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Weldon Spring Site Remedial Action Project

WSSRAP Chemical Plant
Geotechnical Investigations

December 1990

Revision 0

Prepared by

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1 INTRODUCTION

1.1 General

This document has been prepared for the United States Department of Energy (DOE) Weldon Spring Site Remedial Action Project (WSSRAP) by the Project Management Contractor (PMC), which consists of MK-Ferguson Company (MKF) and Morrison Knudsen Corporation Environmental Services Group (MKES) with Jacobs Engineering Group (JEG) as MKF's predesignated subcontractor. MKES provided most of the field and technical support to the PMC for the geotechnical investigations and for the preparation of this document. Since MKF and MKES are both subsidiaries of the Morrison Knudsen Corporation, the distinction between the activities of each organization is not always specified in this document.

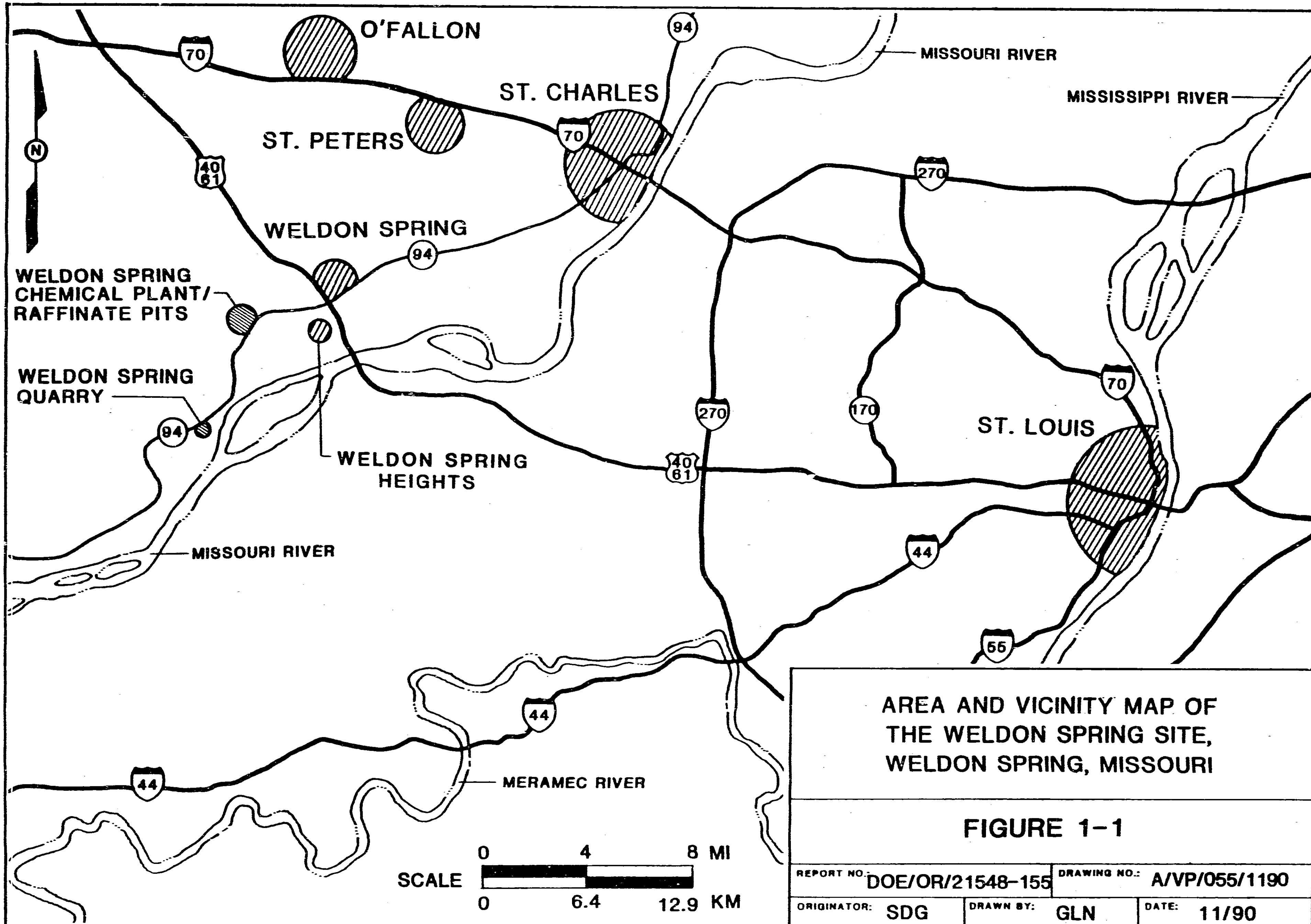
This report presents the results of site geotechnical investigations conducted by the PMC in the vicinity of the Weldon Spring chemical plant and raffinate pits (WSCP/RP) and in potential on-site and off-site clayey material borrow sources. The WSCP/RP is the proposed disposal cell (DC) site.

The proposed disposal cell will be used for long-term storage of contaminated waste materials generated from clean up of the Weldon Spring site. The geotechnical investigations were part of the field data collection for the remedial investigation/feasibility study (RI/FS) process. This process is specified by the National Contingency Plan (NCP) which requires investigation and evaluation of waste sites. The data will also be used to support RI documentation (MKF and JEG 1990c), the site suitability study (MKF and JEG 1990b), and the design of the proposed disposal cell, including temporary facilities. The investigation program generally followed the rationale and procedures identified in the Geophysical/ Geotechnical Investigation Sampling Plan (MKF and JEG 1988b). Results of previous geotechnical investigations of the site and in-progress PMC RI efforts were reviewed and are discussed in sections on geologic and site characterization of the subsurface materials.

As illustrated in Figure 1-1, the Weldon Spring site is located approximately 30 miles west of St. Louis, Missouri, near the town of Weldon Spring in southwestern St. Charles County. The proposed disposal cell site lies just west of Missouri State Highway 94, approximately 2 miles southwest of its junction with U.S. Highway 40 (Interstate 64) near the communities of Weldon Spring and Weldon Spring Heights. The Weldon Spring site also includes the Weldon Spring quarry (QY) located approximately 4 miles south-southwest of the proposed disposal cell site. The geotechnical investigations at the quarry site, which were conducted in conjunction with investigations at the proposed DC site, are addressed in a separate report (MKF and JEG 1990c). Figure 1-2 shows the existing facilities at the site.

The investigations were conducted in two phases. Phase I began in June 1988 and concluded in August 1988. These investigations included drilling in the west dike of raffinate pit No. 4, in the proposed administration building footprint (now constructed), and in the proposed disposal cell area. On-site test pits from which clay or fill material might be borrowed were excavated by a backhoe. Surface geophysical surveys were also performed.

Phase II investigations began in January 1989 and were completed in August 1990. These investigations included drilling angled boreholes in the proposed disposal cell area, drilling



AREA AND VICINITY MAP OF THE WELDON SPRING SITE, WELDON SPRING, MISSOURI

FIGURE 1-1

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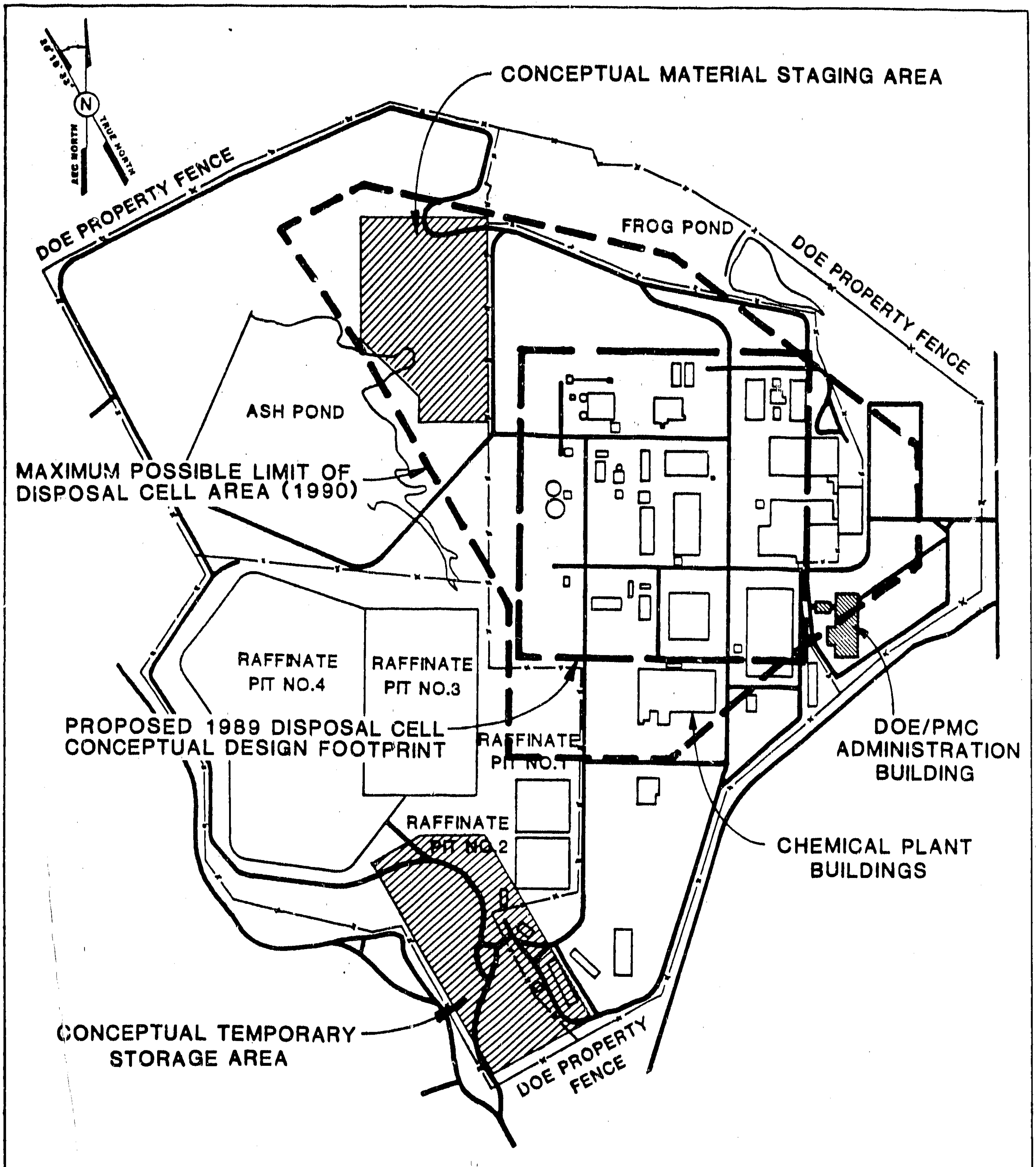

NOTE: FACILITIES SHOWN ARE EXISTING AS OF 1986 UNLESS AS INDICATED

SOURCE: MK-F & JEG, 1990

SCALE 0 500 1000 FT
0 152.4 304.8 M

WELDON SPRING CHEMICAL PLANT/
RAFFINATE PIT SITE PLAN

FIGURE 1-2

REPORT NO. DOE/OR/21548-155		DRAWING NO.: A/CP/099/1190
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boreholes and installing piezometers within and adjacent to the proposed disposal cell area, drilling in the proposed temporary storage area (TSA), and excavating test pits in two potential off-site areas for clay borrow sources. Additional work in Phase II consisced of excavating test pits in the material staging area (MSA) in August 1990.

A summary of these investigations is listed in Table 1-1. Locations of boreholes, piezometers, wells, pits, and geophysical survey lines for Phase I are shown in Figure 1-3; those for Phase II and the additional MSA investigations are shown in Figure 1-4. Off-site borrow test pit locations are shown in Figure 1.5.

In addition to the above activities, the geotechnical site investigations included the relogging of rock cores from boreholes drilled during previous investigations.

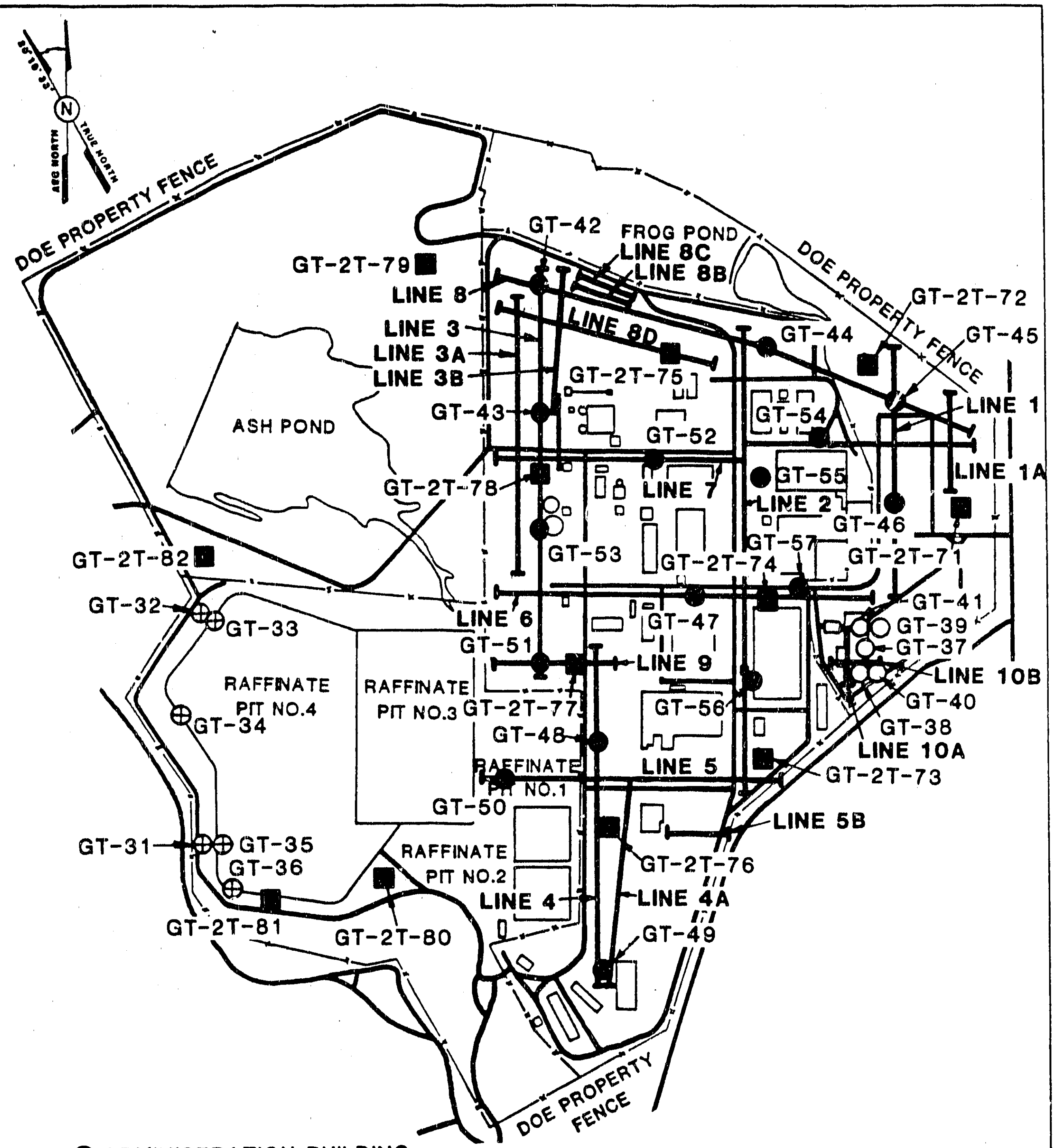
1.2 Purpose and Scope

The purpose of this document is to present the geotechnical field and laboratory data that were obtained to characterize the overburden materials in support of proposed remedial action activities for the disposal cell. Engineering parameters, analyses, and recommendations regarding the subsurface soils and foundation are not part of this report, but will be prepared during a separate special study. The site plan in Figure 1-2 shows the maximum limits and the proposed 1989 conceptual footprint of the disposal cell and related facilities. The footprint shown represents its planned location at the time of planning and execution of the field investigation program (1988-1989). The maximum possible limit shown represents the maximum size of a disposal cell at this site as determined by the PMC in 1990.

TABLE 1-1 Summary of PMC Geotechnical Field Investigations at the Weldon Spring Site

Location	Data Sources	Text References	Purpose
PHASE I (June - Aug, 1988)			
Raffinate pit No. 4	Borings and Piezometers GT-31 to GT-36	Figure 1-3 & Appendix A-1	Dike stability; groundwater monitoring.
Proposed administration building	Borings GT-37 to GT-41	Figure 1-3 & Appendix A-2	Foundation soil parameters.
Proposed disposal cell	Borings GT-42 to GT-57	Figure 1-3 & Appendix A-3	Soil and bedrock parameters; corroborate geophysical survey results.
Various, on site	Test pits GT-2T71 to GT 2T82	Figure 1-3 & Appendix A-4	On-site clay borrow materials.
Proposed disposal cell	Geophysical survey lines #1 to 10	Figure 1-3 *See Note	Subsurface geophysical model.
PHASE II (Jan - Sept, 1989)			
Proposed disposal cell	Angle borings AH-1, AH-2	Figure 1-4 Appendix B-1	Bedrock structure and hydraulic conductivity.
Proposed temporary storage area	Borings GTS-1 to GTS-7	Figure 1-4 Appendix B	Foundation soil parameters.
Proposed disposal site perimeter	Borings and piezometers GT-58 to GT-67	Figure 1-4 Appendix B-3	Soil and bedrock parameters; groundwater monitoring.
Potential off-site clayey borrow source (SSE of DC site)	Test pits TPBS-1 to PBS-30	Figure 1-5 Appendix B-4	Off-site clay borrow materials suitability.
ADDITIONAL WORK TO PHASE II (Aug 1990)			
Proposed materials staging area	Test pits TPMS-1 to TPMS-9	Figure 1-4	Soil parameters.

NOTE * Geophysical survey conducted by Geotechnology Services, Inc., 1988. See reference report.



○ ADMINISTRATION BUILDING BOREHOLE (LOCATION APPROXIMATE)

⊕ RAFFINATE PIT NO.4 DIKE BOREHOLE

● DISPOSAL CELL BOREHOLE/COREHOLE

■ PLANT SITE TEST PITS

GEOPHYSICAL SURVEY LINE

SCALE 0 500 1000 FT

0 152.4 304.8 M

PHASE I FIELD INVESTIGATIONS LOCATION MAP

FIGURE 1-3

REPORT NO.: DOE/OR/21548-155		DRAWING NO.: A/CP/100/1190	
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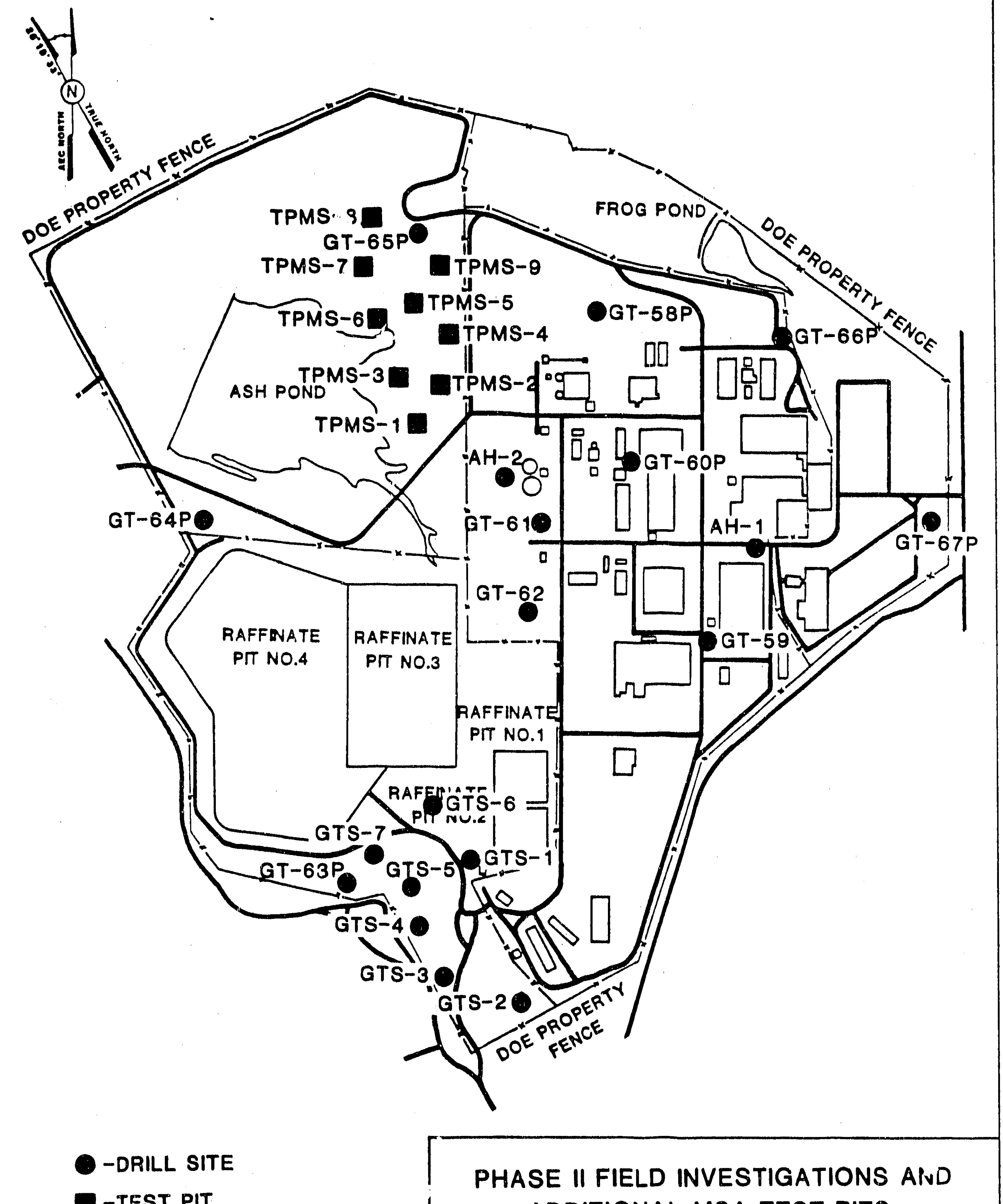

PHASE II FIELD INVESTIGATIONS AND ADDITIONAL MSA TEST PITS LOCATION MAP

FIGURE 1-4

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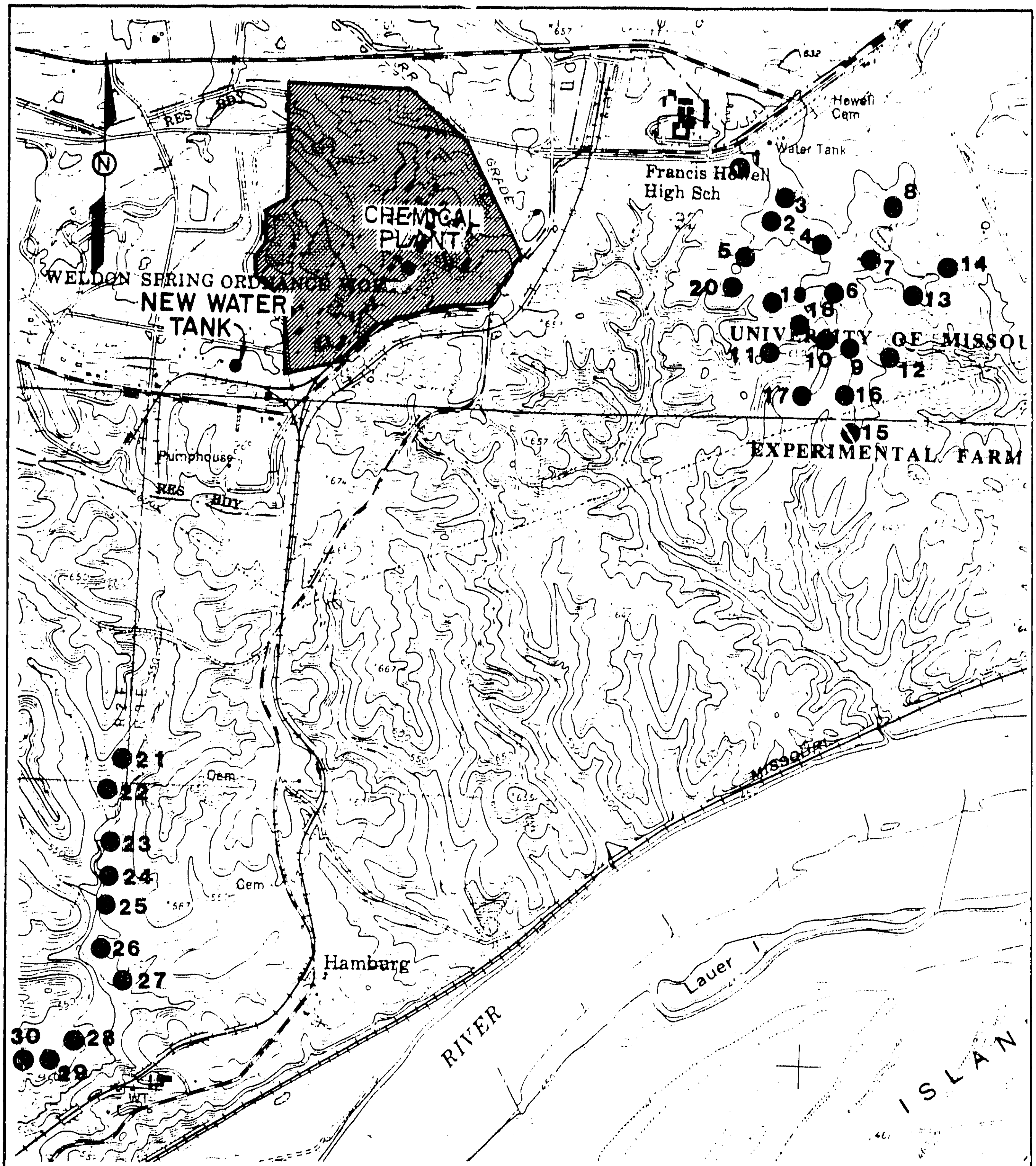

8 ● TEST PIT LOCATION (TPBS-8)

0	2000	4000 FT
0	609.6	1219.2 M

SCALE

SOURCE MAP: USGS QUAD MAP

OFF-SITE CLAY BORROWS SOURCES
TEST PIT LOCATION MAP

FIGURE 1-5

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Geotechnical field investigations conducted by the PMC began in June 1988 and were completed in August 1990. Field activities included overseeing borehole drilling, piezometer installation, test pit excavation, and geophysical survey. Soil samples collected during field investigations were sent to subcontractor laboratories for testing under the direction of the PMC. Laboratory testing began in June 1988 and was completed in October 1990.

The PMC geotechnical investigation program provided data for the following items:

- Foundation engineering parameters for the design of the administration building.

- Subsurface characterization and engineering parameters for the site suitability study and design of the disposal cell (1989 version).

- Foundation engineering parameters for temporary storage area design.

- Foundation engineering parameters for material staging area design.

- Dike stability assessment of raffinate pit Nos. 3 and 4.

- Potential off-site borrow sources for clayey material.

- Potential utilization of uncontaminated on-site soils for construction materials.

- Groundwater elevations.

- Relogging of selected core samples from previous investigations.

A description of the purpose and the number of boreholes or test pits investigated for each item is included in later sections. The original scope has been modified several times to support the changing needs of the RI/FS and RA process as work progresses. With the exception of the preparation of this report, the execution of the material staging area investigation program, and the reduction of laboratory test data and results, most work on the above items was performed between June 1988 and December 1989.

This document was prepared at various stages concurrently, and occasionally in conjunction with, the site RI report (MKF and JEG 1990c) and the site suitability study (MKF and JEG 1990b). Parts of this document may duplicate parts of these two reports. In such cases, references to these reports are not cited.

The geotechnical data and discussions included in this report may be used to support further remedial investigations and design activities at the Weldon Spring site.

1.3 Background and Proposed Construction

The Weldon Spring site contains abandoned buildings, equipment, facilities, and ground surfaces that were used at different times by various organizations to produce or process trinitrotoluene (TNT), dinitrotoluene (DNT), uranium feed materials, and scrap metals. The site has been under DOE custody since 1969 when responsibility for the raffinate pits was first transferred to the DOE. Since then, the DOE has been given management responsibilities for the quarry (in 1981) and for the chemical plant (in 1985). Table 1-2 is a summary chronology of the WSS.

The proposed remedial actions for the WSSRAP include dismantling buildings and facilities; clean up and disposal of wastes from the quarry, raffinate pits, chemical plant, and vicinity properties; and closure of the site. The exact disposal method or methods will not be finalized until the feasibility study report and Record of Decision (ROD) are issued. The geotechnical investigation was planned on the basis of the conceptual design of the disposal cell, which is currently in progress. A brief description of the proposed disposal cell and major temporary facilities (see Figure 1-2) follows to provide readers with background information.

- Disposal Cell - The proposed disposal cell will be designed to receive all solid wastes from the Weldon Spring site, quarry, and vicinity properties, and to isolate them from the environment in compliance with State and Federal requirements. Estimated waste volume in the 1989 conceptual design amounts to approximately 900,000 cubic yards. This estimate will most likely be revised as the RI/FS effort continues. The features of the disposal cell will include bottom liners (earth and synthetic), a leachate collection and detection system, and cover components including a low permeability/radon barrier layer, a bio barrier, a filter layer, and a erosion protection layer. The cell will be designed to have a top slope of approximately 2% to 4% and a side slope of approximately 20%. The cell will be located in the northern part of the chemical plant area.

TABLE 1-2 Summary Chronology of Weldon Spring Site (Sheet 1 of 4)

DATE	SITE STATUS	
1941 to 1945	Property (17,232 acres) acquired by U.S. Department of the Army (USDA), 1941. Atlas Powder Company operates the plant known as Weldon Spring Ordnance Works (WSOW), 1941 - 1944.	Production of explosives TNT and DNT during World War II. Temporary waste storage in seven lagoons; four lagoons drained and filled, three others drained; wastewater treatment plans constructed and product sludges incinerated, 1941-1943.
1946	WSOW declared surplus property; first attempts at decontamination by Atlas Powder Company and by U.S. Army Corps of Engineers (USCOE) Kansas City District.	Processing equipment declared contaminated. USCOE removes over 3,500 cy dirt, burns more than 113,000 lbs TNT and more than 80,000 lbs TNT residues; some buildings removed or destroyed; location of disposed waste material not known.
1947 to 1956	WSOW property disposed of as follows: 1947-1949: - University of Missouri (U of M): 7,920 acres. - Missouri Department of Conservation: 7,200 acres from U of M for Weldon Spring Wildlife Area; 6,944 acres for Busch Wildlife Area	No recorded waste disposal activities (1947-1955). Land transferred to AEC partially decontaminated; WSUFMP land to revert to USDA at end of AEC operations. WSUFMP operations lasted until 1966.

TABLE 1-2 Summary Chronology of Weldon Spring Site (Continued) (Sheet 2 of 4)

DATE	SITE STATUS	
1957 to 1965	- St. Charles County Public Schools: 38 acres for Francis Howell High School. - General Services Administration transfers 2,063 acres to USDA 1950; 205 acres of this transferred to Atomic Energy Commission (AEC) for Weldon Spring Uranium Feed Materials Plant (WSUFMP), 1955. AEC acquires abandoned quarry (WSQ) from USDA for use as low-level radioactive waste disposal site, 1958. After transfer to AEC, the remaining 1,860 acres designated as Weldon Spring U.S. Army Reserve Training Area, 1959.	Mallinckrodt Chemical Works operates WSUFMP producing uranium metal from ore; plant contains numerous buildings and two waste (raffinate) pits, 1957-1966. Third raffinate pit constructed, 1958. Waste from dismantling of Mallinckrodt Destrehan Street Plant disposed of in quarry, 1960-1963. USDA disposes of uranium- and thorium-contaminated wastes from Granite City arsenal until waste purchased for reprocessing and removed, 1963-1965. Fourth raffinate pit constructed on land transferred to AEC, 1964.
1965 to 1966	AEC closes WSUFMP (end of 1966).	Thorium oxide processed at WSUFMP; wastes deposited in raffinate pit No. 4. Thorium residues from Cincinnati (approximately 556 cy) disposed of at site; wastes covered by TNT-contaminated material disposed of by USDA. WSUFMP cleanup wastes disposed of in raffinate pit No. 4.

TABLE 1-2 Summary Chronology of Weldon Spring Site (Continued)
(Sheet 3 of 4)

DATE	SITE STATUS	
1967 to 1970	AEC transfer 205 aces of WSUFMP to USDA. AEC maintains control of Building 438 and 51 acre tract with raffinate pits. In 1967 USCOE contracts for design and construction of herbicide facility known as Weldon Spring Chemical Plant (WSCP) 1967. The plant remained under control of USDA; the raffinate pits (WSRP) declared excess property in 1970.	USDA prepares for production of herbicides. Thompson-Stearns-Roger Corporation designs herbicide plant; in 1968 former WSUFMP facilities to be decontaminated and removed. Approximately 5,560 cy of contaminated wastes dumped at the quarry, 1968-1969. WSCP herbicide project canceled, 1969.
1972	St. Charles County purchases well field previously used for WSOW. Well field is located south of the quarry.	No recorded waste disposal activities.
1975 to 1979	AEC contracts National Lead company of Ohio to do environmental monitoring and maintenance of WSCP and quarry.	Preliminary assessment of environmental impacts at WSCP, 1975. Additional characterization deemed necessary. Phase I - Phase III: Installation Restoration Assessment study by Ryckman, Edgerly, Tomlinson and Associates, 1977-1979.
1981 to 1985	Department of Energy (DOE, successor to AEC) contracts Bechtel National, Inc., for management and maintenance of WSRP and quarry, 1981. DOE assumes custody and accountability of the WSCP from USDA, 1984-1985. DOE designates WSCP and WSRP decontamination as major project, 1985.	No recorded waste disposal activities. Site remedial investigations by Bechtel, Berkeley Geoscience Associates.

TABLE 1-2 Summary Chronology of Weldon Spring Site (Continued)
(Sheet 4 of 4)

DATE	SITE STATUS	
1986 to 1989	MK-Ferguson Company selected as Project Management Contractor for Weldon Spring Remedial Action Project, 1986. DOE project office established on site, 1986. Quarry site included in National Priorities (Superfund) List 1987; WSCP and WSRP added to list and all areas redesignated as Weldon Spring Site, 1989. Source: MK-Ferguson and Jacobs Engineering Group, 1989.	MK-Ferguson assumes control of Weldon Spring site; remedial investigations and characterization started, 1986. Site characterization to be completed in 1989.

- Temporary Storage Area - The proposed temporary storage area (TSA) will be constructed to stockpile wastes and rubble from the quarry until they are placed in the disposal cell. The TSA will be located south of the raffinate pits as shown in Figure 1-2. A low permeability foundation and liner are desirable features for the design of the TSA.

- Material Staging Area - A material staging area (MSA) is proposed near the northern part of the site (Figure 1-2). The MSA will be used for segregating and classifying materials from building dismantlement pending a decision on their ultimate disposition.

- Administration Building - The administration building (constructed in 1989) was designed to house the DOE and PMC site offices. The building is a steel-framed one story building and is located near the eastern part of the site.

- Other pertinent facilities will include a sludge treatment plant, a water treatment plant, holding ponds, and decontamination facilities. The exact locations of these facilities had not been identified at the time of the field investigations.

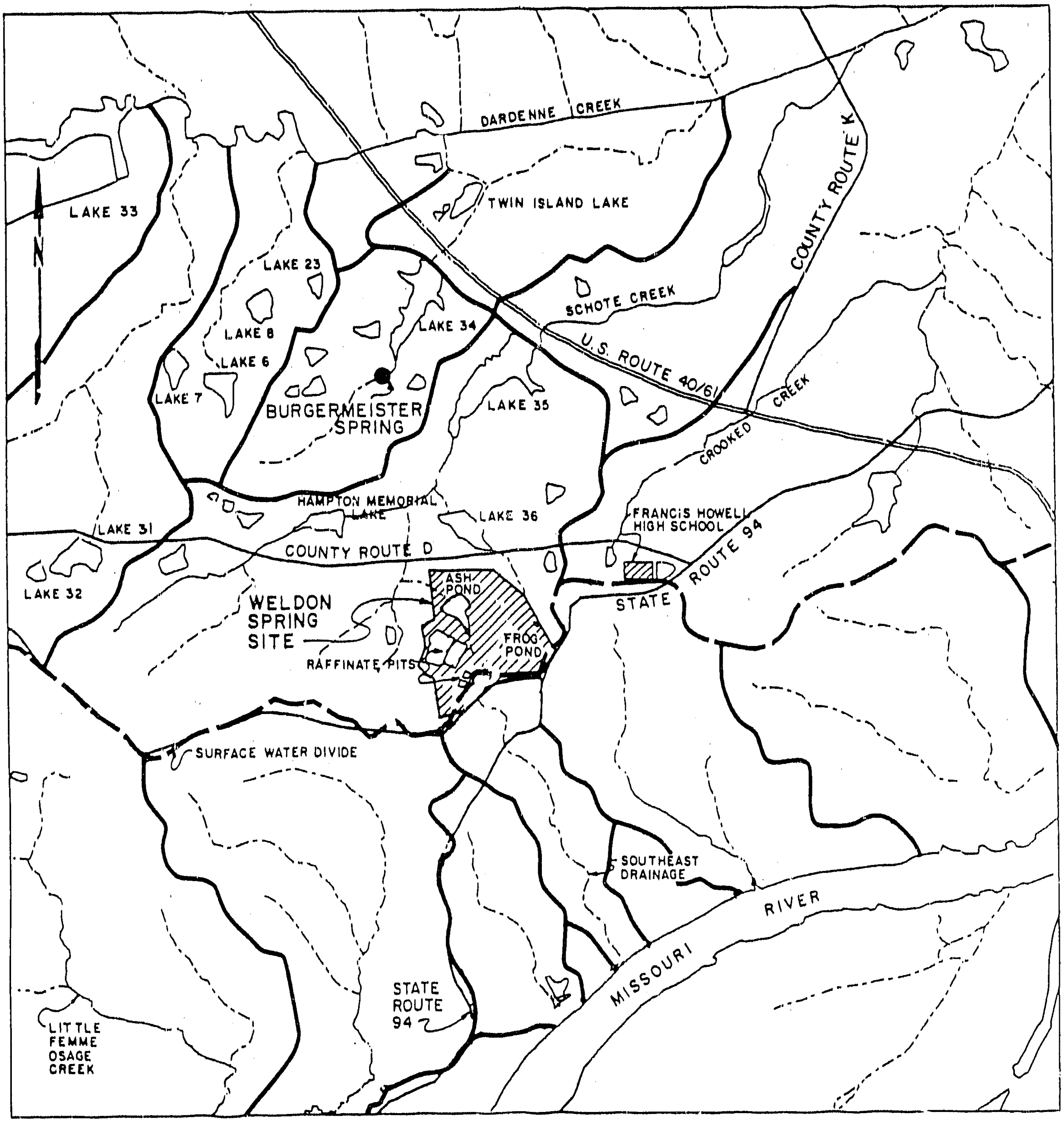
2 SITE CONDITIONS

2.1 Surface Features

The Weldon Spring site is located in southwestern St. Charles County, Missouri, which is a roughly triangular area bounded by the Mississippi River on the north and east and the Missouri River on the south. The uplands in the southwestern half of the county and the area south of the Weldon Spring site (WSS) are rugged and crossed by steep sided, narrow streams (MGS 1977). Figure 2-1 shows the major drainages in the vicinity of the WSS.

The WSS lies on an upland ridge of low relief, with gentle rolling terrain shaped by late Pleistocene (Wisconsinan Stage) glaciation. The furthest advance of these glaciers reached the south edge of the site near the drainage divide between the Mississippi and Missouri Rivers (Figure 2-1). The site straddles this surface drainage divide. Most of the site is drained by Schote Creek and its tributaries, which flow north through a series of lakes into Dardenne Creek, a tributary of the Mississippi River. The Dardenne Creek drainage is characterized by broad floodplains draining gently rolling topography (MGS 1977). The southeastern portion of the site drains southward through several unnamed tributaries to the Missouri River, approximately 1.5 miles to the southeast.

The site topography is generally flat, sloping gently from the higher elevations of about 672 feet mean sea level (MSL) along the south edge of the site to lower elevations of about 608 feet MSL on the north end of the site. Abrupt changes in surface elevation occur in the areas that were disturbed during



LEGEND:

SURFACE WATER DIVIDE BETWEEN MISSISSIPPI RIVER AND MISSOURI RIVER

DRAINAGE BOUNDARY

CREEK OR SURFACE DRAINAGE

POND OR LAKE

0 3200 6400 FT

0 975.4 1950.7 M

SCALE

SURFACE WATER DRAINAGES NEAR THE WSS

FIGURE 2-1

REPORT NO.: DOE/OR/21548-155	EXHIBIT NO.: A/VP/057/1190	
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construction of the raffinate pits and the Ash Pond dikes. The dikes rise up to 20 feet above the surrounding ground surface. The major existing and proposed site features are shown in Figure 1-2.

No natural drainages traverse the site, except for remnants of a filled channel through the Ash Pond area (MKF and JEG 1988a). However, three man-made structures influence site drainage. Ash Pond was formed by construction of a dike across the northwest side of a topographic depression. In 1989, a diversion channel was constructed around the pond to prevent surface water from flowing into it and mixing with contaminated soil and water. When water draining the northern portion of the site reaches the spillway elevation of the Ash Pond collection basin, it flows into the diversion channel and off the WSS via a tributary to Schote Creek. Also, along the north side of the chemical plant area a sloped embankment was built to provide a level area for coal storage and a basin for runoff in the area of the steam plant (BNI 1987). Finally, Frog Pond is a settling basin located near the eastern edge of the site. Overflow water from this pond flows off site to the north into the August A. Bush Memorial Wildlife Area, and eventually into the Schote Creek drainage system (Figures 1-2 and 2-1).

Other man-made features at the WSS consist chiefly of buildings and associated structures used in the chemical plant operations. Many buildings and facilities were destroyed after the completion of each phase of operations at the site, especially those associated with the Weldon Spring Ordinance Works (WSOW). Currently, there are approximately 13 major buildings, 30 support structures, and other miscellaneous facilities and equipment at the WSS. Most of these facilities have been abandoned. The facilities currently in use include office trailers and the newly constructed administration building. Portions of the site other than the buildings, pits, and ponds are generally covered with gravel, asphalt, pavement, grass, or natural vegetation. The wildlife areas surrounding the WSS are predominantly covered with brush, grasses, and trees. The agricultural areas to the east and southeast are open fields with stands of trees and brush generally found along steams.

Flooding at the WSS by heavy spring and early summer rains is unlikely because the site is at the headwaters of most of the small tributary streams that either flow north to Schote Creek and the Mississippi River or south to the Missouri River. Although the Mississippi and Missouri Rivers have in the past often reached flood stage from April through July of each year, the potential for these rivers flooding the site is extremely low because the site elevation rises from 608 feet MSL at its lowest point. The maximum recorded flood on the Missouri River occurred during June 1844 and reached a peak water surface elevation of 473 feet MSL in the stretch of river immediately south and east of the site (MK-Engineers 1988). Since the Missouri River now has numerous flood control facilities, the possibility of a flood of a similar magnitude is remote.

2.2 Geology and Hydrogeology

The materials presented in this section have been compiled and summarized from various references and previous investigations including recent PMC investigations for the RI/FS process.

2.2.1 Regional Setting

The Weldon Spring Site lies on the south edge of the dissected till plains of the Central Lowland Physiographic Province. The site is on an east-west trending low ridge that forms a drainage divide between the Missouri and Mississippi Rivers. This ridge forms the approximate boundary between the glaciated region to the north and the Salem Plateau Physiographic Province to the south. These dissected till plains are broad, flat to gently rolling upland areas. They were eroded nearly flat by glacial scouring and filling during the late Pleistocene (Wisconsinan Stage) glaciation. The Salem Plateau to the south was not glaciated and is characterized by steep, rugged uplands and heavily wooded hills and ridges dissected by narrow streams with steep, rocky slopes (MGS 1977).

2.2.2 Stratigraphy

The stratigraphy of the Weldon Spring site is composed of approximately 2,000 to 3,000 feet of Paleozoic Era marine limestone, dolomite, sandstone, and shale. It has been described in detail in the draft Remedial Investigation Report (MKF and JEG 1990c). Other detailed descriptions are available in the literature (Kleeschulte and Emmett 1986, BNI 1987). This document describes only those soil and bedrock units encountered in the geotechnical investigations because the lower rock units have little or no effect on the proposed remedial action. The reader is directed to the referenced reports for more information on regional site stratigraphy. For immediate reference, Figure 2-2 shows a generalized stratigraphic column for the region. A summary of stratigraphic units discussed here is presented in Table 2-1. The stratigraphic units mentioned on this table are

SYSTEM	SERIES	STRATIGRAPHIC UNIT	TYPICAL THICKNESS (FT)*	LITHOLOGY	PHYSICAL CHARACTERISTICS	AQUIFER DESIG.	HYDROSTRATIGRAPHIC UNIT
QUATERNARY	HOLOCENE	Alluvium	0.5-4		Gravelly, silty loam.		ALLUVIAL AQUIFER
	PLEISTOCENE	Loess and Glacial Drift	15-55	Variable	Silty clay, gravelly clay, silty loam, clay, or loam over residuum from weathered bedrock.		(unsaturated)
MISSISSIPPIAN	MERAMECIAN	Salem Formation	0-15		Limestone, limey dolomite, finely to coarsely crystalline, massively bedded, and thin bedded shale.	*	(unsaturated)
		Warsaw Formation	60-80		Shale and thin to medium bedded finely crystalline limestone with interbedded chert.		
	OSAGEAN	Burlington and Keokuk Limestones	100-200		Cherty limestone, very fine to very coarsely crystalline, fossiliferous, thickly bedded to massive.	SHALLOW BEDROCK AQUIFER	SHALLOW BEDROCK AQUIFER MISSISSIPPIAN-DEVONIAN AQUIFER SYSTEM
		Fern Glen Limestone	45-70		Cherty limestone, dolomitic in part, very fine to very coarsely crystalline, medium to thickly bedded.		
	KINDERHOOKIAN	Chouteau Limestone	20-50		Dolomitic, argillaceous limestone; finely crystalline; thin to medium bedded.		
		Bushberg Sandstone	40-55		Quartz arenite, fine to medium grained, friable.		
DEVONIAN	UPPER	Lower Part of Sulfur Spring Group undifferentiated			Calcareous siltstone, sandstone, oolitic limestone, and hard carbonaceous shale.		
ORDOVICIAN	CINCINNATIAN	Maquoketa Shale	10-30		Calcareous to dolomitic silty shale and mudstone, thinly laminated to massive.	LEAKY CONFINING LAYER	ORDOVICIAN LEAKY CONFINING UNIT
	CHAMPLAINIAN	Kimmswick Limestone	70-100		Limestone, coarsely crystalline, medium to thickly bedded, fossiliferous and cherty near base.		
		Decorah Formation	30-60		Shale with thin interbeds of very finely crystalline limestone.		
		Plattin Limestone	100-130		Dolomitic limestone, very finely crystalline, fossiliferous, thinly bedded.		
		Joachim Dolomite	80-105		Interbedded very finely crystalline, thinly bedded dolomite, limestone; and shale. Sandy at base.		
		St. Peter Sandstone	120-150		Quartz arenite, fine to medium grained, massive.	DEEP BEDROCK AQUIFER	DEEP BEDROCK AQUIFER ORDOVICIAN-CAMBRIAN AQUIFER SYSTEM
	CANADIAN	Powell Dolomite	50-60		Sandy dolomite, medium to finely crystalline, minor chert and shale.		
		Cotter Dolomite	200-250		Argillaceous, cherty dolomite; fine to medium crystalline. Interbedded with shale.		
		Jefferson City Dolomite	160-180		Dolomite, fine to medium crystalline.		
		Roubidoux Formation	150-170		Dolomitic sandstone.		
		Gasconade Dolomite	250		Cherty dolomite and arenaceous dolomite (Gunter Member).		
CAMBRIAN	UPPER	Eminence Dolomite	200		Dolomite, medium to coarsely crystalline, medium bedded to massive.		
		Potosi Dolomite	100		Dolomite, fine to medium crystalline, thickly bedded to massive. Drusy quartz common.		

*Thickness data sources vary. Quaternary unit thicknesses based on on-site drilling and trenching. Burlington and Keokuk through Joachim Dolomite based on USGS wells MW-GS02 and GS05. St. Peter Sandstone and below from Kleeschulte and Emmett (1987). Warsaw and Salem Formations from Missouri DNR-DGLS Geologic Map OFM-89-252 GI (1989).

*The Warsaw and Salem Formations are not believed to be saturated in the WSS vicinity.

GENERALIZED STRATIGRAPHY AND HYDROSTRATIGRAPHY

FIGURE 2-2

REPORT NO.:	DOE/OR/21548-155		EXHIBIT NO.:	A/PI/078/1190	
ORIGINATOR:	SDG	DRAWN BY:	GLN	DATE:	11/90

TABLE 2-1 Stratigraphic Units at the Weldon Spring Site

Stratigraphic Unit		Unit Characteristics	
Age	Name	Thickness Range(ft)[a]	Typical Description
QUATERNARY			
Holocene	Top Soil	0-4	Organic clayey silt to silty clay.
	Fill	0-30	Variable: primarily clayey silt.
Pleistocene	Loess	0-10	Silt to clayey silt, minor sand; low plasticity.
	Ferrelview Formation	0-22	Silty clay to clayey silt, minor sand and fine gravel; stiff and moderately to highly plastic.
	Clay Till	0-30	Silty clay and clayey silt, with sand and gravel (rounded chert, igneous and metamorphic); massive, very stiff, moderately plastic.
	Basal Till	0-10	Sandy, clayey, silty gravel to gravelly silt (angular chert).
(Pre-Pleistocene?)	Residuum	0-26	Gravelly clay to gravelly silt, with weathered chert and limestone fragments
MISSISSIPPIAN	Burlington-Keokuk Limestones	9 to >50	Weathered limestone: argillaceous limestone, with abundant chert as nodules or interbeds; very finely to very coarsely crystalline; fossiliferous; solution features common. Competent limestone: thin to massive; finely to coarsely crystalline; less cherty; stylolitic; fossiliferous.

(a) Thickness data based on-site drilling and trenching.

discussed from the geotechnical engineering perspective in Section 4.

Burlington-Keokuk Limestone (Mississippian) formations comprise the uppermost bedrock stratigraphic unit at the WSS. These formations are undifferentiated at the site and are therefore designated as the Burlington-Keokuk Limestone. Because the Burlington-Keokuk Limestone was the only bedrock unit encountered during the MK-Ferguson investigation, it is the only unit discussed here. This unit is a thin to thick bedded, argillaceous, fine to coarsely crystalline limestone containing abundant chert as nodules and beds. On the basis of borehole stratigraphic data, the formation has been divided into two sub-units primarily according to weathering characteristics. The upper sub-unit is referred to as the weathered limestone; the lower one is referred to as the competent limestone.

The weathered limestone is typically grayish orange to yellowish gray, and argillaceous. It contains up to 60% chert as nodules and interbeds. It is fossiliferous, moderately to highly fractured, and slightly to severely weathered. Solution features are common, ranging from small voids to 5-foot cavities (BNI 1987). Solution features are typically parallel to bedding.

The competent limestone is light gray, finely to coarsely crystalline, stylolitic, and fossiliferous, with less (20-40%) chert than the weathered limestone. This unit is much less fractured than the weathered limestone.

Unconsolidated soil units overlying the limestone bedrock in the WSS area are typically Pleistocene to Holocene glacial and periglacial sediments capped by a layer of organic top soil. At the base of the sequence is the residuum, which has been interpreted as resulting from pre-Quaternary weathering of the youngest bedrock formation (BNI 1987). The residuum is generally a reddish brown gravelly clay to clayey gravel. Thickness and areal extent are variable.

The basal till (early Pleistocene) overlies the residuum. This unit is a yellowish brown, sandy, clayey, silty gravel with angular chert pebbles in a loosely bound matrix. The thinness or absence of this unit in areas of highest bedrock elevations suggests that the deposition of this unit may have been affected by bedrock topography. The basal till is found in the western and north central areas of the site.

The clay till unit overlies the basal till. This early Pleistocene deposit is composed of yellowish brown silty clay to clayey silt. Clay till sediments are massive, very stiff, and moderately to highly plastic. Pebbles in the till are subrounded chert and igneous and metamorphic detritus in contrast with the coarse fraction of the basal till. This may indicate a different source area for the unit. The clay till is widely spread beneath the site.

Overlying the clay till is the Ferrelview Formation, a mid-Pleistocene glacial till plain sediment. This unit is a mottled gray and dark yellowish orange silty clay to clayey silt. It is usually very stiff and plastic. This unit is also found throughout the site subsurface.

Overlying the Ferrelview Formation is a loess unit (late Pleistocene) that occurs sporadically across the site. The spotty distribution may be due to predepositional topography, post depositional erosion, and/or extensive reworking of the upper soils during site construction activities. The loess is primarily silt to clayey silt, with very minor amounts of sand, and has a low plasticity.

The uppermost soil unit is the combined topsoil/fill unit. The topsoil is generally a black, organically rich clayey silt to silty clay. The fill fraction varies in thickness and composition across the site, but is primarily a clayey silt.

2.2.3 Structure and Seismicity

The rocks in the region around the WSS were deformed during the Paleozoic Era vertical crustal movements. These movements resulted in the formation of broad bedrock basins and arches (Eardley 1951). The bedrock flexure nearest to the WSS is the House Springs-Eureka anticline, located approximately 4 miles southwest of the site. This fold trends northwest-southeast (Kleeschulte and Emmett 1986). The bedrock on the northeast limb of the anticline, where the WSS is located, generally strikes in a N60°W direction with gentle dips of 0.5° to 1° to the northeast (Krummel 1956).

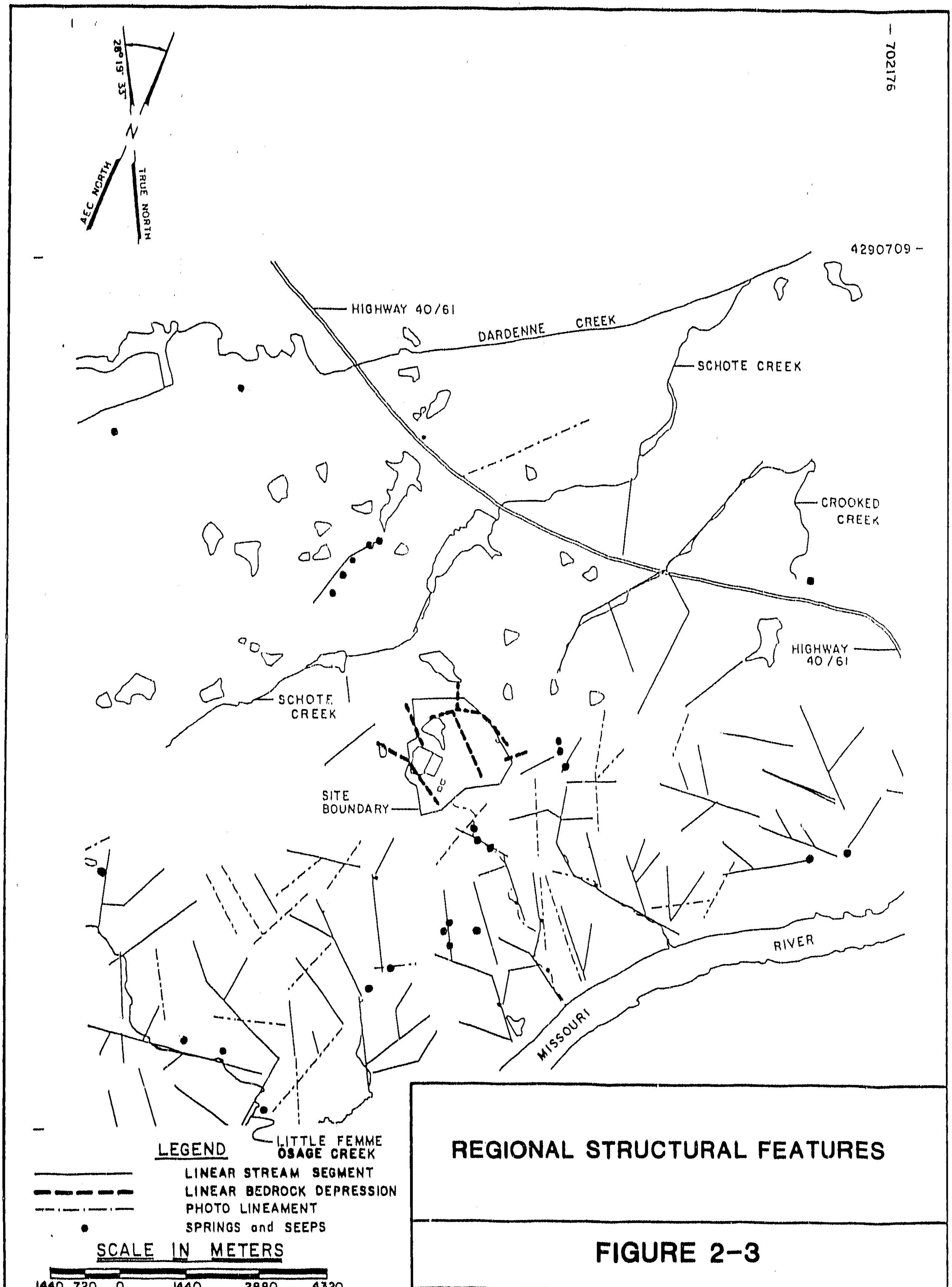
Jointing in the bedrock of the WSS area consists of two major sets. One set trends between N30°W and N65°W; the other between N30°E and N72°E (Roberts 1951). The joints at the WSS and the nearby Weldon Spring quarry dip vertically to nearly vertically (Berkeley Geosciences Associates 1984).

Regional structural features are shown in Figure 2-3. This map reflects a compilation of field measurements of jointing near the WSS and evaluation of the regional topographic features such as linear stream segments and photolineaments. The map is based on review of topographic maps and infrared aerial photographs of the WSS area. The linearity of the streams and features shown in

Figure 2-3 suggests that these valley orientations are structurally controlled along joints. Structural orientations are to the northwest and northeast with a minor northerly trend. The predominant trend of the streams in the unglaciated southern half of the mapped area is a strong orientation to the northwest with a mean bearing of N65°W. A second set of linear stream segments trends N38°E while a third, minor set trends N10°W.

The WSS lies in a tectonically inactive region of the North American continent near the boundary of the Mississippi Embayment (BNI 1983a). No faults are known to cross the site, but geologic mapping of the Weldon Spring 7-1/2 minute quadrangle by the Missouri Department of Natural Resource Division of Geology and Land Survey shows an east-west trending normal fault with up to 60 feet of vertical displacement passing 1 mile to the north of the site. The displacement terminates near Lake 35 of the Busch Wildlife area (Figure 2-1). The fault trends toward Burgermeister Spring which is 1 mile farther to the west. This fault does not show recurrent movement within the last 35,000 years, thus is not considered capable of producing the maximum credible earthquake at the site (JEG 1988).

The nearest seismic source to the WSS is the New Madrid Seismic Zone which is centered approximately 175 miles southeast of the WSS in the extreme southeastern corner of Missouri. This seismic zone has been the locus of some of the largest earthquakes on record for the United States. Between 1811 and 1812 earthquakes with estimated Richter magnitudes of 7.2 to 8.0 occurred there. These quakes rang church bells as far away as Boston. Locally, they reversed the flow of the Mississippi River (McKeown 1982).



REGIONAL STRUCTURAL FEATURES

FIGURE 2-3

REPORT NO.: DOE/OR/21548-155	EXHIBIT NO.: A/VP/058/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90

On the basis of a seismicity evaluation of this source area, Jacobs Engineering Group (JEG 1988) has estimated that the maximum credible earthquake in the New Madrid Seismic Zone would produced Modified Mercalli (MM) intensities in the range of 8.6 to 9.7 (equivalent to a magnitude of 6.1 to 6.6 on the Richter scale). Peak on-site acceleration (bedrock acceleration) from such an earthquake would be 0.26 g.

2.2.4 Hydrogeology

Three principal aquifer systems have been identified in the Weldon Spring area. These are the alluvial aquifers, the shallow bedrock aquifer system, and the deep bedrock aquifer system. In addition, a leaky confining layer is found below the shallow bedrock aquifer system. Individual zones from this layer may also be classified as low yielding aquifers (Kleeschulte and Emmett 1986 and 1987).

The alluvial aquifers include the saturated sands, gravels, and silts that compose the alluvium of the Missouri and Mississippi rivers. Yields from these aquifers along the major rivers range up to 2,600 gallons per minute (gpm) (BNI 1987). Alluvial aquifers are not present beneath the WSS. Some perched groundwater is scattered throughout the vadose zone, which may be a local recharge pathway to the shallow bedrock aquifer system.

The shallow bedrock aquifer system is made up of the saturated Mississippian- and Devonian-age rocks. It ranges in thickness from 250 to 650 feet. This aquifer system includes the Burlington-Keokuk Limestone beneath the Weldon Spring area down through the lower part of the Sulfur Springs Group (Figure 2-2). Reported yields of wells completed in this aquifer range from less than 1 gpm to 50 gpm (BNI 1987). The more productive wells are those that intercept zones of extensive secondary porosity such as joints, fractures, and solution openings.

The deep bedrock aquifer is within Ordovician and Upper Cambrian formations, which include the St. Peter Sandstone through the Potosi Dolomite (Figure 2-2). This aquifer is approximately 1,000 feet thick. Its yields range from 10 gpm to 500 gpm (BNI 1987).

Between the deep and shallow bedrock aquifers is a leaky confining layer from the top of the Maguoketa Shale down through the Joachim Dolomite. These formations correspond to the Ordovician leaky confining unit as depicted in Figure 2-2. Some low yielding aquifers within the leaky confining layer do occur and may produce 10 gpm to 50 gpm of potable water on a local basis. The confining sequence is approximately 400 feet thick (Kleeschulte and Emmett 1986).

3 FIELD INVESTIGATIONS

3.1 Previous Geotechnical Investigations

The PMC has reviewed and evaluated in depth the geotechnical and geophysical investigations conducted at the site before 1988. The findings are summarized and presented in the Geophysical/ Geotechnical Investigation Sampling Plan Report (MKF and JEG 1988b). Previous investigations by various organizations and the years when they were performed are listed below. Numbers in parentheses represent the year in which each report was published.

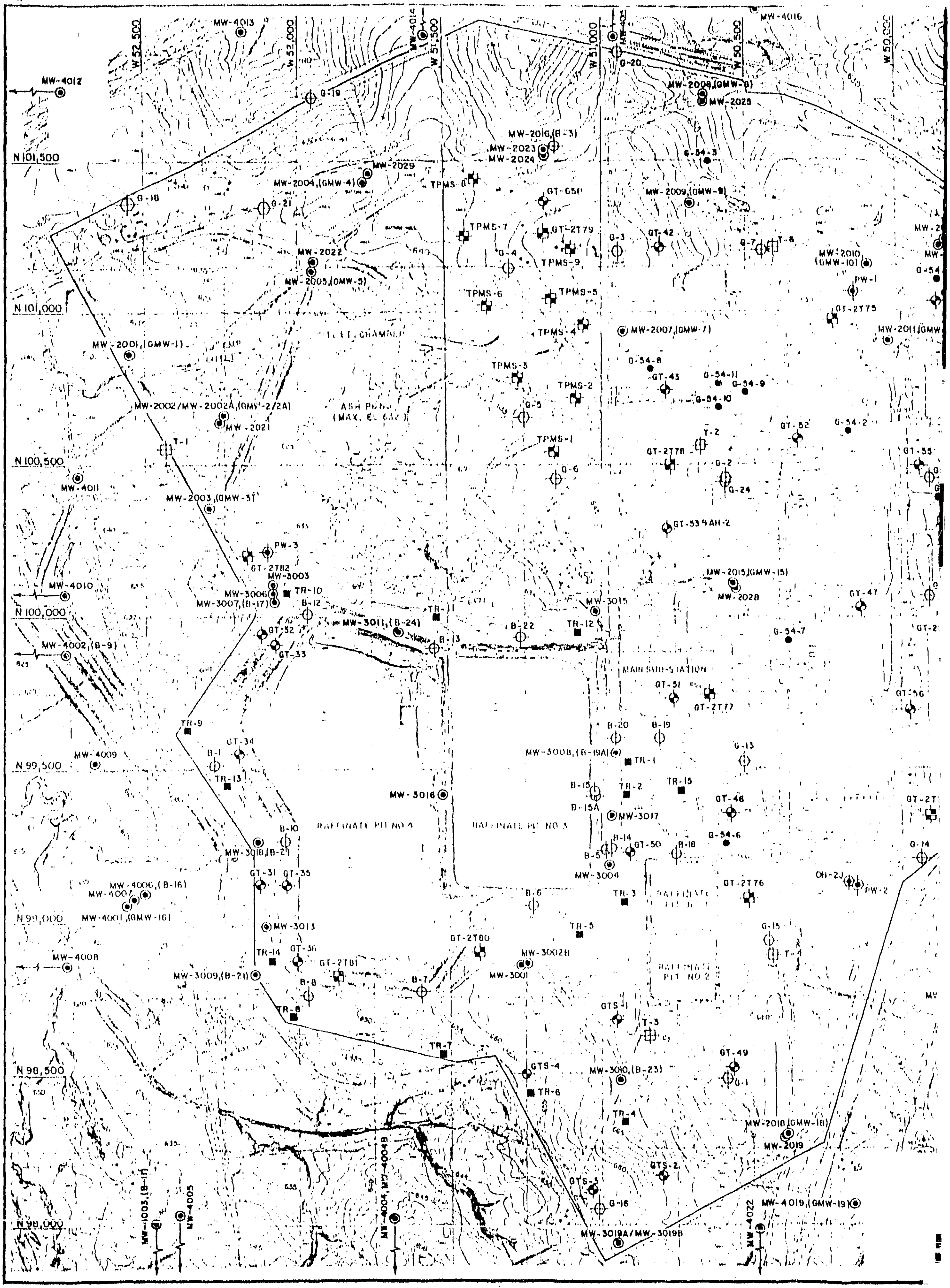
- Geotechnical

  - U.S. Army Corps of Engineers - 1954 (1955, 1970)
  - Henry M. Reitz Consulting Engineers - 1963 (1964)
  - Bechtel National, Inc. - 1982-1984 (1984)
  - Bechtel National, Inc. - 1986 (1987)

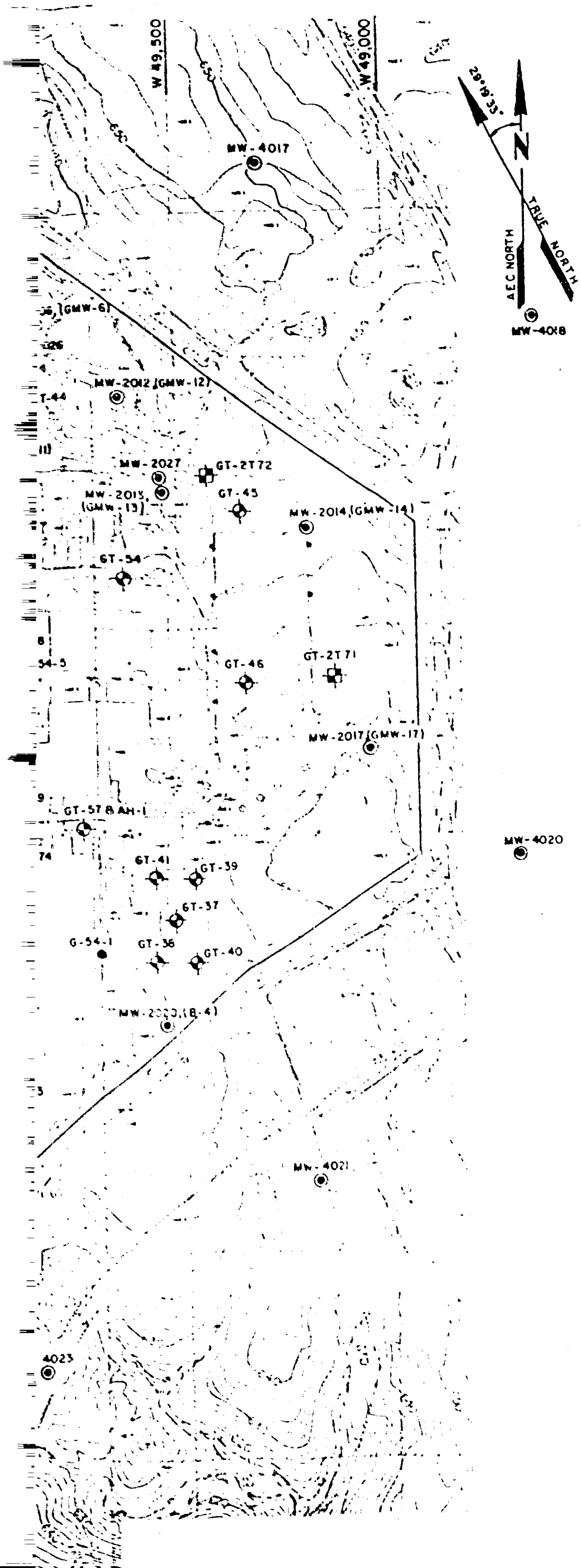
- Geophysical

  - Weston Geophysical Corporation - 1982 and 1984 (1983)
  - Detection Science, Inc. - 1986 (1987)
  - Missouri Dept of Natural Resources - 1988-1989 (1988)

Overall, more than 150 boreholes and monitoring wells have been drilled or installed, and more than 70 test pits and trenches have been excavated in the vicinity of the chemical plant and raffinate pits. Locations of the holes, wells, and test pits resulting from previous investigations, as well as those resulting from the present investigations are shown in Figure 3-1. Table 3-1 shows the numbering system for identifying



MW-4013
MW-4014
MW-4016
MW-4012
W 52,500
W 52,000
W 51,500
W 51,000
W 50,500
N 101,500
N 101,000
N 100,500
N 100,000
N 99,500
N 99,000
N 98,500
N 98,000
G-19
G-20
G-18
G-21
MW-2008,(GMW-8)
MW-2025
MW-2016,(B-3)
MW-2023
MW-2024
MW-2029
MW-2004,(GMW-4)
TPMS-8
GT-65P
MW-2009,(GMW-9)
TPMS-7
GT-2T79
GT-42
G-3
G-4
TPMS-9
MW-2022
MW-2005,(GMW-5)
MW-2010,(GMW-10)
PW-1
TPMS-6
TPMS-5
GT-2T75
TPMS-4
MW-2007,(GMW-7)
MW-2001,(GMW-1)
MW-2011,(GMW
TPMS-3
G-54-8
GT-43
G-54-11
G-54-9
TPMS-2
G-54-10
MW-2002/MW-2002A,(GMW-2/2A)
MW-2021
ASH POND
(MAX. EL. 632)
G-5
T-1
T-2
GT-52
G-54-2
TPMS-1
GT-2T78
GT-55
G-6
G-2
G-24
MW-4011
MW-2003,(GMW-3)
GT-534AH-2
PW-3
GT-2T82
MW-3003
MW-2015,(GMW-15)
MW-2028
MW-4010
MW-3006
TR-10
MW-3007,(B-17)
B-12
MW-3015
GT-47
TR-11
MW-3011,(B-24)
B-22
TR-12
GT-32
B-13
G-54-7
MW-4002,(B-9)
GT-33
MAIN SUB-STATION
GT-51
GT-2T77
GT-56
TR-9
B-20
B-19
GT-34
G-13
B-1
MW-4009
MW-3008,(B-19A)
TR-1
TR-13
TR-15
TR-2
MW-3016
B-15
GT-48
B-15A
MW-3017
RAFFINATE PIT NO 4
RAFFINATE PIT NO 3
B-10
B-14
G-54-6
MW-3018,(B-2)
B-5
GT-50
B-18
G-14
MW-3004
GT-31
GT-35
OH-2J
PW-2
MW-4006,(B-16)
MW-4007
B-6
TR-3
GT-2T76
MW-4001,(GMW-16)
MW-3013
TR-5
G-15
GT-36
GT-2T80
T-4
TR-14
GT-2T81
MW-30028
MW-4008
MW-3009,(B-21)
MW-3001
RAFFINATE PIT NO 2
B-7
B-8
TR-8
GTS-1
T-3
TR-7
GT-49
GTS-4
MW-3010,(B-23)
G-1
TR-6
TR-4
MW-2018,(GMW-18)
MW-2019
GTS-2
GTS-3
G-16
MW-4019,(GMW-19)
MW-4003,(B-11)
MW-4005
MW-4004,MW-4004B
MW-3019A/MW-3019B
MW-4022



LEGEND:

- G-54-11 U.S. C.O.E. BOREHOLE (1954-55)
- TR-15 BNI TEST PIT (1982)
- B-22 BNI MONITOR WELL (1983), OLD I.D IN PARENTHESIS
- T-6 BNI TEST PIT (1983)
- G-24 BNI BOREHOLE (1983)
- (GMW-11) BNI MONITORING WELL (1986), OLD I.D IN PARENTHESIS
- GT-57 MKE PHASE I BOREHOLE (1988)
- MW 4023 MKE AND JEG MONITORING WELL (1988), INCLUDING INCORPORATED BNI WELLS (B AND GMW SERIES)
- PW 3 OR OB-2J MKE PUMPING WELL (1988)
- GT-2T82 MKE PHASE I TEST PIT (1988)
- AH-2 GT-67P OR GTS-7 MKE PHASE II BOREHOLE (1989)
- TPMS-9 MKE TEST PIT (1990)

200 0 200 400
SCALE FEET

TEST PITS, MONITOR WELLS AND BOREHOLES LOCATION MAP

FIGURE 3-1

REPORT NO.:		DRAWING NO.:
ORIGINATOR: ECT	DRAWN BY: RBC	DATE: 12/90

the various borings and test pits. Coordinates and depths of boreholes, monitoring wells, and test pits used in the remedial investigations (RI) database are presented in Appendix C-1. The earliest studies (COE 1955, Henry M. Reitz 1964, Moylan and Elser 1967) obtained soil and rock geotechnical data for construction of site facilities. Later studies (BNI 1983a and 1987) characterized the site geology and hydrogeology and provided long-term groundwater monitoring. On-site geophysical studies prior to 1988 were conducted mainly in the raffinate pits area to refine geologic models of the site subsurface (Weston Geophysical 1983, Detection Science 1987). An off-site geophysical study (Hoffman 1988) was conducted to determine possible sources of water and conduits leading to the Burgermeister Spring.

3.2 PMC Geotechnical Investigations

The geotechnical investigations at the Weldon Spring site by the PMC were based on findings from the Geophysical/Geotechnical Investigation Sampling Plan (MKF and JEG 1988b). These investigations were conducted in phases. Phase I field activities began in June 1988 and were completed in August 1988. Phase II field activities began in January 1989 and were completed in September 1989. Additional Phase II test pit investigations were conducted by MK-Ferguson in August 1990. Table 1-2 summarizes these investigations. Locations of boreholes, test pits, and geophysical surveys are shown in Figures 1-3, 1-4, and 1-5. Individual activities and their purposes are discussed below. Boring and test pit logs are provided in Appendixes (shown in parentheses following each activity). Table 3-2 is a list of all relogged boreholes.

TABLE 3-1 Borehole and Test Pit Numbering System

LETTER DESIGNATION	NUMBER SYSTEM	FIRM/AGENCY	YEAR
B-1, etc.	Borehole number - raffinate pit area	Bechtel National Inc.	1983
B-15A, etc.	Replacement hole for B-15		
G-1	Borehole number - chemical plant area - geotechnical investigations	Bechtel National, Inc.	1986
GMW-3	Borehole completed as a monitoring well	Bechtel National Inc.	1986
G-54-1	Borehole number - chemical plant area	Corps of Engineers	1954
GT-2T-71	Test pit number - geotechnical investigations chemical plant area	PMC	1988
GT-31 to GT-36	Borehole number - geotechnical investigations raffinate pit No. 4	PMC	1988
GT-37 to GT-41	Borehole number - geotechnical investigations Admin. Bldg. area	PMC	1988
GT-42 to GT-57	Borehole number - geotechnical investigations chemical plant area	PMC	1988
AH-1	Angled borehole number - chemical plant area	PMC	1989
GTS-1	Borehole number - temporary storage area	PMC	1989
GT-58P to GT-67P	Borehole number - chemical plant area - P added if hole completed as piezometer	PMC	1989
MW-2001	Monitoring well number - within chemical plant area	PMC	1988-1989
MW-3001	Monitoring well number - wells completed adjacent to the Chemical Plant area	PMC	1988-1889
MW-4001	Monitoring well number - wells completed outside WSS area	PMC	1988-1989
PW-1	Pumping well number	PMC	1989
OB-2j	Observation well adjacent to pumping well	PMC	1989
T-1	Backhoe trench number	Bechtel National, Inc.	1986
TR-1	Trench number	Bechtel National, Inc.	1983
TPBS-1	Test pit number - borrow source area	PMC	1989
TPMS-1	Test pit number - materials storage area	PMC	1990

TABLE 3-2 List of Boreholes with Relogged Rock Cores

BOREHOLE NUMBERS		
B-3 (MW-2016)	GMW-2 (MW-2002)	MW-3002B
B-4 (MW-2020)	GMW-2A (MW-2002A)	MW-3006
B-9 (MW-4002)	GMW-3 (MW-2003)	MW-3019A
B-11 (MW-4003	GMW-4 (MW-2004)	MW-3019B
B-17 (MW-3007)	GMW-5 (MW-2005)	MW-4004
B-19A (MW-3008)	GMW-6 (MW-2006)	MW-4004B
B-21 (MW-3009)	GMW-7 (MW-2007)	MW-4005
B-23 (MW-3010)	GMW-8 (MW-2008)	MW-4007
G-1	GMW-9 (MW-2009)	MW-4008
G-2	GMW-10 (MW-2010)	MW-4009
G-2A	GMW-11 (MW-2011)	MW-4010
G-3	GMW-12 (MW-2012)	MW-4011
G-4	GMW-13 (MW-2013)	MW-4012
G-5	GMW-14 (MW-2014)	MW-4013
G-6	GMW-15 (MW-2015)	MW-4014
G-7	GMW-16 (MW-4001)	MW-4015
G-8	GMW-17 (MW-2017)	MW-4016
G-9	GMW-18 (MW-2018)	MW-4017
G-13	GMW-19 (MW-4019)	MW-4018
G-14	MW-2019	MW-4020
G-15	MW-2021	MW-4021
G-16	MW-2022	MW-4022
G-18	MW-2025	MW-4023
G-19	MW-2026	PW-1
G-20	MW-2027	PW-2
G-21	MW-2028	PW-3
GMW-1 (MW-2001)	MW-3001	OB2j

NOTE: Boreholes B-3 through GMW-19 were drilled by BNI during earlier studies at WSS. Boreholes MW-2019 through MW-4023 and PW-1 through OB2j were drilled as part of the extended monitoring well program by the PMC.

Phase I field activities consisted of the following:

- Six boreholes completed as piezometers (GT-31 to GT-36) along the dike of raffinate pit No. 4. [Appx. A-1]

- Five boreholes (GT-37 to GT-41) in the area of the administration building. [Appx. A-2]

- Sixteen exploratory boreholes (GT-42 to GT-57) in the area of the proposed disposal cell. [Appx. A-3]

- Twelve on-site test pits (GT-2T71 to GT-2T82) for potential on-site clay material sources. [Appx. A-4]

- Relogging of 35 monitoring well logs (MW-2019 to MW-4023, PW-1 to PW-3, and OB2j) using stored rock cores from earlier PMC investigations. [Appx. A-5]

- Surface geophysical survey of the proposed disposal cell site (performed by Geotechnology Services, Inc.). [See GSI 1988]

Phase II field activities consisted of the following:

- Two angled exploratory boreholes (AH-1 and AH-2) in the bedrock beneath the proposed disposal cell. Downhole geophysical surveys were made in these two boreholes. [Appx. B-1]

- Seven exploratory boreholes (GTS-1 to GTS-7) at the site of the proposed temporary storage area (TSA). [Appx. B-2]

- Ten boreholes and piezometers (GT-58P to GT-67P) within, and adjacent to, the proposed disposal cell. [Appx. B-3]

- Thirty test pits (TPBS-1 to TPBS-30) at two potential off-site sources for clay borrow material. [Appx. B-4]

- Relogging of 46 boreholes (B,G, and GMW series) using stored rock cores from previous BNI investigations. [Appx. B-5]

Additional work in Phase II in August 1990 consisted of digging and logging nine test pits (TPMS-1 to TPMS-9) at the proposed Materials Staging Area (MSA). [Appx. B-6]

Appendixes A and B contain brief field reports describing the work, including health and safety precautions taken to protect field personnel.

3.3 Phase I Investigations

Phase I geotechnical investigations began in June 1988 and were completed in August 1988. This section summarizes the purpose, equipment, exploration method, and sampling procedure of each activity in this phase. All explored locations, with the exception of relogged holes, are shown in Figure 1-3. Soil drilling and rock coring were performed by Hannibal Testing Laboratories and logged by MK-Environmental Services Group (MKES) geologists and engineers, except as noted. All drilling was accomplished using a Central Mining Equipment CME-55 drill rig. Drilling was advanced in soil using 6-5/8 inch outside diameter (OD) and 3-1/4 inch inside diameter (ID) hollow-stem augers to auger refusal. In rock, NQ wireline coring (3-inch diameter hole, 1-7/8 inch ID) core bits were used. Soil samples were obtained by using either a standard penetration test (SPT) split spoon sampler (ASTM D1586) or a California modified split-barrel sampler. The SPT sampler had an outside diameter (OD) of 2.0 inches and an inside diameter (ID) of 1.5 inches. The California split-barrel sampler had an OD of 3 inches and an ID of 2.5 inches. The samplers were driven 18 inches into the substratum. The samplers were advanced using a 140-pound safety hammer free-falling 30 inches. The number of blows required to drive each 6-inch advancement was recorded, and the blows for the final two 6-inch drives were summed and expressed as blows per foot. This blow count correlates roughly to various soil properties such as relative density of granular soils and the consistency of cohesive soils. The test is also useful in estimating other soil parameters such as unit weight, angle of internal friction, and unconfined compressive strength. Undisturbed samples were obtained by hydraulically pushing a 3-inch OD, 36-inch long Shelby tube approximately 30 inches. The test pits were excavated by a Caterpillar Company CAT 416 extendable backhoe. All soil samples were described according to the Unified Soil Classification System (USCS). Detailed sampling procedures and edited field logs may be found in Appendixes A-1 through A-4. The soil classifications in the logs were field classified and may not have incorporated laboratory classification results.

3.3.1 Raffinate Pit No. 4 Dike

Boreholes GT-31 through GT-36 were drilled and piezometers installed June 1 and 16, 1988. The holes were drilled to define subsurface conditions and to obtain samples for testing. The testing was needed to determine engineering parameters for dike stability analysis. All boreholes were completed as piezometers to monitor the phreatic surface, if any, within the dike and at the downstream toe. These data were used in stability model analyses. Boreholes GT-31 and GT-32 are located in the western downstream toe of the dike; all others are located on the dike crest (see Figure 1-3). Boreholes GT-31 and GT-32 were sampled every 5 feet or at major lithologic changes. Boreholes GT-33 through GT-36 were sampled continuously throughout the dike material and at every 5 feet within the native soils until auger refusal at the top of bedrock. Screen lengths of piezometers varied depending on the total depth of exposed dike material. Boring dimensions and logs and piezometer construction details are shown in Appendix A-1. All piezometers installed on the pit dike were found to be dry when last observed on August 29, 1988.

3.3.2 Administration Building

Five boreholes (GT-37 through GT-41) were drilled at the proposed location of the new administration building (Figure 1-3) between June 2 and 9, 1988. These holes were drilled to obtain soil samples to be used to determine geotechnical design parameters for the building foundation and to locate the groundwater level in the area. Boreholes were sampled continuously to 15 feet, and every 5 feet thereafter to auger refusal (inferred top of bedrock). All holes were left open to measure the groundwater level. However, the walls of these holes collapsed within a few days to within 8 to 10 feet of the ground surface. Subsequently, on June 17, 1989, all holes were redrilled to the original depth and backfilled with Volclay grout. Detailed drilling procedures and boring logs are shown in Appendix A-2.

3.3.3 Disposal Cell

Sixteen boreholes (GT-41 to GT-47) were drilled in the vicinity of the proposed disposal cell conceptual footprint (1988 and 1989 versions) between June 21 and August 29, 1988. These holes were drilled to obtain soil samples for laboratory testing. The results of these tests were used to determine geotechnical engineering parameters, to characterize the site subsurface in the vicinity of the disposal cell, and to correlate borehole data with the geophysical surveys. Accordingly, several of the boreholes were located along seismic lines or at the intersections of such lines. Locations of borings and seismic survey lines are shown in Figure 1-3. All boreholes were sampled every 2.5 feet in soil until auger refusal at the top of bedrock. Boreholes were typically cored 20 feet into the bedrock, except for boreholes GT-53 and GT-57 (located respectively adjacent to angled boreholes AH-1 and AH-2) and GT-49 (location of another boring previously drilled into bedrock). All boreholes were backfilled with Volclay grout upon completion. Detailed drilling procedures and boring logs are shown in Appendix A-3.

3.3.4 On-Site Test Pits

Twelve test pits (GT-2T71 through GT-2T82) were excavated at the site between July 12 and 15, 1988. These pits provided bulk samples of uncontaminated clayey material from within the site boundaries. This material could be used for the clay cover (radon barrier) and clay liner of the proposed disposal cell and for other backfill purposes. Figure 1-3 shows the locations of the pits. The pits were excavated using a CAT 416 backhoe. Pit dimensions were approximately 2 ft x 8 ft x 12 ft. Bulk samples were stored in 5 mil plastic bags before laboratory testing. Personnel access into the test pits below 5 feet was not permitted because of site safety procedures. Therefore, logging was done during excavation by examination of disturbed samples and visual observation of pit walls. Test pit logs, shown in Appendix A-4, describe soils and the types and locations of soil samples. During pit excavation, contaminated soils were separated from other soil. The pits were backfilled by replacing excavated soils in the reverse order from which they were removed. Backfill in test pit GT-2T81, which is located on top of the raffinate pit No. 1, dike was more thoroughly compacted. The lifts of backfill soil were 6 to 8 inches thick between compactions.

3.3.5 Geophysical Site Investigation

In 1988, an extensive geophysical survey of the proposed disposal cell footprint site was conducted by Geotechnology Services, Inc. (GSI 1988). This investigation was done in conjunction with the Phase I geotechnical program to provide geophysical information for refining the geological model of the site subsurface. Data acquired by the survey are delineated in the Geophysical/Geotechnical Investigation Sampling Plan (MKF and JEG 1988b). They include:

- Shallow and deep electromagnetic induction (EM)
- Gradient magnetometry
- Very Low Frequency (VLF) magnetometry
- Seismic refraction
- Seismic reflection
- Spontaneous potential (SP)
- Direct current vertical electric soundings (VES)

These methods were applied along geophysical survey lines to ensure that data acquired were representative of the site (Figure 1-3). With the exception of seismic refraction, consistent or usable results were generally obtained only where the survey lines were not close to buildings and other sources of interference. A summary of the results is presented in Section

4.1.5. Table 3-3 summarizes each geophysical survey method and the equipment used. Detailed survey data and results are presented in the report by Geotechnology Services, Inc. (GSI 1988).

3.3.6 Relogging of Rock Cores from Previous Investigations

The boreholes drilled in 1988, when the PMC installed monitoring wells, were relogged from September 1988 through January 1989 by a PMC geologist at the site. The purpose of the relogging was to obtain additional data on the physical properties of the overburden soil and bedrock, and to interpret the borehole stratigraphy for the RI effort. Photographs of the cores were taken for archival purposes. The boreholes relogged during this phase are numbered with the prefix MW, PW, or OB and are listed in Table 3-2. The relogged logs are presented in Appendix A-5.

3.4 Phase II Investigations

Phase II geotechnical investigations were conducted from January through September 1989. This section summarizes the work procedures and purpose of each activity conducted. All locations discussed are shown in Figure 1-4. Borehole and test pit logs may be found in Appendix B-1 through B-4.

The subcontractor for drilling vertical boreholes (at the temporary storage area, the disposal cell, and perimeter locations) and for excavating test pits (off-site borrow) was Hannibal Testing Laboratory. Angled boreholes were drilled by Brotcke Engineering. MKES geologists logged all boreholes and test pits. They also supervised the installation of piezometers.

TABLE 3-3 Summary of Geophysical Surveys

GEOPHYSICAL METHOD	PURPOSE	EQUIPMENT USED	SURVEY LOCATIONS	EVALUATION
Shallow Electromagnetic Induction (EM) and Gradient Magnetometry	Locate areas prone to conductive interference. Develop a "layered earth" model of the site subsurface.	Geonics EM-31-DL, Geonics EM-34-3 terrain conductivity meters EDA, OMNI IV magnetometer	Various, on site and site perimeter	On-site results erratic because of buildings, pipes, and/or buried conductors. Perimeter results consistent.
Very Low Frequency (VLF) Magnetometry and Deep Electromagnetic Induction (EM)	Detect deeper conductive features and determine their orientation in the subsurface. Develop "layered earth" model of the site subsurface.	ABEM, VLF/EM Wadi System Geonics EM-34-3 terrain conductivity meter. Measurements at 50-ft intervals; coil spacing: 10,20,10 meters.	Various on site and site perimeter.	On-site results erratic near buildings pipes, and/or buried conductors. Perimeter results consistent.
Seismic Refraction	Detect and delineate subsurface seismic velocity horizons.	Geometrics ES1225 and ES1210 signal enhancement seismographs. Geophone spread length: 220-260 ft.	Various, on site.	On-site results relatively consistent. Perimeter results consistent.
Seismic Reflection	Define subsurface reflecting horizons.	Same as seismic refraction equipment listed above.	Various, on site.	High frequency "noise"; reflection survey discontinued.
Vertical Electric Soundings (VES) and Spontaneous Potential (SP)	Depict resisting layering, depth to bedrock, fractures, and fluid movement. Determine lateral changes in conductivity.	Wenner array with stainless steel and porous pot electrode ABEM, SAS 300B meter.	North of coal storage area. VES soundings based on high anomalies in SP data.	VES and SP on-site results erratic; possibly cherts subsurface material and moisture conditions.

Source: Geotechnology Services, Inc. 1988.

Vertical boreholes in the disposal cell and temporary storage areas were drilled using a CME-55 drill rig. Drilling was advanced using 6-5/8 inch OD, 3-1/4 inch ID hollow stem augers in soil to auger refusal. Where appropriate, bedrock was cored using the NQ wireline coring method. Soil samples were obtained either by driving a standard penetration test (SPT) split spoon sampler or a California modified split-barrel sampler 18 inches into the substructure, or by pushing a Shelby tube approximately 30 inches into the substructure. The dimensions of the sampler and the sampling techniques for vertical boreholes are the same as those described in the discussion of Phase I investigations (Section 3.3). Other sampling methods are described where appropriate. Drilling and sampling of angled boreholes are described below.

3.4.1 Angled Boreholes

Two angled exploratory boreholes, AH-1 and AH-2, were drilled between January 16 and March 14, 1989. These boreholes were drilled to characterize bedrock structure, fracturing, and hydraulic conductivity, and to perform downhole geophysical surveys. Drilling was done by Brotcke Engineering. Figure 1-4 shows the borehole locations. Detailed descriptions of drilling, boring logs, and geophysical logs are presented in Appendix B-1.

The angled boreholes were oriented normal to major regional joint sets trending between N30°-72°E and N30°-65°W in order to intersect these joint sets. Borehole AH-1 was oriented at N42°E and AH-2 was oriented at N39°W. Both holes were drilled approximately 60° from horizontal. Drilling through the soil was accomplished with an Acker Mark IV skid-mounted drill rig equipped with an 8-inch diameter tri-cone bit. The soil portion of the borehole was cased. Coring in bedrock was advanced with a

Longyear 38 wireline rig equipped with a 5-foot NQ-wireline triple tube core barrel. MKES geologists tested bedrock hydraulic conductivity (permeability) at 10-foot intervals in the first 30 feet below top of rock, and after that, at 20-foot intervals. The test methods and results are presented in Appendix B-1. Table 3-4 is a summary of field tested permeability (hydraulic conductivity) values.

Both boreholes were geophysically logged by Wooddell Geophysical Services (Wooddell 1989) at the completion of the drilling. Geophysical logging included the following:

- Caliper
- Deviation
- Temperature
- Natural gamma
- Density
- Resistivity
- Neutron
- Sonic
- Spontaneous potential

Casing of the total depth in borehole AH-1 with 2-inch (ID) PVC pipe was necessary after minor caving occurred during logging. Borehole AH-2 was cased in a similar fashion after the caliper tool was successfully run. Both holes were abandoned after geophysical logging by pumping Volclay grout through the PVC pipe as the pipe was withdrawn from the hole. In borehole AH-1 a portion of the pipe remained stuck and was grouted in place.

TABLE 3-4 Summary of In Situ Permeability Test Results

BOREHOLE NUMBER	TEST INTERVAL OR DEPTH (FEET)[1]	STRATIGRAPHIC UNIT	CALCULATED HYDRAULIC CONDUCTIVITY (cm/sec)[2]
AH-1	3.8 - 12.6	Bedrock	9.3×10^{-4}
	12.4 - 21.3	Weathered limestone	6.1×10^{-4}
	20.6 - 29.5		ND[3]
	29.2 - 46.8		$<1.16 \times 10^{-5}$*
	46.7 - 64.1		$<3.6 \times 10^{-7}$*
	64.0 - 81.5	Competent limestone	8.4×10^{-6}
	79.7 - 100.6		$<6.6 \times 10^{-8}$*
AH-2	4.2 - 12.7	Bedrock	8.7×10^{-4}
	11.5 - 21.4	Weathered limestone	1.7×10^{-4}
	21.0 - 30.0		4.0×10^{-7}
	29.9 - 47.3		$<8.4 \times 10^{-6}$*
	47.2 - 74.7		$<1.9 \times 10^{-6}$*
	64.6 - 82.0	Competent limestone	1.0×10^{-6}
	81.8 - 100.7		2.1×10^{-5}
GTS-1	36.5	Residuum (CL)	ND
GTS-2	10.0	Ferrelview (CL)	ND
	25.0	Clay Till (CL)	ND
GTS-3	7.5	Ferrelview (CH)	ND
	17.5	Clay Till (CL)	ND
	22.5	Residuum (CL)	ND
GTS-4	5.0	Ferrelview (CL)	ND
	24.0	Basal Till (CL)	ND
GTS-5	15.0	Ferrelview (CH)	1.25×10^{-7}*
GT-59	22.5	Clay Till (CH)	4.94×10^{-7}*
GT-61	51.5	Basal Till (ML)	8.56×10^{-5}

NOTES:

1 Hydraulic conductivity tests in bedrock were packer tests under applied water pressure. In soil, tests were constant head permeability tests. For AH-1 and AH-2 the test interval shown is below top of bedrock; for all others, the depth shown is below ground level.

2 Hydraulic conductivity values for AH-1 and AH-2 calculated from Table 1 and 2 in Appendix B-1; for GTS-1 through GTS-5, Appendix B-2; for GT-59 and GT-61, Appendix B-3. Calculated values in bedrock are average of at least three tests; values marked by asterisk are results of single test.

3 ND = results below detection limit (10^{-8} cm/sec).

3.4.2 Temporary Storage Area

Seven boreholes, GTS-1 through GTS-7, were drilled in the temporary storage area between May 5 and June 8, 1989. Drilling was conducted to obtain soil samples for geotechnical laboratory testing, to field test the hydraulic conductivity of in situ soilunits, and to study the physical characteristics of the soil. In addition, bulk samples were taken in the top 5 feet and stored in 5-gallon buckets for laboratory moisture-density (compaction) tests. Boring logs and details of field procedures are shown in Appendix B-2.

The boreholes were drilled through soil by hollow-stem auger until auger refusal at the top of bedrock. Soil samples were obtained during auger drilling every 2.5 feet by driving an SPT sampler or pushing a Shelby tube into the substratum. A constant-head permeability test was performed in boreholes GTS-1 through GTS-5 after each Shelby tube sample was taken. The tests were made by pushing NQ (2-3/8 inch ID) rods through the hollow stem augers 4 inches into the substratum (to provide a seal). The rods were filled with water and water was added as needed to maintain the level. The volume of water added was recorded at typical 10-minute test periods. Table 3-2 summarizes field test soil hydraulic conductivity values. All boreholes were backfilled with Volclay grout after completion of drilling and field testing.

3.4.3 Disposal Cell and Perimeter

Ten boreholes, GT-58P through GT-67P, were drilled June 15 to 30, and July 10 to August 15, 1989. Of these 10, seven were completed as piezometers. In the documentation, these holes are designated by a "P" following the borehole number. These boreholes were drilled to obtain soil samples to be used in determining engineering parameters. These holes were also used in the hydrogeologic investigation program. They provided additional piezometers for monitoring water levels in the shallow bedrock aquifer in the proposed disposal cell area.

The boreholes without piezometers were drilled using 6-7/8 inch OD, 3-1/4 inch ID hollow-stem augers. They were drilled through the overburden to refusal at the top of bedrock. The holes with piezometers were drilled using 7-1/4 inch OD, 4-1/4 inch ID hollow-stem augers. They were drilled through overburden soils. Then NQ-wireline coring was used to core through bedrock to a depth 8 to 10 feet below the projected groundwater table. Borings were sampled 2.5 foot intervals, using an SPT split spoon sampler and/or Shelby tube. Constant-head permeability tests were also conducted in boreholes GT-59 and GT-61 after each Shelby tube sampling.

The piezometers were constructed using 2-inch ID Schedule 40 PVC pipe. Since the water level was typically within the cored bedrock, the piezometers were installed without a sandpack. A 10-foot segment of the pipe straddling the water level encountered during drilling was screened with 0.01 inch slots. In most cases the screen was isolated by using a packer or reduction coupler and sealing off the annular space above it. Piezometers were developed by Hannibal Testing Laboratories between August 21 and September 11, 1989. Development was accomplished using Teflon bailers and a Triloc 1.7-inch PVC hand pump.

Borehole and piezometer locations are shown in Figure 1-4. Boring logs and piezometer development summary data, a field report, and a summary of piezometer completion data may be found in Appendix B-3. Water level elevation data for these and other wells on site are shown in Appendix C.

3.4.4 Off-Site Borrow Test Pits

Thirty test pits, TPBS-1 through TPBS-30, were excavated in two potential clay borrow source areas south and southeast of the WSS site on March 28-30 and April 17-20, 1989. Borrow source No. 1 is located in the experimental farm area owned by the University of Missouri. Borrow source No. 2 is located approximately 0.4 miles west of the town of Hamburg. The locations of the test pits are shown in Figure 1-5. The pits were excavated to provide bulk samples of clayey soils from borrow sources that may be used during the construction of the proposed disposal cell. Samples were tested for engineering characteristics to determine the suitability of the soil for construction and design purposes. In addition to bulk samples, relatively undisturbed samples were obtained from 14 pits by driving a Corps of Engineers sampler approximately 5-foot depths in clays. These undisturbed samples were used to determine in situ dry density and moisture content of the soil. The pits were excavated by Bleigh Construction using a CAT 416 backhoe with an extendable boom and a 24-inch bucket. Pit dimensions were approximately 2 x 8 x 12 feet. MKES geologists logged the test pits and supervised excavation procedures. Test pit logs are presented in Appendix B-4.

3.4.5 Relogging of Rock Cores from Previous Investigations

MKES geologists relogged rock cores from BNI investigations in 1984 and 1986. This relogging took place at the site from October 2 through November 22, 1989. The purpose of the relogging was to (1) obtain geotechnical parameters not recorded by BNI during their drilling programs, (2) identify depths of core loss zones for possible correlation among boreholes, (3) provide detailed lithologic descriptions not provided in the original BNI logs for stratigraphic correlation in the bedrock, and (4) photograph the cores for archival purposes. The boreholes with relogged cores, numbered with prefixes B, G, or GMW, are listed in Table 3-2. The relogged logs are presented in Appendix B-5. Photographs of rock cores are kept in the project archives at the WSS.

3.4.6 Materials Staging Area Investigation

Additional investigations were conducted in August 1990 in the materials staging area (see Figure 1-4). Nine test pits, TPMS-1 through TPMS-9, were excavated on August 6, 1990 using a Ford 755 backhoe furnished by Dave Kolb Excavating. The pits were excavated to provide bulk samples for laboratory testing of soils that may be used during the construction of the material staging area. The pits were approximately 2 x 15 feet wide and up to 15 feet deep. A PMC geologist logged the test pits and supervised excavation procedures. Test pit logs are presented in Appendix B-5.

4 SUBSURFACE CONDITIONS

4.1 General

This section discusses the geotechnical subsurface conditions of the site, including the chemical plant, raffinate pit, temporary storage, and materials storage areas. Accordingly, descriptions of overburden soil material and material thickness and permeability (hydraulic conductivity); bedrock surface depths, fracturing, and hydraulic conductivity; and groundwater flow patterns are included. The conclusions of the geophysical surveys are summarized to provide a verifying model of the site subsurface. All of this information is based on the results of the geotechnical field investigation program conducted by MK-Ferguson from 1988 through 1990, and on the results of previous site characterizations and geologic and geophysical site studies by Bechtel National, Inc. (BNI 1984 and 1987) and Geotechnology Services, Inc. (1988).

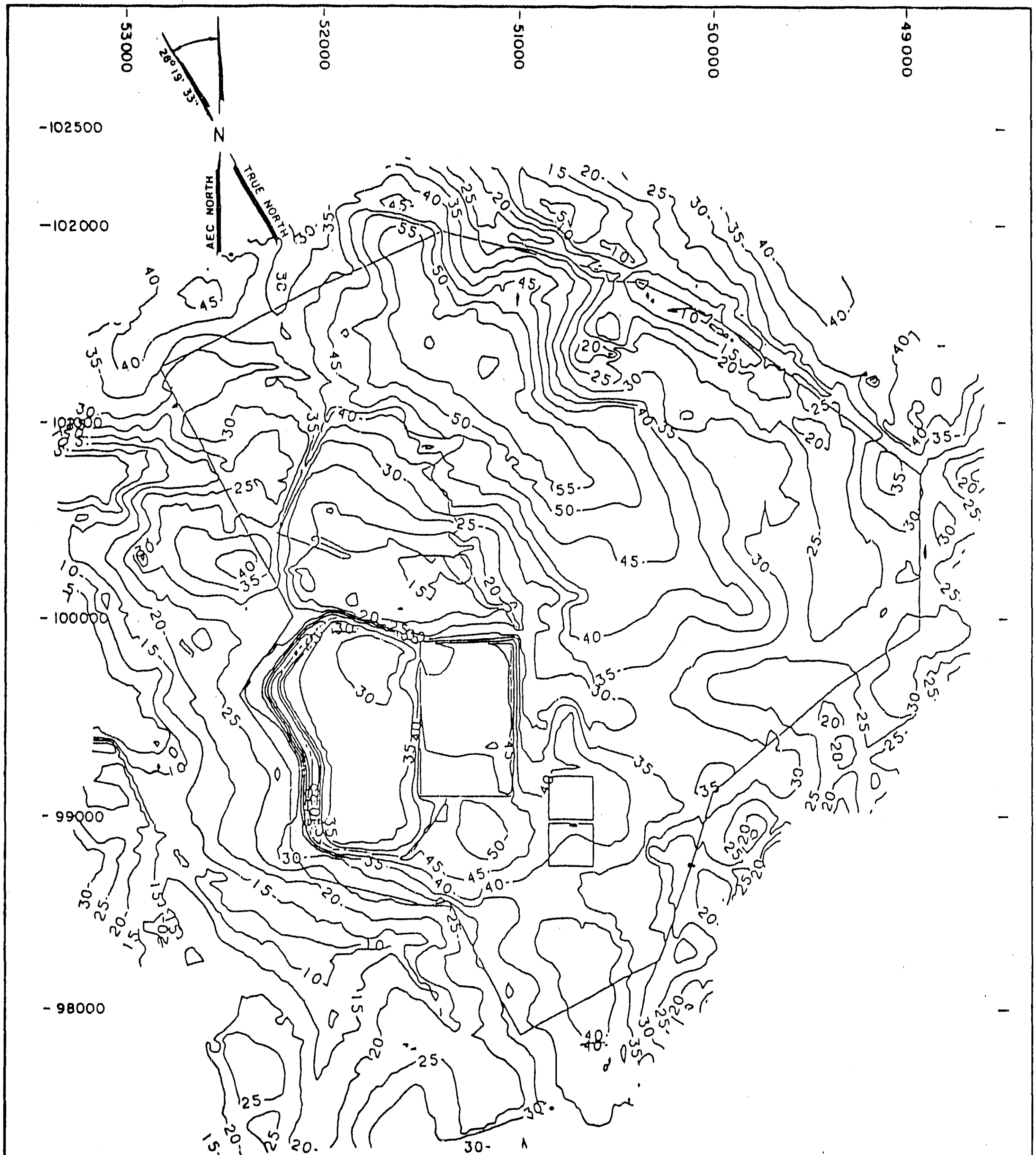
4.2 Soil

General geologic characteristics of the soil units at the Weldon Spring site (WSS) are described in Section 2.2. The physical and geotechnical properties of the soil units are listed in Table 2-1. Averaged geotechnical engineering properties, based on selected test results, have been summarized during remedial investigation (RI) studies performed from 1988 through 1990 (MKF and JEG 1990c) in conjunction with the geotechnical field investigation programs. Results of geotechnical laboratory testing for this study are presented in Section 5.

Following the geotechnical field investigations and the installation of monitoring wells in the chemical plant area, the geologic data from each of the borehole and test pit logs were evaluated. Isopach and contour maps and geologic cross sections were generated by computer using the geological database.

Isopach maps of the various soil units and contour maps of the bedrock units (Figures 4-1 through 4-8) and geologic cross sections (Figures 4-10 through 4-14) were developed during RI documentation efforts (MKF and JEG 1990c). Figure 4-9 is a plan map of all the borehole, test pit, and cross section locations. Geologic cross-sections A-A′ through E-E′ are shown in Figures 4-10 through 4-14, respectively. In the cross sections, each of the soil units and the two bedrock units described below can be seen in their relative positions. These figures indicate the thickness distribution of each unit, particularly the fill. Information from test pits for the material staging area are not included in these figures.

Overall soil (overburden) thickness at the WSS ranges from 15 to 60 feet, and is apparently controlled by both surface erosion and bedrock topography. Figure 4-1 is a soil thickness isopach map. Soil is thickest in the north-central portion of the site and thinnest in the eastern portion. By contrast, the bedrock surface elevation is lowest in the western portion of the site and highest in the eastern portion. This combination of greater soil thickness over areas of lower bedrock surface and vice versa has resulted in a relatively level ground surface across most of the site (Geotechnology 1988, BNI 1987). This corresponds well to eroding of pre-glacial topographic highs and the filling of lower elevations with glacial drift.



LEGEND

——10—— THICKNESS CONTOUR

SCALE IN METERS

120 60 0 120 240 360

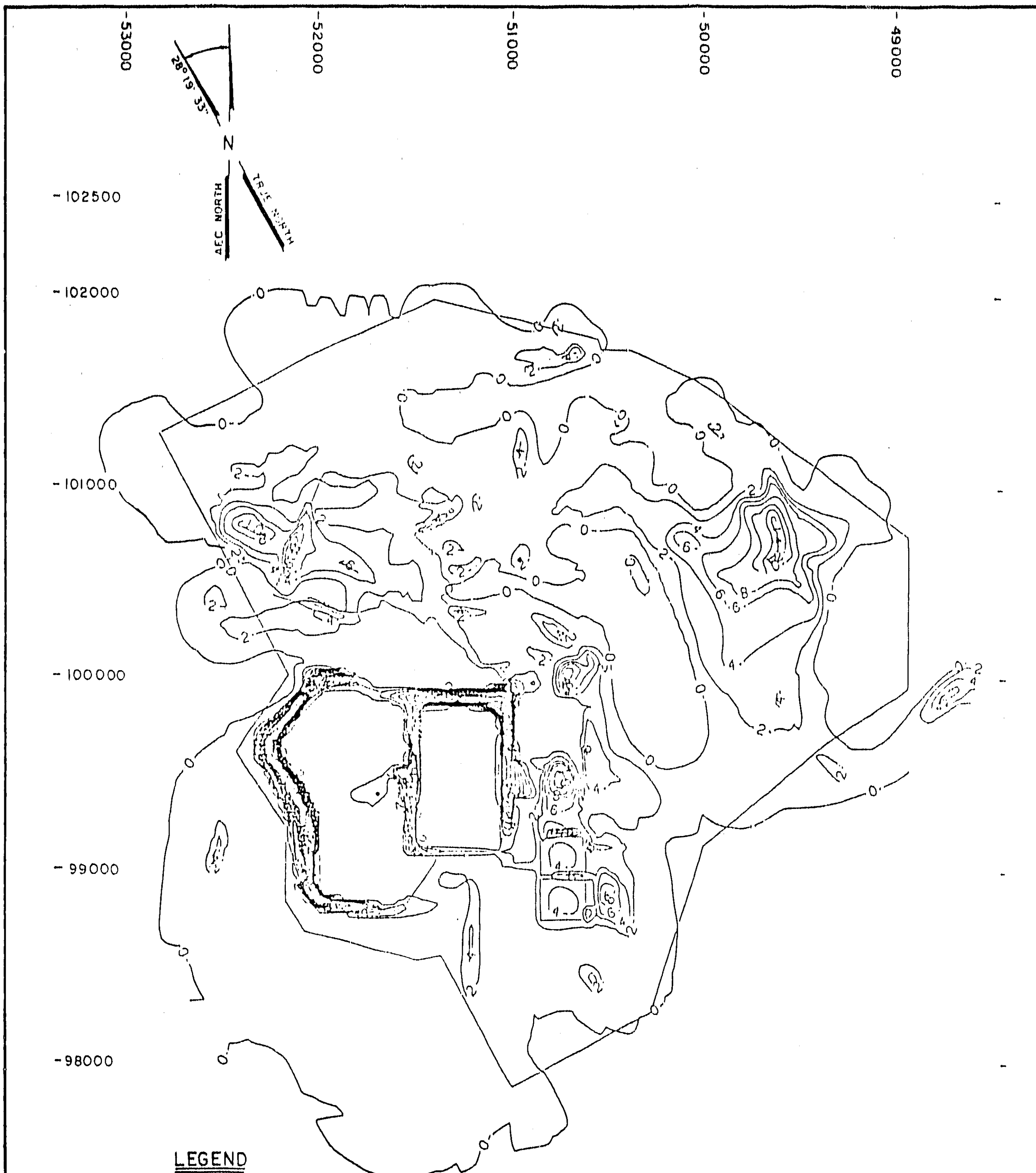
SCALE IN FEET

500 250 0 500 1000

ISOPACH MAP OF TOTAL OVERBURDEN

FIGURE 4-1

REPORT NO. DOE/OR/21548-155		DRAWING NO.: A/CP/102/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



LEGEND

—— 4 —— THICKNESS CONTOUR of TOPSOIL / FILL

CONTOUR INTERVAL 2.0 FT.

SCALE IN METERS

120 60 0 120 240 360

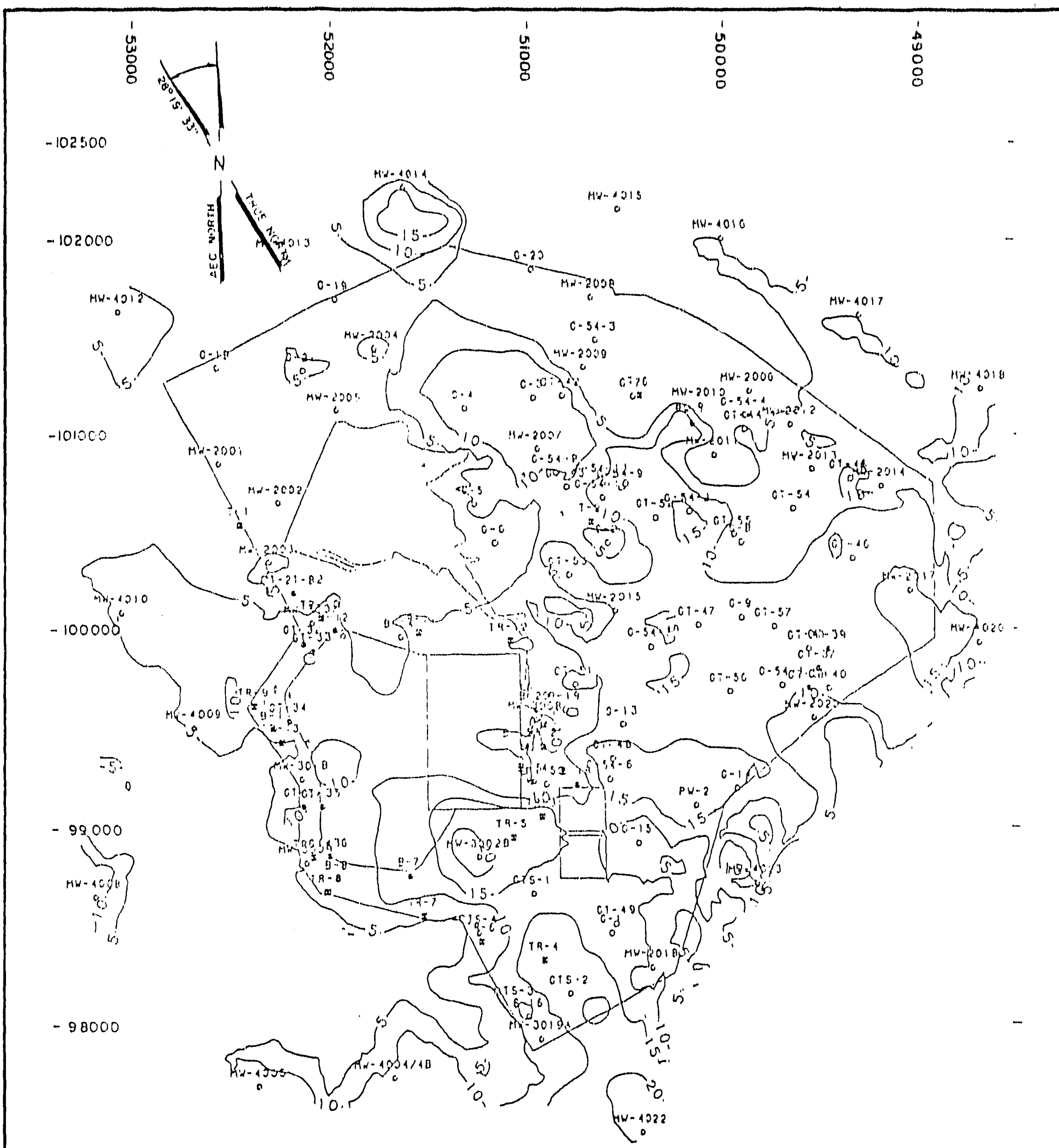
SCALE IN FEET

500 250 0 500 1000

ISOPACH MAP OF TOPSOIL/FILL

FIGURE 4-2

REPORT NO.: DOE/OR/21548-155		EXHIBIT NO.: A/CP/103/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



LEGEND

——10—— THICKNESS CONTOUR of FERRELVIEW FORMATION

CONTOUR INTERVAL 5.0 FT.

SCALE IN METERS

120 60 0 120 240 360

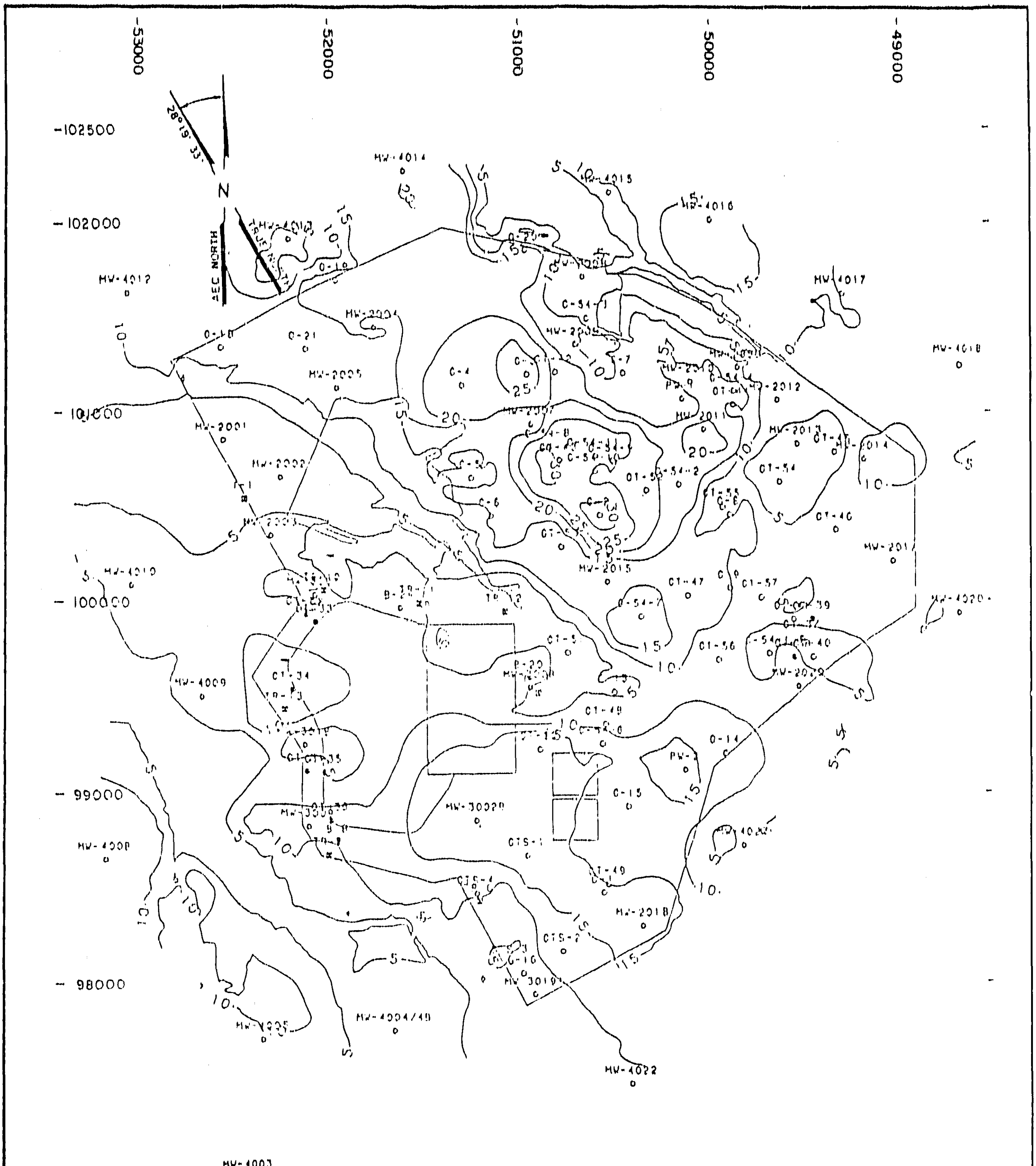
SCALE IN FEET

500 250 0 500 1000

ISOPACH MAP OF FERRELVIEW FORMATION

FIGURE 4-3

REPORT NO.: DOE/OR/21548-155		EXHIBIT NO.: A/CP/104/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



ISOPACH MAP OF CLAY TILL

LEGEND

—— 10 —— THICKNESS CONTOUR of CLAY TILL

CONTOUR INTERVAL 5.0 FT.

SCALE IN METERS

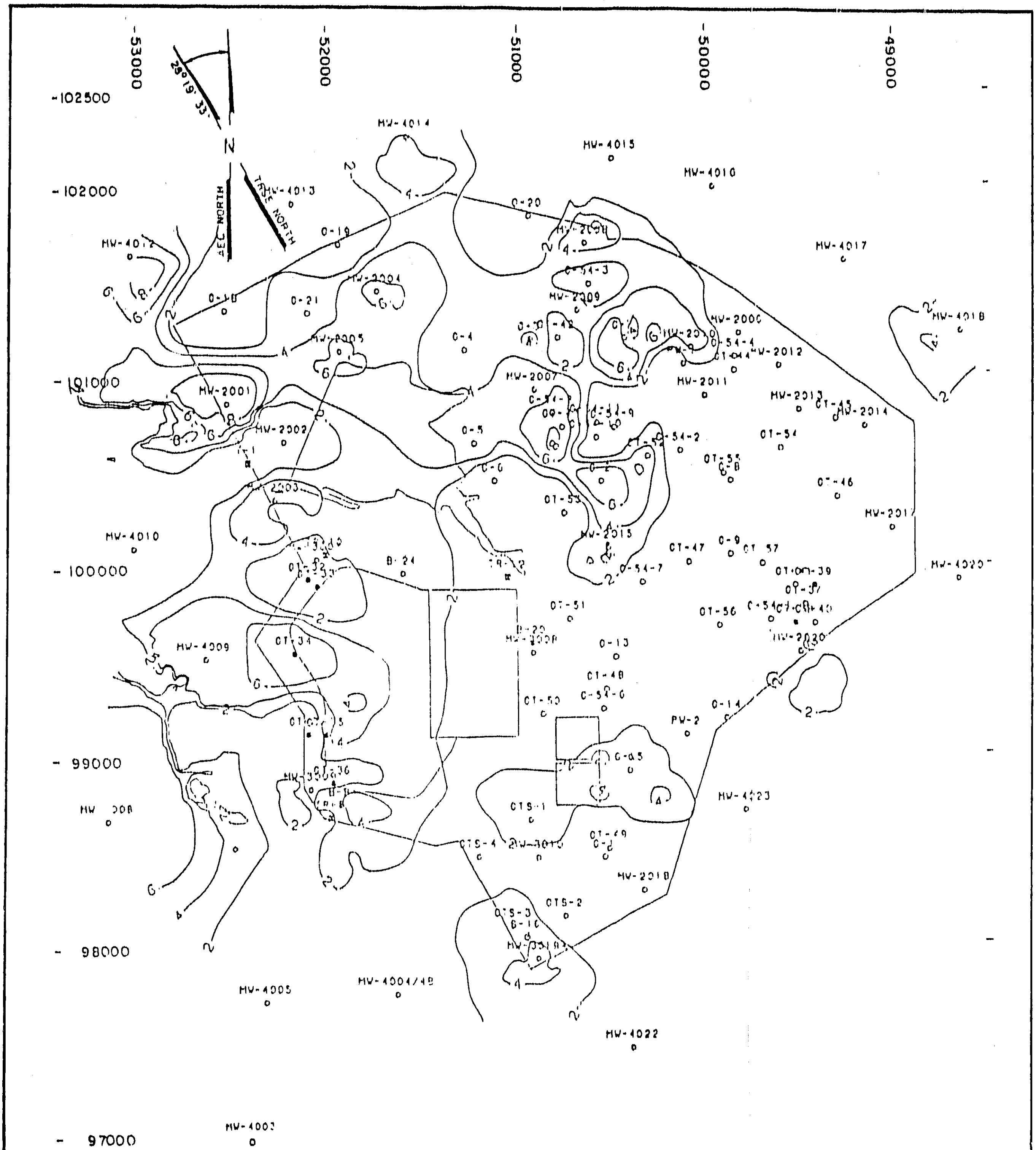
120 60 0 120 240 360

SCALE IN FEET

500 250 0 500 1000

FIGURE 4-4

REPORT NO.: DOE/OR/21548-155		EXHIBIT NO.: A/CP/105/1190
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90



LEGEND

—— 6 —— THICKNESS CONTOUR of BASAL TILL

CONTOUR INTERVAL 2.0 FT.

SCALE IN METERS

120 60 0 120 240 360

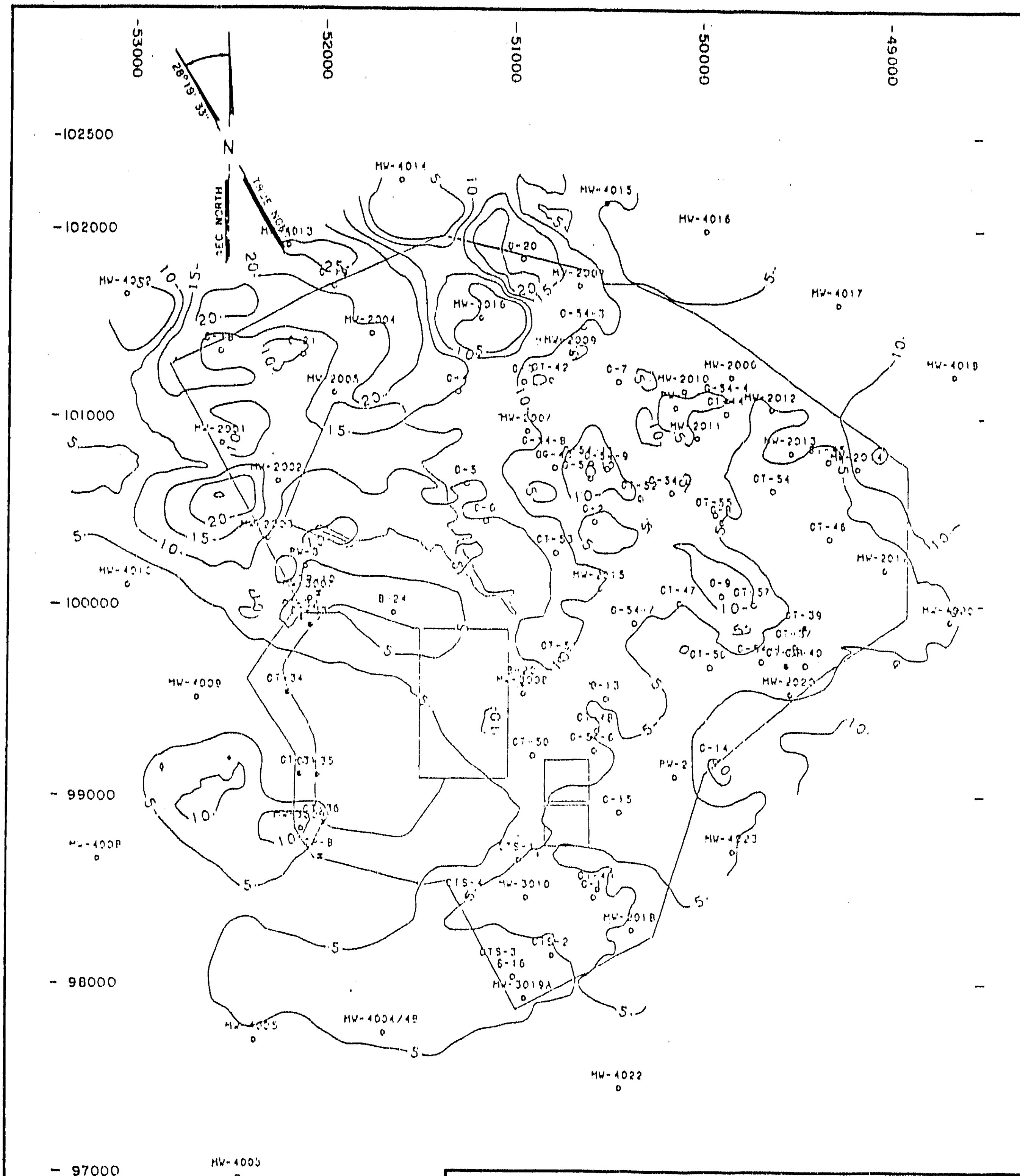
SCALE IN FEET

500 250 0 500 1000

ISOPACH MAP OF BASAL TILL

FIGURE 4-5

REPORT NO.: DOE/OR/21548-155		EXHIBIT NO.: A/CP/106/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



LEGEND

—10——— THICKNESS CONTOUR of RESIDUUM

CONTOUR INTERVAL 2.0 FT.

SCALE IN METERS

120 60 0 120 240 360

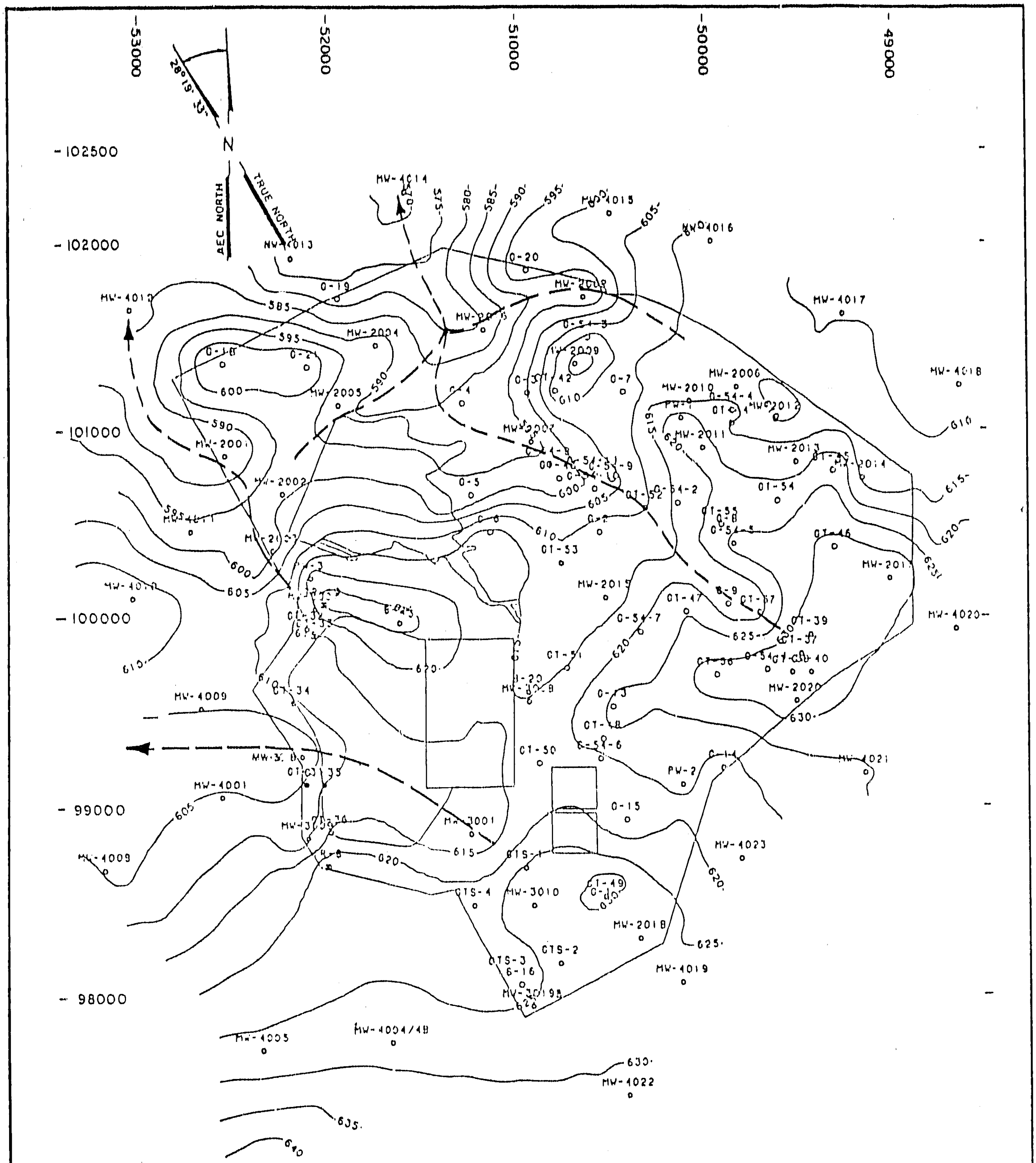
SCALE IN FEET

500 250 0 500 1000

ISOPACH MAP OF RESIDUUM

FIGURE 4-6

REPORT NO.:	DOE/OR/21548-155		EXHIBIT NO.:	A/CP/107/1190	
ORIGINATOR:	SDG	DRAWN BY:	GLN	DATE:	11/90



LEGEND

CONTOUR INTERVAL 5.0 FT.

630 — CONTOUR OF TOP OF WEATHERED LIMESTONE

TREND OF LINEAR DEPRESSION

SCALE IN METERS

120 60 0 120 240 360

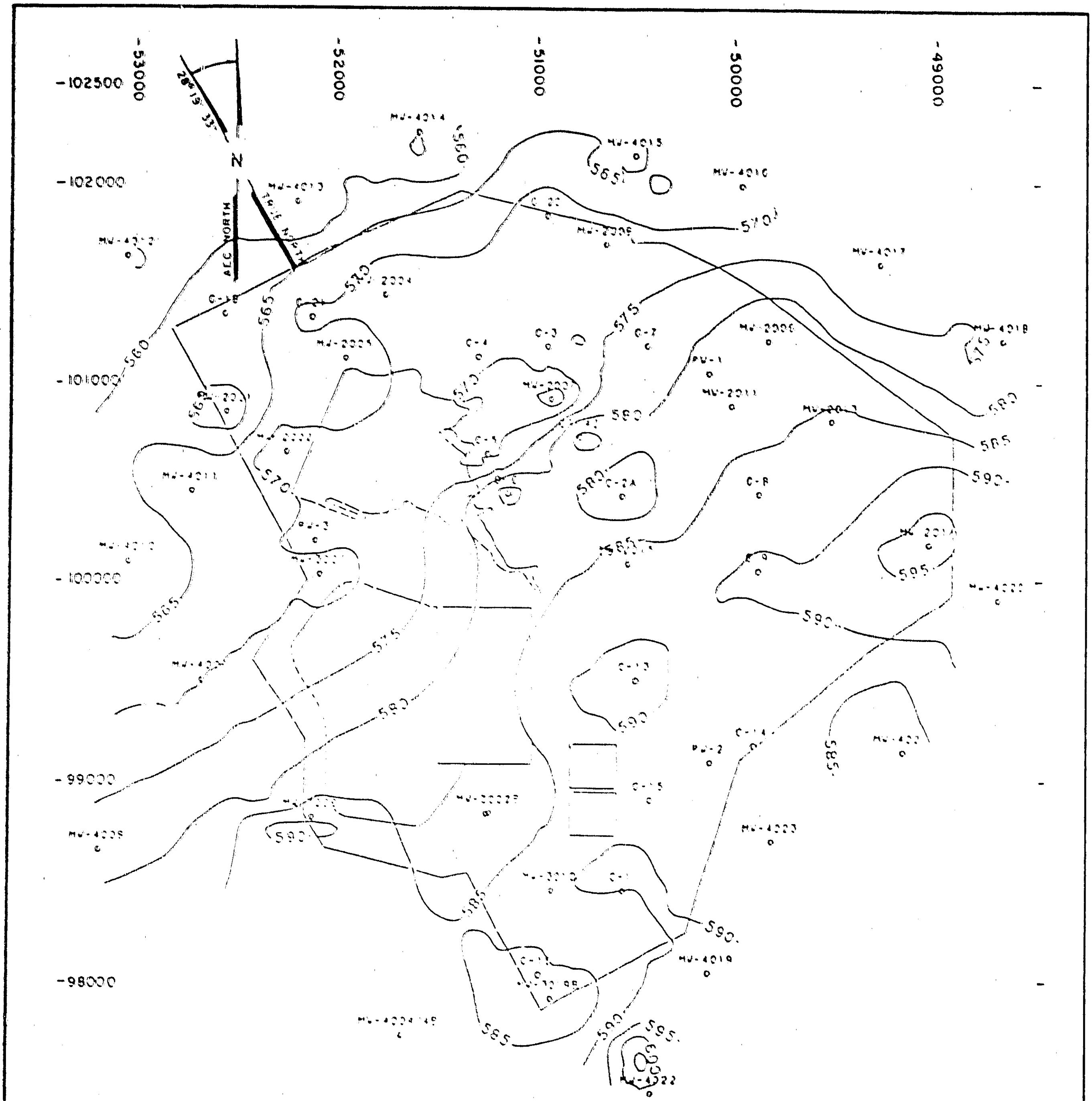
SCALE IN FEET

500 250 0 500 1000

CONTOUR MAP OF TOP OF WEATHERED LIMESTONE

FIGURE 4-7

REPORT NO.:	DOE/OR/21548-155	EXHIBIT NO.:	A/CP/108/1190		
ORIGINATOR:	SDG	DRAWN BY:	GLN	DATE:	11/90



LEGEND

——590—— CONTOUR OF TOP OF COMPETENT LIMESTONE

CONTOUR INTERVAL 5.0 FT.

SCALE IN METERS

120 60 0 120 240 360

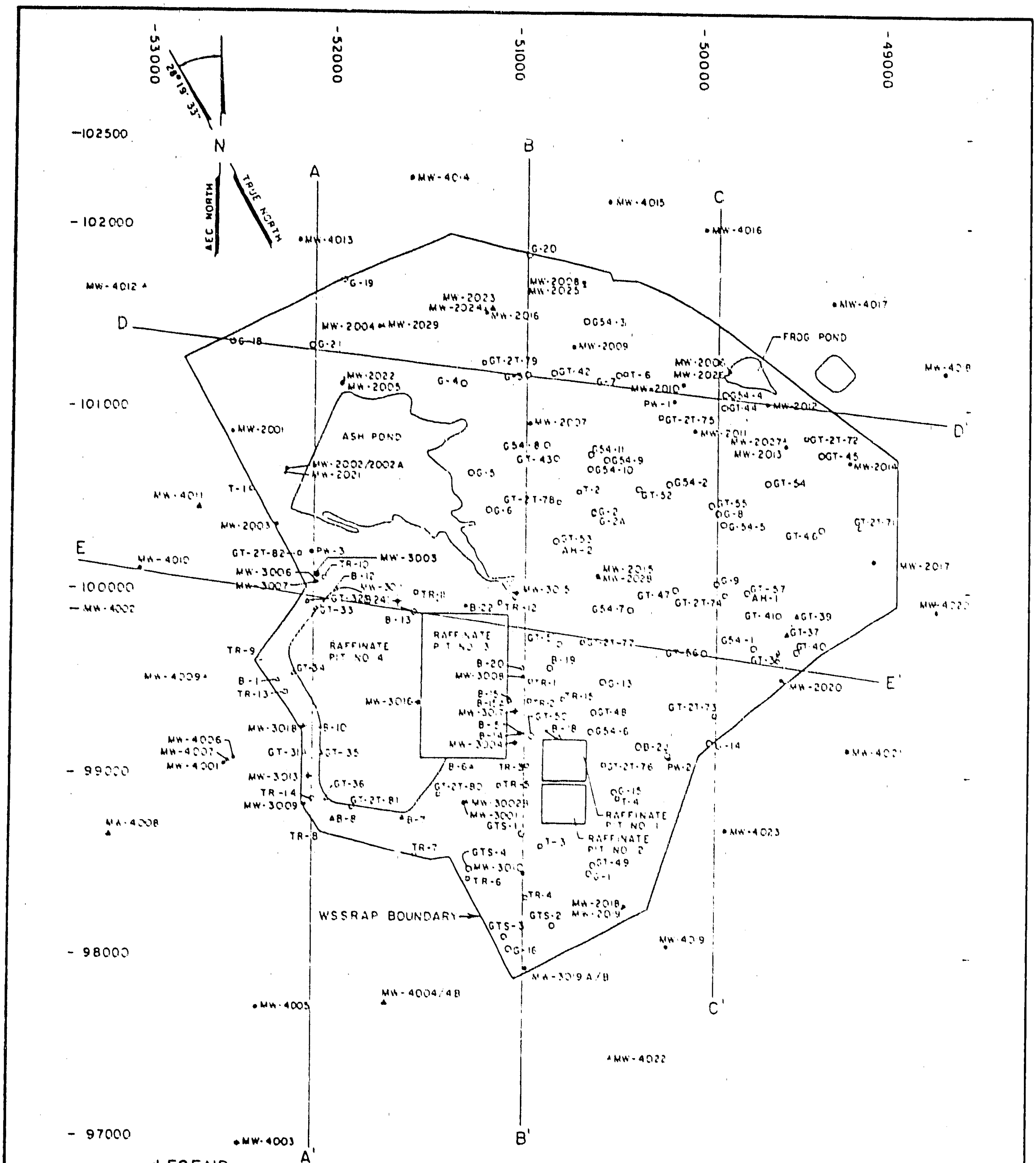
SCALE IN FEET

500 250 0 500 1000

CONTOUR MAP OF TOP OF COMPETENT LIMESTONE

FIGURE 4-8

REPORT NO.:	DOE/OR/21548-155	EXHIBIT NO.:	A/CP/109/1190		
ORIGINATOR	SDG	DRAWN BY.	GLN	DATE	11/90



LEGEND

- • MONITOR WELL (WEATHERED BEDROCK)
- ▲ MONITOR WELL (COMPETENT BEDROCK)
- o BOREHOLE
- ▲ PIEZOMETER
- ✦ OVERBURDEN WELL
- ▫ TRENCH
- A - A' CROSS SECTION LOCATION

SCALE IN METERS

120 60 0 120 240 360

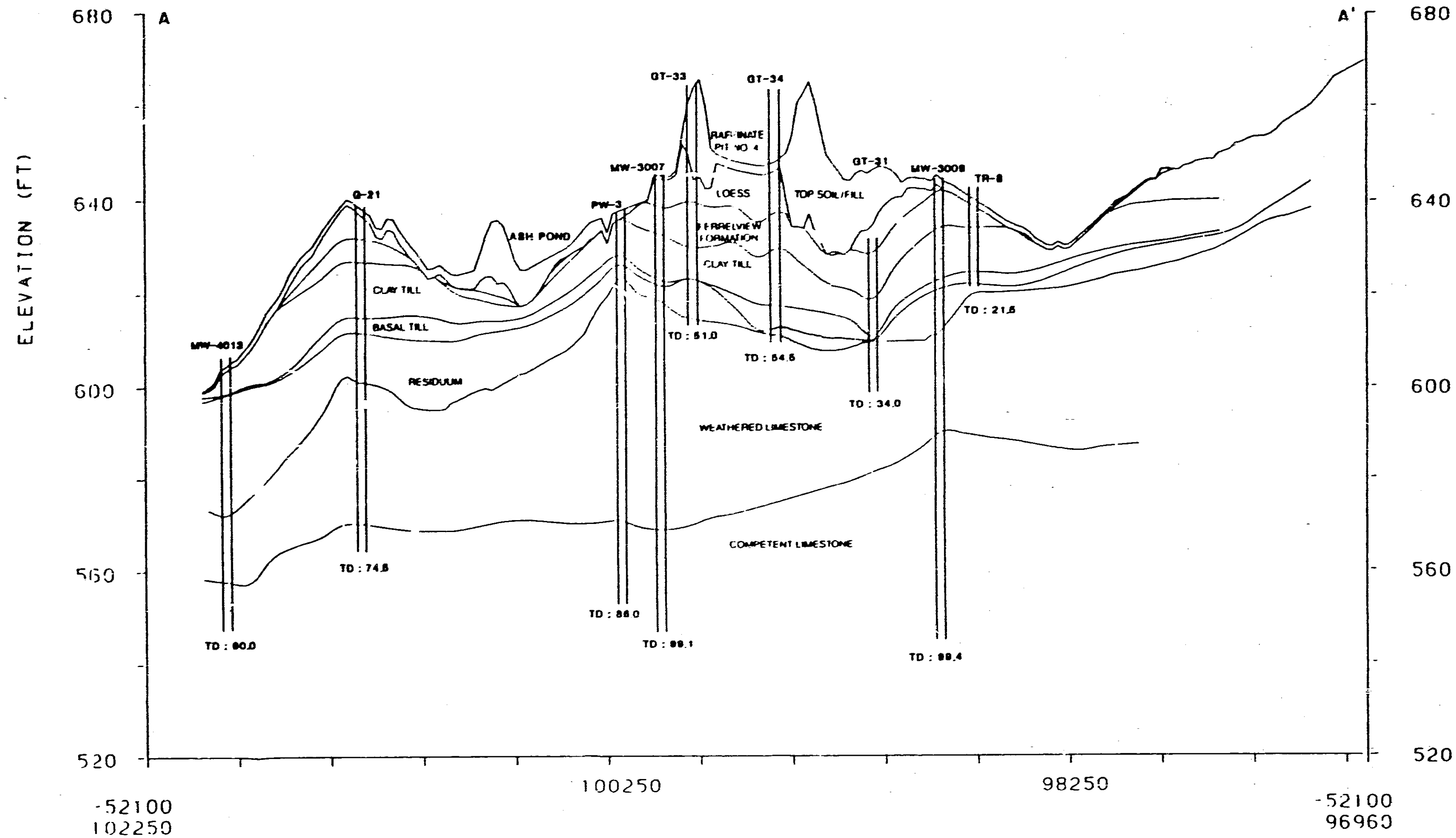
SCALE IN FEET

500 250 0 500 1000

BORHOLE AND TEST PIT LOCATION MAP WITH CROSS SECTIONS

FIGURE 4-9

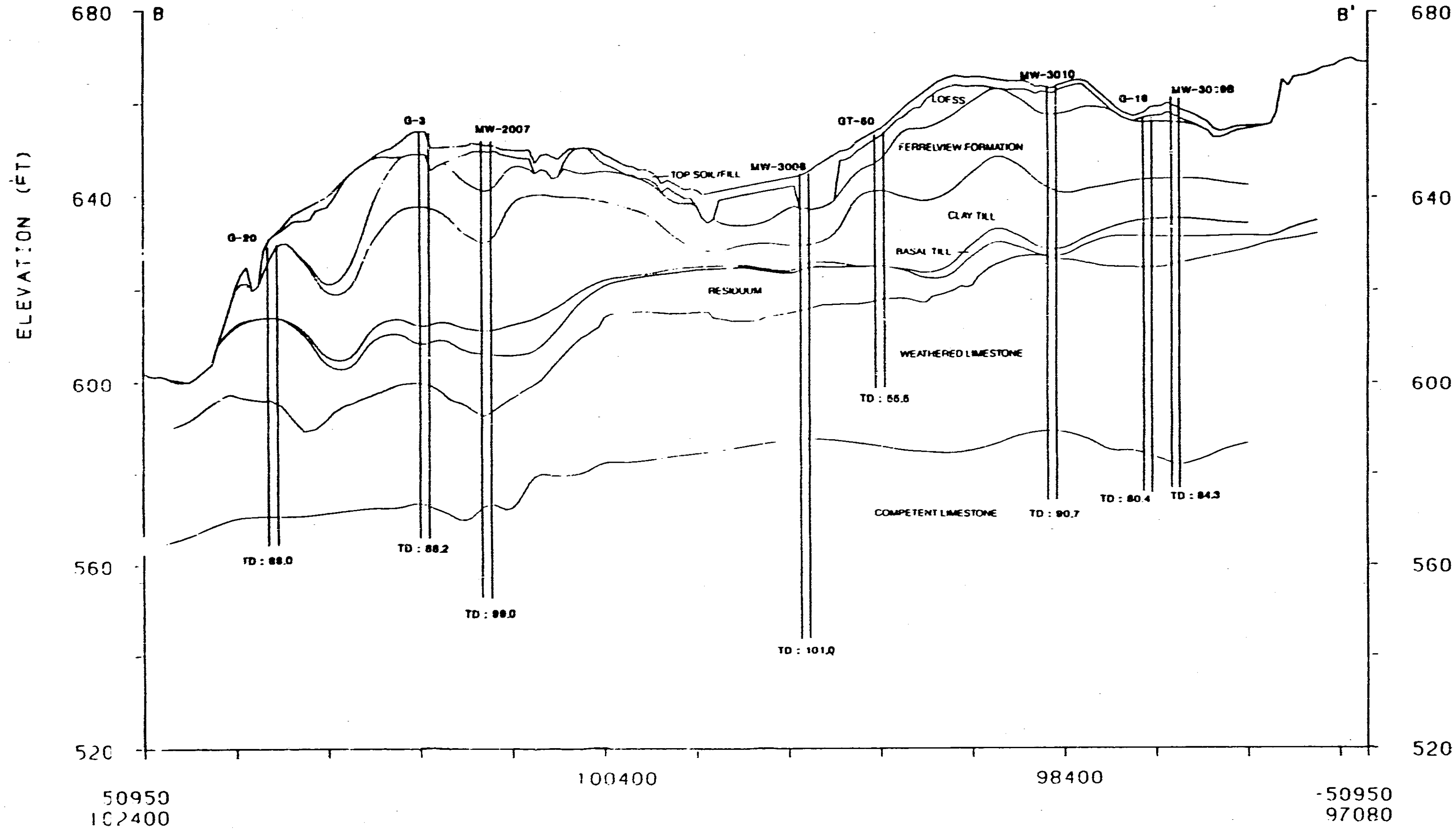
REPORT NO.: DOE/OR/21548-155		EXHIBIT NO.: A/CP/110/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



Scale

Horz Scale 1 unit = 122 M (400 ft)
Vert. Scale 1 unit = 61 M (20 ft)

GEOLOGIC CROSS SECTION A–A'					
FIGURE 4-10					
REPORT NO.	DOE/OR/21548-155		DRAWING NO.:	A/CP/111/1190	
ORIGINATOR:	SDG	DRAWN BY:	GLN	DATE:	11/90

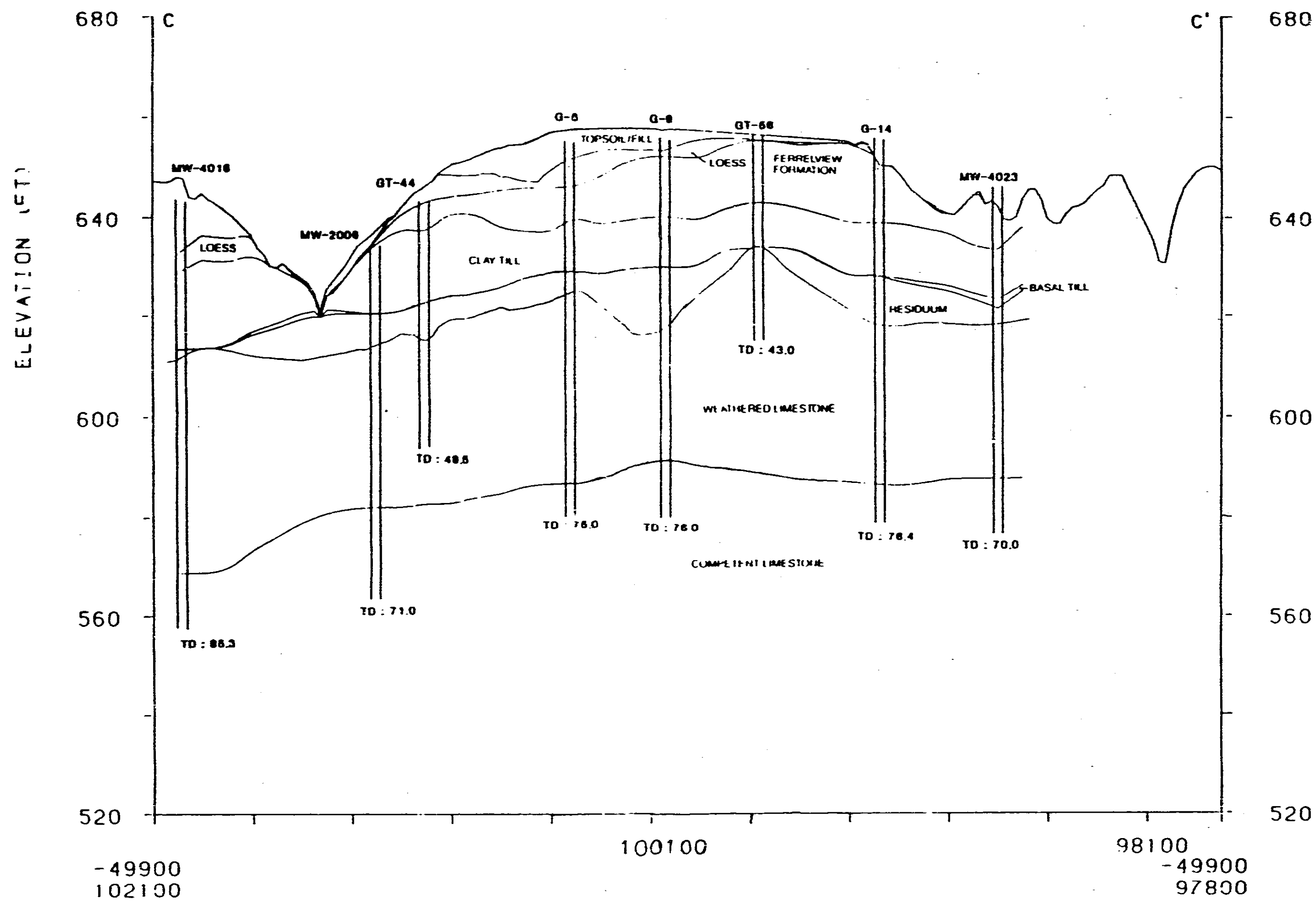


Scale
Horz Scale 1 unit = 122 M (400 ft)
Vert Scale 1 unit = 6.1 M (20 ft)

GEOLOGIC CROSS SECTION B–B'

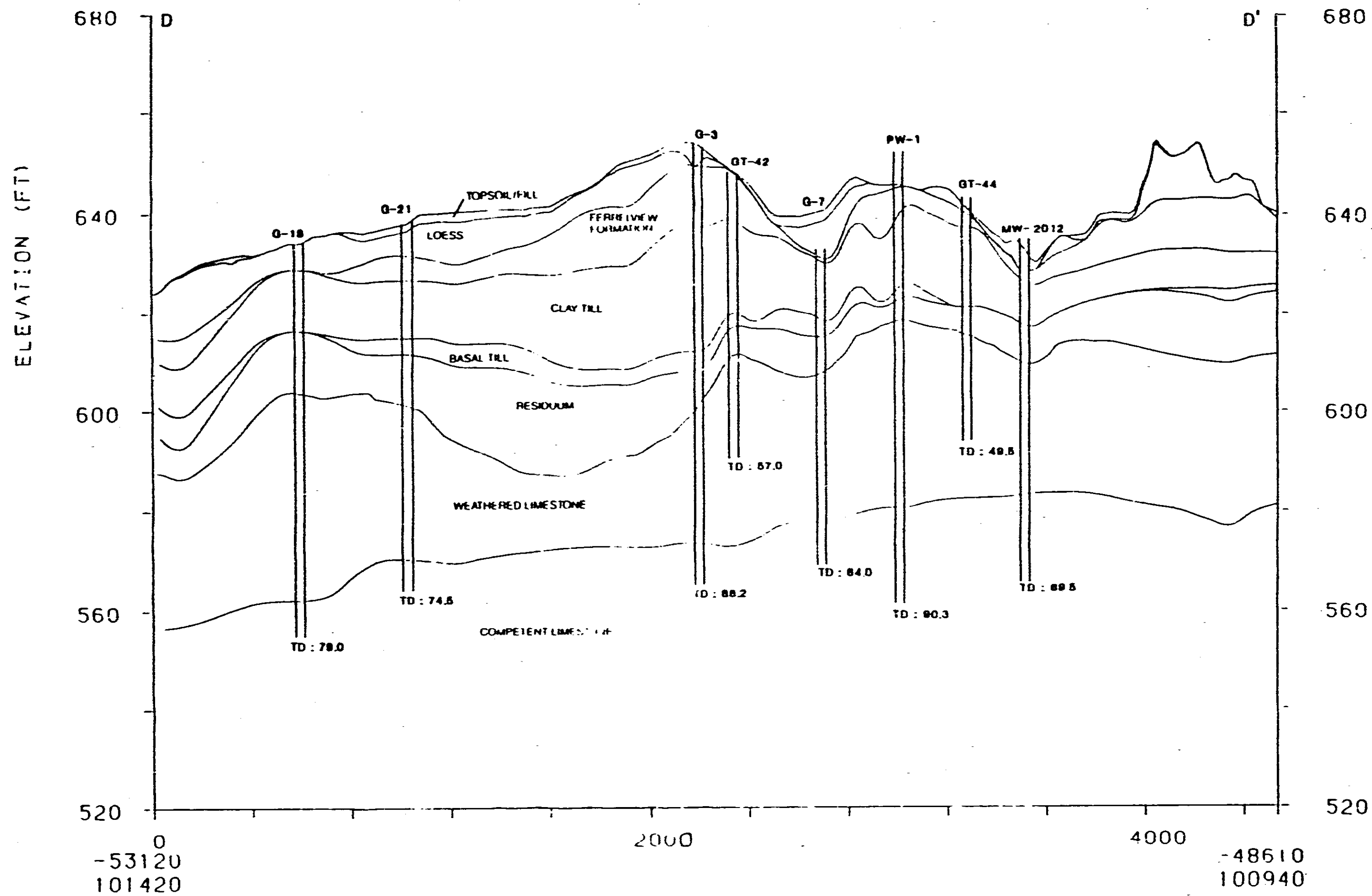
FIGURE 4–11

REPORT NO DOE/OR/21548-155		DRAWING NO.: A/CP/112/1190	
ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90	



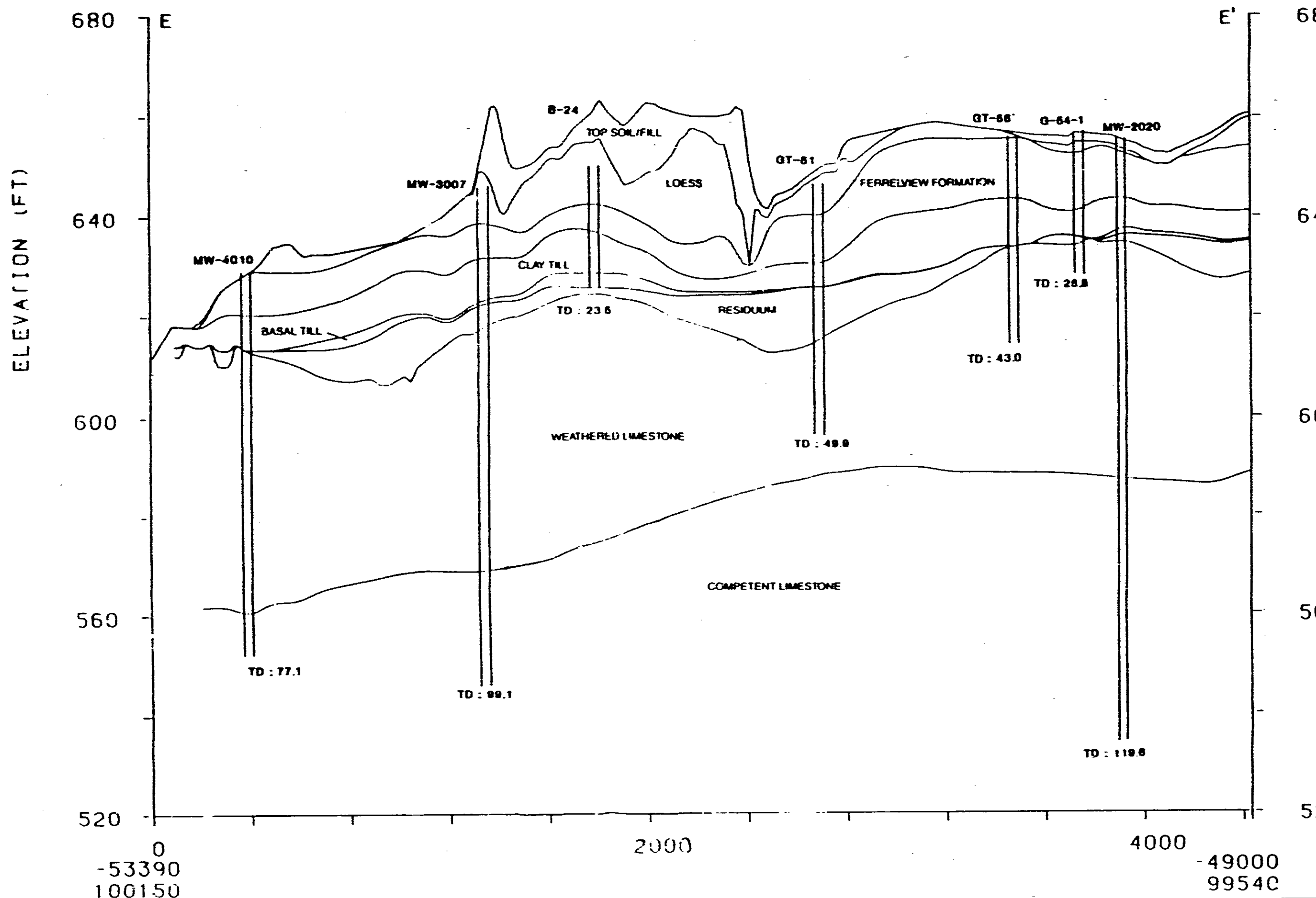
Scale
Horz. Scale 1 unit = 122 M (400 ft)
Vert. Scale 1 unit - 6 1 M (20 ft)

GEOLOGIC CROSS SECTION C-C'	
FIGURE 4-12	
REPORT NO DOE/OR/21548-155	DRAWING NO.: A/CP/113/1190
ORIGINATOR: SDG · DRAWN BY: GLN	DATE: 11/90



Scale
Horz Scale 1 unit 122 M (400 ft)
Vert Scale 1 unit = 6.1 M (20 ft)

GEOLOGIC CROSS SECTION D-D'	
FIGURE 4-13	
REPORT NO DOE/OR/21548-155	DRAWING NO.: A/CP/114/1190
ORIGINATOR: SDG	DRAWN BY: GLN / DATE: 11/90



Scale

Horz. Scale 1 unit = 122 M (400 ft)
Vert. Scale 1 unit = 6.1 M (20 ft)

GEOLOGIC CROSS SECTION E-E'

FIGURE 4-14

REPORT NO. DOE/OR/21548-155	DRAWING NO.: A/CP/115/1190

ORIGINATOR: SDG	DRAWN BY: GLN	DATE: 11/90

<u>Topsoil/Fill</u> - The uppermost soil unit at the site consists of large areas of disturbed fill and a thin veneer of top soil. Fill is highly variable in thickness and composition (Table 2-1). Figure 4-2 is an isopach map of the topsoil/fill. Thickness ranges from 0 to 14 feet, except in the raffinate pit and Ash Pond areas where engineered fills of up to 20 feet are encountered. Large volumes of fill were used in constructing the dikes for the raffinate pits and Ash Pond, where the thickest accumulations were found (BNI 1987). Fill was also used for general site embankment construction, site grading, and drainage control prior to the construction of the chemical plant structures.

The fill material comprising the raffinate pit dikes is unsaturated. Laboratory test results for this material indicate that the moisture content ranges between 15% and 24%, and the dry unit weight is approximately 100 pounds per cubic foot (pcf). Fines content (minus No. 200 sieve) ranges from 78% to 96% and the plasticity index (PI) ranges from 24% to 41%. Permeabilities of the dike fill have been reported at 1.4×10^{-7} cm/sec to 1.6×10^{-9} cm/sec. Piezometers installed in the raffinate pit No. 4 dike during these field investigations have remained dry (Appendix A-1).

<u>Loess</u> - The loess unit underlying the topsoil/fill is randomly scattered across the site, possibly because it is a wind blown deposit (MGS 1977). Loess distribution may also have been affected by extensive reworking of the upper subsurface during site preparation and construction. Loess thicknesses of 10.5 feet were found during these field investigations. Materials from this unit consist primarily of a mottled gray silt to dark yellowish-orange, clayey silt or silty clay. The loess exhibits low plasticity with a PI ranging from 10 to 30. It does

not collapse when wet like other wind deposited soils that are commonly called loess and are typically found in this part of the country. The moisture content of this loess ranges from 15% to 30%; the dry unit weight ranges from 90 to 105 pcf. Fine content usually ranges from 95% to 99%.

Ferrelview - The Ferrelview Formation, which underlies the loess unit is one of the more extensive soil units within the site overburden. This unit is important from the design perspective because it will provide the first natural barrier to contaminant migration below the engineered impermeable liner of the proposed disposal cell. Unit thickness ranges from 0 to 22 feet (Table 2-1 and Figure 4-3). Where encountered in boreholes it is typically dark yellowish-orange to brown with gray mottling. Material composition ranges from a silty clay to a clayey silt, which is often very stiff and exhibits high plasticity (PI from 20 to 50). Laboratory sieve analyses show that most of the material is silt and clay with minor sand. The percent of fines generally ranges from 85% to 98%. The moisture content generally ranges from 15% to 30% and the dry unit weight from 90 pcf to 114 pcf.

The Ferrelview contains a high percentage of silt which may be derived from the underlying clay till. Samples of clay from this formation have a characteristic conchoidal fracture and often exhibit slickensides which may be caused by consolidation and compaction. Permeability test results for the clay portion range between 3.2×10^{-6} and 1.0×10^{-8} cm/sec. In situ permeability tests in boreholes at the proposed temporary storage area have yielded hydraulic conductivities (permeability) less than 1.0×10^{-8} cm/sec within the more clayey zones (Appendix B-2).

Clay Till - The clay till underlying the Ferrelview Formation is the most horizontally extensive soil unit at the WSS. It has been encountered in almost all of the boreholes and test pits on the site. The thickness of this unit ranges up to 30 feet (Figure 4-4). The clay till consists of massive and very stiff yellowish brown silty clay and clayey silt, and has a medium to high plasticity. It contains minor amounts of sand and rounded to subrounded gravel. The PI generally ranges from 25 to 45. The percent of fines (minus No. 200 sieve) ranges from 70% to 95%. The water content ranges from 12% to 35%, and the dry unit weight from 82 pcf to 120 pcf.

Permeability values in the clay till range from 3.0×10^{-6} to 1.4×10^{-8} cm/sec. The unit has a high effective cation exchange ratio.

Basal Till - The basal till unit is found mostly in the western and north-central portions of the site. The unit ranges in thickness from 0 to 8 feet as shown on the isopach map in Figure 4-5. It is typically a yellowish brown, sandy-silty-gravelly clay to clayey gravel. The basal till is the most gravelly soil unit within the site overburden. Therefore, it may have higher permeability than the other overlying soil units. Limited test results (fewer than five tests) show that the PI is generally between 12 and 28, the fines from 28% to 84%, the moisture content approximately 18%, and the dry unit weight from 102 pcf to 120 pcf.

Residuum - The residuum is found immediately above the bedrock throughout the site region. It is laterally extensive on the site. This soil unit is interpreted to be a pre-Pleistocene weathering horizon of the underlying bedrock (BNI 1984). Its thickness varies from 0 to approximately 25 feet as shown in

Figure 4-6. Thicker deposits correspond to bedrock lows; thinner areas are found over bedrock highs. Residuum materials are a distinctive red to yellow. They consist of gravelly clay to gravelly silt, clayey sand, and gravel. The gravel consists of angular, weathered chert or limestone rock fragments. The interstitial clay is often highly plastic and forms a tight matrix around the gravel-sized rock fragments. Samples obtained for testing probably represent residuum with higher fines content because of poor sample recovery from residuum with higher gravel content. Based on the limited test results, the PI for the interstitial clay portion ranges from 28 to 50. Fines in the tested samples range from 35% to 80%. The moisture content and the dry unit weight range from 10% to 25% and 100 pcf to 117 pcf, respectively.

Laboratory testing of a sample from near the top of the residuum yielded a permeability of 5.0 x 10^{-8} cm/sec. In situ permeability, as measured at the soil bedrock interface (base of the residuum), ranges from 6.3 x 10^{-4} cm/sec (BNI 1987). These results most likely reflect hydraulic conductivity values at the residuum/bedrock interface and may not truly represent the residuum unit. Because these permeabilities are much higher than other soil units tested, the residuum is a potential pathway for lateral migration of contaminants. Such migration is possible particularly where abundant rock fragments lie on the bedrock.

4.3 Bedrock

The Burlington-Keokuk Limestone is the first bedrock unit encountered below the unconsolidated overburden materials on the site and is the only bedrock unit discussed in this section. The other bedrock units are not significant from an engineering standpoint because of their great depth. The Burlington-Keokuk

Limestone formation has been sub divided into a weathered limestone horizon and a competent limestone horizon. Information discussed below is presented in Appendixes A-3, B-1, and B-3, and summarized in Table 2-1.

An early investigation (Roberts 1951) reported two major joint sets within the exposed bedrock in the Weldon Spring area: one trends between N30°W and N65°W, and the other trends between N30°E and N72°E. Subsequent regional and site-specific studies have generally confirmed these orientations. Bedrock topography exhibits erosional features interpreted as paleochannels. This interpretation coincides with observed joint set orientations. The joint sets are discussed in more detail in Section 2.

Weathered Limestone - This geotechnical bedrock horizon is highly to moderately weathered and well fractured. The materials are yellowish brown, yellowish gray, grayish orange, and/or white. The limestone contains 40% to 60% chert occurring as distinct interbeds or nodules. It is finely crystalline to coarse grained, thinly to massively bedded, and argillaceous with iron oxide stains in the rock matrix and along fractures. The weathered limestone is characteristically more fractured. It contains numerous solution features. The smaller openings are often lined with calcite and drusy to euhedral quartz, while the larger cavities are often filled with clay or a silt/clay/chert gravel mixture (BNI 1987). These cavities are generally reported as core loss in the boring logs.

The top-of-bedrock contour map (top of weathered limestone, Figure 4-7) shows several depressions. These features are interpreted as pre-glacial, fluvial paleochannels. Their trend is approximately parallel to the regional structure orientations (Figure 2-1) and their location may be structurally controlled.

Bechtel (1983a) identified the largest paleochannel in the subsurface near the power plant (Building No. 401) and the drainage east of Ash Pond. These features were confirmed by geophysical surveys of the area (Geotechnology Services, Inc. 1988).

Core drilling in the weathered limestone for Phase II geotechnical investigations indicated that fracturing is predominantly horizontal and closely spaced. It occurs along the shaley interbeds, bedding planes, and chert interbeds. Fractures are commonly clay filled or mineralized. Rock quality designations (RQD) in boreholes penetrating the weathered limestone show that approximately 73% (see Table 4-1) of this bedrock horizon has very poor to poor RQD values (i.e., it is highly fractured). By contrast about 79% of the rock of the competent limestone bedrock horizon can be classified as having fair to excellent RQD and only 9% as having very poor RQD.

Data from angled borings, cored to intersect the regional northeasterly and northwesterly vertical joint sets, indicate sporadic joints in the weathered limestone immediately below the proposed disposal cell area. Spacing between vertical joints ranges from 2.5 to 18 feet with an average of one every 10 feet. Vertical joints near the weathered bedrock surface are enlarged and often clay filled. Both fracture and joint density in the highly fractured weathered limestone decreases rapidly with depth, and the character of the rock grades into the less fractured competent limestone bedrock horizon.

<u>Competent Limestone</u> - The competent limestone is found 10 to 50 feet beneath the weathered limestone horizon. Figure 4-8 is a contour map of the competent limestone. This fresher rock is gray to light gray, finely to coarsely crystalline, and bedded.

TABLE 4-1 Summary of Rock Quality Designation (RQD) From Coring in Boreholes

STRATIGRAPHIC UNIT	(PERCENTAGE OF CORE RUNS IN GIVEN RANGE) VERY POOR	POOR	FAIR	GOOD	EXCELLENT
Weathered Limestone	41	32	21	5	1
Competent Limestone	9	12	44	25	10

Note: *RQD designations are defined as follows:

Very Poor	0-25%
Poor	26-50%
Fair	51-75%
Good	76-90%
Excellent	91-100%

The competent limestone rock is stylolitic and fossilferous, with 20% to 40% chert nodules and chert beds. Only slightly weathered, if at all, the competent limestone has very little iron oxide staining, and contains fresh pyrite along some fracture surfaces. About 79% of the RQD values can be described as fair to ɾ .llent. Solution features are not common in this unit.

Li'hologic logging indicates that the surface of the competent limestone is much flatter than that of the weathered limestone. It exhibits fewer weat' ring effects and more structural jointing control. Although the surfaces are gentler than the top of bedrock, more frequent vertical fractures may exist in the competent limestone in the areas directly beneath the paleochannels identified in the weathered limestone. These areas may also be the focus of groundwater migration in the shallow bedrock aquifer.

4.4 Groundwater

Groundwater at the WSS is found mainly in the shallow bedrock aquifer comprising an upper zone (weathered limestone) and a lower zone (competent limestone). The groundwater system also includes two important compounds: an unsaturated vadose zone and localized areas of perched groundwater.

Vadose Zone and Soil - The vadose zone is the unsaturated zone of soil and rock between the ground surface and the groundwater table. Because of local and seasonal variations in recharge and discharge, the thickness of the vadose z at the site varies locally, seasonally, and from year to y The variation ranges from less than 35 feet to more than 65 feet. Seasonal variations of groundwater at the site are discussed in more detail in the site Remedial Investigation Report (MKF and JEG 1990a, 1990c).

Water infiltrates from the ground surface through the soil. Since there are no sources of surface water, the chief source of water infiltrating through the vadose zone is precipitation. The quantity of water from this source reaching the vadose zone depends on precipitation intensity and type, the conditions of the soil surface, vegetation, and the physical properties of the soil. Direct infiltration occurs in vegetated areas such as those near Frog Pond, the western portion of the site near Ash Pond and the raffinate pits, and most of the area surrounding the site. The eastern portion of the site is generally paved and/or covered with buildings, which prevents direct infiltration through the surface. Infiltration in this portion is limited to discrete areas such as drainage ditches, deteriorated underground sewer lines, and small exposed areas. Standing bodies of water, such as Ash Pond, Frog Pond, and the raffinate pits may also act as sources for infiltration. The amount of recharge to the subsurface through infiltration is a function of the hydraulic conductivities of the underlying materials.

<u>Perched Groundwater Zones</u> - Areas of perched groundwater are encountered in localized areas. Shallow monitoring wells installed in the chemical plant area have delineated these zones. Perched conditions occur where leakage or infiltration accumulates over low permeability, clay-rich horizons and in discrete permeable lenses. Unsaturated flow is present locally as a result of direct infiltration; leakage from on-site storm water, sewer, and water lines; and leakage from the raffinate pits. Mounded conditions resulting from leakage have been detected beneath the raffinate pits.

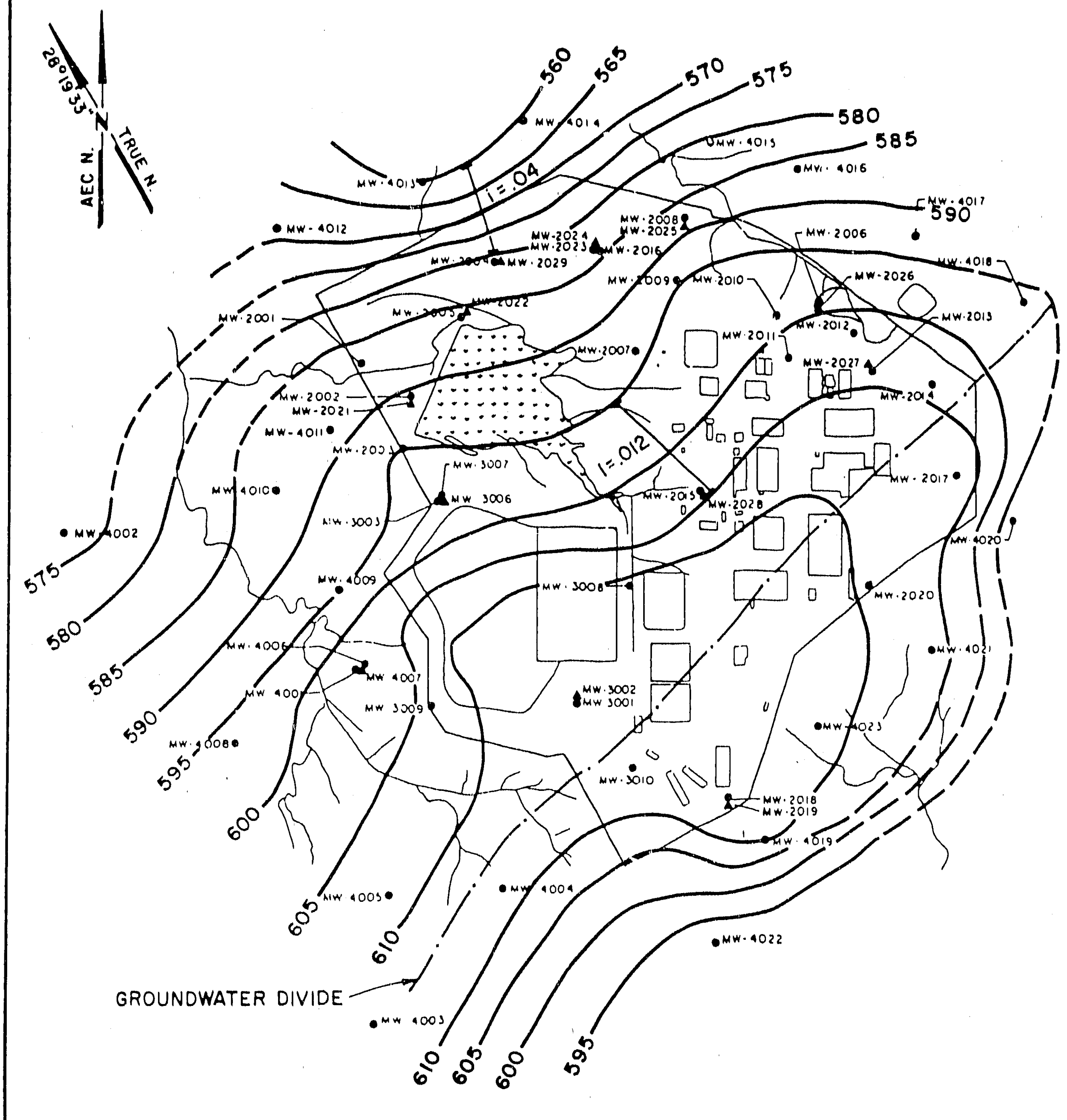
Water-level monitoring from 1986 through June 1990 indicated perched groundwater zones below raffinate pits No. 3 and No. 4 (MKF and JEG 1990a).

Seismic anomalies detected during the geophysical survey in 1984 indicated the possible presence of perched water zones south of raffinate pit No. 3. Geophysical data indicated that the central portion of raffinate pit No. 3 was saturated, and that saturated conditions also existed beneath the unsaturated sludges and soil on the east, west, and south sides of raffinate pit No. 3 (MKF and JEG 1990a).

Perched water was also encountered in boreholes drilled in June 1988 near the administration building. This water originated from a leaking water line. It is stored near the surface, possibly in the pipe trench backfill.

<u>Shallow Bedrock Aquifer</u> - The shallow bedrock aquifer at the WSS is contained within the Burlington-Keokuk bedrock unit. Groundwater elevation (potentiometric surface) contours for the shallow bedrock are shown in Figure 4-15. Potentiometric contours for other aquifers are presented in the site Remedial Investigation Report (MKF and JEG 1990c). Groundwater elevation data for the period August 1989 to August 1990, including piezometers installed for this investigation, are listed in Appendix C.

The potentiometric contour maps show fluctuations in groundwater elevation. The changes may be due to seasonal changes in precipitation and infiltration at the site. In general, precipitation is lowest and evaporation highest in summer (July), and vice versa in winter (December). This is



LEGEND:

● SHALLOW MONITORING WELL

▲ DEEP MONITORING WELL

—600— POTENTIOMETRIC CONTOUR (FT-MSL)

0 800 1600 FT

0 243.8 487.7 M

SCALE

POTENTIOMETRIC SURFACE OF SHALLOW AQUIFER

FIGURE 4-15

REPORT NO.: DOE/OR/21548-156		EXHIBIT NO.: A/CP/116/1190	
ORIGINATOR: SDG	DRAWN BY: GLN		DATE: 11/90

reflected in lower summer groundwater levels for a given location and higher winter levels. In either case, the level of the piezometric surface appears to be below planned excavation depths.

The potentiometric surface elevation for deep wells is generally lower than that for shallow wells. This indicates a net downward vertical gradient within the shallow bedrock aquifer. Another feature of this aquifer is the groundwater divide passing through the eastern portion of the site. Groundwater to the east of this divide flows east-southeast toward the Missouri River. Groundwater to the west of this divide flows north-northwest. The groundwater divide essentially mimics the regional surface water divide. Surface water on the eastern part of the site flows toward the Missouri River; that on the western part flows toward a network of streams and lakes north of the site. Eventually this portion flows into the Mississippi River.

A detailed hydrogeologic study by the PMC is presented in the Draft Aquifer Characterization Data Report (MKF and JEG 1990a).

4.5 Geophysical Model

Results of the geophysical investigation (not including geophysical logging of angled holes) are summarized below and shown in Table 3-1. With the exception of seismic refraction, results of the geophysical surveys are generally restricted to the site perimeter. Despite their limitations, the geophysical surveys indicates a model of the site subsurface that generally corresponds to the geology encountered in boreholes (Geotechnology Services, Inc. 1988).

Generally, three seismic layers were defined by the refraction data. The top (surface) seismic layer ranges in thickness between 1.5 and 15 feet; its velocity varies between 50 and 1,920 feet per second (fps). The second seismic layer ranges in thickness from 22 to 60 feet and has a velocity between 2,220 fpc and 5,880 fpc. This layer may correspond to an increase in moisture content near the top of the Ferrelview Formation. The deepest seismic layer has a seismic velocity of 11,600 fpc to 18,200 fpc and generally corresponds to the top of moderately weathered to fresh limestone. However, this layer may also include basal till and residuum at some locations. This layer dips generally toward the north-northeast and undulates. Local relief of 15 feet is commonly observed along the seismic lines. The higher velocities are thought to indicate zones of dense competent limestone.

The electromagnetic (EM) resistivity data generally indicate four layers. The surface layer ranges in thickness from 5.5 to 18 feet and has resistivities ranging from 8 to 45 ohm-meter (ohm-m). This layer is composed of fill, loess, and/or the Ferrelview Formation. The second layer is 4.5 to 16.5 feet thick and ranges from 2 to 170 ohm-m in resistivity. It usually corresponds to zones in the clay till, but may also indicate areas of higher clay content in the Ferrelview Formation. The bottom of this layer may correspond to a decrease in clay content. The third EM layer ranges in thickness from 3.5 to 33 feet and in resistivity from 13 to 24 ohm-m.

The only area where usable spontaneous potential (SP) data were collected is near the north corner of the site. The SP data contain numerous anomalies which may be the result of varying soil thicknesses, conductivity, moisture content, and/or features in the limestone bedrock such as clay filled fractures. The anomalies generally range in magnitude from 4 to 12 volts.

Models based on the results of the four vertical direct current electrical sounding (VES) surveys show five (sometimes six) layers. The surface resistivity layer varies both in resistivity and thickness due to the moisture content, type of soil, and the vegetation present on the surface. The resistivities of the second and third layer range, from 20 to 35 ohm-m and 3 to 20 ohm-m, respectively. Both of these layers may include the clay till. Variations in resistivity are probably due to changes in clay and/or moisture content. The fourth layer ranges in resistivity from 22 to 120 ohm-m. This layer may correspond to the basal till or highly weathered limestone. Moderately weathered limestone may be indicated by the fifth layer which ranges in resistivity from 480 to 1,000 ohm-m.

4.6 On-Site Borrow Sources

Overburden soils from the ground surface to shallow depths (approximately 12 feet) were investigated as possible sources of borrow material. These soils are representative of the upper soil units such as fill, loess, and Ferrelview clay that are generally found on the site. The soils encountered consist of silty clays and gravelly clays (fill) with medium to high plasticity and a consistency from medium stiff to very stiff. The soils are generally classified as CL and CH according to the Unified Soil Classification System. The moisture content ranges from 14% to 28% and the specific gravity from 2.51 to 2.74. The plasticity index is between 6 and 56. The proportion of fines ranges from 89% to 99%. Compaction test results using the Standard Proctor (ASTM D 698) method show an optimum moisture content ranging from 12% to 21% and a maximum dry unit weight ranging from 102 pcf to 116 pcf. If these soils are uncontaminated they are, with few exceptions, suitable for use as earth fill materials.

4.7 Off-Site Borrow Sources

Except for the 1-foot thick topsoil layer, the soils encountered at borrow source No. 1 are mainly silty clay and clayey silt with occasional thin layers (1 to 2 feet) of gravelly clay. The clays and silts are yellowish brown to yellow mottled with gray and brown colorings, stiff to hard, and low to highly plastic. The plasticity index from laboratory test results, with the exception of one non-plastic sample, ranges from 10 to 44. The moisture content usually ranges from 11% to 28% and the specific gravity from 2.55 to 2.74. The percent of fines passing the No. 200 sieve ranges from 62% to 98%. Standard Proctor compaction test (ASTM D 698) results show that the optimum moisture content and the maximum dry unit weight range from 15% to 21% and 100 pcf to 116 pcf, respectively. Preliminary evaluation suggests that this is a good source for clayey materials for disposal cell construction. It is tentatively estimated that the amount of clayey soils available for construction exceeds the required design volume required by the 1989 conceptual design.

The soils encountered at borrow source No. 2 contain a high percentage of sands and gravels. The soils generally consisted of clayey silt; gravelly, clayey silt; and silty, sandy gravel. Due to the high proportion of coarse grained particles, this source is considered to be unsuitable for clayey soil borrow. However, the upper clayey silts may be utilized as a source of material to be used with vegetative cover or as general backfill material. Limited test results on this layer show the soil is classified as CL or ML according to the Unified Soil Classification System with a PI of 12 to 21. The moisture content ranges from 23% to 28% and specific gravity from 2.62 to 2.71. The proportion of fines ranges from 90% to 97%. Standard Proctor compaction tests show that the optimum moisture content ranges from 15% to 18% and the maximum dry unit weight from 105 pcf to 110 pcf.

5 GEOTECHNICAL LABORATORY TESTING

5.1 Testing Programs

Geotechnical laboratory tests were used to determine the engineering properties of the overburden units within the chemical plant and raffinate pit areas, and at potential on-site and off-site clayey borrow sources. These data are needed for design studies of the proposed disposal cell and related facilities. In addition, geotechnical data including the engineering properties of the soil units were necessary for the site suitability study (MKF and JEG 1990b) which will lead to a decision on the location of the disposal cell at the Weldon Spring site.

Since the start of the MKF geotechnical field investigations in May 1988, a total of 11 laboratory testing programs have been implemented. These programs were planned in stages to facilitate laboratory testing as samples were obtained during the field investigations. Each test program specified which soil samples would be tested, the types of tests, the standard testing methods, and special testing instructions. Table 5-1 summarizes these programs, including the designated testing laboratories and the dates the programs were transmitted for testing. Three of these testing programs were prepared for soil samples from the staging and water treatment facility areas at the quarry. Detailed information concerning these programs is presented in the Quarry Geotechnical Investigations Report (MKF and JEG 1990c).

TABLE 5-1 Summary of Soil Testing Programs

TEST PROGRAM #	REQUEST NO.	DATE REQUESTED	PURPOSE	LABORATORY
1	1	06/23/88	Administration building	Chen-Northern, Inc.
2	2	07/27/88	Raffinate pit No. 4	Chen-Northern, Inc.
3	3	10/14/88	Disposal cell	Chen-Northern, Inc.
4	4	05/11/89	Quarry staging area	Geotechnology Services, Inc.
5	2A	05/26/89	Disposal cell	Geotechnology/metaTRACE/Chen-Northern
6	3A	06/26/89	Borrow sources	Geotechnology/metaTRACE/Chen-Northern/ Hannibal Testing
7	2B	08/25/89	Disposal cell	Geotechnology Services, Inc.
8	5	10/11/89	Temporary storage area	Geotechnology Services, Inc.
9	6	05/07/90	Quarry staging area	Geotechnology Services, Inc.
10	7	06/26/90	Quarry staging area	Geotechnology Services, Inc.
11	8	08/03/90	Material staging area	Geotechnology Services, Inc.

The four laboratories utilized for the testing were:

- Chen-Northern, Inc., Denver, CO

- Geotechnology Engineering and Environmental Services Inc., St. Louis, MO (Geotechnology).

- MetaTRACE, Inc., Earth City, MO

- Hannibal Testing Laboratories, Inc., Hannibal, MO

Chen-Northern was retained to perform the first three programs in 1988. Geotechnology was awarded the contract for laboratory testing in 1989 and 1990. Because of limited ability to perform some tests, Geotechnology subcontracted cation exchange capacity tests to metaTRACE, Inc., and the capillary moisture tests to Chen-Northern, Inc.

Soil samples were shipped in specially constructed wooden boxes to Chen-Northern in Denver by overnight air freight. All samples tested by Chen-Northern were discarded following the testing.

Samples designated for Geotechnology were transported to their St. Louis location in site vehicles. The samples from the materials staging area were returned to the site for storage for possible future testing. The rest were discarded after testing.

Fourteen in situ samples taken by drive tube methods from the vertical walls of off-site clay borrow test pits by Hannibal Testing Laboratory were tested at their laboratory for moisture content and density.

Most of the laboratory soil tests were performed in accordance with the American Society for Testing and Materials (ASTM) standard test procedures (ASTM 1990). The triaxial shear tests, which included the consolidated undrained compression tests with pore pressure measurements (R tests); the unconsolidated undrained compression tests (Q tests); and the triaxial constant-head permeability tests were all performed in accordance with the methods described in U.S. Army Corps of Engineers Manual EM 1110-2-1906 (COE 1970). The cation exchange capacity test was performed in accordance with Soil Conservation Service Method 5A3A.

Special instructions for slight modifications to the standard test methods were included in the test program. These modifications usually included specified confining pressures for triaxial shear and consolidation tests, consolidation loading increments, and density for remolded specimens.

Requested laboratory tests and methods used in the testing programs included (ASTM 1990, COE 1970, SCS 1967):

- Sieve Analysis - ASTM C136
- Sieve Analysis with Hydrometer - ASTM D422
- Atterberg Limits - ASTM D4318
- Shrinkage Limit - ASTM D427
- Unit Weight/Void Ratio - ASTM C29
- Specific Gravity - ASTM D854
- Moisture Content - ASTM D2216
- Dry Density - ASTM D2937
- One Dimensional Consolidation Test - ASTM D2435
- Triaxial (R) Shear Strength Test - EM 1906
- Triaxial (Q) Shear Strength Test - EM 1906
- Triaxial Permeability - EM 1906

- Capillary-Moisture Relationships - ASTM D2325 and D3152
- Moisture-Density Relationships - ASTM D698
- Cation Exchange Capacity - SCS 5A3A

Table 5-2 shows the number of tests by each test method on samples from the administration building area, the material staging area, the raffinate pit No. 4 dike, the clay borrow sources, and the chemical plant area including the TSA. A brief discussion of the significance and applicability of each type of test follows:

- Gradation Test (Sieve Analysis) with or without Hydrometer - The gradation test, or sieve analysis, determines the distribution of the various particle sizes in soil. Standard sieves are used to separate particles of all sizes except silts and clays, for which a hydrometer analysis is used. Particle size distribution is used to classify soils, estimate permeability (using empirical correlations), and analyze the potential for particle migration at the interface of two soils.

- Atterberg Limits and Shrinkage Limit Tests - Atterberg limits are index properties, and include liquid limit, plastic limit, and plasticity index. These properties are used in classifying soils and estimating relative plasticity, consistency, and activity. In some cases, these properties also correlate with compressibility, permeability, compactability, shrink-swell, and shear-strength characteristics.

  The shrinkage limit is typically assumed to be the amount of water required to fill the voids of a cohesive soil at

TABLE 5-2 Summary of Laboratory Tests

LABORATORY SOIL TEST	CHEMICAL PLANT	MATERIALS STAGING AREA	RAFFINATE PIT #4 DIKE	BORROW SOURCES	ADMINISTRATION BUILDING
Sieve Analysis, ASTM C136	12	-	1	0	-
Sieve Analysis with Hydrometer, ASTM D422	51	17	15	18	6
Atterberg Limits, ASTM D4318	60	16	14	20	10
Shrinkage Limit, ASTM D427	4	-	-	4	7
Specific Gravity, ASTM D854	35	-	3	13	3
Moisture Content, ASTM D2216	91	-	16	20	11
Dry Density, ASTM D2937	57	-	10	0	8
1-D Consolidation, ASTM D2435	21	-	-	3	3
Triaxial (R) Test, EM 1906	11	-	5	6	-
Triaxial (Q) Test, EM 1906	17	-	-	6	2
Triaxial Permeability, EM 1906	35	17	1	13	-
Capillary Moisture, ASTM D2325 & D3152	8		-	12	-
Moisture-Density, ASTM D698	-	16	-	12	1
Cation Exchange Capacity, SCS 5A3A	8	-	-	6	-

the minimum void ratio obtained by drying. It is used todetermine the shrinkage potential or possibility of development of cracks in cohesive soils.

- Specific Gravity Test - Specific gravity is used to determine the unit weight of the solid particles of a soil with respect to the unit weight of water. This factor is used in many equations expressing relationships among air, water, and soil solids.

- Moisture Content Test - The moisture content test determines the pore water content (% of dry soil weight) of a soil-water mixture. It is used extensively in determining soil index properties, soil behavior correlations, and soil consistency. It is used in most equations expressing relationships among air, water, and solids in a soil. When used with Atterberg limits, moisture content can indicate whether a soil is preconsolidated.

- Dry Density Test - The dry density test is used to determine the dry unit weight of a soil. It is the basis for determining overburden stresses and other soil properties useful in settlement calculations. In situ density can also be used as an indicator of past stresses and loadings on the soil. It also helps to determine soil horizons and volume-weight relationships.

- One-Dimensional Consolidation Test - The one-dimensional consolidation test determines consolidation characteristics for a soil. A sample is axially loaded and allowed to drain while being restrained laterally. The decrease in void ratio is measured as a function of loading and time. The consolidation characteristics obtained are used to estimate the rate and magnitude of consolidation under actual loading conditions. It is also used to estimate the permeability of the soil.

- Unconsolidated, Undrained Triaxial Compression (UU or Q) Test - The UU triaxial compression or Q test determines the total shear strength parameters and stress/strain characteristics of a soil. The test simulates short-term or end-of-construction conditions by not allowing excess pore pressures resulting from confining and axial stresses to dissipate. A series of samples of a soil are tested at different confining pressures to develop the parameters and characteristics.

- Consolidated, Undrained Triaxial Compression (CU or R) Test - The CU triaxial compression or R test determines total and effective shear strength parameters and stress/strain characteristics of a soil. The test simulates long-term conditions by allowing consolidation of the soil sample before uniaxial loading is applied. A series of samples of a soil are tested at different consolidation/confining pressures to develop the parameters and characteristics.

- Constant-Head Triaxial Permeability Test - The constant-head triaxial permeability test measures the permeability, or hydraulic conductivity, of a saturated soil that is subjected to a constant head and hydraulic gradient. The tests are performed at different confining pressures to simulate in situ loading conditions. The hydraulic conductivity of a soil is used to determine seepage and groundwater flow rates. It is also used as a measure of relative imperviousness.

- Capillary-Moisture Relationships - The capillary-moisture relationship test determines the moisture content retained in the soil subjected to a given soil-water tension. The tensions applied usually range from 1 to 15 atmospheres (atm). The test results are used to estimate the long-term moisture content of the soil, for radon barrier design, and for correlating unsaturated hydraulic conductivities with soil saturation.

- Moisture-Density Relationships (Compaction Test) - The compaction test develops moisture-density characteristics for a soil that is compacted at a specified energy level. Each soil has unique moisture-density characteristics. The test utilizes a standard compaction mold and a hammer to compact the soil. A series of tests at varying moisture contents yields a compaction curve from which the maximum dry density and the optimum moisture content (OMC) can be determined. The minimum density and range of water content to be used in the field can then be specified.

- Capillary-Moisture Relationships - The capillary-moisture relationship test determines the moisture content retained in the soil subjected to a given soil-water tension. The tensions applied usually range from 1 to 15 atm. The test results are used to estimate the long-term moisture content of the soil for radon barrier design and for correlating unsaturated hydraulic conductivities with soil saturation.

- Cation Exchange Capacity - The cation exchange capacity (CEC) is the charge or electrical attraction for cations per unit mass as measured in milequivalents per 100 grams of soil. The CEC indicates various clay minerals such as kaolinite, illite, or montmorillinite in the soil. The clay mineral types are used in evaluating the expansiveness of a soil.

5.2 Soil Test Results

The first group of soil test results were received by the PMC from Chen-Northern, Inc., in August 1988. These were for samples taken during investigations regarding the foundation of the administration building. Additional test results were received from Chem-Northern, Inc., in October 1988 and again in January 1989. These were from the tests of soil samples from the raffinate pit No. 4 dike and the chemical plant site boreholes drilled in 1988.

During 1989 and into 1990 the soil samples selected for testing were sent to Geotechnology, Inc., in St. Louis. Results were transmitted periodically to the PMC until October 1990. These results have been filed by the PMC, and the number of tests per method has been entered into a database which can be sorted by sample location and sample number, and by test method.

The results of tests for physical soil characteristics are presented in Table 5-3. These characteristics include sieve analysis, Atterberg limit, shrinkage limit, specific gravity, natural moisture content, and dry unit weight. Properties such as optimum moisture content and dry unit weight, which are the results of compaction tests, and coefficient of permeability are also listed in the table. Soil test data sheets, including

TABLE 5-3 Average Values of Laboratory Test Results of Overburden Soil Samples (Based on test results from Oct. 88 to May 89)

Overburden Unit	% of Grain Size				Atterberg Limit		Unified Soil Classif.	Specific Gravity	Unit Weight (pcf)		Moisture Content (%)
	Gravel	Sand	Silt	Clay	Liquid Limit (%)	Plasticity Index (%)			Dry	Wet	
Loess	0	2	68	30	41	22	CL	2.71	100	113	22.5
Ferrelview	0	5	60	35	48	31	CL-CH	2.68	101	124	23.0
Clay Till	1	19	40	40	52	36	CL-CH	2.71	108	128	19.8
Basal Till/ Clay Till	0*	26	39	35	37	20	CL	2.61	106	125	18.3
Residuum	28	24		48	72	50	GC	2.79	88	118	22.9
Raffinate Pit Dike Fill	0	12	48	40	49	31	CL	2.63	102	125	20.8

NOTES: 1. Values in table are average values based on available laboratory test results as of May 1989. (--) denotes result not available yet or no test was performed. Average values from additional testing subsequent to May 1989 will be published in addendum or separate report.

2. Average values were calculated using arithmetic mean method unless otherwise noted.

3. All tests are performed on disturbed or undisturbed samples.

* Gravel portions of the basal till usually not obtainable from conventional sampling methods used. Therefore, gradation values are for the non-gravel size particles.

TABLE 5-3 Summary of Laboratory Testing Soil Samples (Based on test results from October 88 to May 89) (Continued)

Overburden Unit	Hydraulic Conductivity (Permeability) (cm/sec)	Consolidation	Coeff. of Consolidation (cm^2/sec)	Shear Strength: Total (from UU test)	Shear Strength: Effective (from CU test)
Loess	6.2×10^{-6}	$C_c = 0.144$ (a) $C_r = 0.022$ (b)	4.1×10^{-4}	c = 1160 psf Ø = 0	c\` = 240 psf Ø\` = 35°
Ferrelview Formation	8.9×10^{-8}	$C_c = 0.173$ $C_r = 0.033$	6.0×10^{-4}	c = 1380 psf Ø = 0	c\` = 250 psf Ø\` = 19°
Clay Till	2.6×10^{-8}	$C_c = 0.153$ $C_r = 0.042$	1.4×10^{-4}	c = 1140 psf Ø = 0	c\` = 110 psf Ø\` = 26°
Basal Till/Clay Till	3.8×10^{-8}	-- --	--	c = 1060 psf Ø = 0	-- --
Residuum	5.0×10^{-8}	$C_c = 0.142$ $C_r = 0.052$	1.0×10^{-4}	--	c\` = 260 psf Ø\` = 15°
Raffinate Pit Dike Fill	--	-- --	--	--	c\` = 310 psf Ø\` = 17°

NOTE: See also notes shown on Page 1 of Table

C_c = virgin compression ratio
C_r = recompression ratio
Values averaged using geometric mean method.

results from triaxial-shear strength and consolidation tests, are presented in Appendix D. Minor corrections such as typographical errors, mislabelling, and laboratory originated revision values, have been incorporated into the data sheets for the sake of consistency and are noted accordingly on the revised data sheets. A summary of permeability test result corrections is included in Appendix D-1.

Test results from the Phase I investigation program (tests performed by Chen-Northern) were evaluated for conceptual design of the disposal cell in 1989. Average values for each overburden soil unit were calculated for each test type and are shown in Table 5-4.

Test results for the Phase II investigation program will be evaluated in detail 1991.

TABLE 5-4 Summary of Soil Test Results
(10 Sheets)

SUMMARY OF SOIL TEST RESULTS

(sheet 1 of 10)

Hole or Trench Number	Sample Number	Depth [ft] From	Depth [ft] To	Laboratory Classificatn USCS	Mechanical Analysis [%] Gravel	Mechanical Analysis [%] Sand	Mechanical Analysis [%] Fines	Atterberg Limits [%] LL	Atterberg Limits [%] PL	Shrnkage Limit [%]	Specfic Gravity G	Natural W [%]	Natural γ_d [PCF]	Std. Proctor Compaction (ASTM D698) Optimum W [%]	Std. Proctor Compaction (ASTM D698) Optimum γ_d [PCF]	Triaxial Permeability γ_d [PCF]	Triaxial Permeability (conf. pressure) [KSF]	Triaxial Permeability k [cm/sec]	Consolidation C_c	Consolidation C_r	Consolidation P'_o [KSF]	Soil Unit
GT-31	ST-02	4.0	6.5	CL	0	10	90	36	20		2.68	20.9	103.1									F
	ST-06	18.0	20.5		0	4	96					20.1	104.6									F
GT-32	ST-01	1.0	3.5	CL	0	4	96	32	13			20.1	101.4									L
	ST-02	9.0	10.5	CH	0	15	85	64	46			26.4	90.5									F
	ST-05	18.5	21.0		3	8	89					36.3	82.9									C
GT-33	ST-04	5.0	7.5	CL	0	4	96	45	27													DF
	ST-14	22.0	24.5	CL	0	6	94	30	10			15.7	100.7			90.4		1×10^{-7}				L
GT-34	SS-05	7.0	8.5	CH	0	20	80	55	38			20.8										DF
	SB-17	27.0	28.5	CL	0	7	93	40	23			15.4	111.0									F
GT-35	SS-02	3.0	4.5	CH	0	10	90	59	41			22.5										DF
	SS-08	13.0	14.5	CL	0	14	86	49	31		2.63	18.9										DF
	SS-11	17.5	19.0									16.0										DF
	SB-15	25.0	26.5	CL	0	5	95	42	25			20.8	104.6									DF
	SS-20	31.0	32.5	CL	0	1	99	45	25			19.8										L
GT-36	ST-06	8.5	11.0	CL	0	22	78	48	32			22.6	100.2									DF
	SS-09	14.0	15.5	CL	0	5	95	43	24			23.8										DF
	ST-12	18.5	21.0	CL	0	1	99	46	29		2.70	21.5	97.8									L
GT-37	SB-01	1.5	3.0	CH				58	34	8.3												L
	ST-03	5.0	7.5	CL	0	3	97	45	29	11.3	2.67	24.4	101.2									L
	SS-05	9.0	10.5									20.7										F
	ST-06	10.5	13.0	CH	0	3	97	55	39			25.1	100.3									F
GT-38	SS-02	3.0	4.5	CL	0	2	98	37	18	19.4		21.8										L
	ST-03	4.5	7.0	CL	0	1	99	48	29	18.4		27.3	96.8									L
GT-39	SS-02	4.0	5.5	CL	0	1	99	38	19	15.9		26.8	105.0									L
	ST-03	5.5	8.0								2.66	23.2	100.6	17.9	107.5							F
	ST-04	9.5	12.0																			F

SUMMARY OF SOIL TEST RESULTS

(sheet 2 of 10)

Hole or Trench Number	Sample Number	Depth [ft]		Laboratory Classificatn	Mechanical Analysis [%]			Atterberg Limits [%]		Shrnkage Limit	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698)		Triaxial Permeability		Consolidation			Soil Unit
		From	To	USCS	Gravel	Sand	Fines	LL	PL	[%]		w [%]	γ_d [PCF]	Optimum w [%]	Optimum γ_d [PCF]	γ_d [PCF]	(conf. pressure) [KSF] k [cm/sec]	C_c	C_r	P'_o [KSF]	
GT-39	ST-05	12.0	14.5	CH	0	6	94	65	48		2.68	25.1	99.4								F
GT-41	SS-02	3.5	4.0	CL				48	28	11.8		27.9	94.4								
	SS-02	4.0	4.5	CL				48	25	12.3		28.6	95.1								
	SS-04	6.0	7.5	CL				40	23			24.5									F
GT-42	ST-05	9.5	12.0									16.9	113.3								F
	SB-08	17.0	18.5									12.4	119.4								C
GT-43	SS-02	1.5	3.0									14.5									L
	ST-04	6.0	8.5					48	32		2.66	24.4	98.6			95.7 to 91.6	(0.07) < 9 x 10^{-4}** (0.29) < 7 x 10^{-8}**				F
	ST-10	21.5	23.75	CL	0	26	74	47	32		2.71	18.6	106.5								C
	SB-17	40.0	41.5					31	12		2.59	17.6	109.9			106.3 to 106.8	(0.43) < 6 x 10^{-8}** (0.72) < 4 x 10^{-8}**				B
	SS-19	50.0	51.0		29	46	25					20.4									R
GT-44	ST-03	5.0	7.5									17.6	106.2								C
	SB-10	22.5	24.0									45.0	76.5								R
GT-45	SS-01	1.0	2.5									18.1									
	ST-04	7.5	10.0									21.3	103.3								F
	ST-07	15.0	17.5									22.1	104.5			106.5 to 102.8	(0.21) < 8 x 10^{-5}** (0.58) < 2 x 10^{-8}**				C
GT-46	ST-03	5.0	7.5									22.1	99.6			103.7 to 103	(0.07) < 5 x 10^{-4}** ((0.43) < 3 x 10^{-6}**	0.202	0.027	6.6	F
GT-47	ST-05	10.0	12.5	CL	0	2	98	45	28		2.69	22.9	99.5					0.216	0.06	6.5	F
	SB-09	20.0	21.5					51	36			20.5	72.4								C
	SS-14	32.3	34.0									14.8									R

B = Basal Till C = Clay Till DF = Dike Fill
F = Farrelview Clay L = Loess/Fill R = Residuum

SUMMARY OF SOIL TEST RESULTS

(sheet 3 of 10)

Hole or Trench Number	Sample Number	Depth [ft] From	Depth [ft] To	Laboratory Classificatn USCS	Mechanical Analysis [%] Gravel	Mechanical Analysis [%] Sand	Mechanical Analysis [%] Fines	Atterberg Limits [%] LL	Atterberg Limits [%] PI	Shrinkage Limit [%]	Specfic Gravity G	Natural w [%]	Natural γ_d [PCF]	Std. Proctor Compaction (ASTM D698) Optimum w [%]	Std. Proctor Compaction (ASTM D698) Optimum γ_d [PCF]	Triaxial Permeability γ_d [PCF]	Triaxial Permeability (conf. pressure) [KSF] k [cm/sec]	Consolidation C_c	Consolidation C_r	Consolidation P'_o [KSF]	Soil Unit
GT-48	ST-03	7.5	10.0	CL	0	3	97	41	23			27.7	90.3								F
	ST-05	12.5	15.0									22.4	101.3			101.3 to 99.2	(0.14) $< 6 \times 10^{-4}$** (0.50) $< 1 \times 10^{-8}$**				F
	ST-07	17.5	20.0	CH	0	2	98	60	43		2.67	27.8	93.9					0.335	0.086	6.1	F
	SS-12	30.0	31.5									18.5									R
																					-
GT-53	ST-02	2.5	5.0	CL	0	1	99	47	29		2.72	21.1	104.7			107.7 to 98.2	(0.07) $< 3 \times 10^{-4}$** (0.43) $< 2 \times 10^{-5}$**	0.23	0.032	30.0	L
	ST-06	12.5	15.0	CH	0	5	95	63	45		2.74	26.9	95.8					0.26	0.075	15.0	C
	SB-08	17.5	19.0									14.0	114.3			113.8 to 100	(0.22) $< 2 \times 10^{-7}$** (0.58) $< 7 \times 10^{-8}$**				C
	SS-14	32.5	34.0									18.0									R
GT-51	SS-01	0.0	1.5					33	11			22.9									L
	ST-03	5.0	7.5	CL	0	2	98	44	28		2.63	21.4	101.6								F
	ST-07	15.0	17.5	CL	0	30	70	39	25		2.68	17.5	112.6			113.2 to 108.6	(0.22) $< 3 \times 10^{-6}$** (0.58) $< 2 \times 10^{-8}$**				C
	SB-09	20.0	21.5	CH	4	15	81	74	50		2.79	33.7	88.7			92.7 to 87.7	(0.29) $< 3 \times 10^{-7}$** (0.65) $< 5 \times 10^{-8}$**	0.265	0.097	5.0	R
	SS-11	25.0	26.5									10.1									R
GT-52	ST-04	10.0	12.5					51	44			23.9	101.3								F
	ST-06	15.0	17.5								2.79	23.8	100.5			100.1 to 91.7	(0.22) $< 3 \times 10^{-6}$**				C
	SS-09	22.5	24.0									18.2									C
	SS-11	27.5	29.0									15.6	115.0								C
	SB-14	35.0	36.5	CL	0	26	74	42	28		2.68	19.0	101.6								B
	SS-17	42.5	44.0									25.7									R
GT-53	ST-02	2.5	4.0									21.4	102.3								L
	ST-04	7.5	10.0	CH	0	3	97	54	38		2.73	22.3	103.8								F
	ST-08	17.5	20.0	CH	0	16	84	58	41		2.70	21.6	104.8								C
	SS-15	35.9	36.5		22	25	53					23.1									R
GT-54	ST-03	4.5	7.0	CL	0	3	97	42	25			21.8	100.1								L
	SS-08	17.0	18.5									21.8									L

B = Basal Till
F = Ferrelview Clay
C = Clay Till
L = Loess/Fill
DF = Dike Fill
R = Residuum

**Unreliable results due to extremely low confining pressure used in testing.

SUMMARY OF SOIL TEST RESULTS

(sheet 4 of 10)

Hole or Trench Number	Sample Number	Depth (ft)		Laboratory Classificatn USCS	Mechanical Analysis (%)			Atterberg Limits (%)		Shrnkage Limit (%)	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698) Optimum		Triaxial Permeability		Consolidation			Soil Unit
		From	To		Gravel	Sand	Fines	LL	PI			W (%)	γ_d (PCF)	W (%)	γ_d (PCF)	γ_d (PCF)	(conf. pressure) (KSF) K (cm/sec)	C_c	C_r	P'_o (KSF)	
GT-55	ST-02	2.5	[illegible]									22.8	96.0								L
	ST-04	7.5	10.0	CL/CH	0	1	99	50	29		2.75	30.7	89.4			94.0 to 92.8	(0.14) < 1 x 10^{-5}** (0.50) < 2 x 10^{-6}**	0.26	0.041	6.7	L
	ST-06	12.5	15.0	CH	0	1	99	51	33		2.82	22.8	103.7					0.462	0.042	18.5	F
	ST-08	17.5	20.0					42	27			22.5	103.0								F
	SB-12	27.5	29.0					70	50			25.2	99.6					0.268	0.096	7.6	R
GT-56	ST-02	5.0	7.5	CL	0	2	98	30	12		2.62	20.4	101.1					0.380	0.033	9.9	F
	ST-04	10.0	12.5									26.5	98.1			96.2 to 92.6	(0.14) < 2 x 10^{-7}** (0.50) < 3 x 10^{-8}**				F
	ST-06	15.0	17.0	CL	0	26	74	44	29			19.0	109.9					0.248	0.061	30.0	C
	SB-08	19.5	21.0		0	19	81	42	27		2.66	21.5	105.6								C
	SS-09	22.0	23.5	CH	4	6	90	71	50			27.1									C
GT-57	SS-02	5.0	6.5									26.3									L
	ST-03	7.5	10.0					42	25			27.4	93.5								F
	ST-06	15.6	16.5									22.7									F
	SB-11	27.5	29.0		55	10	35					17.1									R
GT-58P	ST-01	2.5	5.0	CH	0	4	96	54	23			T 19.8 [illegible]					(1) 2 x 10^{-7} (3) 4 x 10^{-7}				L
	ST-03	7.5	10.0	CH	0	5	95	66	46		[illegible]	21.5 B 11.7				104.8	(1) 6 x 10^{-8} (3) < 1 x 10^{-9}				F
	ST-09	22.5	24.0								2.65						(2) 4 x 10^{-8} (4) < 1 x 10^{-9}				C
	SB-11	28.5	29.0	GC	50	10	40					1.3	117.4								R
GT-59	ST-02	5.0	7.5													102.4	(1) 4 x 10^{-7} (3) 2 x 10^{-7}				F
	ST-06	15.0	17.5													110.5	(2) < 1 x 10^{-9}				C
GT-60P	ST-04	12.5	15.0								2.72										F
	ST-10	27.5	30.0	CL	0	21	79	44	27		2.70					109.1	(3) 5 x 10^{-8} (6) < 1 x 10^{-9}				C
	SB-15	41.0	41.5	CL	7	16	77	35	13							105.5 105.1	(4) 7 x 10^{-8} (7) 2 x 10^{-7}				C

B = Basal Till C = Clay Till DF = Dike Fill
F = Ferrelview Clay L = Loess/Fill R = Residuum

** Unreliable results due to extremely low confining pressure used in testing.

Hole or Trench Number	Sample Number	Depth (ft)		Laboratory Classificatn	Mechanical Analysis (%)			Atterberg Limits (%)		Shrnkage Limit (%)	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698)		Triaxial Permeability		Consolidation			Soil Unit
		From	To	USCS	Gravel	Sand	Fines	LL	PI			w (%)	γ_d (PCF)	Optimum w (%)	Optimum γ_d (PCF)	γ_d (PCF)	(conf. pressure) (KSF) k (cm/sec)	C_c	C_r	P'_o (KSF)	
GT-60P	SS-16	45.0	46.5	SC	33	37	30														B
	SB-17	48.5	49.0	SC	22	52	28									105.1	(4) 3×10^{-7} (7) 2×10^{-7}				B
GT-61	SS-13	32.5	34.0	CH	0	27	73														C
	SB-18	50.5	51.5	GC	36	34	30					11.0	118.9								B
GT-62	ST-07	17.5	20.0									T 30.9 B 25.0				100.3	(2) 5×10^{-8} (4) $<1 \times 10^{-9}$				F
	SB-13	32.8	33.3													100.1	(3) 3×10^{-5} (5) 6×10^{-8}				B
	SB-15	38.0	39.0	GC	43	17	40				2.76										R
GT-63P	ST-01	2.5	5.0	CH	6	7	87	56	32							100.4	(1) 7×10^{-9}				F
	ST-09	22.5	25.0	CL	0	25	75	45	28		2.75	16.4	113.9			110.0	(2) 9×10^{-9}				C
	SB-15	38.0	38.5	SC-GC	40	45	15														C
GT-65P	SB-16	40.0	41.5	GC	42	24	34									81.3	(4) 8×10^{-7} (7) 4×10^{-7}				R
GT-67P	ST-04	10.0	12.5																		F
	ST-08	20.0	22.5													110.2	(2) 1×10^{-7}				7

B = Basal Till
F = Ferrelview Clay
C = Clay Till
L = Loess/Fill
DF = Dike Fill
R = Residuum

**Unreliable results due to extremely low confining pressure used in testing.

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3'SM-STR.PG8

SUMMARY OF SOIL TEST RESULTS

(sheet 6 of 10)

Hole or Trench Number	Sample Number	Depth (ft)		Laboratory Classificatn	Mechanical Analysis (%)			Atterberg Limits (%)		Shrnkage Limit	Specfic Gravity	Natural		Std. Proctor Compaction (ASTM D698)		Triaxial Permeability		Consolidation			Soil Unit
		From	To	USCS	Gravel	Sand	Fines	LL	PI	(%)	G	W (%)	γ_d (PCF)	Optimum W (%)	Optimum γ_d (PCF)	γ_d (PCF)	(conf. pressure) (KSF) k (cm/sec)	C_c	C_r	P'_o (KSF)	
GTS-1	ST-03	5.0	7.5	CL	0	3	97	40	20			T 26.7 M 23.3 B 22.8	97.5	17.5 (comp. GTS-3,	102.8 with ST-02)		(1) 2×10^{-7} (composite with GTS-3, ST-02)				F
GTS-2	ST-02	2.5	5.0									T 29.2 M 25.5 B 26.6	95.0	18.3 (comp. GTS-4,	103.0 with ST-04)		(1) 8×10^{-8} (composite with GTS-4, ST-04)				F
	ST-04	7.5	10.0									T 20.5 B 20.7	103.0 102.1								F
	COMP.	2.5	10.0	CL	0	3	97	44	24												
GTS-3	ST-02	5.0	7.5	CH	1	3	96	53	34			T 28.9 M 29.6 B 37.4	89.9	(composite see	GTS-1, ST-03)		(composite see GTS-1, ST-03)				F
GTS-4	ST-02	2.5	5.0	CH	8	4	88	51	28			T 28.2 M 28.4 B 28.1	92.3	17.7 (comp.)	105.3		(1) 2×10^{-7} (composite)				F
	ST-04	7.5	10.0	CL	0	6	94	41	21			T 26.7 M 21.5 B 21.5	102.9								F
GTS-5	BULK	0.0	5.0	CH	1	7	92	53	35			21.7		17.5	106.4		(1) 2×10^{-8}				DF
GTS-6	BULK	0.0	5.0	CL	0	1	99	43	21			19.7		18.3	104.6		(1) 1×10^{-7}				F
GTS-7	BULK	0.0	5.0	CH	0	5	95	59	35			23.8		19.2	102.7		(1) 5×10^{-8}				DF

B = Basal Till C = Clay Till DF = Dike Fill

SUMMARY OF SOIL TEST RESULTS

(sheet 7 of 10)

Hole or Trench Number	Sample Number	Depth [ft]		Laboratory Classificatn	Mechanical Analysis [%]			Atterberg Limits [%]		Shrnkage Limit [%]	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698)		Triaxial Permeability		Consolidation			Soil Unit
		From	To	USCS	Gravel	Sand	Fines	LL	PL			W [%]	γ_d [PCF]	Optimum W [%]	Optimum γ_d [PCF]	γ_d [PCF]	(conf. pressure) [KSF] k [cm/sec]	C_c	C_r	P'_o [KSF]	
GT-2T71	BU-01A	3.5	6.5	CL	0	2	98	39	17		2.69	28.3		15.4	108.0						
	BU-02A	6.5	12	CH	0	3	97	52	37	8.3	2.51	17.3		18.5	106.9	104.4	(1) 2 x 10^{-8}				
GT-2T72	BU-01A	7.5	10.5	CL-ML	0	5	95	27	6		2.68	22.1		12.5	116.1						
GT-2T74	BU-01A	7.0	10.5								2.74										
	BU-01A	4.0	8.5	CL	0	3	97	42	24			25.9		16.1	106.3						
GT-2T75	BU-01A	2.5	6.5								2.61	20.8									
	BU-02B	6.5	12.0	CH	0	3	97	77	56	7.3	2.71	23.1		20.1	102.3	97.5	(1) <1 x 10^{-9}				
	BU-01A	4	8.5	CL	0	1	99	39	17					17.6	108.0						
GT-2T77	BU-01B	1.5	4.0	CH	0	1	99	50	33		2.55	16.6		16.6	104.3						
	BU-02A	7.0	10.5	CL	0	3	97	49	26		2.66	24.4		19.2	104.4	103.0	(1) 2 x 10^{-8}				
GT-2T78	BU-01A	4.0	6.5	CL	0	2	98	30 28	11 10		2.71	14.7		11.7	114.9						
	BU-02B	6.5	12.0	CH	0	3	97	58	39		2.69	20.1		14.6 21.3	115.6 104.2	99.2	(1) 4 x 10^{-8}				
GT-2T79	BU-01A	1.0	6.5	CL	0	2	98	49	26	10.7	2.72	14.0									
	BU-02A	6.5	12.0	CH	0	3	97	62	43			23.1									
GT-2T80	BU-01A	0.5	8.0	CL	0	4	96	38	18			18.1									
GT-2T81	BU-01A	0.5	6.5	CL	1	10	89	36	18	17.5		20.4									
	BU-02A	6.5	12.0	CL	0	4	96	40	23			21.0									
GT-2T82	BU-01A	0.5	3.5	CL	0	4	96	42	23			13.8									
	BU-02A	3.5	6.0	CH	0	3	97	53	31			25.2									
	BU-03A	6.0	10.0	CL	2	21	77	48	27			16.2									

B = Basal Till
F = Ferrelview Clay
Visual Classification

C = Clay Till
L = Loess/Fill

DF = Dike Fill
R = Residuum

**Unreliable results due to extremely low confining pressure used in testing.

SUMMARY OF SOIL TEST RESULTS

(sheet 8 of 10)

Hole or Trench Number	Sample Number	Depth [ft]		Laboratory Classificatn	Mechanical Analysis [%]			Atterberg Limits [%]		Shrnkage Limit [%]	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698)		Triaxial Permeability			Consolidation			Soil Unit
		From	To	USCS	Gravel	Sand	Fines	LL	PI			w [%]	γ_d [PCF]	Optimum w [%]	Optimum γ_d [PCF]	γ_d [PCF]	(conf. pressure) [KSF]	k [cm/sec]	C_c	C_r	P'_o [KSF]	
TPMS-1	BU-01	0.0	6.0	CH	0	7	93	51	32			20.0		17.4	103.0	98.6	(1)	7×10^{-8}				
TPMS-2	BU-01	0.0	2.0	CL	0	6	94	49	29			18.3		18.7	104.3		(1)	2×10^{-8}				
	BU-02	2.0	6.0	CH	0	20	80	67	46			21.9		21.0	100.6		(1)	$< 1 \times 10^{-9}$				
TPMS-3	BU-01	1.0	6.0	CL	0	16	84	39	19			22.4		17.1	104.0	99.5	(1)	3×10^{-7}				
TPMS-4	BU-01	1.0	3.5	CL	0	10	90	49	29			20.0		18.6	105.4		(1)	9×10^{-9}				
	BU-02	4.0	6.5	CH	0	14	86	73	49			27.5		22.8	95.5		(1)	$< 1 \times 10^{-9}$				
TPMS-5	BU-01	1.0	7.0	CL	0	4	96	37	20			19.8		15.9	108.1		(1)	2×10^{-6}				
	BU-02	8.0	10.0	CH	0	2	98	73	49			26.6		21.7	97.8		(1)	9×10^{-9}				
TPMS-6	BU-01	1.0	5.0	CL	0	2	98	40	22			15.7		16.4	104.0		(1)	3×10^{-7}				
	BU-02	5.5	7.5	CH	0	4	96	72	47			19.4		21.3	100.4		(1)	$< 1 \times 10^{-9}$				
TPMS-7	BU-01	1.0	7.0	CL	0	4	96	37	20			19.7		17.4	107.2		(1)	1×10^{-7}				
	BU-02	7.0	9.5	CH	0	3	97	82	58			20.3		21.1	94.7		(1)	1×10^{-7}				
TPMS-8	BU-01	1.0	5.0	CL	0	3	97	30	11			9.9		16.3	108.0		(1)	6×10^{-8}				
	BU-02	5.0	10.5	CH	0	6	94	63	44			23.1		21.4	98.6		(1)	$< 1 \times 10^{-9}$				
	BU-03	12.0	14.0		0	45	55					11.1										
TPMS-9	BU-01	1.0	7.0	CL	0	3	97	38	18			16.5		15.4	104.9							
	BU-02	8.0	15.0	CH	0	5	95	69	45			15.1		24.1	95.9							

B = Basal Till C = Clay Till DF = Dike Fill
F = Ferrelview Clay L = Loess/Fill P = Residuum
* Visual Classification

**Unreliable results due to extremely low confining pressure used in testing.

SUMMARY OF SOIL TEST RESULTS

(sheet 9 of 10)

Hole or Trench Number	Simple Number	Depth [ft] From	Depth [ft] To	Laboratory Classificatn USCS	Mechanical Analysis [%] Gravel	Mechanical Analysis [%] Sand	Mechanical Analysis [%] Fines	Atterberg Limits [%] LL	Atterberg Limits [%] PI	Shrnkage Limit [%]	Specfic Gravity G	Natural W [%]	Natural γ_d [PCF]	Std. Proctor Compaction (ASTM D698) Optimum W [%]	Std. Proctor Compaction (ASTM D698) Optimum γ_d [PCF]	Triaxial Permeability γ_d [PCF]	Triaxial Permeability (conf. pressure) [KSF] k [cm/sec]	Consolidation C_c	Consolidation C_r	Consolidation P'_o [KSF]	Soil Unit
TPBS-1	2A	2.6	7.1	CL	0	9	91	31	13		2.65	11.1		15.6	110.4	108.2	(1) 3×10^{-7}				
	3A	7.1	9.2	MH				60	21		2.70	26.4		20.1	105.6						
	4A	9.2	14.1	CL	1	37	62	42	18	22.2	2.55	20.5		14.6	116.1	111.0	(1) 4×10^{-8}				
TPBS-6	1A	0.6	9.1	ML	0	3	97	38	11		2.64	24.6		19.3	102.9	99.9	(1) 2×10^{-8}				
	2A	9.1	12.6	CH				64	44		2.74	24.9		20.9	100.3	94.6	(1) 1×10^{-8}				
TPBS-8	1A	0.6	4.3	CL	0	2	98	34	16		2.73	21.8		17.4	108.0	102.9	(1) 3×10^{-8}				
	2A	4.3	8.5	CL	0	2	98	31	12	15.9	2.64	13.3		15.6	109.4	106.0	(1) 2×10^{-7}				
	3A	8.5	11.9	CL-CH	0	2	98	50	30		2.59	21.0		20.5	101.9	97.3	(1) 5×10^{-9}				
TPBS-10	1A	0.5	6.1	ML				49	17			23.5				102.1	(1) 1×10^{-7}				
	2A	8.2	10.9	ML	0	3	97	43	11			20.1				101.7	(1) 2×10^{-8}				
	3A	10.9	13.4	CH	0	5	95	61	43	8.5		23.1				97.0	(1) $<1 \times 10^{-9}$				
TPBS-11	1A	0.6	10.4	CL	0	3	97	33	12		2.72	25.3				99.5	(1) 3×10^{-8}				
	2A	10.4	12.3	CH	0	2	98	57	27	11.3	2.69	27.4				98.5	(1) 3×10^{-8}				
TPBS-14	2A	2.9	8.4	CL	0	2	98	35	15			25.2									
	3A	8.4	11.8	ML	0	2	98		NP			22.4									
TPBS-15	1A	2.4	10.8	ML	0	3	97	41	9			25.5									
	2A	10.8	12.9	CH	2	4	94	71	50			28.1									
	3A	7.1	9.2		1	13	86														
TPBS-23	1A	0.7	13.1	CL	0	3	97	37	21		2.71	26.2		18.2	106.3	102.0	(1) 4×10^{-7}				
TPBS-28	1A	0.7	9.9	CL	0	5	95	38	19		2.65	23.2		14.9	109.8						
TPBS-30	1A	1.8	12.4	ML	0	10	90	37	12		2.62	28.0		18.3	104.6						

B = Basal Till
F = Ferrelview Clay
NP = nonplastic

C = Clay Till
L = Loess/Fill

DF = Dike Fill
R = Residuum

**Unreliable results due to extremely low confining pressure used in testing.

Hole or Trench Number	Sample Number (Test No.)	Depth [ft]		Laboratory Classificatn	Mechanical Analysis [%]			Atterberg Limits [%]		Shrnkage Limit [%]	Specfic Gravity G	Natural		Std. Proctor Compaction (ASTM D698) Optimum	
		From	To	USCS	Gravel	Sand	Fines	LL	PI			W [%]	γ_d [PCF]	W [%]	γ_d [PCF]
TPBS-13	1	4.6	5.0	CL*								27.7	92.2		
TPBS-14	2	4.9	5.3	CL*								25.9	93.6		
TPBS-15	3	5.9	6.3	CL*								23.8	89.1		
TPBS-16	4	5.4	5.8	CL*								24.3	98.7		
TPBS-17	5	6.0	6.4	CL*								24.8	96.8		
TPBS-18	6	4.8	5.2	CL*								19.9	97.5		
TPBS-19	7	4.9	5.3	CL*								24.0	99.0		
TPBS-20	8	5.4	5.8	CL*								25.6	96.7		
TPBS-23	9	5.7	6.1	CL*								28.3	91.8		
TPBS-26	10	4.9	5.3	CL*								24.8	98.9		
TPBS-25	11	4.1	4.5	CL								23.5	100.1		
TPBS-28	12	3.8	4.2	CL*								26.4	95.2		
TPBS-29	13	4.8	5.2	CL*								28.0	96.1		
TPBS-30	14	4.9	5.3	CL*								26.4	95.3		

B = Basal Till C = Clay Till DF = Dike Fill
E = Ferrelview Clay L = Loess/Fill R = Residuum

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