/96	MW-5R	0.056	ND	0.024
11/11/96	MW-5R	0.143	0.109	ND
11/13/96	MW-5R	0.02	ND	0.012
11/15/96	MW-5R	0.014	ND	0.008
11/19/96	MW-5R	0.024	ND	ND
11/26/96	MW-5R	0.038	ND	0.031
12/4/96	MW-5R	0.011	ND	0.006
12/17/96	MW-5R	0.014	ND	0.01
12/31/96	MW-5R	0.089	ND	0.041
1/23/97	MW-5R	0.01	ND	0.013
2/7/97	MW-5R	0.055	ND	0.01
2/19/97	MW-5R	0.026	ND	0.013
3/12/97	MW-5R	0.038	ND	0.026
11/11/96	MW-5R	0.036	ND	0.011

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/7/96	MW-6	0.006	ND	0.011
11/11/96	MW-6	0.057	ND	0.023
11/13/96	MW-6	0.013	ND	0.009
11/15/96	MW-6	0.006	ND	0.009
11/19/96	MW-6	ND	ND	ND
11/26/96	MW-6	0.022	ND	0.031
12/4/96	MW-6	0.007	ND	0.009
12/17/96	MW-6	ND	ND	0.011
12/31/96	MW-6	0.007	ND	0.01
1/23/97	MW-6	0.007	ND	0.012
2/7/97	MW-6	ND	ND	0.008
2/19/97	MW-6	ND	ND	0.011
3/13/97	MW-6	0.026	ND	0.03
11/7/96	MW-7	0.007	ND	0.015
11/11/96	MW-7	0.084	ND	ND
11/13/96	MW-7	0.015	ND	0.008
11/15/96	MW-7	0.01	ND	0.012
11/19/96	MW-7	ND	ND	ND
11/26/96	MW-7	NS	NS	NS
12/4/96	MW-7	ND	ND	ND
12/17/96	MW-7	ND	0.039	ND
12/31/96	MW-7	ND	0.009	ND
1/23/97	MW-7	ND	ND	0.008
2/7/97	MW-7	ND	ND	0.006
2/19/97	MW-7	ND	ND	0.007
3/13/97	MW-7	0.022	ND	0.019

DYE TEST RESULTS
(CONT)

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/7/96	MW-8	0.006	ND	0.011
11/11/96	MW-8	0.052	ND	0.011
11/13/96	MW-8	0.026	ND	0.009
11/15/96	MW-8	0.008	ND	0.009
11/19/96	MW-8	0.009	ND	ND
11/26/96	MW-8	0.024	ND	0.029
12/4/96	MW-8	0.007	ND	0.007
12/17/96	MW-8	0.006	ND	0.01
12/31/96	MW-8	0.009	ND	0.01
1/23/97	MW-8	0.011	ND	0.011
2/7/97	MW-8	0.008	ND	0.01
2/19/97	MW-8	0.014	ND	0.037
3/13/97	MW-8	0.03	ND	0.026
11/7/96	MW-10	0.007	ND	0.017
11/11/96	MW-10	0.082	ND	0.029
11/13/96	MW-10	0.028	ND	0.022
11/15/96	MW-10	0.006	ND	0.011
11/19/96	MW-10	0.011	ND	ND
11/26/96	MW-10	0.034	ND	0.039
12/4/96	MW-10	0.01	ND	0.009
12/17/96	MW-10	0.015	ND	0.009
12/31/96	MW-10	0.015	ND	0.011
1/23/97	MW-10	0.015	ND	0.006
2/7/97	MW-10	ND	0.01	ND
2/19/97	MW-10	0.009	ND	0.01
3/13/97	MW-10	0.059	ND	0.081

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/7/96	MW-11	0.006	ND	0.012
11/11/96	MW-11	0.036	ND	0.009
11/13/96	MW-11	0.009	ND	0.01
11/15/96	MW-11	0.006	ND	0.007
11/19/96	MW-11	0.009	ND	0.007
11/26/96	MW-11	0.016	ND	0.019
12/4/96	MW-11	0.006	ND	0.007
12/17/96	MW-11	ND	ND	ND
12/31/96	MW-11	0.009	ND	0.011
1/23/97	MW-11	0.007	ND	0.013
2/7/97	MW-11	ND	ND	0.007
2/19/97	MW-11	0.006	ND	ND
3/13/97	MW-11	0.024	ND	0.026
11/6/96	PZ-2	0.025	ND	0.01
11/11/96	PZ-2	0.577	ND	ND
11/13/96	PZ-2	0.261	ND	0.007
11/15/96	PZ-2	0.213	ND	0.006
11/19/96	PZ-2	0.055	ND	ND
11/26/96	PZ-2	0.3	ND	0.023
12/4/96	PZ-2	0.118	ND	ND
12/17/96	PZ-2	0.052	ND	0.02
12/31/96	PZ-2	3.551	ND	ND
1/23/97	PZ-2	0.199	ND	0.009
2/7/97	PZ-2	0.011	ND	0.019
2/19/97	PZ-2	0.038	ND	0.018
3/13/97	PZ-2	6.093	ND	ND
11/11/96	PZ-2 DUP	0.314	ND	ND

DYE TEST RESULTS
(CONT)

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	PZ-3	0.014	ND	0.01
11/11/96	PZ-3	0.018	ND	0.01
11/13/96	PZ-3	0.043	ND	0.01
11/15/96	PZ-3	0.007	ND	0.007
11/19/96	PZ-3	0.013	ND	ND
11/26/96	PZ-3	NS	NS	NS
12/4/96	PZ-3	0.017	ND	0.01
12/17/96	PZ-3	0.015	ND	0.01
12/31/96	PZ-3	0.019	ND	0.011
1/23/97	PZ-3	0.019	ND	0.01
2/7/97	PZ-3	0.009	ND	0.009
2/19/97	PZ-3	ND	0.021	0.013
3/13/97	PZ-3	0.03	ND	0.02
11/6/96	DOM-1	0.418	ND	0.032
11/11/96	DOM-1	3.621	ND	ND
11/13/96	DOM-1	2.139	ND	0.008
11/15/96	DOM-1	2.555	ND	ND
11/19/96	DOM-1	5.191	ND	ND
11/26/96	DOM-1	4.255	ND	ND
12/4/96	DOM-1	7.157	ND	ND
12/17/96	DOM-1	14.83	ND	ND
12/31/96	DOM-1	5.636	ND	ND
1/23/97	DOM-1	5.514	ND	ND
2/7/97	DOM-1	1.525	ND	ND
2/7/97	DOM-1T	0.016	ND	0.025

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	DOM-2	0.004	ND	0.01
11/11/96	DOM-2	0.009	ND	0.015
11/13/96	DOM-2	0.011	ND	0.015
11/15/96	DOM-2	0.007	ND	0.012
11/19/96	DOM-2	0.008	ND	0.011
11/26/96	DOM-2	0.018	ND	0.04
12/4/96	DOM-2	0.01	ND	0.006
12/17/96	DOM-2	0.007	ND	0.016
12/31/96	DOM-2	0.006	ND	0.01
1/23/97	DOM-2	0.016	ND	0.041
2/7/97	DOM-2	ND	0.016	ND
2/19/97	DOM-2	0.015	ND	0.017
11/6/96	SP-1	2.41	ND	ND
11/11/96	SP-1	0.694	ND	ND
11/13/96	SP-1	0.782	ND	ND
11/15/96	SP-1	0.465	ND	ND
11/19/96	SP-1	0.587	ND	ND
11/25/96	SP-1	0.592	ND	ND
12/4/96	SP-1	0.265	ND	ND
12/17/96	SP-1	1.479	ND	ND
12/31/96	SP-1	0.411	ND	ND
1/23/97	SP-1	0.489	0.303	ND
2/19/97	SP-1	0.675	ND	ND
3/13/97	SP-1	0.276	0.108	ND

DYE TEST RESULTS
(CONT)

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	SP-2	0.583	1.374	ND
11/11/96	SP-2	0.048	0.52	ND
11/13/96	SP-2	0.315	ND	0.013
11/15/96	SP-2	0.222	ND	ND
11/19/96	SP-2	0.401	ND	ND
11/26/96	SP-2	0.191	ND	0.021
12/4/96	SP-2	0.181	ND	ND
12/17/96	SP-2	0.644	ND	ND
12/31/96	SP-2	0.005	0.742	ND
1/23/97	SP-2	0.158	0.121	ND
2/7/97	SP-2	0.142	0.028	ND
2/19/97	SP-2	0.217	0.005	ND
3/13/97	SP-2	0.207	0.22	ND
11/6/96	SP-3	0.232	0.095	ND
11/11/96	SP-3	0.518	0.053	ND
11/13/96	SP-3	0.551	ND	0.052
11/15/96	SP-3	0.555	ND	0.014
11/19/96	SP-3	0.483	ND	0.016
11/26/96	SP-3	0.711	ND	0.03
12/4/96	SP-3	0.69	ND	0.017
12/31/96	SP-3	0.354	0.075	0.047
1/23/97	SP-3	0.086	0.81	0.005
2/7/97	SP-3	0.206	0.027	0.094
2/19/97	SP-3	0.509	0.005	0.1
3/13/97	SP-3	0.018	0.212	0.005

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	SP-4	0.014	ND	0.012
11/11/96	SP-4	ND	0.062	ND
11/13/96	SP-4	0.034	ND	0.003
11/15/96	SP-4	0.003	0.007	0.003
11/19/96	SP-4	0.028	0.056	ND
11/26/96	SP-4	0.024	0.034	0.026
12/4/96	SP-4	0.027	ND	ND
12/17/96	SP-4	0.006	0.007	ND
12/31/96	SP-4	0.008	0.009	ND
1/23/97	SP-4	0.007	0.012	ND
2/7/97	SP-4	ND	ND	0.007
2/19/97	SP-4	ND	ND	ND
3/13/97	SP-4	0.074	0.135	ND
11/6/96	SP-5	0.144	ND	ND
11/11/96	SP-5	0.007	0.19	ND
11/13/96	SP-5	0.017	ND	ND
11/15/96	SP-5	0.023	0.014	ND
11/19/96	SP-5	0.068	0.04	ND
11/26/96	SP-5	0.073	ND	0.034
12/4/96	SP-5	0.056	0.11	ND
12/17/96	SP-5	0.018	0.054	ND
12/31/96	SP-5	ND	0.078	ND
1/23/97	SP-5	0.01	0.025	ND
2/7/97	SP-5	0.013	0.022	ND
2/19/97	SP-5	ND	ND	ND
3/13/97	SP-5	0.112	0.207	ND

DYE TEST RESULTS
(CONT)

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/6/96	SP-6	0.004	ND	0.011
11/11/96	SP-6	0.015	0.101	ND
11/13/96	SP-6	ND	0.076	ND
11/15/96	SP-6	0.002	0.01	ND
11/19/96	SP-6	0.066	0.174	ND
11/26/96	SP-6	NS	NS	NS
12/4/96	SP-6	NS	NS	NS
12/17/96	SP-6	0.091	0.205	ND
12/31/96	SP-6	0.049	0.032	ND
1/23/97	SP-6	0.048	0.044	ND
2/7/97	SP-6	ND	0.015	ND
2/19/97	SP-6	ND	ND	ND
3/13/97	SP-6	0.106	0.342	ND
11/6/96	GLASS BG	0.023	0.064	ND
11/11/96	GLASS	0.079	ND	0.052
11/13/96	GLASS	0.146	ND	ND
11/15/96	GLASS	0.059	0.029	ND
11/19/96	GLASS	0.145	0.024	ND
11/26/96	GLASS	0.177	0.048	0.015
12/4/96	GLASS	0.081	0.038	ND
12/17/96	GLASS	0.454	ND	ND
12/31/96	GLASS	0.237	ND	ND
1/23/97	GLASS	0.561	ND	ND
2/7/97	GLASS	0.345	ND	ND
2/19/97	GLASS	0.574	ND	ND
3/13/97	GLASS	0.229	0.191	ND

DATE COLLECTED	WELL NUMBER	FLUORESCEIN PPB	EOSINE PPB	RHODAMINE WT PPB
11/11/96	GLASS D	0.067	ND	0.053
11/15/96	GLASS D	0.125	ND	0.065
11/19/96	GLASS D	0.073	0.021	ND
11/26/96	GLASS D	0.111	0.071	0.009
12/4/96	GLASSD	0.085	0.05	ND
12/17/96	GLASSD	0.484	ND	ND
12/31/96	GLASS D	0.702	ND	0.157
2/7/97	GLASS D	0.353	ND	ND
2/19/97	GLASS D	0.65	ND	ND
3/13/97	GLASS D	0.198	0.122	ND
11/7/96	POND	ND	0.106	ND
11/11/96	UP CLEAR	1.582	ND	0.042
11/13/96	UP CLEAR	0.644	ND	0.056
11/15/96	UP CLEAR	0.632	ND	0.023
11/19/96	UP CLEAR	0.47	ND	0.016
11/26/96	UP CLEAR	0.907	ND	0.041
12/4/96	UP CLEAR	0.533	ND	ND
12/17/96	UP CLEAR	2.243	ND	0.159
12/31/96	UP CLEAR	0.251	ND	ND
1/23/97	UP CLEAR	0.506	ND	0.134
2/7/97	UP CLEAR	0.128	0.358	ND
2/19/97	UP CLEAR	0.363	ND	0.061
3/13/97	UP CLEAR	ND	0.003	0.016

TO BE ADDED AS APPENDIX O

ANALYSIS OF GROUND WATER TRACING

AT THE

SUNRAY SERVICES, INC., TONTITOWN LANDFILL

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March 29, 1997

ANALYSIS OF GROUND WATER TRACING AT THE SUNRAY SERVICES, INC., TONTITOWN LANDFILL

INTRODUCTION

The Arkansas Department of Pollution Control and Ecology mandated that ground water tracing be conducted at the Sunray Services, Inc., Tontitown Landfill in accordance with conditions for permitted expansion. The activities conducted for the three ground water traces using dyes and an analysis of the results are described in detail below.

LOCATION AND GEOLOGY

The Sunray Services, Inc., Tontitown Landfill is located on the Springfield Plateau of the Ozarks Highlands Province in a valley north of Clear Creek of the northwest-central part of Washington County, Arkansas. The landfill is located approximately 3 miles south-southwest of Tontitown, Arkansas in the northwest quadrant of Section 23, Township 17 North, Range 31 West (Figure 1). Topographically, the area consists of a gently undulating uplands surface that is dissected by ephemeral tributaries that lead to the floodplain valleys of Clear and Little Wildcat creeks. The landfill is located at the head of one of the ephemeral stream valleys that empties into Clear Creek.

The site is underlain by a characteristically red residuum soil composed of silty-clay, silt loam, and clay with a high content of chert fragments. Soil types are within the Captina, Nixa, Clarksville, Razort, Elsah, and Johnsburg Soil Series. Underlying the residuum is the Boone Limestone of Mississippian age. The Boone-St. Joe aquifer was first studied in detail in northwest Arkansas and the area around the landfill by Ogden(1980). He found that the aquifer is primarily unconfined with local semi-confinement by dense chert and limestone beds. Short term, single-well aquifer tests performed by Ogden (1980) on two wells located along Clear Creek near the landfill yielded specific capacity values of 0.54 and 0.94 gpm/ft and transmissibility values of 257 and 301 gpd/ft. The detailed potentiometric surface map of the landfill site produced by Genesis Environmental, Inc. (Drawing 8), generally corresponds with the regional water table map constructed by Ogden (1980). A large spring named Glass Spring is located approximately 1 mile south of the landfill along Clear Spring. The potentiometric surface maps suggest that this spring is the most likely candidate to drain most of the site.

GROUND WATER TRACING METHODOLOGY

The standard method of conducting qualitative dye tracing is to use passive activated charcoal packets referred to as "traps" that are capable of absorbing dyes and concentrating them approximately 400 times the levels that would be found in a water sample. The traps are constructed by simply weighing out a certain amount of charcoal and wrapping it in nylon window screen "envelope". Although the same amount of charcoal is placed within each screen envelope, it is important to note that the absorptive capability of each trap is different due to the size, shape, and arrangement of the grains. Crawford and Associates, Inc., provided the traps and performed the analyses.

Prior to injecting the dyes, background traps were placed in monitoring wells, piezometers, and springs in and around the landfill site. Twenty-one of the background traps were left in the water for 7 to 8 days prior to dye injection while two were only in the water for 1 day. Glass Spring and Upper Clear Creek were the two sites with only 1 day of background information. At this point it is also important to note that the concentration of dye found from an elutriated charcoal sample is dependent on the amount of time at which the dissolved dye in water passes through the charcoal. All of the background traps were retrieved and replaced on November 8th, 1996.

Dye injection occurred on November 9th, 1996. Two pounds of fluorescein powder were premixed with water and transported to PZ-1 (see Drawing 2). Injection began at 11:25 am. The dye was then flushed with approximately 150 gallons of water. Then 1 pound of eosine powder was premixed with water and transported to MW-1. Injection began at 12:45 pm followed by flushing with approximately 125 gallons water. Injection of the flush water was slowed by the low permeability of the well compared to PZ-1. Finally, 1 pound of rhodamine WT liquid dye was injected in MW-5 at 2:00 pm followed by approximately 150 gallons of water. Injection of the flush water was again slowed by low aquifer permeability. MW-5 did accept the water at a faster rate than MW-1.

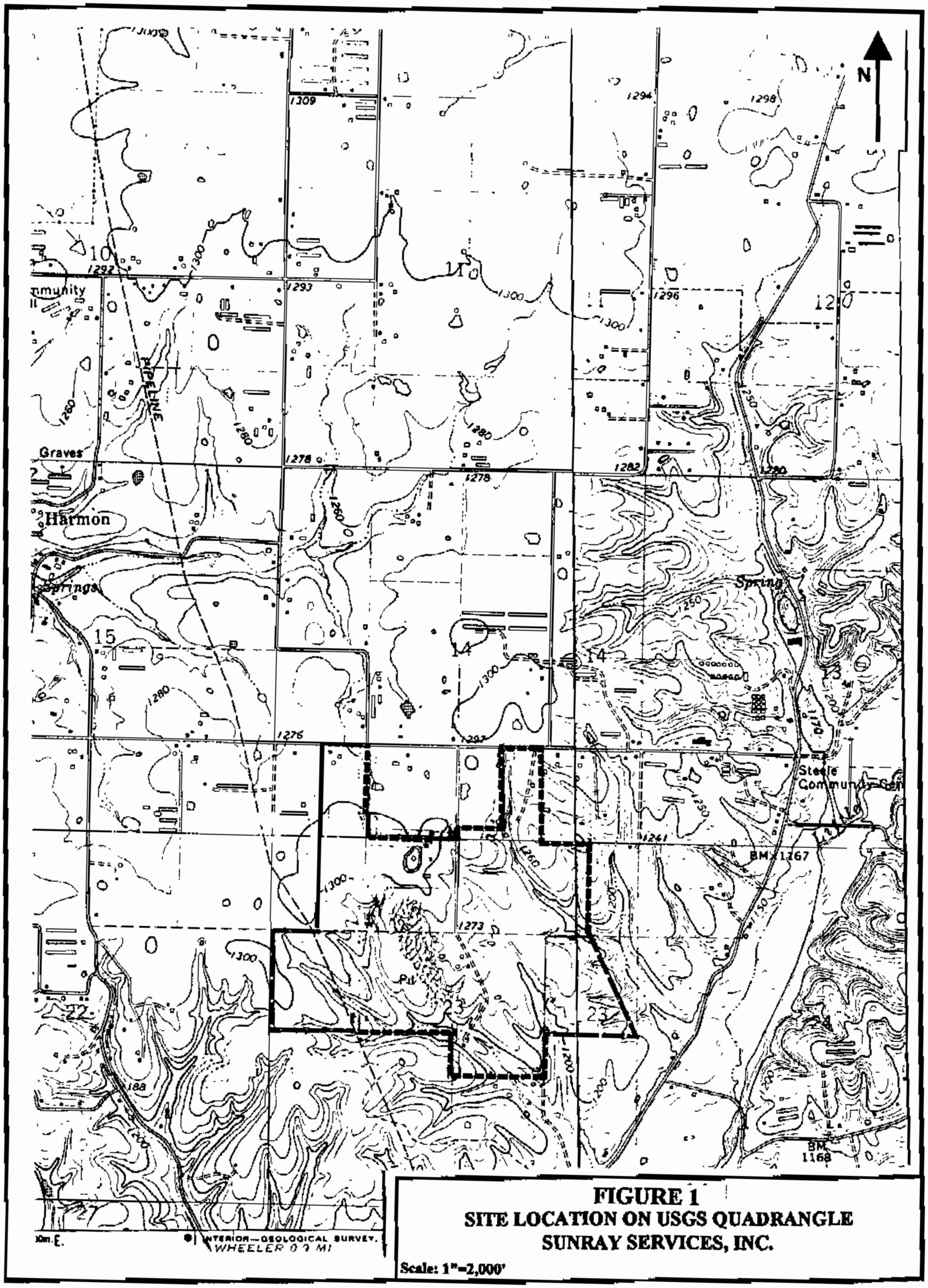

FIGURE 1
SITE LOCATION ON USGS QUADRANGLE
SUNRAY SERVICES, INC.

Scale: 1"=2,000'

RESULTS

Introduction

At only two sites (PZ-2 and DOM-1) did the concentration of any dye from the elutriated charcoal traps exceed 1 part per billion. Using the laboratory's 400 to 1 estimate of the ability of a charcoal trap to concentration dye suggests that in the other 21 sampling sites, the concentration of dye in the water probably did not exceed 5 to 10 parts per trillion. Dye concentrations from the charcoal traps fluctuated considerably in the part per trillion range at the 21 sites. This is certainly to be expected in a landfill environment where sources for the dyes likely occur with fluctuations in concentrations being related to recharge events. Therefore, it is extremely difficult to pinpoint a low concentration positive when background levels vary so radically.

Water sampling of the three injection wells on March 12, 1997 (4 months after injection) provided the following results: 1)Fluorescein-504 ppb, 2)Eosine-40 ppb, and 3)Rhodamine WT-589 ppb. These results indicate that much of the dyes have moved into the aquifer from the injection points, but that permeability is not high otherwise concentrations of this magnitude would not be seen.

Interpretation of the Fluorescein Trace

Two pounds of fluorescein was injected into PZ-1 on 11/9/96. A positive indication occurred in DOM-1, located approximately 500 ft to the north, beginning just two days later, with concentrations increasing until late December. This result indicates flow opposite to that predicted by the potentiometric surface map at a rate of approximately 250 per day. A positive for fluorescein dye also occurred in PZ-2, located approximately 150 feet southeast of the injection point, but not until 12/31/96. This indicates an approximately flow velocity of 3 feet per day. The results also suggest that PZ-1 is located on the ground water divide since dye was seen in wells in nearly opposite directions. The difference in flow velocities indicates that hydraulic conductivity of the aquifer is significantly less to the southeast and/or a majority of ground water flow is to the north. Many of the other monitoring stations such as MW-4, MW-7, and MW-10 which are located at the toe of the landfill showed an increase in fluorescein on 11/11/96. Of all the tracing agents used, a source of fluorescein probably exists buried in the landfill. Therefore, the changes are likely a result of background fluctuations related to the landfill. If the dye concentrations at the distant sites were a result of dye injection, a systematic increase of concentration would be seen at monitoring stations closer to the injection point. Also, note that Glass Spring shows an increase in fluorescein soon after injection, but the concentrations are low and nearly mimic MW-4, MW-7, and MW-10. It is not until 12/17/96 that Glass Spring shows a repeated increase over background. This is not considered a positive because Glass Spring is reported by the owner to be influenced by Clear Creek. At times, Glass Spring is actually inundated by Clear Creek. The fluorescein levels in the upper Clear Creek samples are very similiar to Glass Spring, strongly suggesting that the creek water is mixed with the spring's water.

Interpretation of the Eosine Trace

One pound of eosine dye was injected into MW-1 on 11/9/96. A positive indication for dye appears to have occurred at MW-1R on 2/19/97 yielding an approximate flow velocity of 4 feet per day in a northwest direction. This is contrary to the flow direction indicated by the potentiometric surface map, but corresponds to the results of the fluorescein trace. It is also possible that eosine reached MW-2R on 2/19/97. MW-2R is located nearly due west of the injection point, but at approximately the same distance as MW-1R. This would yield a similar flow velocity. Since no eosine was found in PZ-2, it indicates that MW-1 is on the northern side of the ground water divide.

Interpretation of the Rhodamine WT Trace

One pound of rhodamine WT was injected into MW-5 on 11/9/96. It is my opinion that no sampling station indicates a positive for the dye. This is likely related to the amount of dye injected. Rhodamine WT is only approximately one quarter as strong as fluorescein or eosine. Based on the low levels of eosine and fluorescein that were detected, it would have required at least 4 pounds of rhodamine to achieve a positive result.

CONCLUSIONS

The ground water tracing activities at the Sunray Services, Inc., Tontitown Landfill have produced important results that should prove helpful in the permit expansion and delineating an appropriate monitoring program. The results of the dye traces demonstrate that the aquifer is very anisotropic and heterogeneous, thus producing differing flow velocities. For a carbonate aquifer, the flow rates are quite low. The very low concentrations of dye, particularly fluorescein, demonstrate that the fractures and bedding planes contain significant amounts of clay that is highly absorptive. The dye trace results further indicate that a ground water divide exists on the landfill property that has not been depicted by the potentiometric surface map.

CITED REFERENCE

Ogden, A.E., 1980, Hydrogeologic and geochemical investigation of the Boone-St. Joe Limestone Aquifer in Benton County, Arkansas: Arkansas Water Resources Research Center, Publ. No. 68, 2 plates, 168 p.