TENNESSEE VALLEY AUTHORITY
River System Operations & Environment
Research & Technology Applications
Environmental Engineering Services - East

KINGSTON FOSSIL PLANT – PENINSULA SITE

HYDROGEOLOGIC EVALUATION OF COAL-COMBUSTION BYPRODUCT DISPOSAL FACILITY

WR2005-1-36-133

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TABLE OF CONTENTS

List of Figures

List of Tables

EXECUTIVE SUMMARY

The construction of a new of Class II coal-combustion byproduct (CCB) disposal facility proposed at the Kingston Fossil Plant (KIF) may occur in two separate phases. Both phases would involve disposal of gypsum derived from flue-gas desulfurization (FGD). Phase 1 would be constructed pending successfully marketing of the FGD derived gypsum. The footprint for Phase 1 includes an area of approximately 35 acres. If efforts to market the gypsum are unsuccessful, the disposal facility will be expanded laterally under Phase 2. Phase 2 includes additional area adjacent to the site and encompasses approximately 80 acres (total for both Phase 1 and 2). If approved, approximately 1 million cubic yards (CY) of gypsum is tentatively scheduled to be deposited in Phase 1 between 2009 and 2029. If the facility is expanded to include Phase 2, approximately 8 million CY of gypsum would be deposited in the facility between 2009 and 2029. Estimates of FGD wastes for disposal are approximate, and depend on the sulfur content of coal utilized by the plant, as well as TVA's ability to successfully market the FGD derived gypsum for other uses. Current design plans for the disposal facility include a low-permeability liner and under-drain system. Hydrogeologic evaluations of the proposed facility were performed to examine its suitability relative to the appropriate standards of Tennessee Department of Environment and Conservation (TDEC) Rule 1200-1-7. Evaluations addressed effects of proposed disposal facilities on local groundwater and surface water resources.

Hydrogeologic data used to support the site evaluation were derived from recent geotechnical investigations at the site conducted by MACTEC Engineering and Consulting, Inc., from single-well aquifer testing, and from several previous site investigations. Recent investigations included 26 geotechnical soil borings, bedrock coring at 14 locations, and installation of 13 wells for the purposes of single-well aquifer testing and to supplement water level data provided by five existing piezometers. Cone penetrometer surveys were performed at 10 locations and 55 Geoprobe borings were installed within the proposed disposal site to supplement boring data.

The proposed disposal site is topographically bounded by a relatively high ridge along the northeast margin and hydraulically by the Clinch River along the S-SE. A mantle of predominantly residual soil resides above bedrock. Soil thickness is highly variable, ranging from 8.5 to 120 ft and averaging 40.5 ft based on all available data (139 holes) within the confines of the proposed disposal area. Residuum primarily consists of clay and silt with variable chert gravel content. Silty alluvial soils (clayey to sandy silt) were encountered along a small low-lying area on the western margin of the site.

The Knox Group comprises bedrock beneath the proposed disposal area, and the general variation in lithology of the Knox is from massive, crystalline, very cherty dolomite at the base to generally less massively bedded, dense to fine crystalline, less cherty dolomite at the top. Core samples of the Knox bedrock at the site (Appendix C) exhibit slight to highly fractured conditions. Most cavities and joints were also observed to be completely or partially filled with clays or sands. An exception was at NB-66 where open cavities were observed. Cavity thicknesses ranged from 0.4 to 8.0 ft. Cavities of measurable thickness were observed at half of the corehole locations.

Groundwater movement at the site generally follows topography with groundwater flowing southeasterly from the site ridge-line toward the Clinch River. All groundwater originating on, or flowing beneath, the proposed disposal site ultimately discharges to the Clinch River without traversing private property.

Hydrogeologic conditions at the proposed disposal site appear to satisfy geologic and hydrologic standards for Class II disposal facilities. Key findings and recommendations are summarized as follows:

- A survey of water use in June 2005 indicates that there are no surface or groundwater supplies located within a one-mile radius of the site. Furthermore, considering that the site is hydraulically bounded on virtually all sides, there is no potential for offsite impacts to residential or municipal groundwater supplies. The facility poses no risk to existing or future groundwater users since there are no existing groundwater wells downgradient of the proposed facility, and there is no potential for future development of such wells since all downgradient property between the disposal site and surface water boundaries lies within the plant reservation.

- There is no evidence of Holocene-age faulting within the 200-ft facility exclusion zone. Although topographic expressions of dolines are exhibited at the site, these features do not possess open throats or avenues for reception of incipient recharge. Rather, the dolines are thickly mantled by soil thicknesses ranging from about 35 to 75 ft. Visual and laboratory classifications of these soils indicated that they are of residual origin except in the area of NB-21 and NB-44 (site pond) where alluvial deposition has occurred. There were no voids detected immediately above bedrock that would indicate stoping of soil into the deeper bedrock system.

- Two small areas within the proposed facility boundary reside within the 100-yr flood stage of the Clinch River and the natural geologic buffer zone within these areas is lacking. However, the proposed facility design includes plans for filling of these areas with suitable borrow soil. Furthermore, the current facility plan includes a bottom liner residing above the seasonal high groundwater elevation and an under-drain system to intercept leachate.

- Groundwater monitoring for potential CCB leachate contaminants is anticipated to include several discrete locations within the geologic buffer zone immediately beneath the landfill liner. Although design of the complete groundwater monitoring network is dependent on the features of the final landfill design, it is expected that monitoring ports beneath the landfill will be situated at centroid and peripheral locations with horizontal conduit runs to sampling ports. Perimeter monitoring wells will be installed at critical locations to complement those monitoring locations beneath the landfill. Upgradient wells are currently being installed at higher elevations of the site (ridge-line) that should serve to gage background groundwater quality. The final groundwater monitoring plan will be detailed in the facility operations plan.

1. INTRODUCTION

1.1 Background

The Kingston Fossil Plant (KIF) is located at the base of a peninsula formed by the Clinch and Emory River embayments of Watts Bar Lake. Construction of KIF began in 1951 and commercial operation began in 1955. Land acquisition for KIF included approximately 550 acres east of the current plant operational area, commonly referred to as the KIF Peninsula site. The area was originally devoted to agricultural and residential use. These cultivated fields are currently used by the Tennessee Wildlife Resources Agency (TWRA) to support an onsite wildlife management program (i.e., hunting).

The proposed coal-combustion byproduct (CCB) facility at TVA's Kingston Fossil Plant is located on a peninsula landform at the confluence of the Clinch and Emory Rivers in Roane County, Tennessee (Figure 1-1). The Emory River enters the Clinch River at Clinch River Mile (CRM) 4.36 along the eastern margin of the peninsula. Existing ground surface across the proposed disposal site ranges from approximately elevation 735 to 860 ft-msl, and the 100-year flood stage elevation is 747.6 ft. Modern day floods near the mouth of the Clinch River (CRM 0.7) suggest that the highest modern (1903) flood stage was near 746 ft-msl. Potential fluvial deposition associated with ancestral flooding has not been mapped at the KIF Peninsula site.

The CCB disposal facility proposed at KIF may occur in two separate phases. Both phases would involve disposal of gypsum derived from flue-gas desulfurization (FGD). Phase 1 would be constructed pending successfully marketing of the FGD derived gypsum. The footprint for Phase 1 includes an area of approximately 35 acres. If efforts to market the gypsum are unsuccessful, the disposal facility will be expanded laterally under Phase 2. Phase 2 includes additional area adjacent to the site and encompasses approximately 80 acres (total for both Phase 1 and 2). If approved, approximately 1 million CY of gypsum is tentatively scheduled to be deposited in Phase 1 between 2009 and 2029. If the facility is expanded to include Phase 2, approximately 8 million CY of gypsum would be deposited in the facility between 2009 and 2029. Estimates of FGD wastes for disposal are approximate, and depend on the sulfur content of coal utilized by the plant, as well as TVA's ability to successfully market the FGD derived gypsum for other uses. Current design plans for the disposal facility include a low-permeability liner and under-drain system. Hydrogeologic evaluations of the proposed facility were performed to examine its suitability relative to the appropriate standards of Tennessee Department of Environment and Conservation (TDEC) Rule 1200-1-7. Evaluations addressed effects of proposed disposal facilities on local groundwater and surface water resources.

Figure 1-1. Site Map Showing Location of Proposed CCB Disposal Facility

1.2 Purpose and Scope

The objective of this report is to evaluate the suitability of the proposed CCB disposal facility relative to the appropriate standards of TDEC Rule 1200-1-7. Hydrogeologic data used to support the site evaluation were derived from recent geotechnical investigations at the site conducted by MACTEC Engineering and Consulting, Inc. (MACTEC, 2005), from single-well aquifer testing (Section 2.4), and from several previous site investigations (Section 1.3). A survey of private water wells and public water supplies within one mile of the site was conducted to establish local water use.

Results of recent geotechnical investigations supporting disposal facility design and the hydrogeological evaluation are reported in MACTEC (2005). Recent investigations included 26 geotechnical soil borings (MACTEC, 2005) drilled to refusal at locations in and around the proposed site to characterize overburden stratigraphy and to acquire samples for laboratory testing. Seven additional off-set borings were installed at primary borehole locations for the purposes of sample collection. Sample collection and standard penetration resistance testing was performed at approximately 5-ft intervals. Bedrock coring was conducted at 14 boring locations. Thirteen wells were installed at the site for the purposes of single-well aquifer testing and to supplement water level data provided by five existing piezometers. Nine of the new wells were developed in soil and four of the wells were developed in bedrock. Cone penetrometer surveys were performed at 10 locations (Figure 1-2) within the proposed disposal site to supplement boring data. Complete descriptions of sampling and testing procedures used in these investigations are presented in MACTEC (2005). Geoprobe measurements were performed by TVA at 55 locations with probing extending from ground surface to refusal to supplement top of rock data.

1.3 Previous Site Investigations

Three previous investigations at the KIF site have included the collection of subsurface data at the proposed disposal site. The first was a siting study by Benziger and Kellberg (1951). This study was primarily focused within the area currently occupied by the main plant site. However, five of these borings (V-8+00, V-6+00, V-4+00, T-6+00, R-6+00; Figure 1-3) were installed on the western margin of the proposed disposal area. These boreholes were primarily used to identify top of bedrock and to visually classify the character of bedrock (e.g., weathering). Detailed boring logs were not produced.

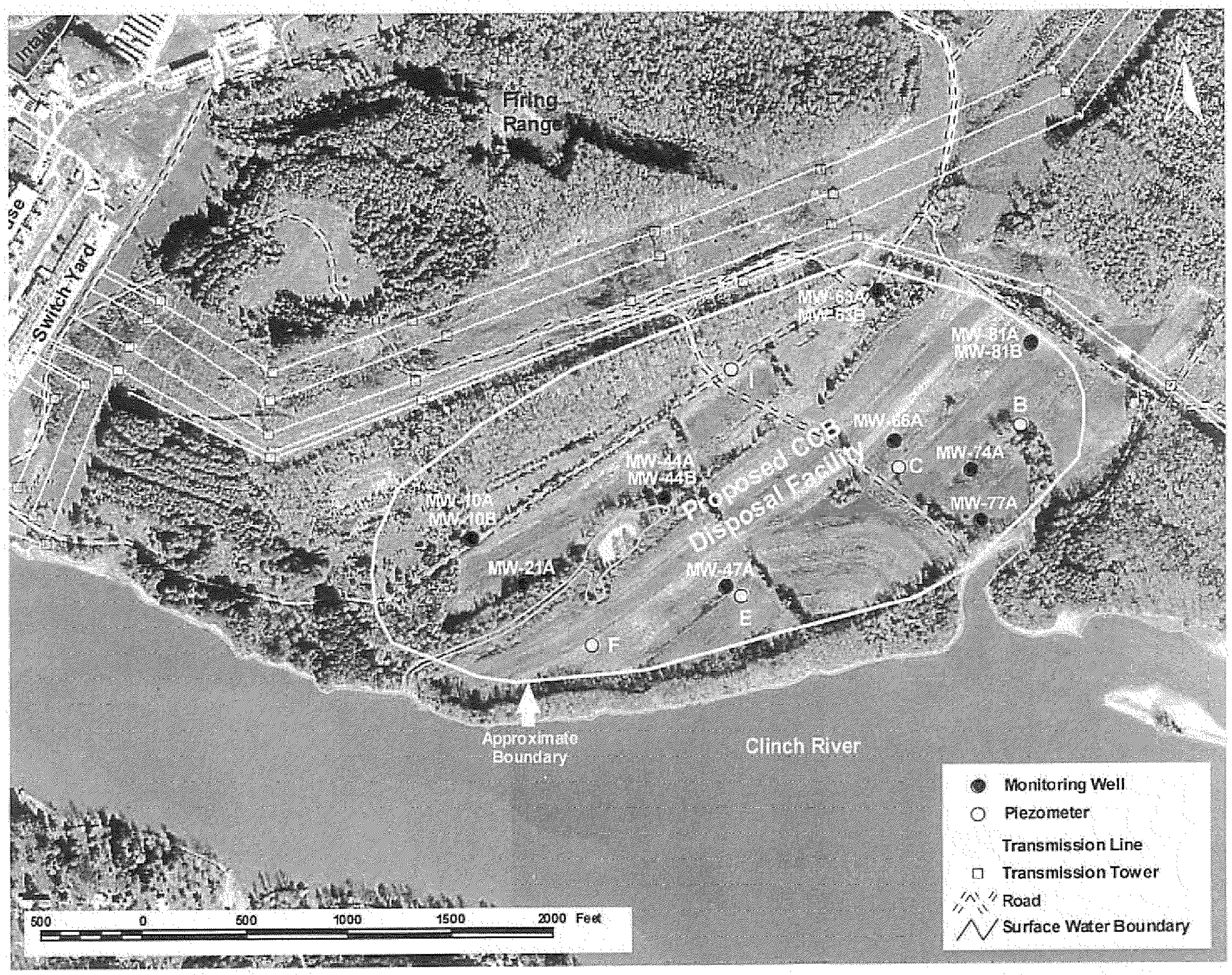

Figure 1-2. Site Map Showing Locations of Wells and Piezometers. "A" and "B" Wells are Developed in Soil and Bedrock, Respectively

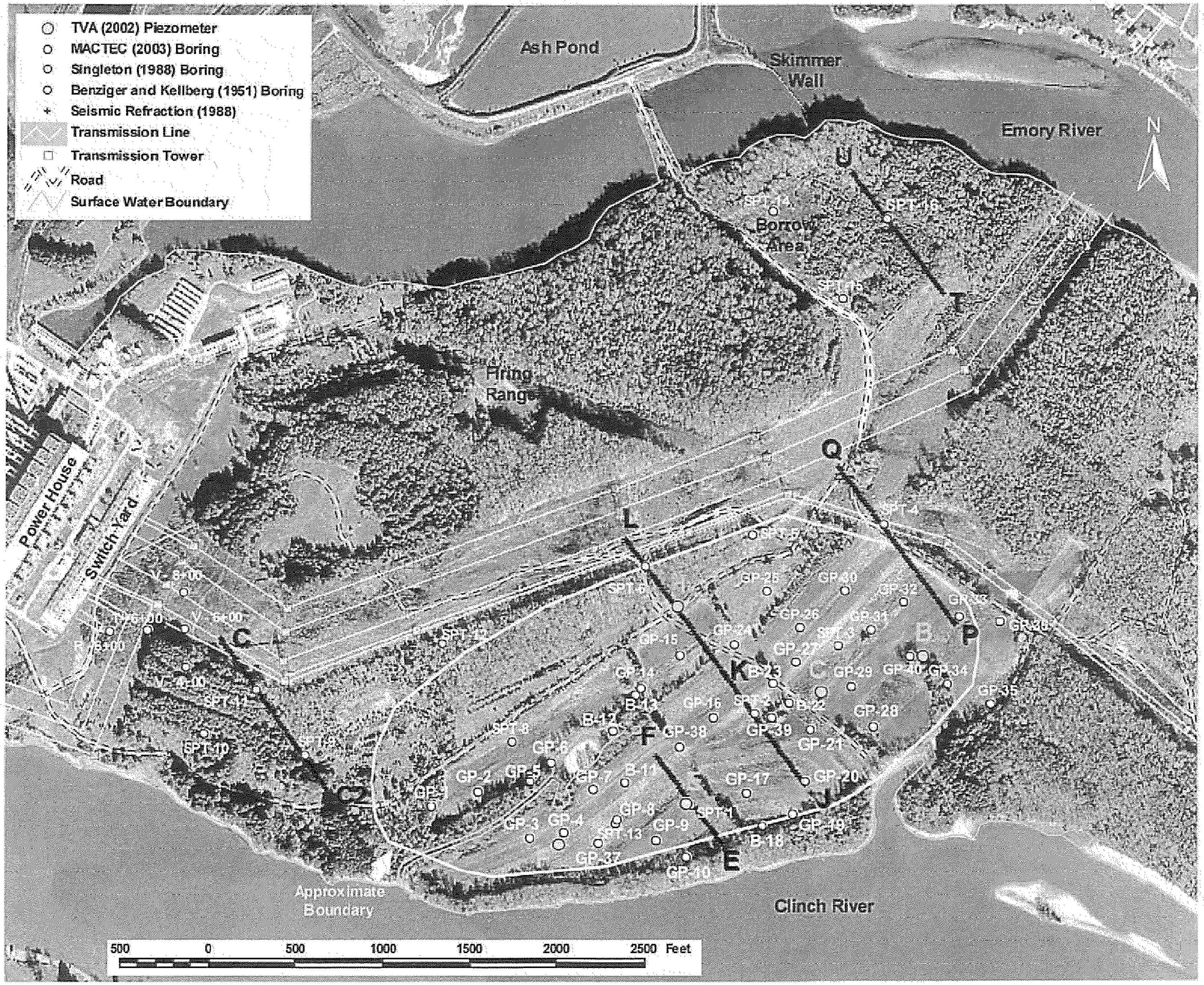

Figure 1-3. Site Map Showing Locations of Historical Subsurface Data

Carpenter and Bohac (1988) performed a subsurface investigation at the KIF Peninsula site in 1988. As part of this investigation, fifteen exploratory borings were drilled to top of bedrock (SPT-1 to SPT-16; except SPT-7) by Singleton Material Engineering Laboratory (Singleton, 1988) and seismic refraction surveys were completed along four transects (Figure 1-3) to define variations in top of bedrock (Figure 1-3). Overburden thickness ranged from 10.3 to 52 feet and averaged 30 feet based on SPT borings. Carpenter and Bohac (1988) suggested that medium to fat clays are the predominant soil type at the Peninsula site, but ranged from fat to silty with colors from dark brown to light yellow. Boring logs from Singleton (1988) also identified alluvial layers of sandy clay to clean, fine sand at borings SPT-2 and SPT-3. Lab permeameter testing of two silty-sandy-clay soil samples provided vertical hydraulic conductivity estimates of 3.5E-08 and 8.6E-08 cm/s, and porosity values of 0.41 and 0.44. These values are typical of residuum. X-ray powder diffraction and polarized-light microscopy were used to determine primary soil minerals of the two silty-sandy-clay soil samples. Table 1-1 depicts the mineral composition of test soils.

Table 1-1. Mineral Composition of Soils (from Carpenter and Bohac, 1988)

	Fraction (%)	
Mineral	**Sample 1**	**Sample 2**
Quartz	70-80	50-60
Kaolinite (primary) Halloysite (secondary)	20-30	40-50
Goethite	5-10	5-10

Velasco and Bohac (1991) performed a site-wide assessment of groundwater conditions at the KIF main plant site. Their investigations included an evaluation of the potential of geochemical attenuation of ash-related contaminants. Mineralogical analyses were conducted on 20 soil samples collected adjacent to main plant site monitoring wells 1 through 6 (alluvium and Consauga residuum). X-ray diffraction analysis indicated that clay minerals predominantly consisted of kaolinite and illite with trace amounts of other minerals, all of which tend to adsorb cations present in groundwater. Iron oxides were detected at contents of 0.33 to 0.60 percent. Soil cation exchange capacities ranging from 6.6 to 34 meq/100 g were reported.

Subsequent boring work by TVA and MACTEC (2003) included the installation of 31 Geoprobe borings to refusal (GP-1 to GP-40), six auger borings to refusal (B-11, B12, B-13, B-18, B22, and B-23), and two bedrock corings into the upper 30 feet of bedrock (B-22 and B-23; Figure 1-3). These boring logs are provided in Appendix A. MACTEC (2003) describes the subsoil encountered in all test borings for the full depth as "residuum." However, the log for boring B-11 indicates the presence of fine subrounded gravel. These observations, coupled with those of Singleton (1988), suggest a possibility that certain portions of the site may have been subjected to fluvial deposition. However, these data may simply represent weathering products (e.g., sandy facies) associated with the Knox.

Groundwater level monitoring at the site has been conducted at piezometers B, C, E, F, and I (Figure 1-2) since January 13, 2003. These 1-inch diameter piezometers (Table 1-2) were installed in October/November 2002 using Geoprobe methods and extend to top of bedrock. Piezometer diagrams are included in Appendix B. Continuous water level measurements were sporadically collected at piezometers B, C, E, and F using pressure transducers and dataloggers. Manual water level measurements at site piezometers have been obtained at approximately monthly intervals from January 2003 to 2004 and from September 2004 to present.

Table 1-2. Wells and Piezometers

Piezometer/ Well	NAD83 (ft)		*TOG Ele (ft-msl)*	*TOC Ele (ft-msl)*	*Total Depth (ft)*	*Bottom Ele (ft-msl)*	*Screen Length (ft)*	*Screened Interval (ft-msl)*	
	Easting	Northing						*from*	*to*
B	2412581.3	571957.0	744.0	746.12	22.5	721.5	10.0	721.5	731.5
C	2411991.9	571753.8	761.9	763.84	41.0	720.9	15.0	720.9	735.9
E	2411221.4	571123.2	764.6	767.53	34.7	729.9	10.0	729.9	739.9
F	2410489.4	570887.5	749.6	752.75	56.0	693.6	35.0	693.6	728.6
I	2411169.8	572238.8	786.7	789.61	52.0	734.7	15.0	734.7	749.7
MW-10A	2409890.9	571411.8	768.2	771.87	56.2	712.0	34.4	747.5	713.1
MW-10B	2409897.0	571414.0	768.2	771.61	72.4	695.8	24.6	722.6	698.0
MW-21A	2410148.7	571188.9	757.7	762.34	50.4	707.3	29.6	739.2	709.6
MW-44A	2410846.1	571607.4	742.4	745.00	40.5	701.9	34.5	739.4	704.9
MW-44B	2410844.4	571612.4	742.7	744.04	104.2	638.5	49.5	693.6	644.1
MW-47A	2411145.9	571170.7	762.9	766.38	44.4	718.5	19.6	740.4	720.8
MW-63A	2411894.3	572623.7	780.2	781.96	48.8	731.4	29.4	763.1	733.7
MW-63B	2411865.5	572611.6	780.9	784.94	82.3	698.6	28.5	728.5	700.0
MW-66A	2411965.8	571887.2	752.9	756.39	38.8	714.1	24.5	740.4	715.9
MW-74A	2412338.3	571743.6	752.0	756.01	59.3	692.7	44.4	739.9	695.5
MW-77A	2412389.8	571490.6	749.9	754.37	35.4	714.5	19.6	738.1	718.5
MW-81A	2412640.2	572363.5	763.4	765.25	39.8	723.6	14.4	742.4	728.0
MW-81B	2412636.5	572358.4	762.9	764.27	61.1	703.2	24.4	729.4	705.0

TOG = Top of Ground; TOC = Top of Casing

1.4 Current Site Investigation

Results of recent geotechnical investigations supporting disposal facility design and the hydrogeological evaluation are reported in MACTEC (2005; Appendix C). Thirteen monitoring wells (Table 1-2) were installed at the site for the purposes of single-well aquifer testing and to supplement water level data provided by five existing piezometers (Figure 1-2). Nine of the new wells ("A" wells) were developed in soil and the upper 1.5 to 5 ft of bedrock (epikarst zone) and four of the wells were developed in bedrock ("B" wells). Monitoring wells consisted of 2-inch diameter, schedule 40 PVC pipe with double-density, 0.010-inch slotted screens. The screened intervals of overburden wells spanned the approximate groundwater depths to top of bedrock. The screened intervals of bedrock wells spanned the entire corehole thickness of bedrock (30 to 60 ft). All monitoring wells were developed in a cyclic fashion using overpumping/backwashing methods in conjunction with manual surge-blocking. Monitoring well installation logs are provided in Appendix C.

Investigations also included 24 geotechnical soil borings drilled to refusal at locations in and around the proposed site to characterize overburden stratigraphy and to acquire samples for laboratory testing (Figures 1-4 and 2-1). Boring logs and summaries are provided in Appendix C. Sample collection and standard penetration resistance testing was performed at approximately 5-ft intervals. Bedrock coring was conducted at 12 borings locations. Cone penetrometer surveys were performed at 10 locations (Figures 1-4 and 2-1) within the proposed disposal site to supplement boring data. Complete descriptions of sampling and testing procedures used in these investigations are presented in Appendix C (MACTEC, 2005). Geoprobe measurements were performed by TVA at 55 locations with probing extending from ground surface to refusal to supplement top of rock data (Figures 1-4 and 2-1). Overall, new and existing boring locations at the site are within the 200-ft spacing recommended by TDEC in all areas except where extreme topography (e.g., steep slopes) prevented access (Figure 2.1). Likewise, these borings were advanced to bedrock at all locations. Fourteen bedrock corings (12 new and 2 existing) were installed at the site to meet TDEC guidance of one boring per 10 acres for minimum drilling at depth.

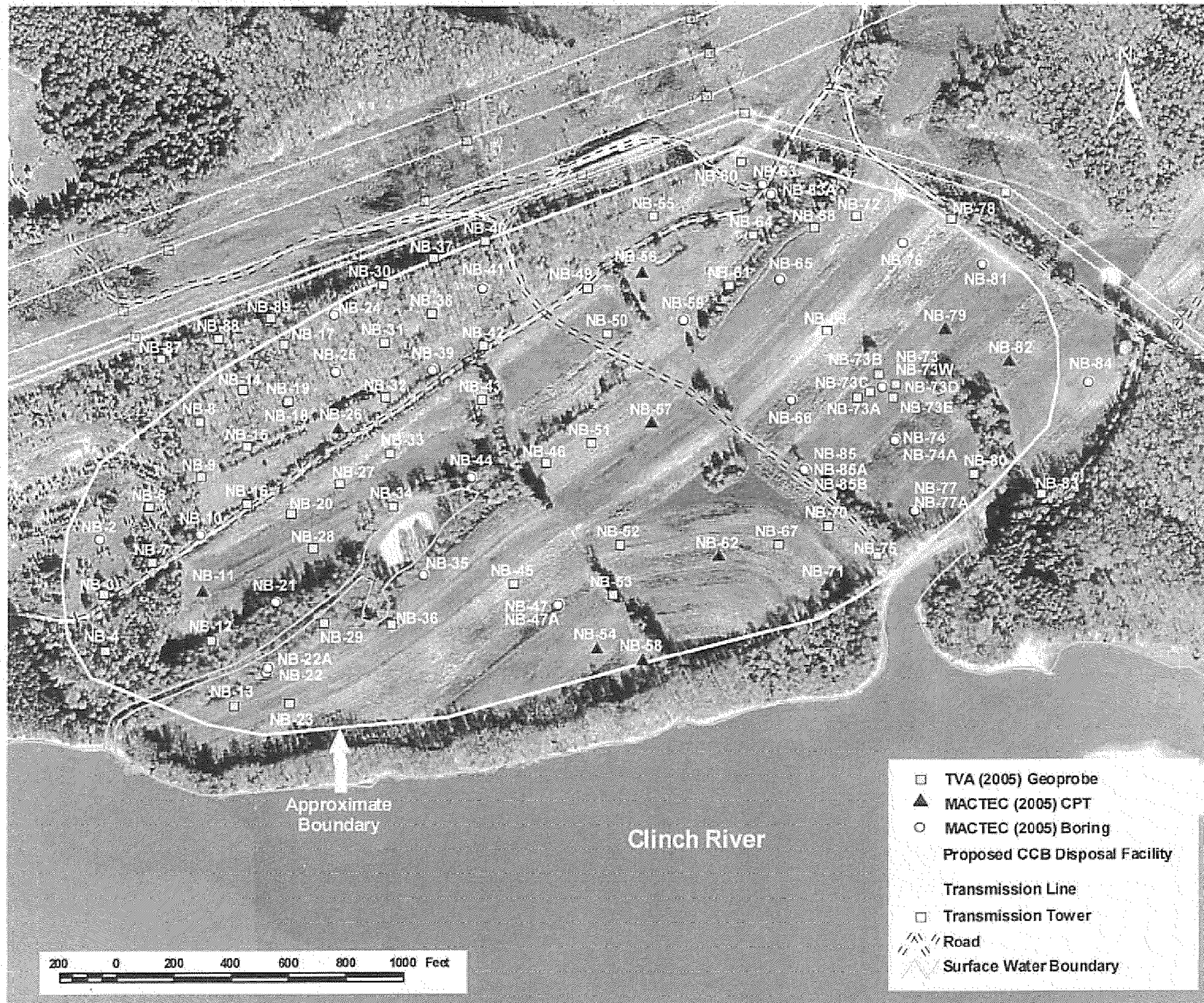

Figure 1-4. Site Map Showing Locations of Subsurface Data from Recent Geotechnical Investigation

2. HYDROGEOLOGIC CONDITIONS

2.1 Topography and Karst Features

The plant site resides within the Valley and Ridge physiographic province, a region characterized by narrow, subparallel ridges and valleys trending northeast-southwest. As shown in Figure 2-1, the proposed disposal site is topographically bounded by a relatively high ridge along the northeast margin and hydraulically by the Clinch River along the S-SE. Existing ground surface exceeds 860 ft-msl along the northeast margin of the proposed disposal area footprint and topographic lows are near 735 ft-msl within the pond and drainage channel on the southwest portion of the site. A 1924 pre-impoundment map (USACE, 1924) of the Clinch and Emory Rivers (Figure 2-2) indicates that erosion of the KIF Peninsula site primarily occurred along eastern margins of the site (i.e., along the Emory River) and along southeastern parts of the site just downstream of the confluence of the Clinch and Emory Rivers.

Within site bedrock (Knox Group), some rock fractures are enlarged by solution activity. As solution developed in the upgradient direction of the water table, some conduits, as they intercepted smaller solution features, became dominant flow channels. Relative to other bedrock types in the region, the Knox and the adjacent Maynardville Limestone possess more highly developed and aerially extensive cavity systems. With regard to the bedrock flow system, structural features are of importance because they host and guide almost all parts of the solutioned fracture networks. Overall, it is these entities where rock is absent that determine much of the variety of form and behavior that occurs in the bedrock groundwater flow system.

Figure 2-3 shows topography of the KIF Peninsula site based on pre-disturbed mapping by TVA in 1950. Seven dolines are highlighted for easy viewing. The largest doline (near the centroid of the proposed footprint) was subsequently drained to the Clinch River via a large canal (presumably for mosquito control). This doline currently impounds water at shallow depths during wet weather and the channel directs drainage from the pond to the reservoir during this time. During periods when the reservoir is at summer pool (near 741.0 ft-msl), the pond and channel remain filled.

A visual inspection of all dolines within the proposed disposal site indicates no direct drainage to bedrock; rather, the dolines are all closed. This is supported by existing boring data that indicates a minimum of 35 ft of overburden material between the base of dolines and the top of bedrock. There were no voids detected immediately above bedrock that would indicate stoping of soil into the deeper bedrock system. Due to the absence of natural karst features (e.g., sinkholes, sinking streams, and springs) directly integrated into the subsurface, dye tracing at the site would prove to be difficult since it would require dye injection and monitoring via groundwater wells alone. Further discussions related to karst features appear in ensuing sections of this report.

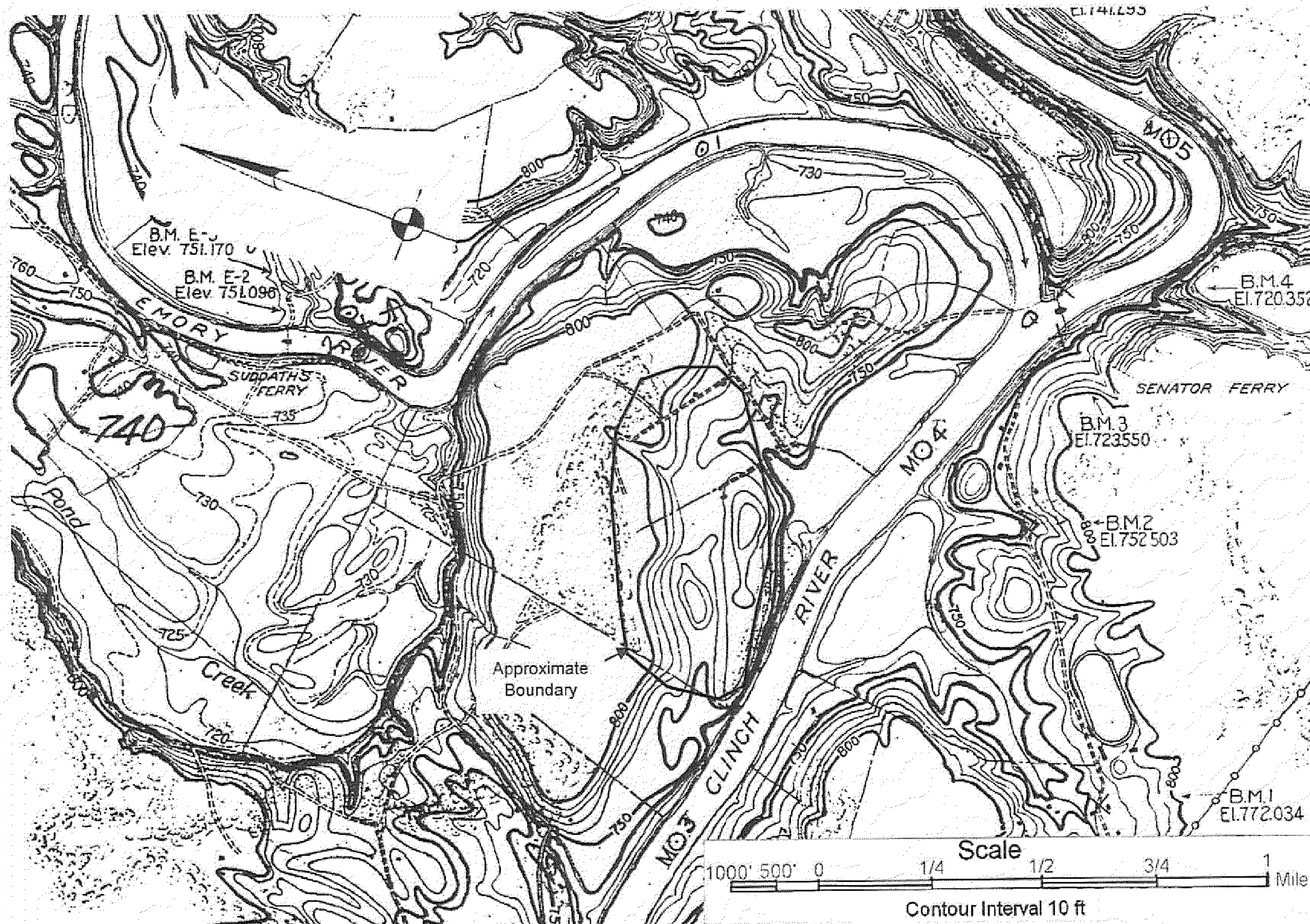

Figure 2-2. 1924 Pre-Impoundment Topography (adapted from USACE, 1924)

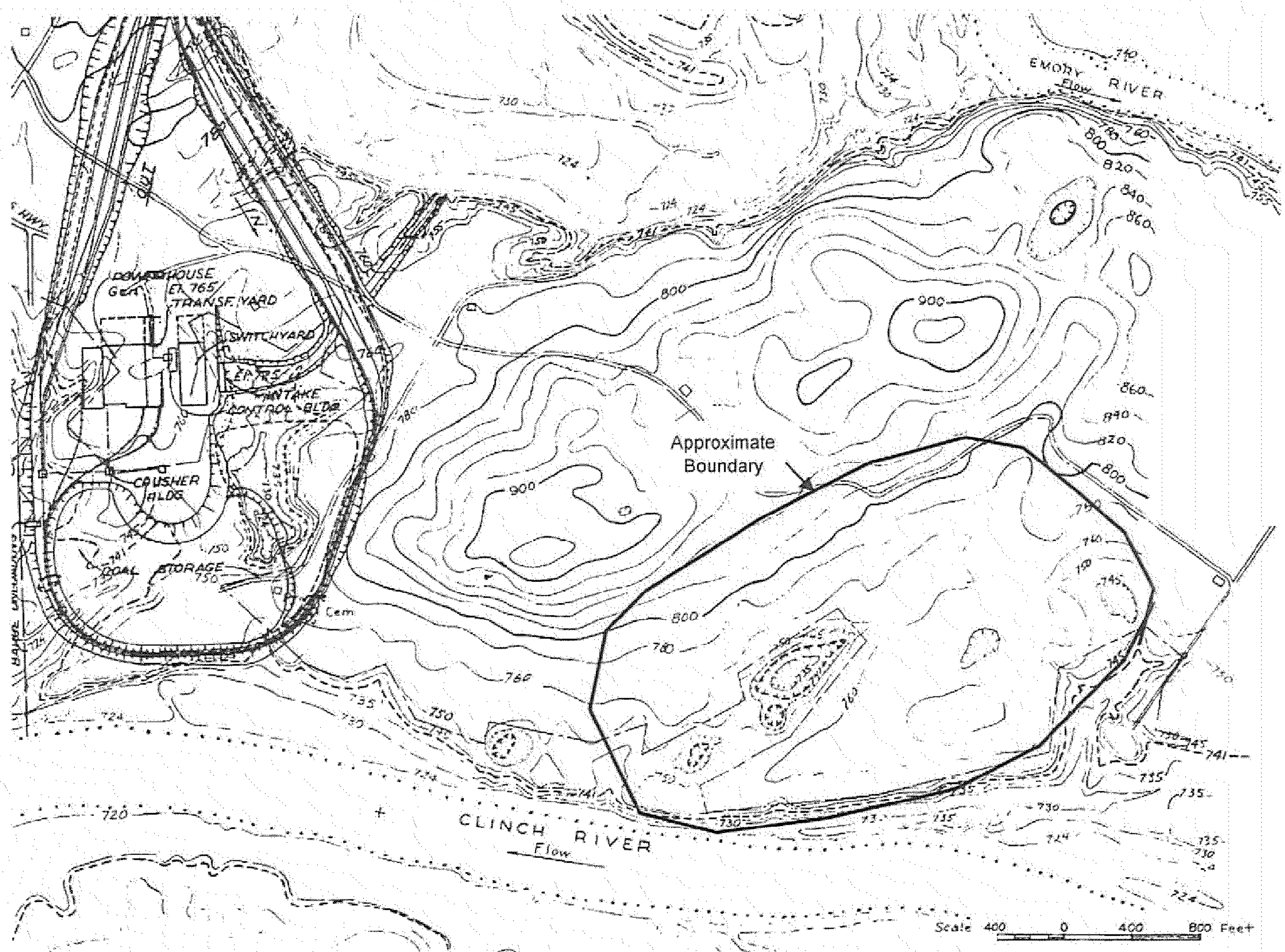

Figure 2-3. 1950 Pre-Disturbed Topography (adapted from TVA, 1950)

2.2 Geology

2.2.1 Bedrock

The area proposed for the future CCB disposal area is underlain exclusively by the Knox Group as shown in Figure 2-4. The Conasauga Group borders the Knox on both sides. The Knox Group is one of the most widely distributed lithologic units in East Tennessee. The Knox Group in the region can be subdivided into five formations but cannot be differentiated at the site based on bedrock cores. Formation contacts of the Knox shown in Figure 2-4 are proximal, based on average thicknesses of units reported by Solomon et al. (1992). Table 2-1 stratigraphically depicts the Knox in relation to other geologic formations. The Knox consists of a massive sequence of Lower Ordovician and Upper Cambrian dolomite and dolomitic limestones that were deposited on a vast stable craton in a near-equator environment (Lumsden and Caudle, 2001). The general variation in lithology of the Knox is from massive, crystalline, very cherty dolomite at the base to generally less massively bedded, dense to fine crystalline, less cherty dolomite at the top. Thin beds of dense limestone are present in the upper part of the Knox, and thin beds of relatively pure sandstone occur about 1000 ft above the base of the group (at base of Chepultepec dolomite).

Table 2-1. Stratigraphic Description (adapted from Solomon et al., 1992)

Unit	Age	Thickness (ft)	Lithology
Knox Group			
Mascot Dolomite	Lower Ordovician, Upper Cambrian	250 - 400	Massive dolomite, siliceous dolomite, bedded chert, limestone, some clastics
Kingsport Formation		300 - 500	
Longview Dolomite		130 - 200	
Chepultepec Dolomite		500 - 700	
Copper Ridge Dolomite		800 - 1100	
Conasauga Group			
Maynardville Limestone	Middle, Upper Cambrian	400 - 480	Dolomitic limestone, limestone
Nolichucky Shale		320 - 500	Shale, siltstone, calcareous siltstone and shale, shaly limestone, limestone
Dismal Gap Formation (formerly Maryville Limestone)		310 - 400	
Rogersville Shale		60 - 120	
Rutledge Formation		100 - 130	
Pumpkin Valley Shale		300 - 330	
Rome Formation	Upper Cambrian	300 - 410	Interbedded shale, siltstone, sandstone, local dolomite lenses

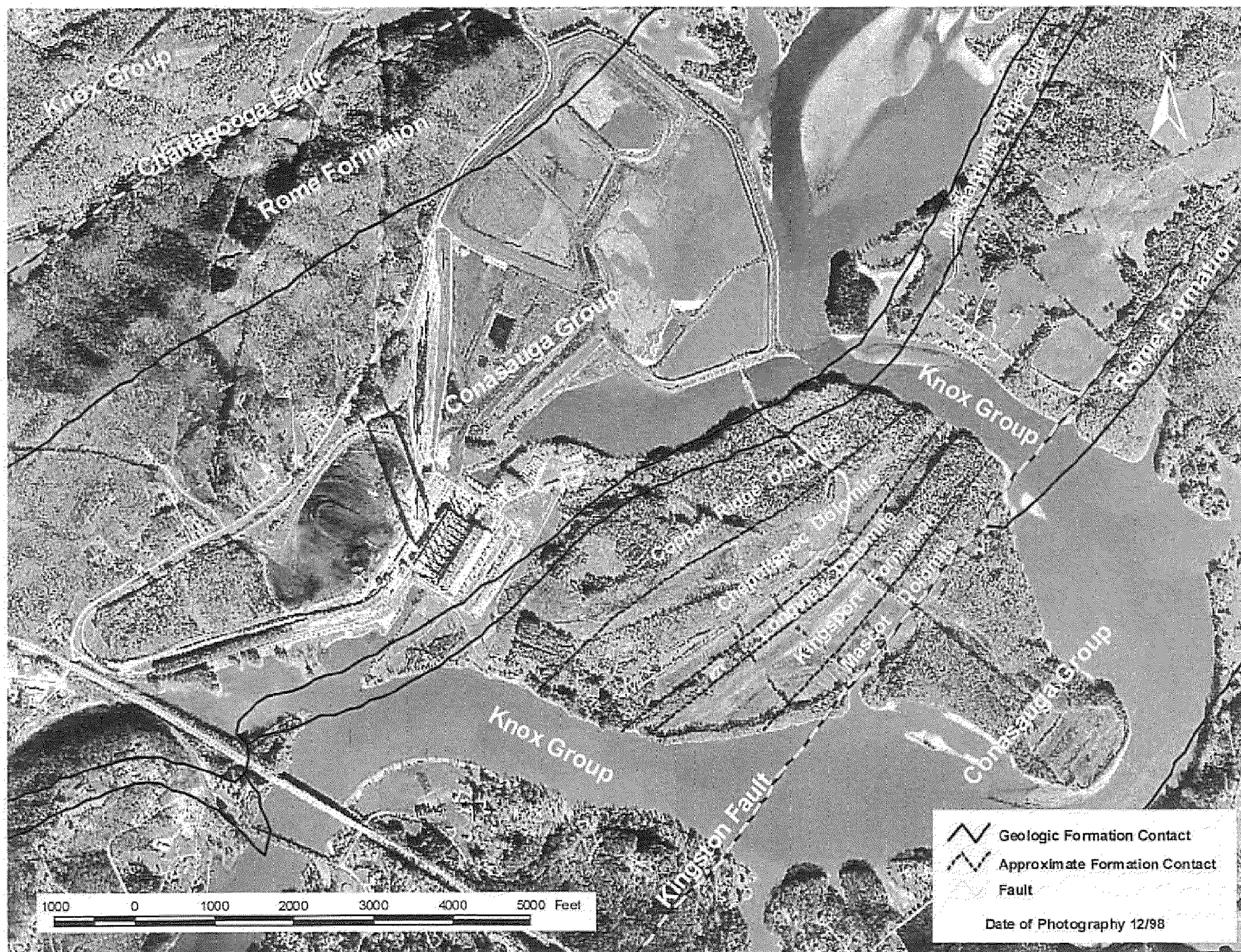

Figure 2-4. Site Geologic Map

The controlling geologic structure of the region is a series of northeast-striking thrust faults which have forced older rocks from the southeast over younger units. Bedrock units of the Conasauga Group (Cambrian age) and the Knox Group (Cambro-Ordovician) subcrop beneath the KIF Peninsula in northeast-trending bands (Figure 2-4). Bedrock strike is expected to average ~N55°E based on regional measurements (Dreier et al., 1987; Sledz and Huff, 1981). Bedrock bedding within the site generally dips to the southeast at angles averaging 45 to 50 degrees (Bensiger and Kellberg, 1951). Although bedrock does not outcrop within the confines of the proposed disposal facility, the Knox outcrops just south of the site along the Clinch River. Visual observations of these outcrops suggest similar angles of bedrock dip. Bedrock cores from within the site (Appendix C) indicate that bedrock dip is generally on the order of 40 to 50 degrees southeast with orthogonal joints occurring at angles ranging from about 55 degrees to vertical.

The Kingston fault (a thrust fault) crosses the southeastern margin of the site as shown in Figure 2-4. Drilling logs for borings NB-47 and NB-84 (nearest the Kingston fault) showed evidence of slickensides in bedrock cores (Appendix C). Bedrock core samples of the Knox from boring NB-10 indicated brecciation. The Kingston fault is an ancient structure and further movement along the fault is highly improbable. The hydrologic importance of this fault in the study area is not evident from the study performed herein.

The East Tennessee Seismic Zone (ETSZ) is a 300-km long, northeast-southwest trending concentration of earthquakes that has been well-delineated in recent years by regional seismograph networks (Powell et al., 1994). In recent years, except for the New Madrid Seismic Zone, the rate of earthquake activity in the ETSZ has been higher than any location in the United States east of the Rocky Mountains. However, the Kingston and associated faults were formed approximately 300 million years ago (Bailey, 2000) and further movement along these faults is improbable. Modern day earthquakes in East Tennessee tend to occur several miles beneath the surface and no recent movement has been observed on other surface faults in East Tennessee.

Joints are simple fractures without significant vertical or lateral displacement of strata. They occur during diagenesis, later tectonism, and erosional loading and unloading. They may be the result of tensional shear forces. Most joints, like those at KIF, are oriented normal to bedding planes. In plan view a majority is straight, but sinuous and curved joints may be found. Parallel joints constitute a joint set. Joint fracture openings may be passive or tiny and impermeable by water, or larger but filled with secondary dolomite/calcite that renders them impermeable. Most large joints and cross-joints will be permeable. Before any solutional modification, it appears that such openings are angular and jagged, with many irregular points and patches of rock contact. Under lithostatic pressure, joints are more readily closed to impermeable dimensions than are bedding planes (Ford and Williams, 1989).

All of the site bedrocks exhibit numerous joints. Solution weathering has progressed along many of these steeply dipping joints, which has resulted in relatively thick residuum, making it difficult to approximate their trends from the surface exposures. However, from indications afforded by drill cores at least two joint sets are obvious. The most prominent set strikes N 55^{o}E, parallel to strike, and the second is orthogonal to strike (bedding-plane parallel or strike-parallel). The frequency, spacing, and length of joints are further expected to be complex functions of bed thickness and lithology. Core samples of the Knox bedrock at the site (Appendix C) exhibit slight to highly fractured conditions. Most cavities and joints were also observed to be completely or partially filled with clays or sands. An exception was at NB-66 where open cavities were observed. Cavity thicknesses ranged from 0.4 to 8.0 ft. Cavities of measurable thickness were observed at half of the corehole locations.

Figure 2-5 displays top of bedrock at the site based on all available data (seismic, Geoprobes, and borings). Although this figure is useful in depicting the top of rock, parts of the map may be biased by Geoprobe pushes that stopped on large residual gravel and/or relatively deep data that might be related to the intersection of near-vertical bedrock joints. The elevation of the top of rock directly beneath the entire proposed disposal area ranges from 677 to 851 ft-msl (Figure 2-5). Within the lower elevations of the site (SE of the ridge), bedrock ranges from 677 to 748 ft-msl and averages about 719 ft-msl. Outside this area the bedrock surface rises steeply to the NW – toward the ridgeline. In general, bedrock is a subdued reflection of topography. Apparent depressions at the top of bedrock do not necessarily coincide with surface depressions (e.g., near piezometer F and at B-22). The ground surface depression at B-22 appears to be an old man-made farm pond (i.e., berms along sides). However, bedrock depressions are coincident with ground surface depressions (dolines) in the vicinity of NB-73 and at the site pond (near NB-44; Figure 2-5).

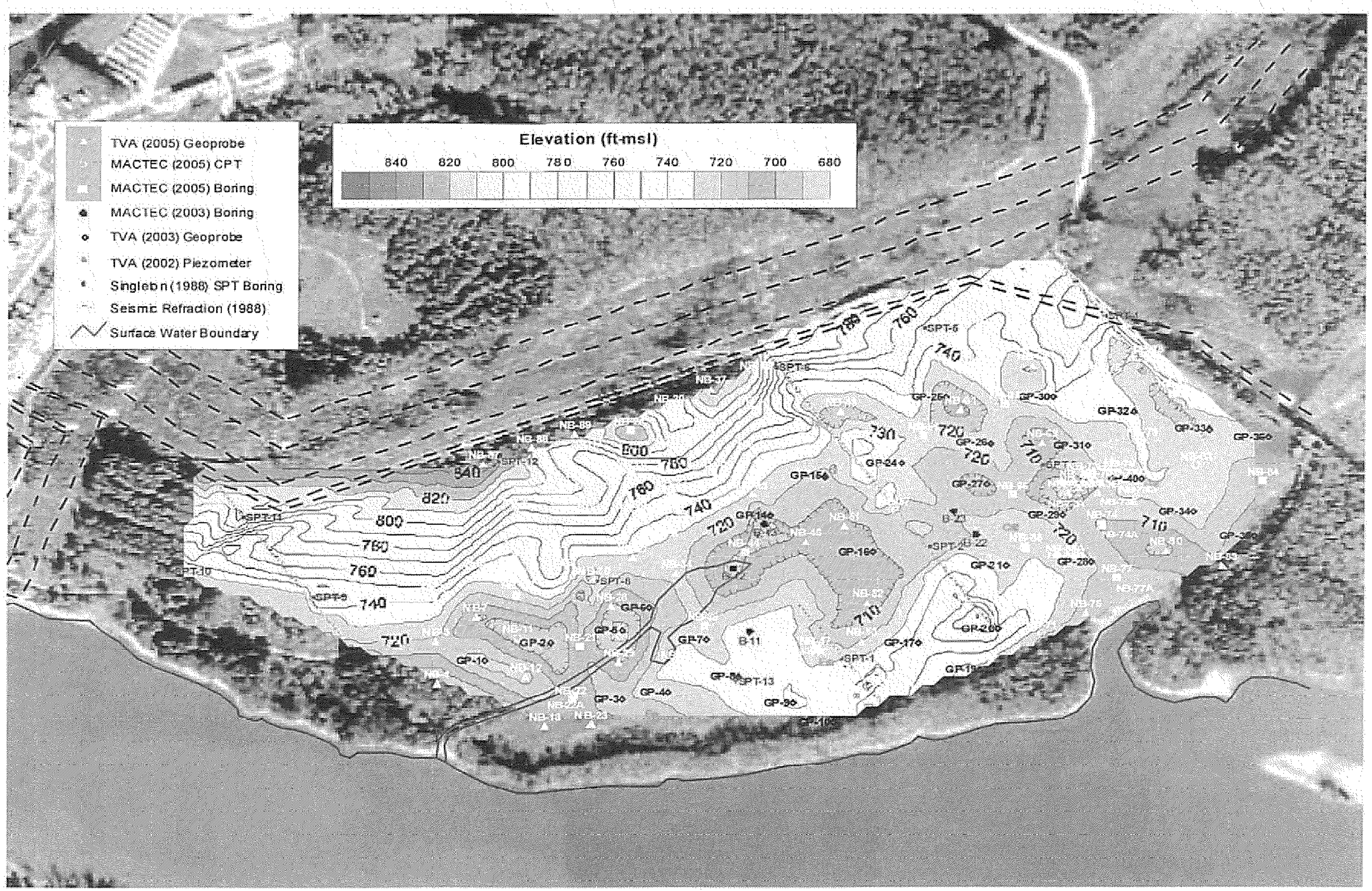

Figure 2-5. Top of Bedrock (ft-msl)

2.2.2 Soils

A mantle of predominantly residual soil lies above bedrock in the proposed CCB disposal area (Appendix C). Figure 2-6 displays soil thickness at the site based on all available data (seismic, Geoprobes, and borings). Although this figure is useful in soil thickness, the reliability of the map may be uncertain due to those factors described in the previous paragraph. Soil thickness is highly variable, ranging from 8.5 to 120 ft and averaging 40.5 ft based on all available data (139 holes) within the confines of the proposed disposal area.

Depressions/dolines depicted at the top of bedrock (Figure 2-5) are mantled by soil thicknesses (Figure 2-6) ranging from 35.2 – 74.5 ft. Visual and laboratory classifications of these soils indicated that they are of residual origin except in the area of NB-44 (site pond) where alluvial deposition has occurred. There were no voids detected immediately above bedrock that would indicate stoping of soil into the deeper bedrock system. Alluvial soils were observed at test borings NB-21, NB-22, NB-35, and NB-44. Alluvium was encountered at depths ranging from 2.5 ft (NB-22 and NB-44) to 47.8 ft (NB-21). Alluvium consists primarily of clayey to sandy silt with sand and chert fragments. Alluvial deposition observed in the vicinity of these borings is apparently associated with higher levels of the Clinch River along this relatively low topographic area of the site.

Natural moisture content test were performed on many of the undisturbed soil samples. Atterberg limits, specific gravity tests, grain size distributions with hydrometer analyses, and unit weight tests were performed for selected undisturbed soil samples. Appendix C (Table E-1) summarizes the test results. Residual soils are primarily silty clays and were encountered at all MACTEC (2005) test borings except NB-21 based on visual classifications at the time of drilling. Laboratory classification of soil samples (Appendix C; Table E-1) indicates soils are predominantly fine-grained consisting of silt and clay.

Liquid limits of site soil samples ranged from 35 to 81; plastic limits ranged from 18 to 42; and plasticity indices ranged from 12 to 47. Natural moisture contents ranged from 17.7 (NB-41) to 54.2% (NB-44). Optimum moisture content in the standard Proctor test ranged from 17.7 to 26.8%. Specific gravities of soil specimens ranged from 2.62 to 2.78 and unit weight ranged from 103.6 to 125.1pcf (Appendix C; Table E-1).

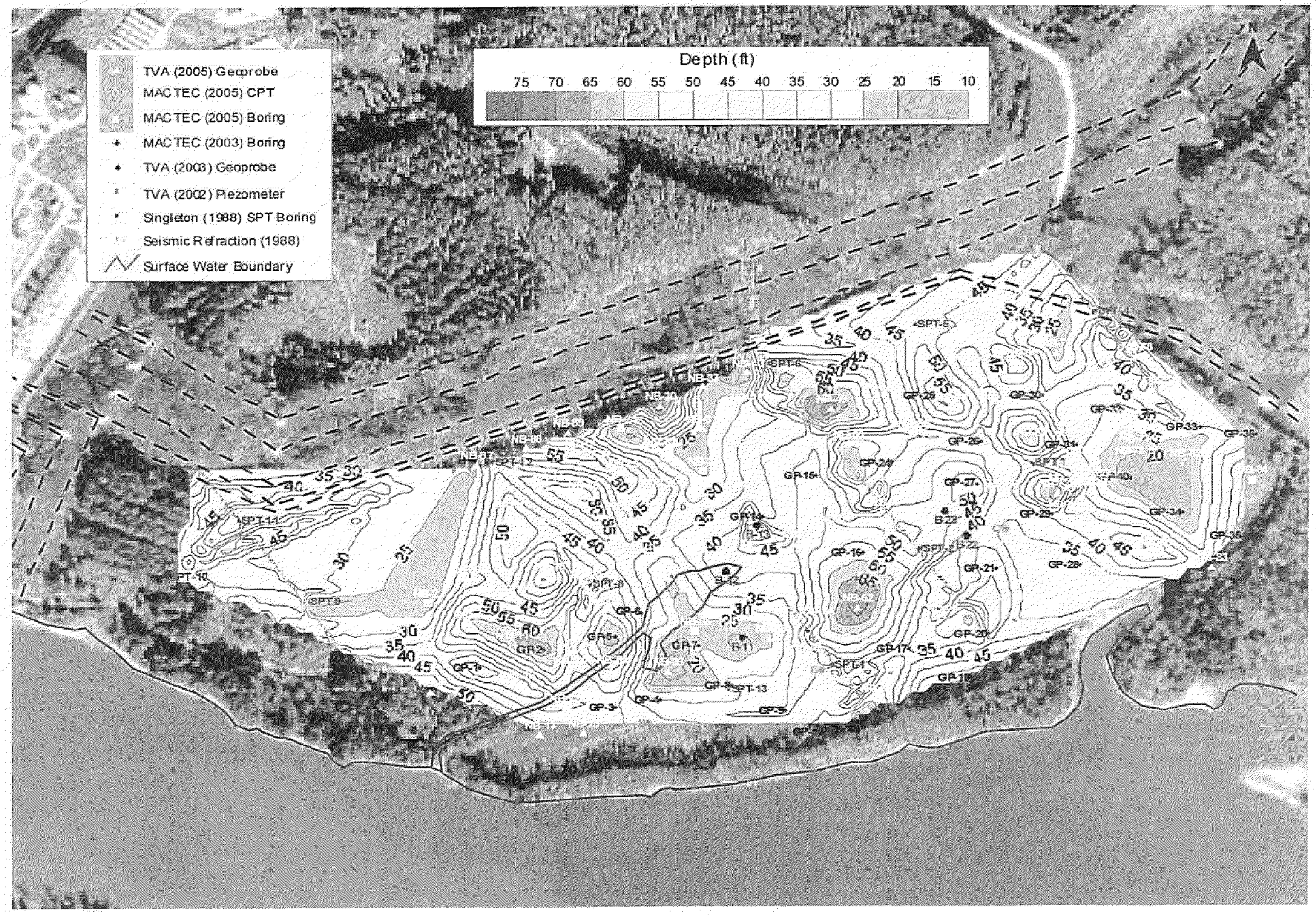

Figure 2-6. Overburden Thickness (ft)

A total of ten constant-head permeability tests (ASTM D5084) were performed on undisturbed and remolded (bulk) samples obtained from selected site borings (Table 2-2). Bulk samples were remolded to 95% of their respective standard Proctor maximum dry densities and about 2% over optimum moisture content. The effective confining pressures applied to various soil specimens varied according to estimates of conceptual landfill design. As shown in Table 2-2, resulting K_v values for undisturbed samples ranged from 10^{-4} to 10^{-8} cm/s. K_v values for remolded samples were on the order of 10^{-6} to 10^{-7} cm/s. Although the number of soil samples included for lab permeability testing does not strictly adhere to TDEC guidelines (1 per 3 acres; 11 samples for Phase 1 disposal area), single-well aquifer testing and borehole flowmeter surveys (Section 2.4) provide additional in-situ measurements of hydraulic conductivity for the natural geologic buffer. Additional lab permeability measurements have also been planned for borrow areas that will serve as geologic buffer fill.

Table 2-2. Summary of Constant-Head Permeability Tests (ASTM D5084)

Boring ID	Depth (ft)	Moisture (%)	K_v (cm/s)	Sample Type
NB-21A	33.0 - 35.0	26.6	1.5E-08	Shelby (undisturbed)
NB-44	16.5 - 18.5	28.2	4.6E-08	Shelby (undisturbed)
NB-44	21.5 - 23.5	25.7	1.6E-04	Shelby (undisturbed)
NB-47A	30.0 - 32.0	32.8	5.5E-08	Shelby (undisturbed)
NB-76	19.0 - 20.5	23.9	2.0E-07	Shelby (undisturbed)
NB-84	32.5 - 34.5	27.1	5.9E-08	Shelby (undisturbed)
NB-22	2.0 - 10.0	19.2	1.3E-06	bulk (remolded)
NB-76	5.0 - 15.0	23.0	2.5E-06	bulk (remolded)
NB-84	2.0 - 10.0	23.8	1.4E-07	bulk (remolded)
NB-59	5.0 - 15.0	22.4	1.1E-07	bulk (remolded)

2.3 Groundwater Occurrence

The first occurrence of groundwater below the proposed CCB disposal areas is generally within residual soils. Under present conditions, shallow groundwater is derived from infiltration of precipitation. It is possible that limited lateral inflow may occur into the bedrock flow system along the river margin of the site; e.g., during periods of rapid increases in reservoir elevations. However, the lateral outflow from the bedrock flow system to the Clinch River is expected to be the norm. Groundwater movement, as inferred from potentiometric map developed from water level measurements (July 2005) in soil monitoring wells located inside of the proposed disposal area, is generally S-SE from the ridgeline toward the Clinch River (Figure 2-7). During wetter periods of the year, when groundwater levels are higher, the site pond and drainage canal are likely recipient of shallow groundwater recharge.

Based on three snapshot water level measurements since July 18, 2005, downward vertical hydraulic gradients between soil and shallow bedrock were less than two percent at wells MW-10A&B and MW-63A&B, which are located near the base of the ridgeline (Figure 1-3). Vertical hydraulic gradients are nonexistent at wells MW-81A&B on the eastern margin of the site. These data suggest that unconfined conditions exist for the soil and shallow fractured bedrock system at the site.

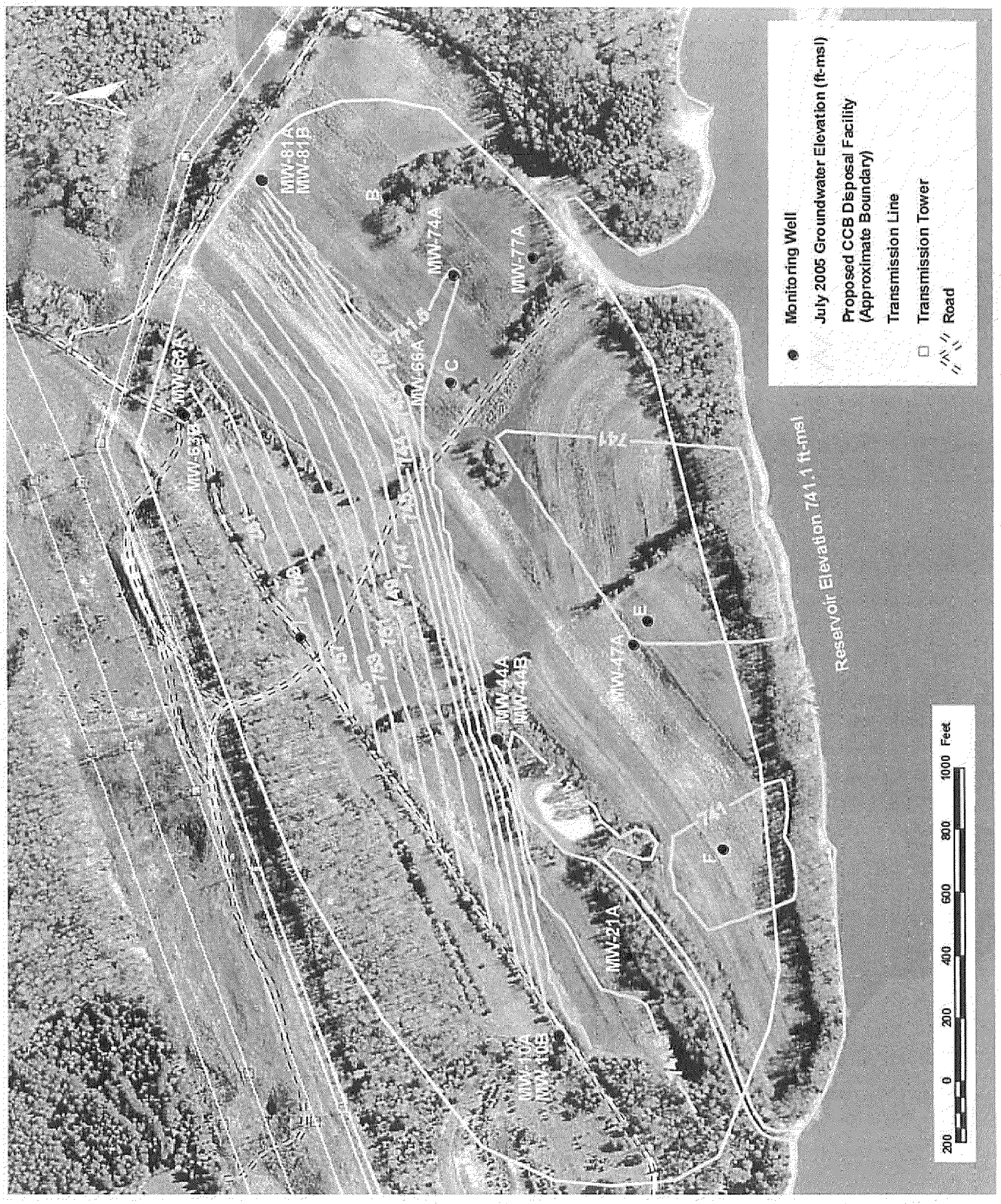

Figure 2-7. Potentiometric Surface from July 2005 Groundwater Level Measurements

Long-term hydrographs for the five site piezometers (B, C, E, F, and I) are presented in Figure 2-8. Continuous measurements were collected using in-situ pressure transducers connected to automatic dataloggers. Due to uncertainties in selection of the Peninsula site as a disposal area, data collection was sporadic and gaps occur in the water level measurements. Likewise, river elevation data was not available until February 2005, at which time a continuous recorder was installed on the Emory River side of the intake channel skimmer wall (Figure 1-1). Natural seasonality in groundwater level trends is not discernible in Figure 2-8, perhaps due in part to the infrequency of measurements. However, continuous water level data collected after March 2005 suggest that water levels of the Emory and Clinch Rivers significantly influence average groundwater levels in the lower elevations of the site; i.e., within about 1,000 ft of the Clinch River bank. As shown in Figure 2-9, groundwater levels observed during the period after March 2005 are highly correlated with Emory River elevations. During this time period, the Watts Bar Pool was being increased to its summer pool elevation near 741 ft-msl – noting that the Emory and Clinch River elevations are essentially the same at the Peninsula site. Water level data for the March – May 2005 time interval indicate correlation coefficients ranging from 0.84 to 0.98 for water levels of the Emory River and piezometers B, C, E, and F. High resolution (30-minute) rainfall data collected at the site during January – March 2005 (Figure 2-10) would suggest that groundwater levels in the lower elevations of the site also respond in a lagged fashion to rainfall events and to elevations exceeding those of adjacent surface waters. However, the Emory River is uncontrolled (i.e., no dams in headwater reaches) such that river elevations near the site also fluctuate with rainfall events.

Figure 2-11 shows maximum-mean-minimum values in groundwater levels at all site wells and piezometers for the period January 2003 to present. The plot also illustrates the relationship between groundwater and ground surface elevations at the site. A transitional curve-fit (groundwater level as function of ground surface elevation) was developed for mean groundwater level data as shown in Figure 2-11 and this curve was then repositioned (raised by 8.5 ft) to the equivalent highest water level measurement observed in lower elevations of the site since January 2003; i.e., maximum value at piezometer F. The resulting relationship was then used to calculate seasonal high groundwater elevations at the site as a sole function of ground surface elevation. The predicted seasonal high potentiometric map (Figure 2-12) was produced using this relationship. In general, lower ground surface elevations of the site would be expected to experience a seasonal high groundwater elevation near 749.2 ft-msl utilizing this approach. Although this estimate is somewhat dependent on a single surrogate well (i.e., piezometer F), the approximation is considered conservative with a seasonal high groundwater elevation (749.2) at lower topographic elevations exceeding the 100-yr flood elevation of 747.6 ft-msl (Figure 2-1).

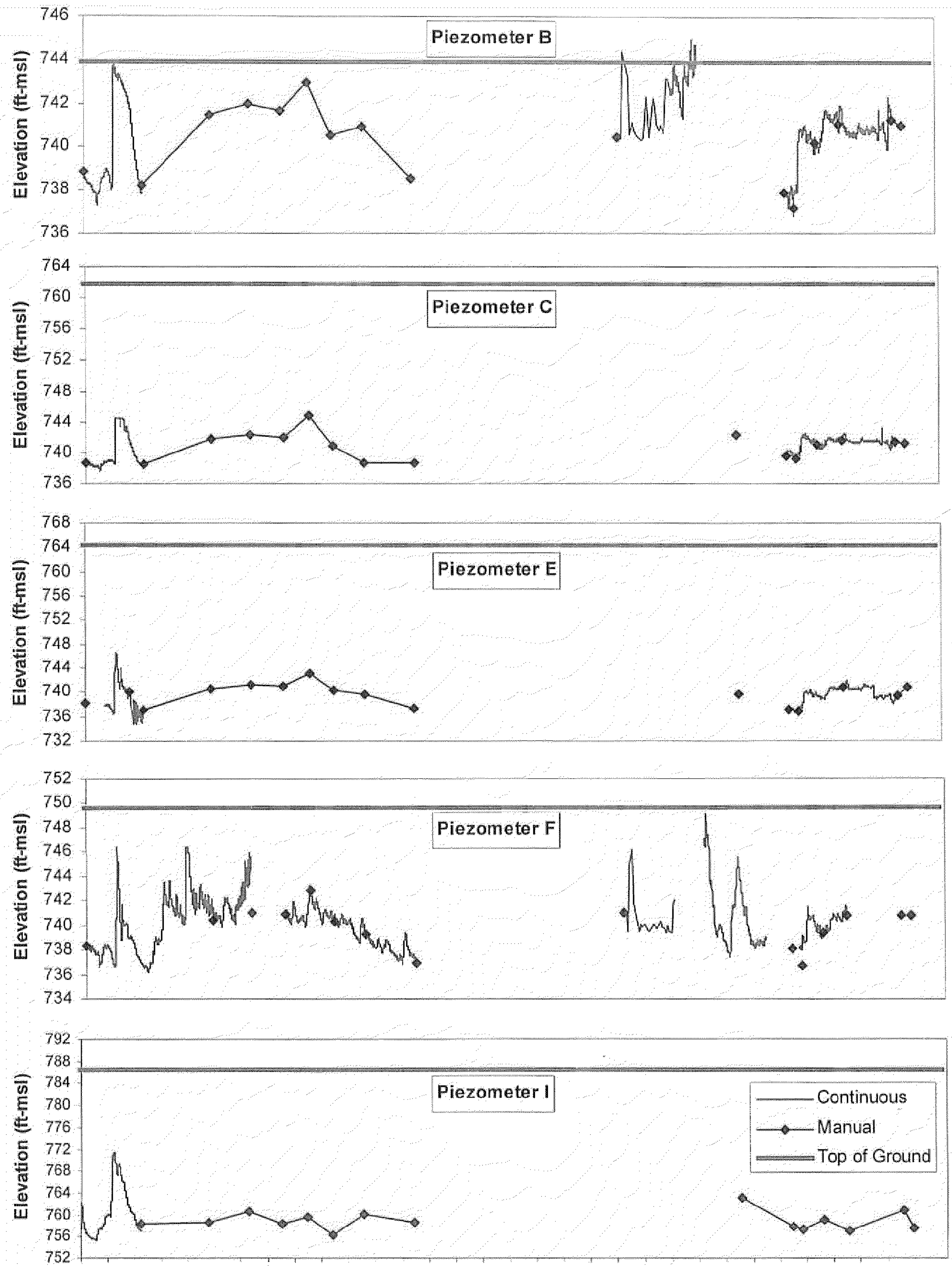

Figure 2-8. Time Series Plots of Groundwater Levels at Site Piezometers

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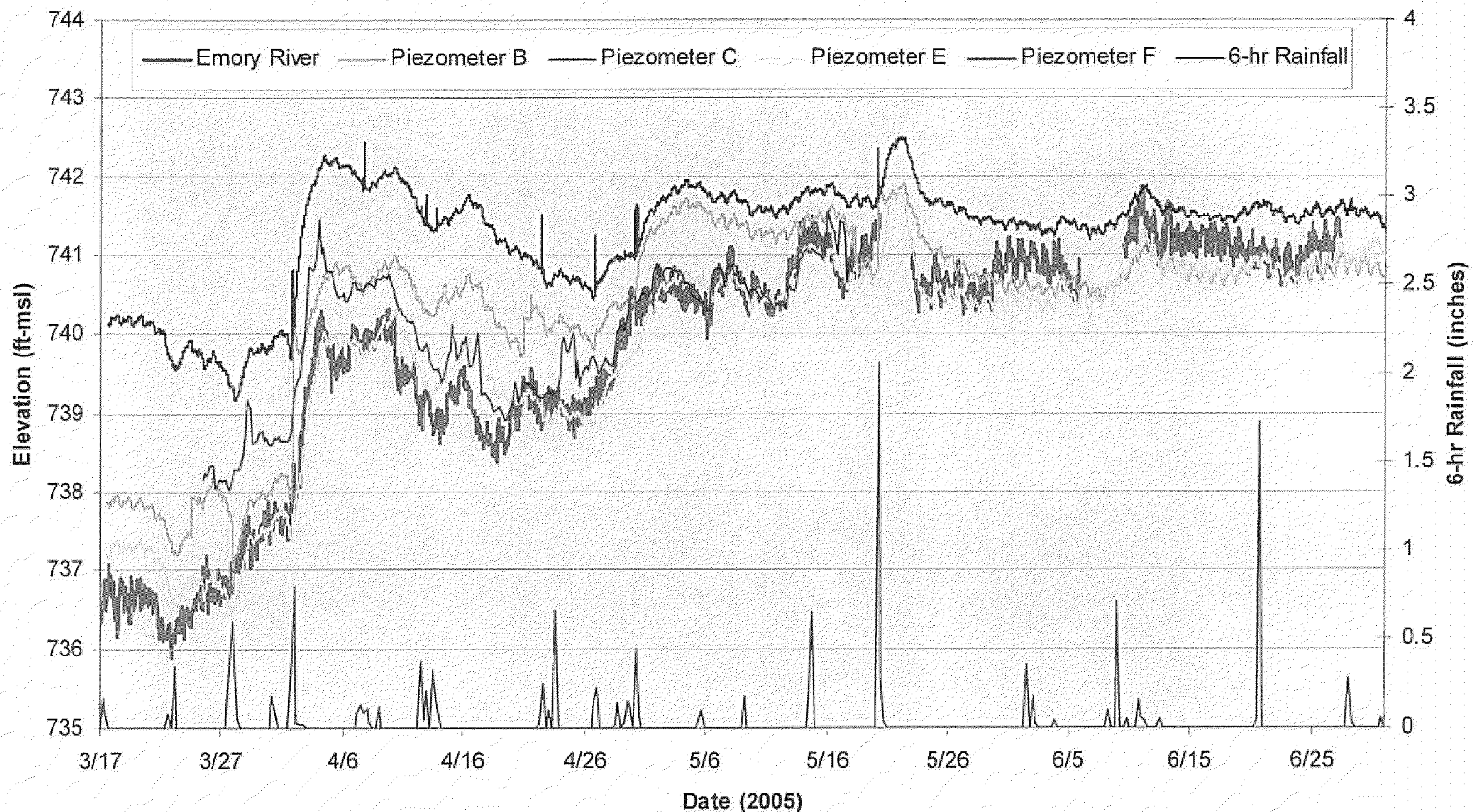

Figure 2-9. Time Series Plot of Groundwater Levels, Emory River, and 6-hr Rainfall from March – April 2005

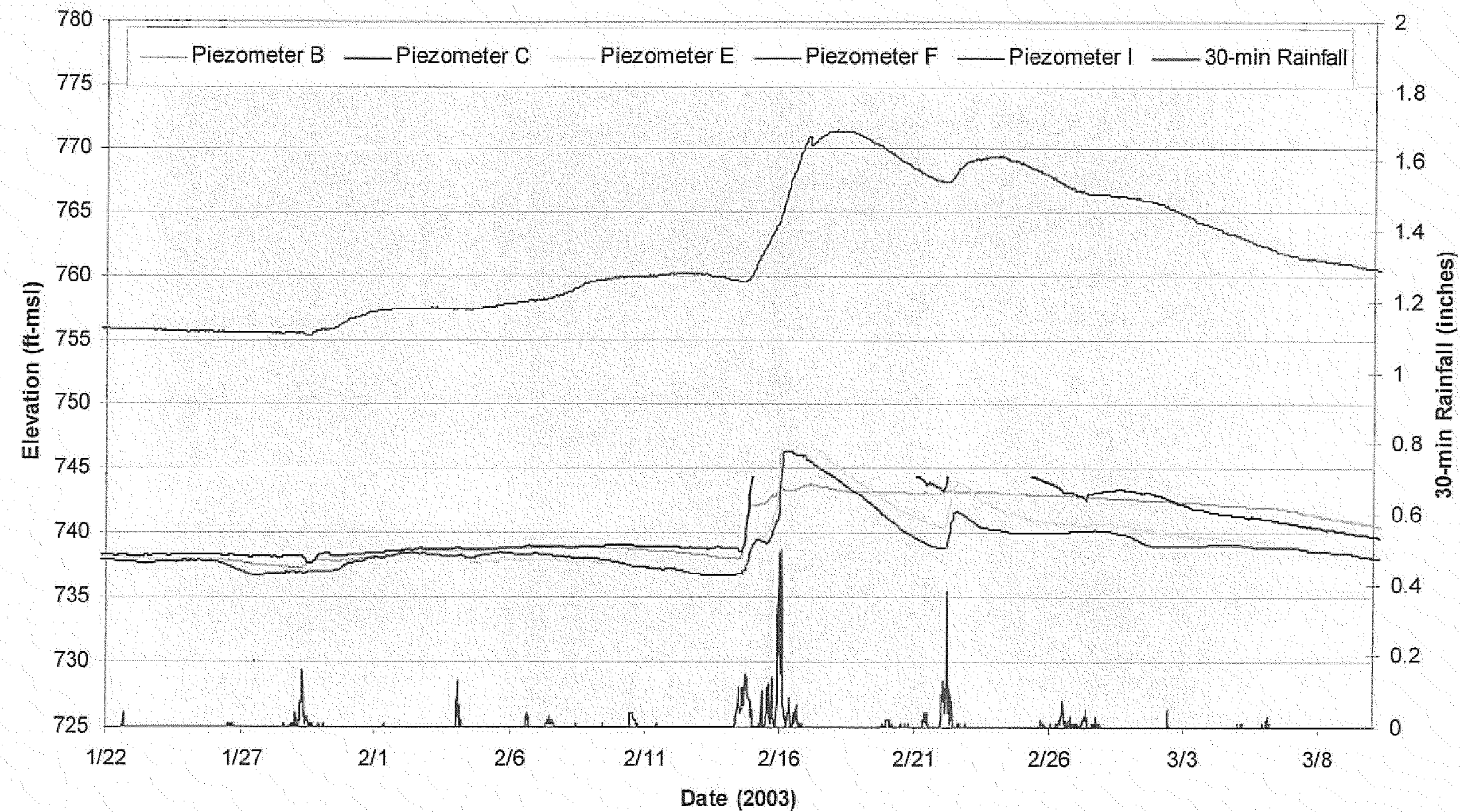

Figure 2-10. Time Series Plot of Groundwater Levels and 30-Minute Rainfall from January – March 2005

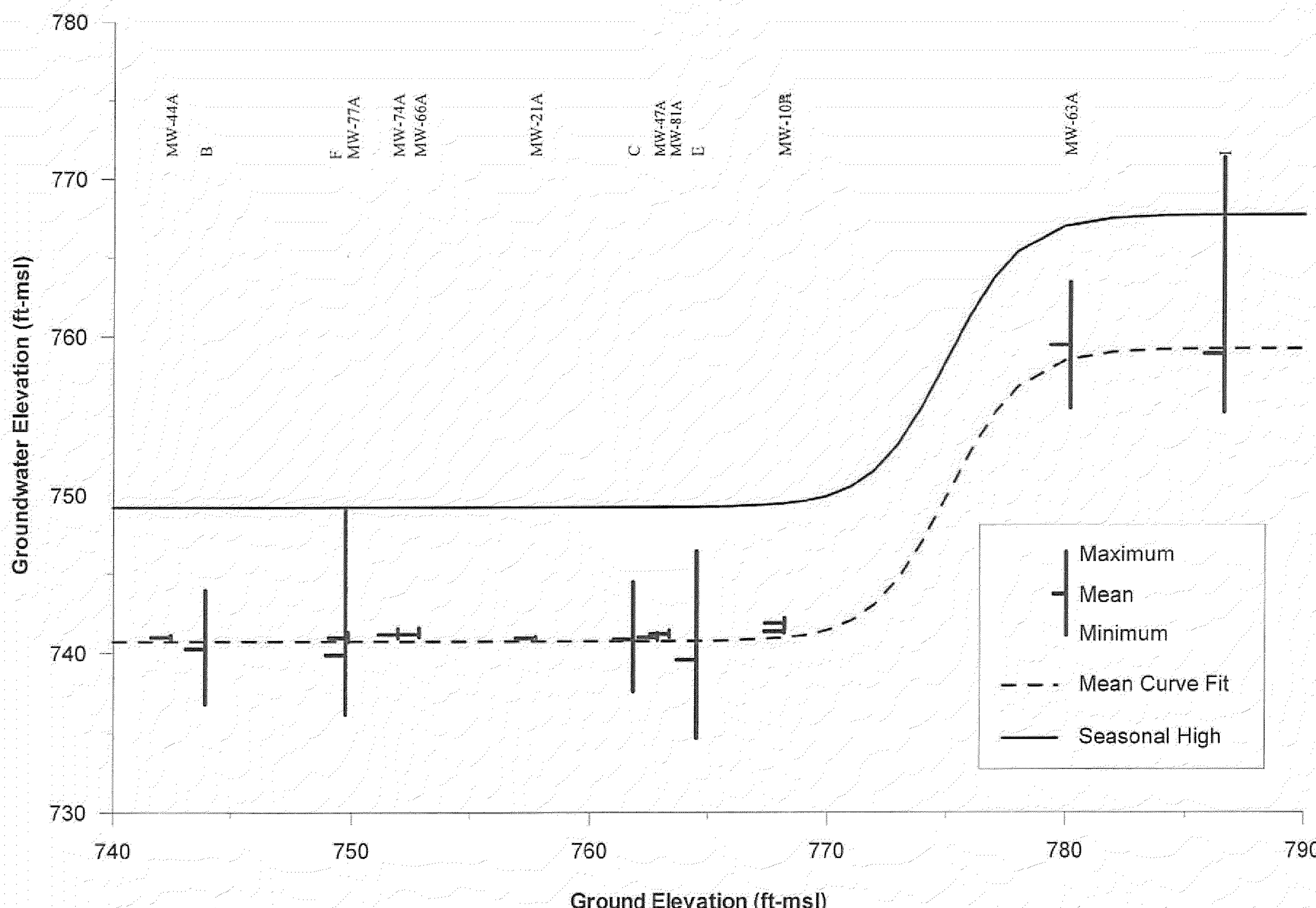

Figure 2-11. Relationship between Groundwater and Ground Surface Elevations at Site Wells and Piezometers

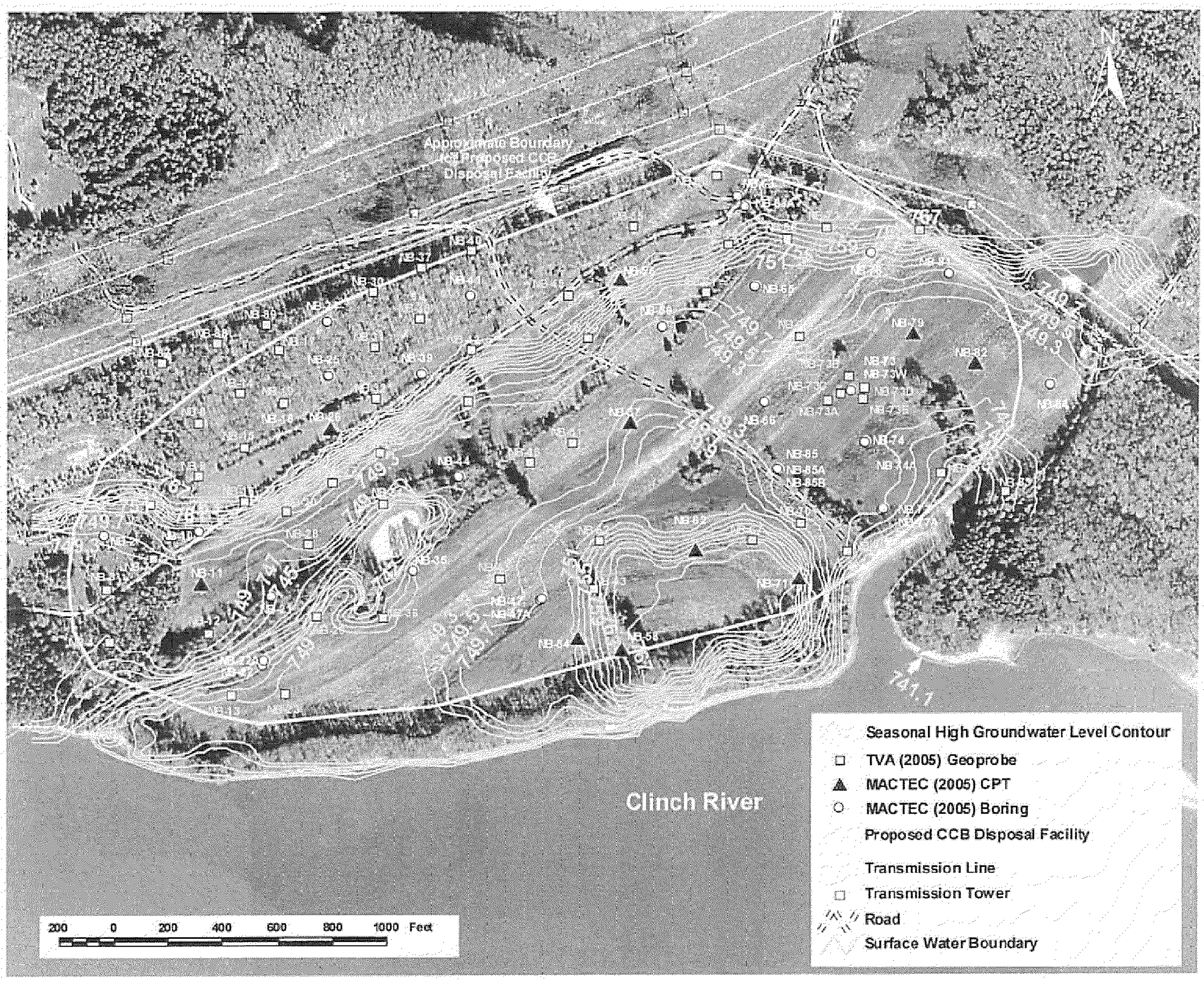

Figure 2-12. Seasonal High Potentiometric Map Predicted from Ground Surface Elevations

Two small areas within the proposed facility boundary reside within the 100-yr flood plain and the natural geologic buffer zone within these areas is minimal. The first area is in the immediate vicinity of the site pond and drainage channel and the second area is on the SE corner of the site at the head of a small river cove. However, the proposed facility design includes plans for filling of these areas with suitable borrow soil. Furthermore, the current facility plan includes a bottom liner residing above the seasonal high groundwater elevation and an under-drain system for interception of leachate.

2.4 Hydraulic Properties

2.4.1 Main Plant Site Hydraulic Conductivity Data

A summary of field and laboratory measurements of hydraulic conductivity (K) for ash, alluvial soils, and shallow bedrock derived from previous investigations at the main plant site is presented in Table 2-3. Vertical hydraulic conductivities (K_v) for nine fly ash samples range from $3.6x10^{-6}$ to $8.3x10^{-5}$ cm/s and exhibit a median value of $2.0x10^{-5}$ cm/s. The two field measurements of fly ash horizontal conductivity (K_h) generally fall within the range of data reported for K_v. Laboratory-derived K_h and K_v data for alluvial clay-silt samples show little difference and average about $5x10^{-7}$ cm/s. Field measures of K_h for this unit are about an order of magnitude higher, averaging approximately $7x10^{-6}$ cm/s. The difference reflects the larger measurement scale associated with field tests, as well as the tendency for higher K values in the horizontal direction. Field testing performed in three wells completed in the upper Conasauga shale yielded K_h values averaging $2x10^{-5}$ cm/s.

2.4.2 KIF Peninsula Single-Well Aquifer Tests

Single-well aquifer testing at the KIF Peninsula site consisted of single-well pumping tests, injection tests, slug tests, and electromagnetic borehole flowmeter (EMFM) surveys. Drawdown measurements during testing were continuously measured using pressure transducers connected to automatic dataloggers. Discharge measurements were manual and involved time to fill calibrated containers. All slug tests involved instantaneous introduction of water into a well via a calibrated container. Test results from pumping and injection tests were analyzed using the Theis (1935) Forward solution and Copper-Jacob (1946) Time-Drawdown analyses. Slug test results were analyzed using Bouwer (1989), Bouwer and Rice (1976), and Hvorslev (1951) methods. Table 2-4 provides a summary of test results.

Table 2-3. Summary of Main Plant Site Hydraulic Conductivity Data

Media	Location	K_h (cm/s)	K_v (cm/s)	Test Method	Reference
Fly Ash	Ash Dredge Cell 1	--	8.3E-05	ASTM D-5084	Law, 1995
Fly Ash	Ash Dredge Cell 3	--	3.4E-05	ASTM D-5084	Law, 1995
Fly Ash	B-1	1.4E-05	5.1E-06	ASTM D-6391	MACTEC, 2004
Fly Ash	B-2	3.7E-06	3.6E-06	ASTM D-6391	MACTEC, 2004
Fly Ash	B-2A	--	1.67E-05	ASTM D-5084	MACTEC, 2004
Fly Ash	B-1A, 1B	--	1.87E-05	ASTM D-5084	MACTEC, 2004
Fly Ash	--	--	2.0E-05	ASTM D-2434-68	Young, et al., 1993a
Fly Ash	--	--	2.1E-05	ASTM D-2434-68	Young, et al., 1993a
Fly Ash	--	--	2.2E-05	ASTM D-2434-68	Young, et al., 1993a
Bottom Ash	(?)	--	9.3E-03	ASTM D-5084	Law, 1995
Alluvial Clay-Silt	Well 2	7.4E-08	6.3E-08	(note 1)	Milligan and Ruane, 1980
Alluvial Clay-Silt	Well 4	6.6E-08	2.8E-07	(note 1)	Milligan and Ruane, 1980
Alluvial Clay-Silt	Well 5	2.8E-07	4.0E-07	(note 1)	Milligan and Ruane, 1980
Alluvial Clay-Silt	Well 6	2.5E-06	4.4E-07	(note 2)	Milligan and Ruane, 1980
Alluvial Clay-Silt	Well 2	9.1E-06	--	(note 2)	Velasco and Bohac, 1991
Alluvial Clay-Silt	Well 4B	6.1E-06	--	(note 2)	Velasco and Bohac, 1991
Alluvial Clay-Silt	Well 5	9.1E-06	--	(note 2)	Velasco and Bohac, 1991
Alluvial Clay-Silt	Well 13A	3.0E-06	--	(note 2)	Velasco and Bohac, 1991
Conasauga Shale	Well 9B	6.1E-06	--	(note 2)	Velasco and Bohac, 1991
Conasauga Shale	Well 13B	2.1E-05	--	(note 2)	Velasco and Bohac, 1991
Conasauga Shale	Well 15A	3.0E-05	--	(note 2)	Velasco and Bohac, 1991

Notes:

1. *Laboratory constant-head test of undisturbed sample in triaxial cell; exact method unknown.*
2. *Field constant-rate pumping test in single well.*

The most representative test results are the longest tests, with highest drawdowns, and typically higher discharge rates. Likewise, pumping test results are typically preferred over injection tests. There were little differences between Bouwer (1989), Bouwer and Rice (1976), and Hvorslev (1951) analytical results. Hence, Bouwer and Rice (1976) are reported in Table 2-4. Likewise, similar results were obtained using the Theis (1935) Forward solution and Copper-Jacob (1946) Time-Drawdown analyses. The Cooper-Jacob (1946) Time-Drawdown result is reported in Table 2-4.

Bulk K values from single-well testing at soil wells were higher than anticipated at several locations. Results at soil wells ranged from 10^{-3} to 10^{-6} cm/s, and the geometric mean K is $3x10^{-4}$ cm/s. Bulk K values for bedrock wells are simply estimates since apertures and dimensions of solutioned fractures are unknown and analytical solutions are based on porous media assumptions. Results at bedrock wells ranged from 10^{-2} to 10^{-6} cm/s, and the geometric mean K is $4x10^{-4}$ cm/s.

Table 2-4. Summary of Single-Well Aquifer Test Results

Well	*Test Type*	*Q (gpm)*	EMFM Test	Analytical Results	
				K (ft/s)	K (cm/s)*
MW-10A	*slug*	6 gallons		1.93E-06	**5.88E-05**
MW-10B	*slug*	3 gallons		2.34E-06	**7.13E-05**
MW-21A	*slug*	6 gallons		1.25E-06	**3.81E-05**
MW-21A	*pump*	0.18	X	1.86E-05	5.67E-04
MW-44A	*pump*	6.40 & 4.88	X	3.17E-04	**9.66E-03**
MW-44B	*pump*	18.6 & 4.90	X	6.03E-04	**1.84E-02**
MW-47A	*slug*	6 gallons		2.71E-05	8.26E-04
MW-47A	*pump*	4.17		1.25E-04	**3.81E-03**
MW-47A	*injection*	1.54		1.35E-03	4.11E-02
MW-63A	*slug*	3.5 gallons		2.64E-07	**8.05E-06**
MW-63B	*slug*	3 gallons		3.46E-07	1.05E-05
MW-63B	*injection*	0.20	X	2.10E-07	**6.40E-06**
MW-66A	*slug*	6 gallons		1.46E-05	4.45E-04
MW-66A	*pump*	0.35	X	2.13E-04	6.49E-03
MW-66A	*pump*	3.26		1.78E-05	**5.43E-04**
MW-66A	*injection*	0.76	X	8.14E-05	2.48E-03
MW-74A	*slug*	6 gallons		1.68E-06	5.12E-05
MW-74A	*pump*	0.28	X	1.02E-05	3.11E-04
MW-74A	*pump*	1.05		7.65E-06	**2.33E-04**
MW-77A	*slug*	3.5 gallons		1.14E-05	3.47E-04
MW-77A	*pump*	3.00	X	2.89E-05	8.81E-04
MW-77A	*pump*	2.50		3.89E-05	**1.19E-03**
MW-81A	*slug*	6 gallons		6.00E-06	**1.83E-04**
MW-81A	*injection*	0.65	X	2.00E-04	6.10E-03
MW-81B	*slug*	6 gallons		1.03E-04	**3.14E-03**

*Bold values considered most representative

2.4.3 Electromagnetic Borehole Flowmeter (EMFM) Surveys

Development of borehole flowmeters for environmental and hydrogeological applications resulted from a growing recognition beginning in the 1960s that transport and dispersion of groundwater contaminants are controlled by the spatial variability of hydraulic conductivity [Skibitzke and Robertson, 1963; Dagan, 1982, 1984; Gelhar and Axness, 1983]. The theoretical advancements of these researchers in dispersive transport modeling necessitated development of practical methods for characterizing the hydraulic conductivity distributions of aquifers to support modeling of contaminant transport and remediation. A theoretical model for estimating vertical variations in hydraulic conductivity from borehole flowmeter data by Hufschmied [1983] represented a major step towards the development of such methods. Flowmeters had long been recognized by the petroleum industry as a practical tool for delineating productive oil-bearing zones in tests wells. However, mechanical impeller flowmeters available before the early 1980s lacked sufficient sensitivity and precision for the relatively low well flow rates typical of environmental applications. One of the first sensitive impeller flowmeters designed for environmental and hydrogeological applications was developed by INTEGRO (Zug, Switzerland) as reported by Hufschmied [1983, 1986] and Rehfeldt et al. [1989a, 1989b]. A series of papers by Hess [1982, 1986], Morin et al. [1988], and Hess and Paillet [1989, 1990] describe advancements in heat-pulse type flowmeters and their application to fractured rock hydrology.

Taylor et al. [1990] reviewed various techniques, both direct and indirect, for developing flow or hydraulic conductivity information in screened wells and/or boreholes. They concluded that methods relying on direct hydraulic measurements of some type, such as transient pressure changes or flowrates, offer the most promising methodologies for determining accurate logs of horizontal K and/or fracture locations in aquifers. Boggs et al. [1989], Rehfeldt et al. [1989a, 1989b], and Molz et al. [1989a, 1989b, 1990] also evaluated alternative methods for measuring the vertical variation of hydraulic conductivity. Among the different methods are small-scale tracer tests, multilevel slug tests, laboratory permeameter tests, equations based on grain-size distributions, and borehole flowmeter tests. All three groups concluded that the borehole flowmeter test is the most promising method for measuring the spatial variability in an aquifer's hydraulic conductivity field.

The EMFM was developed and patented by the TVA Engineering Laboratory [Young and Waldrop, 1989]. EMFM logging has been successfully conducted at several research facilities, RCRA/Superfund sites, and numerous TVA sites. Initial applications of the EMFM were targeted at characterizing the heterogeneous alluvial aquifer of the groundwater research facility at Columbus AFB, Mississippi [Rehfeldt et al., 1989b, 1992]. Well development and performance testing with the EMFM have been conducted by Julian and Young [1994] to gauge development requirements of wells used for hydraulic characterization of aquifers. Field demonstration of the prototype EMFM was conducted at three Superfund sites selected by the EPA [Young et al., 1997]. Further demonstrations of the EMFM include applications at Oak Ridge National Laboratories [Moore and Young, 1992; Julian, 1996b], the Paducah Gaseous Diffusion Plant [Young et al., 1993b], and numerous other TVA facilities [Julian, 1994, 1996a; Julian et al., 1993; Julian and Young, 1994,]. More recent applications related to the EMFM can be found in publications by Boggs and Julian (1998), Julian et al. (2001), Chen et al. (2001), Elci et al. (2003), and Flach et al. (2004). The "User's Guide for Application of the Electromagnetic Borehole" (Young et al., 1997) is provided in Appendix D.

Flowmeter testing at the KIF Peninsula site was conducted at specific wells identified in Table 2-3. The resulting K_h profile associated with EMFM testing at soil wells is shown in Figure 2-13. Note that the lower K_h threshold of the EMFM is 10^{-6} cm/s. At well MW-21A, K_h profiles suggest that relatively high K_h alluvium at this location is interbedded with lower K_h silty clay. The high K_h zone (4E-3 cm/s) above elevation 736 ft-msl is associated with silty sand (Appendix C). At depth (<719 ft-msl), silty clay to clayey silt with interbedded sandy layers dominates the soil profile (Appendix C). K_h values for the silty clay zones are $<10^{-6}$ cm/s. The higher K_h intervals observed at depth are associated with thin layers of fine to medium sand observed in soil samples. EMFM testing at NB-44A displayed the highest K_h values (Figure 2-13) observed for soil wells at the site, with some K_h values approaching 8E-2 cm/s. This well is also situated in alluvium and higher K values are associated with layers of medium to coarse sand. The K_h profiles for wells MW-66A, MW-74A, and MW-77A (Figure 2-13) indicate K_h values ranging from $<10^{-6}$ cm/s to 10^{-3} cm/s. Although these values are not necessarily indicative of residual soils, field observations, lab classifications, and laboratory permeability measurements indicate that these wells are developed in residuum with K_v values ranging from 10^{-6} to 10^{-8} cm/s. Hence, anisotropy of residual soils may be expected to range from 10 to 1000.

Incremental K_h values for bedrock wells are simply estimates since apertures and dimensions of solutioned fractures are unknown. Figure 2-14 depicts K_h profiles at bedrock wells. In general, flowmeter profiles suggest a relatively transmissive horizon (perhaps a few feet thick) near the bedrock interface (epikarst zone) at both test well locations. Similar high flow zones were observed at the epikarst zone at well MW-81B but the flowmeter survey at this location was not quantitative. The EMFM profiles at MW-63B suggest relatively competent bedrock across the available bedrock profile with only one small active fracture zone near 713.5 ft-msl – test discharge rates at this location were very low (0.2 gpm). In contrast, EMFM profiles at well MW-44B indicate at least three active fracture zones across the available bedrock profile. Discharge rates of 18.6 gpm at this well produced drawdowns of less than two feet. These results are not surprising given the location of MW-44B relative to the site pond, alluvium zones within the soil profile, and historical mapping indicating that this was a relatively large doline.

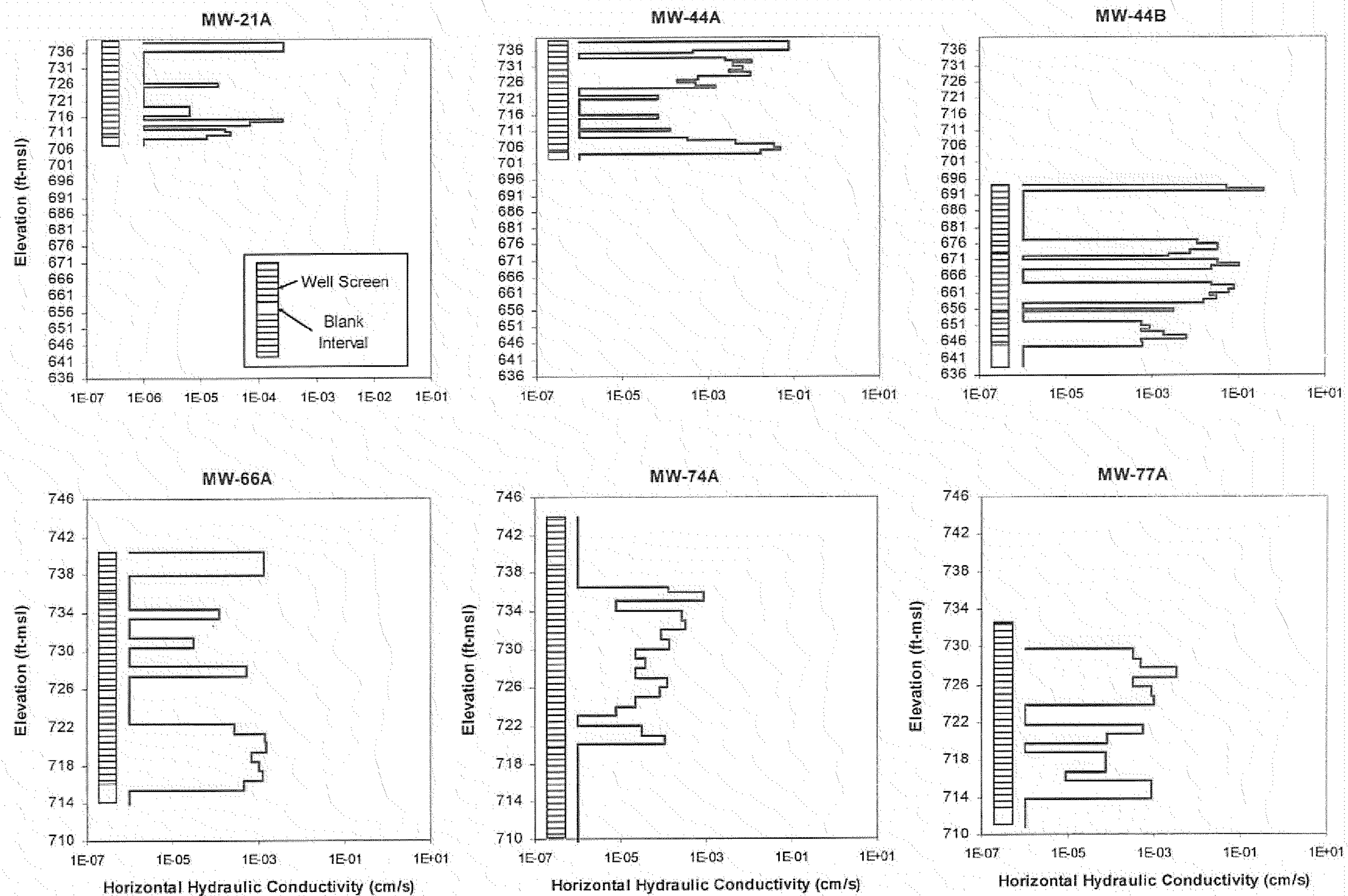

Figure 2-13. K_h Profiles from EMFM Testing at Soil Wells (MW-44B included for comparison)

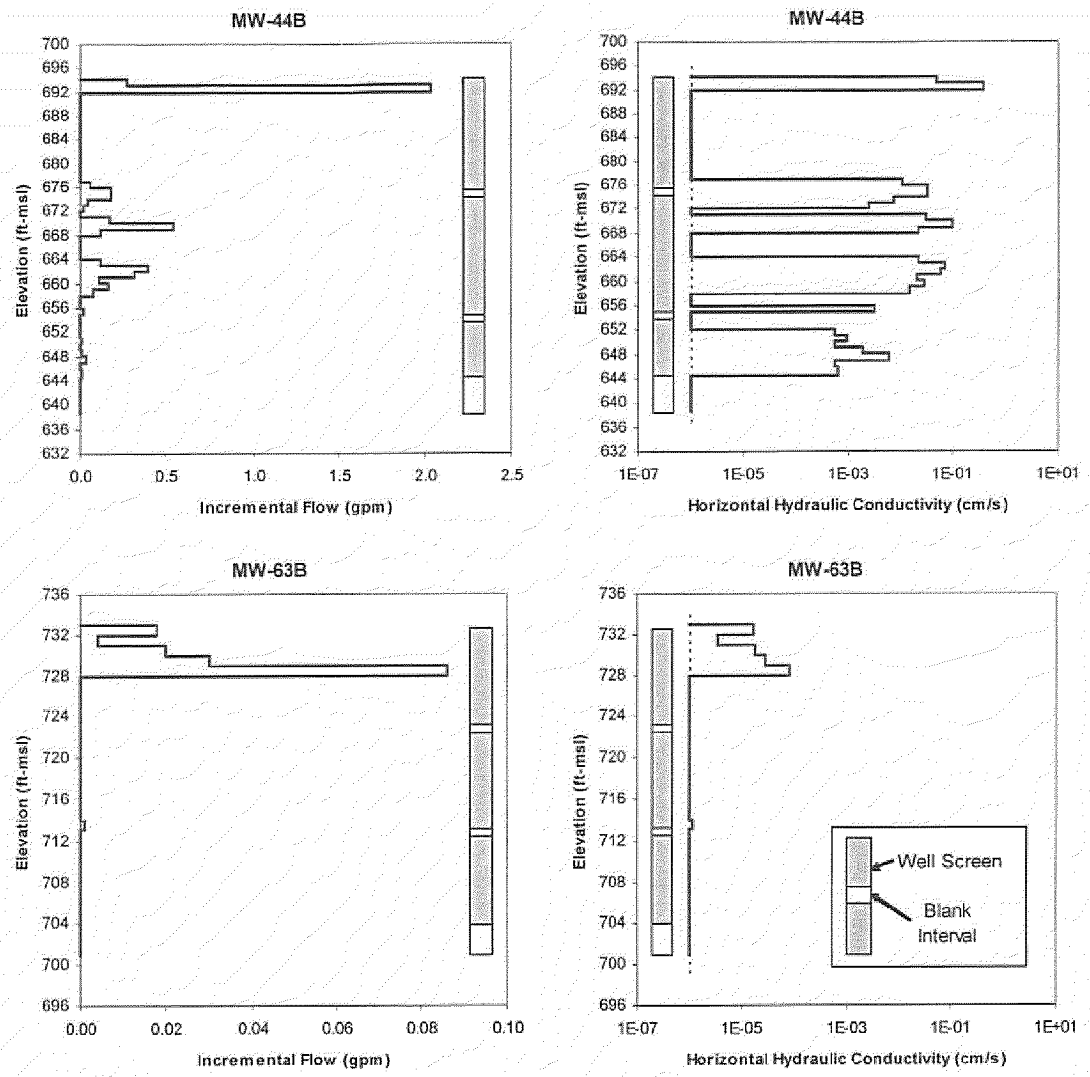

Figure 2-14. K_h Profiles from EMFM Testing at Bedrock Wells

2.5 Precipitation

In the absence of long-term precipitation records for the KIF site, precipitation data were obtained from the National Oceanic and Atmospheric Administration (NOAA) station in Oak Ridge, Tennessee, located some 20 miles northwest of the site. A continuous 20-year period (1968-87) of daily precipitation data was selected. Annual precipitation for the period ranged from 38.8 to 76.3 inches and averaged approximately 52.9 inches.

Thirty-minute precipitation data was collect locally at the Peninsula site for the period of January to October 2003. This data was supplemented with data from the KIF rain gage and another located at Melton Hill Dam. Figure 2-15 shows these data in time series.

3. LOCAL GROUNDWATER USE

A survey of local water use within an approximate two-mile radius of the center of the ash pond area was conducted in June 2005. The survey included interviews with local residents and utility district managers. Water well records maintained by the State of Tennessee were also examined for wells within the survey region. This survey identified a total of 32 residential wells. A listing of these wells and their coordinate locations is given in Table 3-1. Note that wells were numbered 1 through 32 with no well 15. One spring (Spring 1) was identified which provides untreated water for 10 to 12 residences along Swan Pond Road and for several residents of the Kingston Heights subdivision. The spring emanates from aquifers of the Knox Group. This spring appeared to be the only spring in the survey region used for water supply. Other residents within the survey region were served by one of the four local water utilities listed in Table 3-1. These utilities provide treated water from intakes on Watts Bar Lake or the Emory River. Figure 3-1 shows the Peninsula site and locations of water supplies in the region. As shown in the figure, there are no water supplies located within a one-mile radius of the site. Furthermore, considering that the site is hydraulically bounded on virtually all sides, there is no potential for offsite impacts to residential or municipal groundwater supplies.

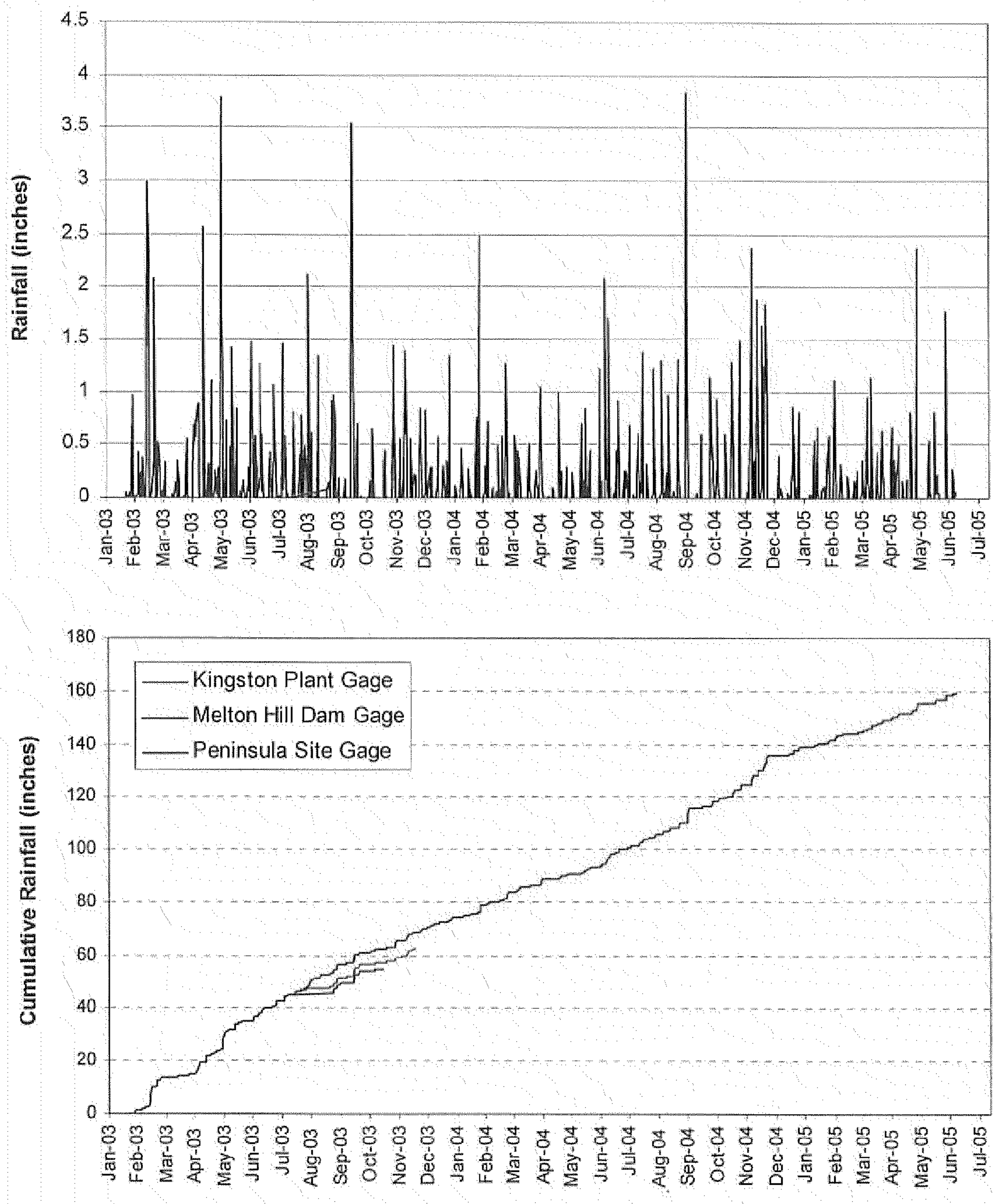

Figure 2-15. Rainfall Data

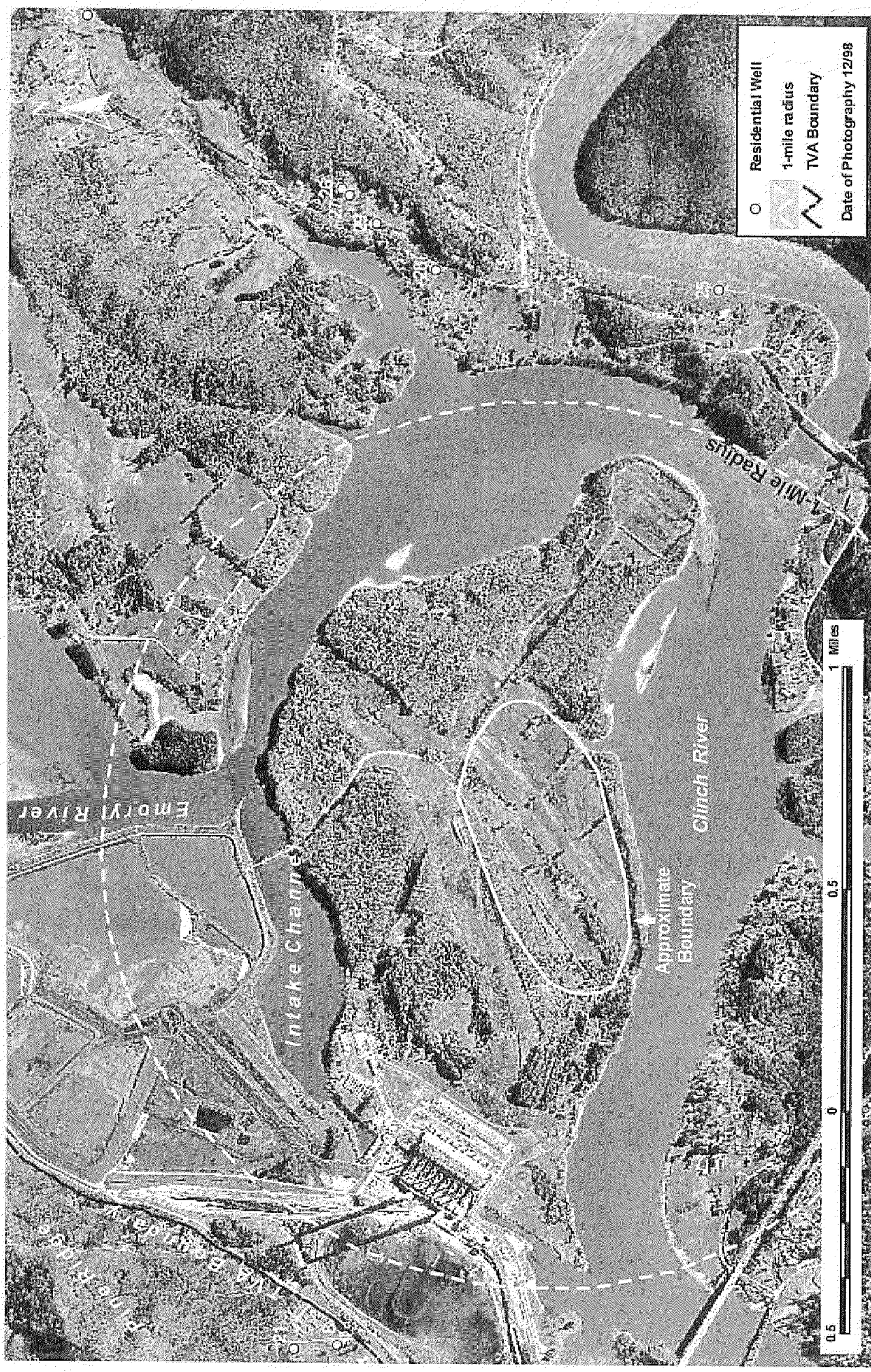

Figure 3-1. Offsite Well Map

Table 3-1. Offsite Wells, Springs, and Public Water Supplies in Site Vicinity

TVA Well ID	Location Description	Latitude (dd-mm-ss)	Longitude (dd-mm-ss)
Well 1	Swan Pond Rd south of Hwy 70	35-53-35 N	84-32-05.5 W
Well 2	Swan Pond Rd south of Hwy 70	35-53-34 N	84-32-09 W
Well 3	Swan Pond Rd south of Hwy 70	35-53-33 N	84-32-10.5 W
Well 4	North of Hwy 70, South of I-40	35-53-41.5 N	84-32-14 W
Well 5	Swan Pond Rd north of Hwy 70	35-53-44.5 N	84-32-09.5 W
Well 6	Swan Pond Rd north of Hwy 70	35-53-45 N	84-32-06 W
Well 7	Swan Pond Circle north of Swan Pond Rd	35-55-18 N	84-31-04.5 W
Well 8	Swan Pond Rd north of Hwy 70	35-54-06 N	84-31-31 W
Well 9	Swan Pond Rd north of Hwy 70	35-54-07 N	84-31-37 W
Well 10	Swan Pond Rd north of Hwy 70	35-54-00.5 N	84-31-41 W
Well 11	Swan Pond Rd north of Hwy 70	35-53-58.5 N	84-31-46 W
Well 12	Swan Pond Rd north of Hwy 70	35-54-00.5 N	84-31-50.5 W
Well 13	Could not locate, not able to confirm	35-53-52 N	84-31-47 W
Well 14	Swan Pond Rd north of Hwy 70	35-53-55 N	84-31-50 W
Well 16	Swan Pond Rd north of Hwy 70	35-53-53 N	84-31-53 W
Well 17	Swan Pond Rd north of Hwy 70	35-53-55 N	84-31-56 W
Well 18	Swan Pond Rd north of Hwy 70	35-53-52 N	84-31-58.5 W
Well 19	Swan Pond Rd north of Hwy 70	35-53-56 N	84-32-00 W
Well 20	Swan Pond Rd west of Swan Pond Circle	35-55-06.5 N	84-31-09 W
Well 21	Swan Pond Rd north of Hwy 70	35-54-11 N	84-31-31.5 W
Well 23	Hassler Mill west of Swan Pond Rd	35-54-43 N	84-31-54 W
Well 24	Sugar Grove Valley Rd east of KIF across the Emory River	35-54-34 N	84-28-19 W
Well 25	Sugar Grove Valley Rd east of KIF across the Emory River	35-53-20 N	84-28-59 W
Well 26	Sugar Grove Valley Rd east of KIF across the Emory River	35-54-04 N	84-28-44 W
Well 27	Sugar Grove Valley Rd east of KIF across the Emory River	35-54-03 N	84-28-45 W
Well 28	Sugar Grove Valley Rd east of KIF across the Emory River	35-53-53 N	84-28-56 W
Well 29	Sugar Grove Valley Rd east of KIF across the Emory River	35-54-00 N	84-28-49 W
Well 30	Youngs Creek Way east of the City of Kingston south of the Clinch River	35-53-05 N	84-28-06 W
Well 31	Dickey Valley Rd east of KIF across the Emory River	35-54-54 N	84-28-08 W
Well 32	Duncan Hollow Rd south of Hwy 70 west of the Clinch River	35-52-30 N	84-32-30 W
Spring 1	Near intersection of Swan Pond Rd and Frost Hollow Rd (used for portion of municipal supply by city of Kingston)	35-55-07 N	84-31-54 W
Rockwood Water System	Intake On Watts Bar Lake near Post Oak Creek	35-50-07 N	84-41-30 W
Cumberland Utility District	Watts Bar	35-58-01.9 N	84-27-58 W
Harriman Utility Board	Intake on Emory River Near Mile 13	35-56-07 N	84-33-32 W
Kingston Water System	Intake off Hwy 58 south of Kingston on Watts Bar Lake	35-51-24.9 N	84-31-50 W

4. CONCLUSIONS

Hydrogeologic conditions at the proposed disposal site appear to satisfy geologic and hydrologic standards for Class II disposal facilities. There is no evidence of Holocene-age faulting within the 200-ft facility exclusion zone. Although topographic expressions of dolines are exhibited at the site, these features do not possess open throats or avenues for reception of incipient recharge. Rather, the dolines are thickly mantled by soil thicknesses ranging from about 35 to 75 ft. Visual and laboratory classifications of these soils indicated that they are of residual origin except in the area of NB-21 and NB-44 (site pond) where alluvial deposition has occurred. There were no voids detected immediately above bedrock that would indicate stoping of soil into the deeper bedrock system. The facility poses no risk to existing or future groundwater users since there are no existing groundwater wells downgradient of the proposed facility, and there is no potential for future development of such wells since all downgradient property between the disposal site and surface water boundaries lies within the plant reservation. Two small areas within the proposed facility boundary reside within the 100-yr flood stage of the Clinch River and the natural geologic buffer zone within these areas is lacking. However, the proposed facility design includes plans for filling of these areas with suitable borrow soil. Furthermore, the current facility plan includes a bottom liner residing above the seasonal high groundwater elevation and an under-drain system to intercept leachate.

Groundwater monitoring for potential CCB leachate contaminants is anticipated to include several discrete locations within the geologic buffer zone immediately beneath the landfill liner. Although design of the complete groundwater monitoring network is dependent on the features of the final landfill design, it is expected that monitoring ports beneath the landfill will be situated at centroid and peripheral locations with horizontal conduit runs to sampling ports. Perimeter monitoring wells will be installed at critical locations along strike (east and west) of the disposal facility to complement those monitoring locations beneath the landfill. Upgradient wells are currently being installed at higher elevations of the site (ridge-line) that should serve to gage background groundwater quality. The final groundwater monitoring plan will be detailed in the facility operations plan.

5. REFERENCES

Bailey, C. M., "Major Faults & High-Strain Zones in Virginia," College of William and Mary, 2000.

Benziger, C.P., and J.M. Kellberg, 1951, "Preliminary Geological Investigations for Eastern Area Steam Plant," Division of Water Control Planning, Geologic Branch, Tennessee Valley Authority, Knoxville, Tennessee.

Boggs, J.M., S.C. Young, D.J. Benton, and Y.C. Chung, 1989, "Hydrogeologic Characterization of the MADE Site," Electric Power Research Institute Interim Report, EN-6915, Palo Alto, CA.

Boggs, J.M., and H.E. Julian, 1998, "Demonstration of the Electromagnetic Borehole Flowmeter at Federal Laboratories Area 15A, Saltsburg, Pennsylvania," TVA Engineering Laboratory Report WR98-1-520-200, February, Norris, Tennessee.

Bouwer, H., 1989, "The Bouwer and Rice Slug Test - An Update," Ground Water, vol.27, No. 3, pp. 304-309.

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

Carpenter, W.C., and C. Bohac, 1988, "Kingston Environmental Assessment for Fly Ash Disposal," TVA Engineering Laboratory Report, Norris, Tennessee.

Chen, J., S. Hubbard, and Y. Rubin, 2001, "Estimating the Hydraulic Conductivity at the South Oyster Site from Geophysical Tomographic Data Using Bayesian Techniques Based on the Normal Linear Regression Model," Water Resources Research, 37(6): 1603–1613, June.

Cooper, H.H., and C.E. Jacob, 1946, "A generalized graphical method for evaluating formation constants and summarizing well field history," Am. Geophys. Union Trans., vol. 27, pp. 526-534.

Dagan, G., 1982, "Analysis of Flow through Heterogeneous Random Aquifers, 2: Unsteady Flow in Confined Formation," Water Resources Research, 18(5):1571-85.

Dagan, G., 1984, "Solute Transport in Heterogeneous Porous Formations," Fluid Mechanics, 145(3):241-248.

Drier, R.B., D.K. Solomon, and C.M. Beaudoin, 1987, "Fracture Characterization in the Unsaturated Zone of a Shallow Land Burial Facility," Flow and Transport Through Fractured Rocks, American Geophysical Union Monograph 42.

Elci, A., G.P. Flach, and F.J. Molz, 2003, "Detrimental Effects of Natural Vertical Head Gradients on Chemical and Water Level Measurements in Observation Wells: Identification and Control," Journal of Hydrology, 281(70).

Flach, G.P., S.A. Crisman, and F.J. Molz III, 2004, "Comparison of Single-Domain and Dual-Domain Subsurface Transport Models," Ground Water, Vol. 42(6), November–December.

Ford, Derek, and Paul Williams, 1989, "Karst Geomorphology and Hydrology," Unwin Hyman, Ltd.

Gelhar, L.W., and C.L. Axness, 1983, "Three-dimensional Stochastic Analysis of Macrodispersion in Aquifers," Water Resources Research, 19(1):161-180.

Hess, A.E., 1982, "A Heat-Pulse Flowmeter for Measuring Low Velocities in Boreholes," U.S. Geological Survey Open-File Report 82-699, 40 pp, 1982.

Hess, A.E., 1986, "Identifying Hydraulically Conductive Fractures with a Slow-Velocity Borehole Flowmeter," Canadian Geotechnical Journal, 23, pp 69-78.

Hess, A.E., and F.L. Paillet, 1989, "Characterizing Flow Paths and Permeability Distribution in Fractured Rock Aquifers Using a Sensitive Thermal Borehole Flowmeter," in Proceedings of the Conference on New Field Techniques for Quantifying the Physical and Chemical Properties of Heterogeneous Aquifers, F.J. Molz, O. Güven, and J.G. Melville, ed., Water Resources Research Institute, Auburn University, AL.

Hess, A.E., and F.L. Paillet, 1990, "Applications of the Thermal-Pulse Flowmeter in the Hydraulic Characterization of Fractured Rocks," in ASTM STP 1101, pp 99-112.

Hufschmied, P., 1983, Ermittlung der Durchlassigkeit von Lockergesteins-Grundwasserleitern, eine vergleichende Untersuchcung verschiednener Feldmethoden, Ph.D. Dissertation 7397, Eidgenossische Technische Hochschule, Zurich, Switzerland.

Hufschmied, P., 1986, "Estimation of Three-Dimensional Statistically Anisotropic Hydraulic Conductivity Field by Means of a Single Well Pumping Test Combined with Flowmeter Measurements," Hydrogeologie, 2, 163-174.

Hvorslev, M.J., 1951, "Time Lag and Soil Permeability in Ground-Water Observations," Bul. No. 26, Waterways Experiment Station, Corps of Engineers, U.S. Army, Vicksburg, Mississippi.

Julian, H.E., 1994, "Site Assessment and Remediation Plan for TVA Centerville 161 kV Substation, Centerville, Tennessee," Tennessee Valley Authority, Resource Group, Chattanooga, Tennessee, November.

Julian, Hank E., 1996a, "Assessment of Groundwater Impacts from Releases of Diesel Fuel Oil at Bellefonte Nuclear Plant," TVA Engineering Laboratory Report, WR28-1-88-120, June.

Julian, H.E., 1996b, "Electromagnetic Borehole Flowmeter Application at Solid Waste Storage Area 5-North, Oak Ridge National Laboratory," TVA Engineering Laboratory Data Report, March.

Julian, H.E., and S.C. Young, 1994, "Measuring Well Development With an Electromagnetic Borehole Flowmeter," Proceedings of the Eight National Outdoor Action Conference, NGWA, Minneapolis, Minnesota, May.

Julian, H.E., S.C. Young, C. Lu, J.C. Herwiejer, and K.E. Richter, 1993, "NFERC Regional Groundwater Investigation," TVA Engineering Laboratory Report, WR28-1-520-191, April.

Julian, H.E., J.M. Boggs, C. Zheng., and C.E. Feehley, 2001, "Numerical Simulation of a Natural Gradient Tracer Experiment for the Natural Attenuation Study: Flow and Physical Transport," Ground Water, 39(4): 534-545, July-August.

LAW Engineering, Inc., 1995, "Use of Coal Combustion By-Products as Engineered Fills," LAW Engineering Project Nos. 50385-5-0400 (Phase 1) and 581086010, Atlanta, Georgia.

Lumsden, D.N., and G.C. Caudle, 2001, "Origin of Massive Dolostone: The Upper Knox Model," *Journal of Sedimentary Research*, 71(3): 400-409, May.

MACTEC Engineering and Consulting, Inc., 2003, "Report of Geotechnical Exploration - Proposed Scrubber Stack Disposal Area – Kingston Fossil Plant - Kingston, Tennessee," MACTEC Project 3043031008/0001, Knoxville, Tennessee.

MACTEC Engineering and Consulting, Inc., 2004, "Report of Geotechnical Exploration - Ash Disposal Area - Kingston Fossil Plant - Kingston, Tennessee," Knoxville, Tennessee.

MACTEC Engineering and Consulting, Inc., October 2005, "Report of Geotechnical Exploration – Proposed Gypsum Disposal Area – Kingston Fossil Plant – Kingston, Tennessee," MACTEC Project 3043051021.01, Knoxville, Tennessee.

Milligan, J.D., and R.J. Ruane, 1980, "Effects of Coal-Ash Leachate on Ground Water Quality," EPA-600/7-80-066.

Molz, F.J., R.H. Morin, A.E. Hess, J.G. Melville, and O. Güven, 1989a, "The Impeller Meter for Measuring Aquifer Permeability Variations: Evaluations and Comparison With Other Tests," Water Resources Research, Vol. 25, pp 1677-1683.

Molz, F.J., O. Güven, J.G. Melville, and C. Cardone, 1989b, "Hydraulic Conductivity Measurement at Different Scales and Contaminant Transport Modeling: Dynamics of Fluids in Hierarchical Porous Media", J.H. Cushman, ed., Academic Press, New York, pp 37-59.

Molz, F.J., O. Güven, and J.G. Melville (with contributions by I. Javandel, A.E. Hess and F.L. Paillet), 1990, "A New Approach and Methodologies for Characterizing the Hydrogeologic Properties of Aquifers," Robert S. Kerr Environmental Research Laboratory, Ada, OK, EPA/600-2-90/002, 205 pp.

Moore, G.K., and S.C. Young, 1992, "Borehole Flowmeter Data and Interpretation," ORNL/ER-91, Oak Ridge, Tennessee.

Morin, R.H., A.E. Hess, and F.L. Paillet, 1988, "Determining the Distribution of Hydraulic Conductivity in Fractured Limestone Aquifers by Simultaneous Injection and Geophysical Logging," Ground Water, Vol. 26, pp 587-595.

Powell, C. A., G. A. Bollinger, M. C. Chapman, M. S. Sibol, A. C. Johnston and R. L. Wheeler, "A Seismotectonic Model for the 300 Kilometer Long Eastern Tennessee Seismic Zone," Science, v. 264, pp. 686-688, 1994.

Rehfeldt, K.R., L.W. Gelhar, J.B. Southard, and A.M. Dasinger, 1989a, "Estimates of Macrodispersivity Based on Analysis of Hydraulic Conductivity Variability at the MADE Site," EPRI Interim Rep., EN-6405, Electric Power Research Institute, Palo Alto, California.

Rehfeldt, K.R., P. Hufschmied, L.W. Gelhar, and M.E. Schaefer, 1989b, "Measuring Hydraulic Conductivity with the Borehole Flowmeter," EPRI Top. Rep., EN-6511, Electric Power Research Institute, Palo Alto, California.

Rehfeldt, K.R., J.M. Boggs, and L.W. Gelhar, 1992, "Field Study of Dispersion in a Heterogeneous Aquifer, 3: Geostatistical Analysis of Hydraulic Conductivity," Water Resources Research, 28(12):3309-3325.

Singleton, 1988, "Kingston Steam Plant Ground Water Assessment," SME-SOI-88-006, Division of Nuclear Engineering, Materials Technology Branch, Singleton Materials Engineering Laboratory, Knoxville, Tennessee.

Skibitzke, H.E. and G.M. Robertson, 1963, "Dispersion in Groundwater Flowing Through Heterogeneous Materials," U.S. Geological Survey Prof. Paper 386-B.

Sledz, J.J., and D.D. Huff, 1981, "Computer Model for Determining Fracture Porosity and Permeability in the Conasauga Group," ORNL/TM-7695, Oak Ridge National Laboratory.

Solomon, D.K., G.K. Moore, L.E. Tovan, R.B. Dreier, and W.M. McMaster, 1992, "Status Report - A Hydrogeologic Framework for the Oak Ridge Reservation," ORNL/TM-12026, Oak Ridge National Laboratory, May.

Taylor, K., S.W. Wheatcraft, J. Hess, J.S. Hayworth, and F.J. Molz, 1990, "Evaluation of Methods for Determining the Vertical Distribution of Hydraulic Conductivity," Ground Water, 27, pp 88-98.

Theis, C.V., 1935, "The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage," Am. Geophys. Union Trans., vol. 16, pp. 519-524.

TVA, 1950, "Preliminary report on the Kingston site for Steam Plant A," Tennessee Valley Authority, Knoxville, Tennessee.

USACE, 1924, "Topographic Map of Clinch and Emory River Valleys," U.S. Army Corp of Engineers Map, TVA Map Files.

Velasco, M.L., and C.E. Bohac, 1991, "Kingston Groundwater Assessment," TVA Engineering Laboratory, Report No. WR28 1-36-115.

Young, S.C., and W.R. Waldrop, 1989, "An Electromagnetic Borehole Flowmeter for Measuring Hydraulic Conductivity Variability," in Proceedings of the Conference on New Field Techniques for Quantifying the Physical and Chemical Properties of Heterogeneous Aquifers, F. J. Molz, O. Güven, and J. G. Melville, eds., Water Resources Research Institute, Auburn University, AL.

Young, S.C., R. Schmidt-Petersen, M. Ankeny, and D.B. Stephens, 1993a, "Physical and Hydraulic Properties of Fly Ash and Other By-Products from Coal Combustion," Electric Power Research Institute Report TR 101999, Project 2485-05.

Young, S.C., H.E. Julian, and M.J. Neton, 1993b, "Application of the Electromagnetic Borehole Flowmeter and Evaluation of Previous Pumping Tests at Paducah Gaseous Diffusion Plant," TVA Engineering Laboratory, Report No. WR28-1-520-187, January.

Young, S.C., H.E. Julian, H.S. Pearson, F.J. Molz, and J.K. Bowman, 1997, "User's Guide for Application of the Electromagnetic Borehole," U.S. EPA report, Robert S. Kerr Environmental Research Laboratory, Ada, OK, Report in Press.