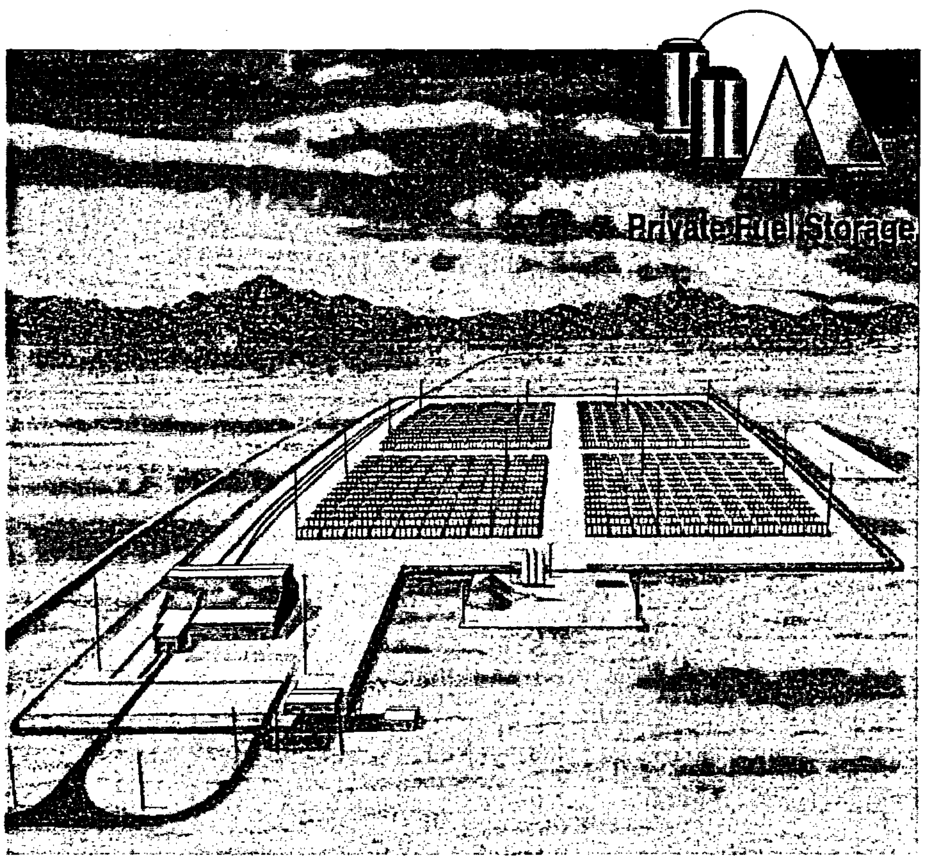
Private Fuel Storage, LLC

Final
Safety Analysis Report

License # SNM 2513

Book 1 of 2



No Document Control List Will Be Included At This Time

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

TABLE OF CONTENTS (cont.)

TABLE OF CONTENTS (cont.)

CHAPTER 1

INTRODUCTION AND GENERAL DESCRIPTION OF FACILITY

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

LIST OF FIGURES

CHAPTER 1

INTRODUCTION AND GENERAL DESCRIPTION OF FACILITY

1.1 INTRODUCTION

Electric utilities operating nuclear power plants in the United States are rapidly reaching their maximum capacity for onsite storage of spent fuel. The Nuclear Waste Policy Act (NWPA) of 1982 mandated that the Department of Energy (DOE) was responsible for the permanent disposal of spent nuclear fuel from the nation's commercial nuclear power plants. The NWPA obligated DOE, beginning not later than January 31, 1998, to dispose of the spent fuel. In a December 17, 1996 letter to all utilities, DOE stated that it would not meet the 1998 deadline. As a result, utilities have had to plan for alternate means of interim storage for their spent fuel beyond 1998.

One such alternate means of spent fuel storage includes dry cask storage. Using this concept, a consortium of utilities have joined in a cooperative agreement through the Private Fuel Storage L.L.C. (PFSLLC) with the Skull Valley Band of Goshute Indians (Band) to undertake the development, licensing, construction, and operation of an Independent Spent Fuel Storage Installation (ISFSI) called the Private Fuel Storage Facility (PFSF). The PFSF will be built on the Skull Valley Indian Reservation and will provide timely, centralized, cost-effective spent fuel storage capacity to meet the needs of the utilities and provide long-term, stable financial income, employment, and training opportunities for Band members and the surrounding community. Preservation of the site and surrounding environment has resulted in the adoption of a "Start Clean / Stay Clean" philosophy that will permit utilization of the land and all buildings constructed in this project for other traditional industrial uses after the facility is decommissioned.

The PFSF will utilize the dry cask storage technology. Dry cask storage safely stores spent nuclear fuel inside of sealed canisters rather than in a spent fuel pool. The storage system technology is compatible with the long-term plans of the DOE interim storage facility and permanent repository. The PFSF is designed to store spent fuel for up to 40 years by which time it is anticipated that all of the spent fuel will be transferred offsite and the facility ready for decommissioning. The initial request for a license is for a term of 20 years. Prior to the end of the initial license term an application for license renewal will be submitted.

The PFSF is located on the Skull Valley Indian Reservation in Tooele County, Utah, approximately 27 miles west-southwest of Tooele City[1] (see Figure 1.1-1). There are no major towns within 10 miles of the PFSF site. The Skull Valley Band of Goshute Indian village is approximately 3.5 miles east-southeast of the site. This village has approximately 30 residents.

The reservation consists of approximately 18,000 acres, of which the PFSF site area is approximately 820 acres. Interstate Highway 80 and the Union Pacific Railroad main line are about 24 miles north of the site. Due to the proximity of the PFSF to the railroad mainline, the shipping cask will either be off-loaded at an intermodal transfer point 1.8 miles West of Timpie, Utah, and loaded onto a heavy haul tractor/trailer for transporting to the PFSF, or transported via a new railroad line connecting the PFSF directly to the Union Pacific mainline at Low Junction. The PFSF will be accessed by a new road from the Skull Valley Road as shown on Figure 1.1-2.

It is anticipated that the PFSF will be issued a specific license to receive, transfer and possess spent fuel, in accordance with the requirements of 10 CFR 72 (Reference 1), in spring of 2002. Construction would began thereafter with construction and preoperational testing completed in time to support operation of the facility in the latter part of 2003. As part of the license application, this Safety Analysis Report (SAR) has

been prepared in accordance with the guidelines contained in NRC Regulatory Guide 3.48 (Reference 2) and NRC NUREG-1567, draft Standard Review Plan for Spent Fuel Dry Storage Facilities, (Reference 3).

[1] Tooele City is used to distinguish the City of Tooele from Tooele County.

THIS PAGE INTENTIONALLY LEFT BLANK

1.2 GENERAL DESCRIPTION OF FACILITY

The PFSF is designed to store up to 40,000 Metric Tons of Uranium [2](MTU) of spent fuel from U.S. commercial power reactors in sealed metal canisters (approximately 4,000 storage casks). A detailed description of the fuel types that can be stored is provided in Chapter 3 and 10. The canister-based spent fuel storage system selected for use at the PFSF utilizes sealed metal canisters to store multiple spent fuel assemblies. Each canister is placed inside of a storage cask. The storage system is passive and relies on natural convection for cooling. The system is an integral part of the facility "Start Clean / Stay Clean" philosophy which eliminates the need to handle individual fuel assemblies at the site. The system assures there is negligible contamination or radioactive waste generated at the site and facilitates the ease of decommissioning at the end of the life of the facility. Design criteria are described in more detail in Chapter 3.

The passive nature of the storage systems results in a relatively simple facility as shown on Figure 1.2-1. The Restricted Area (RA), is approximately 99 acres and is surrounded by a chain link security fence and an outer chain link nuisance fence with an isolation zone and intrusion detection system between the two fences. The cask storage area within the RA is surfaced with compacted gravel that slopes slightly to allow for runoff of storm water. The cask storage area consists of concrete cask storage pads that support the storage casks. Each pad is designed to support up to 8 storage casks in a 2 by 4 array. The Canister Transfer Building, as well as the Security and Health Physics Building, are also located within the RA. An overhead bridge crane and a semi-gantry crane are located within the Canister Transfer Building to facilitate shipping cask load/unload operations and canister transfer operations.

---

[2] Metric Tons of Uranium (initial uranium). This includes the small amount of mixed oxide fuels that are anticipated to require storage.

A main gate is provided for vehicular access to the RA. Light poles are located within the RA inside the security fence. Outside the RA is an Administration Building, and an Operations and Maintenance Building. The overall site or owner controlled area (OCA) is approximately 820 acres and is bounded by a range fence. The fence will be a typical 4-strand wire range fence, which will serve to identify the limit of PFSF activities and to keep out any stray livestock. Specifications for the fence, such as wire type and spacing, and pole type and spacing will meet the requirements of the BLM Manual Handbook H-1741-1 for Fencing and/or other applicable requirements identified by the BLM and BIA. PFS will consult with the BLM and BIA prior to construction of the fence to make sure the fence meets the latest BLM/BIA requirements.

The PFSF is designed to accommodate the storage system and the transportation of spent fuel canisters to and within the facility. The amount of yard area provided within the RA is sized to limit the radiation dose outside of the RA from the storage casks to less than 2 mrem/hr per 10 CFR 20.1301(a)(2). The yard area is also sized to provide adequate space for maneuvering the onsite cask transporter used during storage cask placement. The area of the OCA is based on 10 CFR 72.104(a) requirements of maintaining the annual dose to any real individual outside the OCA during normal operations to less than 25 mrem whole body, as well as 10 CFR 72.106(b) requirements of maintaining a minimum distance of 100 meters (328 ft) from spent fuel storage and handling areas to the OCA boundary and limiting the dose to 5 rem, to the whole body or any organ, from any design basis accident.

In compliance with 10 CFR 72.122 (d), the PFSF does not share structures, systems, or components with other facilities.

1.3 GENERAL SYSTEMS DESCRIPTION

In order to store spent fuel at a centralized facility, an appropriate system is required to accommodate transfer of the spent fuel from nuclear power plants to the PFSF and to provide storage of the spent fuel at the PFSF.

The PFSLLC has selected the canister-based system for use at the PFSF because it eliminates the need to handle individual spent fuel assemblies once a canister is loaded and is sealed at the originating nuclear power plant. In addition, the canister-based system is expected to be compatible with the final DOE system for spent fuel management (Reference 4).

The canister-based system utilizes a sealed metal canister to store multiple spent fuel assemblies in a controlled environment. The sealed metal canister is placed in casks that provide radiation shielding and physical protection of the fuel during fuel transfer, transportation, storage, and disposal. The vendor selected to provide canister-based storage systems at the PFSF is Holtec International (Holtec). Holtec is supplying its Holtec International Storage and Transfer Operation Reinforced Module Cask System (HI-STORM 100) (Reference 5). This canister-based storage system and its components are shown on Figure 1.3-1.

The metal canister is a cylindrical shell with a lid and closure ring, shell assembly, and bottom plate, that houses a spent fuel basket. The canister is designed to accommodate PWR or BWR fuel types, including failed and mixed oxide fuel. The spent fuel basket provides structural support for the spent fuel assemblies and a path for the transfer of heat generated by ~~the spent fuel to the canister shell. The spent fuel~~ basket also provides criticality control to ensure that a nuclear fission reaction can not be sustained. The cylindrical shell provides structural support for the fuel basket structure and spent fuel assemblies. During storage, the cylindrical shell, bottom plate, and the canister lid provide the primary confinement boundary to prevent the release of radioactive material from the spent fuel. The canister is designed to provide radiation

shielding and physical protection for the spent fuel and uses casks to provide additional shielding and protection. Spent fuel assembly capacities of the canisters are as follows:

	Holtec HI-STORM 100 Cask System
PWR fuel	24
BWR fuel	68

After the spent fuel is loaded into a canister at the originating nuclear power plant, the canister is partially drained and the lid is welded to the canister. The canister is then drained and vacuum dried, filled with helium, and sealed closed by welding the vent and drain port cover plates. The closure ring is then welded onto the top of the canister, providing a redundant closure. Additional details of the canister design are discussed and shown in Chapter 4.

There are three types of casks used to transfer and store the canister. These are a shipping cask, transfer cask, and storage cask.

The shipping casks are used to transport the spent fuel canisters from the originating power plants to the PFSF. The shipping casks are designed to provide a complete confinement barrier, canister cooling, and to protect the canister from the effects of environmental conditions and natural phenomena. Each shipping cask is fitted with impact absorbing devices and is designed to withstand postulated transportation accidents. After the spent fuel canister is unloaded, the empty shipping cask is returned to the power plants for reloading with another canister of spent fuel.

The metal transfer cask is used to provide radiation shielding, physical protection, and canister cooling for the spent fuel canister when transferring the canister from the shipping cask to the storage cask at the PFSF.

The storage cask is a concrete and steel cylindrical structure which provides structural support for the spent fuel canister, physical protection, radiation shielding, and provides for natural convection cooling of the canister to remove decay heat while in storage at the PFSF.

At the PFSF, the shipping cask is lifted off the transport vehicle and placed in a shielded area of the Canister Transfer Building, called a transfer cell, using the overhead bridge crane. The canister is transferred from the shipping cask to the transfer cask, then transferred from the transfer cask into the storage cask using either the overhead bridge crane or the semi-gantry crane. The storage cask, loaded with the canister, is then sealed closed and moved to the storage area using a cask transporter and placed on a concrete pad. Details of the HI-STORM 100 cask are discussed and shown in Chapter 4.

The design of the canister system and the loading procedures minimizes the potential for contamination of the canister at the originating nuclear power plant. Contamination that does occur is removed to within acceptable limits at the power plant. The system assures there is negligible contamination or radioactive waste generated at the site and facilitates the ease of decommissioning at the end of the life of the facility. Site generated waste confinement and management is discussed in Chapter 6.

THIS PAGE INTENTIONALLY LEFT BLANK

1.4 SPENT FUEL TRANSPORTATION TO THE PFSF

Although transportation activities are not part of the 10 CFR 72 license application for the PFSF, the transportation process is briefly described herein to provide an understanding of the overall program. Spent fuel enroute to the PFSF will be transported in accordance with applicable U.S. Department of Transportation (DOT) regulations (49 CFR 173-Shippers General Requirements for Shipments and Packaging, Subpart A-"General" and Subpart I - "Radioactive Materials", 49 CFR 171-"General Information, Regulations, and Definitions", 49 CFR 172-"Hazardous Materials Tables and Hazardous Materials Communications Regulations", 49 CFR 174-"Carriage by Rail", 49 CFR 177-"Carriage by Public Highway"), and NRC regulations (10 CFR 71 - "Packaging and Transportation of Radioactive Material"). Holtec is supplying its Holtec International Storage, Transport, and Repository Cask System (HI-STAR 100) (Reference 7).

As a result of adherence to strict controls, utilities and carriers have a long history of safe spent fuel transportation. In more than 30 years of shipping fuel in the United States, no accident has caused a release of radioactive material. Moreover, no deaths or serious injuries to the public or to transportation industry personnel have ever occurred as the result of the radioactive nature of any radioactive material shipment (Reference 9).

Currently there is no direct rail line to the PFSF. Therefore the PFSF will be designed to employ two transport modes to ship the cask from the railroad mainline to the site. The preferred mode is to ship the shipping cask the final 32 miles by rail on a new rail line. The alternate mode is to transfer the shipping cask from the rail car to a heavy haul transport tractor/trailer at an intermodal transfer point located 1.8 miles West of Timpie and haul the shipping cask the final 26 miles by road to the PFSF. The PFSF

is expected to handle approximately 100 to 200 shipments of loaded spent fuel canisters annually. The PFSF will accept delivery and perform receipt inspection of the spent fuel shipping casks at the PFSF.

PFS may contract directly with its utility customers to perform as a rail or motor carrier to transport the casks from the main rail line to the PFSF. To the extent that PFS acts as a carrier, PFS will comply with the applicable DOT statutes and regulations pertaining to rail or motor carriers, as appropriate, and the related hazardous material transportation requirements. Specifically, if PFS operates as a rail carrier for the utility customers, PFS will meet the requirements applicable to rail carriers, including, without limitation, the applicable requirements set forth in 45 U.S.C. Chapters 2, 8, 9,and 11; 49 U.S.C. Subtitle IV (Part A); Subtitle V, Subtitle X; and associated implementing regulations contained in 49 C.F.R. Parts 200, 1000 through 1300. If the heavy haul option is used and PFS operates as a motor carrier for the utility customers, PFS will meet the requirements for motor carriers including, without limitation, the applicable requirements set forth in 49 U.S.C. Subtitle Part IV Part B; Subtitle VI; Subtitle X and associated implementing regulations contained in 49 C.F.R. Parts 300, 1000, 1090-1099, 1200 and 1300. The commitments in this paragraph are made as part of a Settlement between PFS and the State of Utah in the NRC licensing proceeding for the PFSF and, as agreed to by PFS, the State and the NRC Staff, do not constitute a license condition or licensing commitment under the 10 CFR Part 72 license for the PFSF and, as further agreed, their incorporation into the SAR does not render the commitments subject to 10 CFR 72.48, or obligate the NRC Staff to enforce the above requirements, or undertake enforcement action with respect to violation of these requirements, under the 10 C.F.R. Part 72 license for the PFSF.

1.5 IDENTIFICATION OF AGENTS AND CONTRACTORS

Holtec International is providing the spent fuel storage systems for use at the PFSF. Holtec has a NRC-approved QA program and is responsible for the design and licensing of their canisters, casks, and transfer equipment.

Stone & Webster Engineering Corporation is providing the engineering and supporting preparation of the 10 CFR 72 license documentation of the PFSF site.

The PFSLLC has responsibility for construction management and construction of the PFSF. The prime construction contractor for the PFSF, with experience on nuclear projects, will either have its own NRC-approved QA program, or work under the PFSLLC NRC-approved QA program. This contractor, and other subcontractors for non-nuclear portions of the facility, will be selected at an appropriate time to support construction of the PFSF.

The PFSLLC will be responsible for operation of the PFSF.

THIS PAGE INTENTIONALLY LEFT BLANK

1.6 MATERIAL INCORPORATED BY REFERENCE

The Safety Analysis Report for the Holtec HI-STORM 100 Storage System (Reference 5) was submitted to the NRC for approval and is incorporated by reference into this document. The NRC has issued a Certificate of Compliance for Holtec's HI-STORM 100 storage cask system (Reference 12), as well as a Certificate of Compliance for its HI-STAR 100 shipping cask system (Reference 13).

The PFSLLC QA Program (Reference 11) was approved by the NRC on November 3, 1996 for use under 10 CFR 71, Subpart H (Docket 71-0829) and is incorporated by reference into this document. It is proposed that this PFSLLC QA Program be used to satisfy the requirements of 10 CFR 72, Subpart G. Chapter 11 provides a detailed discussion of the PFSLLC QA program.

THIS PAGE INTENTIONALLY LEFT BLANK

1.7 REFERENCES

1. 10 CFR 72, Licensing Requirements for the Independent Storage of Spent Nuclear Fuel and High Level Radioactive Waste.

2. U.S. Nuclear Regulatory Commission, Regulatory Guide 3.48, Standard Format and Content for the Safety Analysis Report for an Independent Fuel Storage Installation or Monitored Retrievable Storage Installation (Dry Storage), August 1989.

3. U.S. Nuclear Regulatory Commission, NUREG-1567, draft Standard Review Plan for Spent Fuel Dry Storage Facilities, October, 1996.

4. DOE/RW-0445, Multi-Purpose Canister System Evaluation, U.S. Department of Energy Civilian Radioactive Waste Management, September 1994.

5. Final Safety Analysis Report for the Holtec International Storage and Transfer Operation Reinforced Module Cask System (HI-STORM 100 Cask System), Holtec Report HI-2002444, Revision 0, NRC Docket 72-1014, July 2000.

6. (deleted)

7. Topical Safety Analysis Report for the Holtec International Storage, Transport, and Repository Cask System (HI-STAR 100 Cask System), Holtec Report HI-951251, Revision 9, Docket 71-9261, April 2000.

8. (deleted)

9. U.S. Department of Energy, "Transporting Spent Nuclear Fuel: An Overview, DOE/RW-0065 (1986).

10. (deleted)

11. PFSLLC QA Program Manual, Current Revision, Docket 71-0829.

12. 10 CFR 72 Certificate of Compliance 1014, Rev. 0, HI-STORM 100 System, May, 2000.

13. 10 CFR 71 Certificate of Compliance 9261, Rev. 0, HI-STAR 100 System, March, 1999.

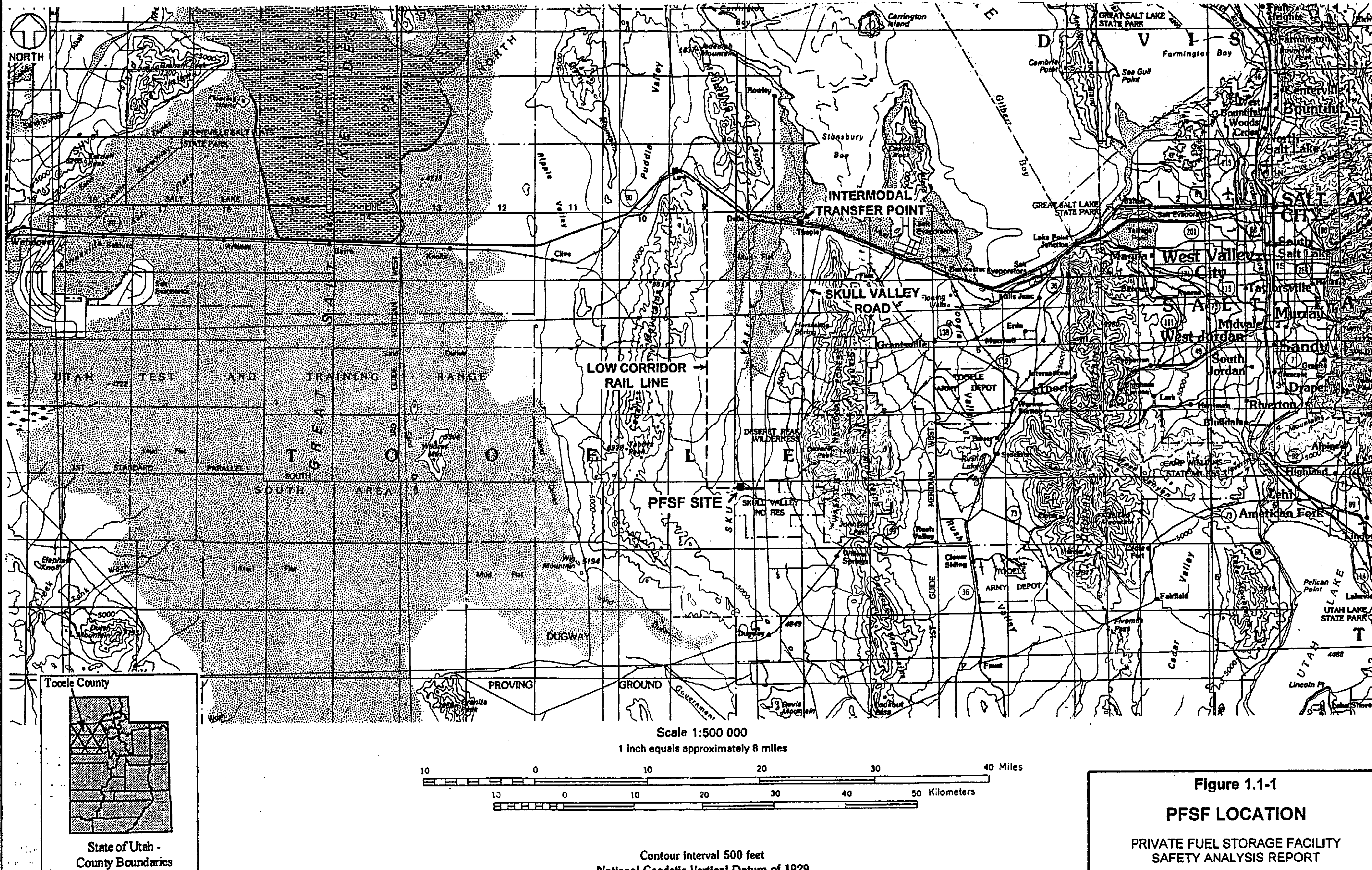


Figure 1.1-1

PFSF LOCATION

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

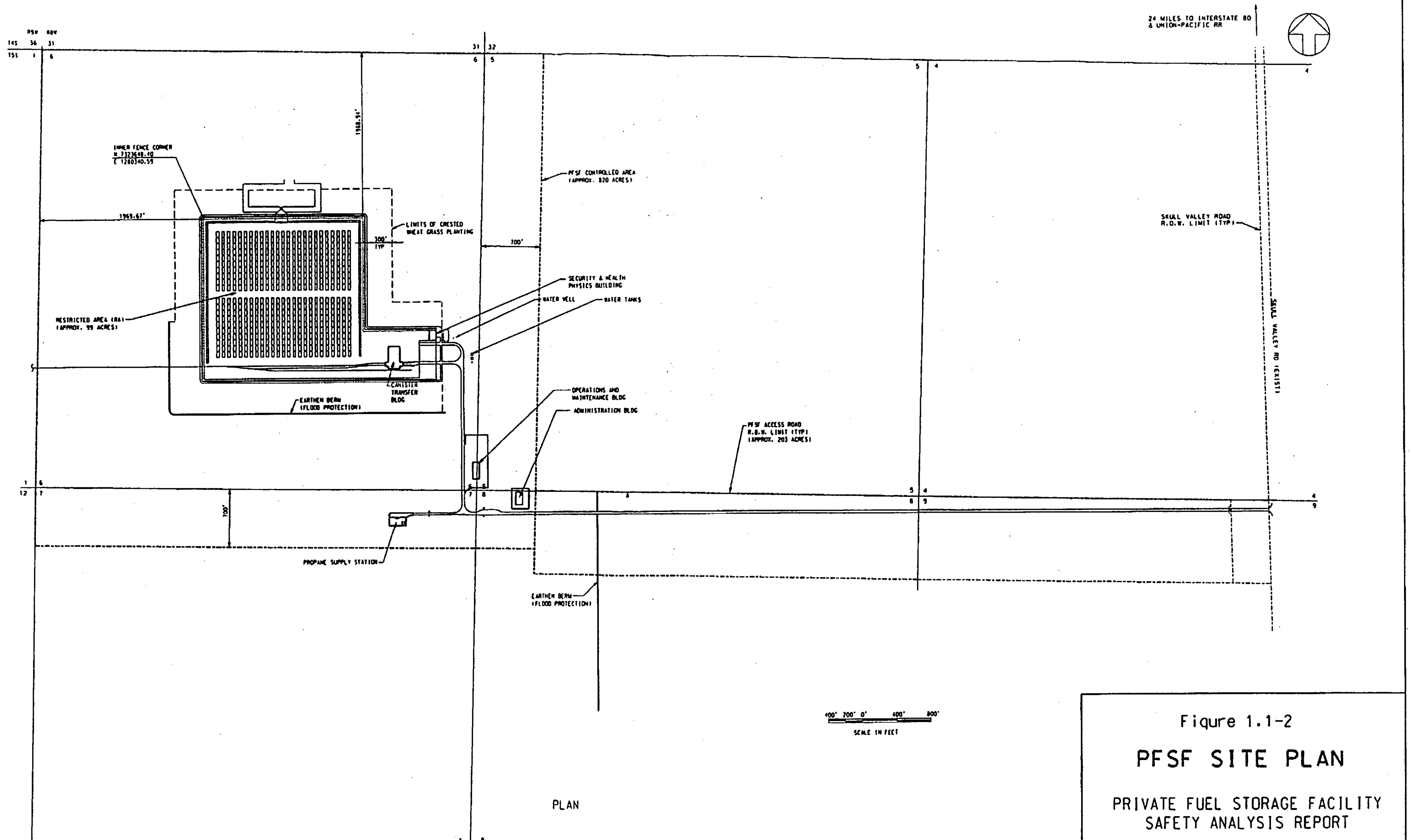


Figure 1.1-2

PFSF SITE PLAN

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

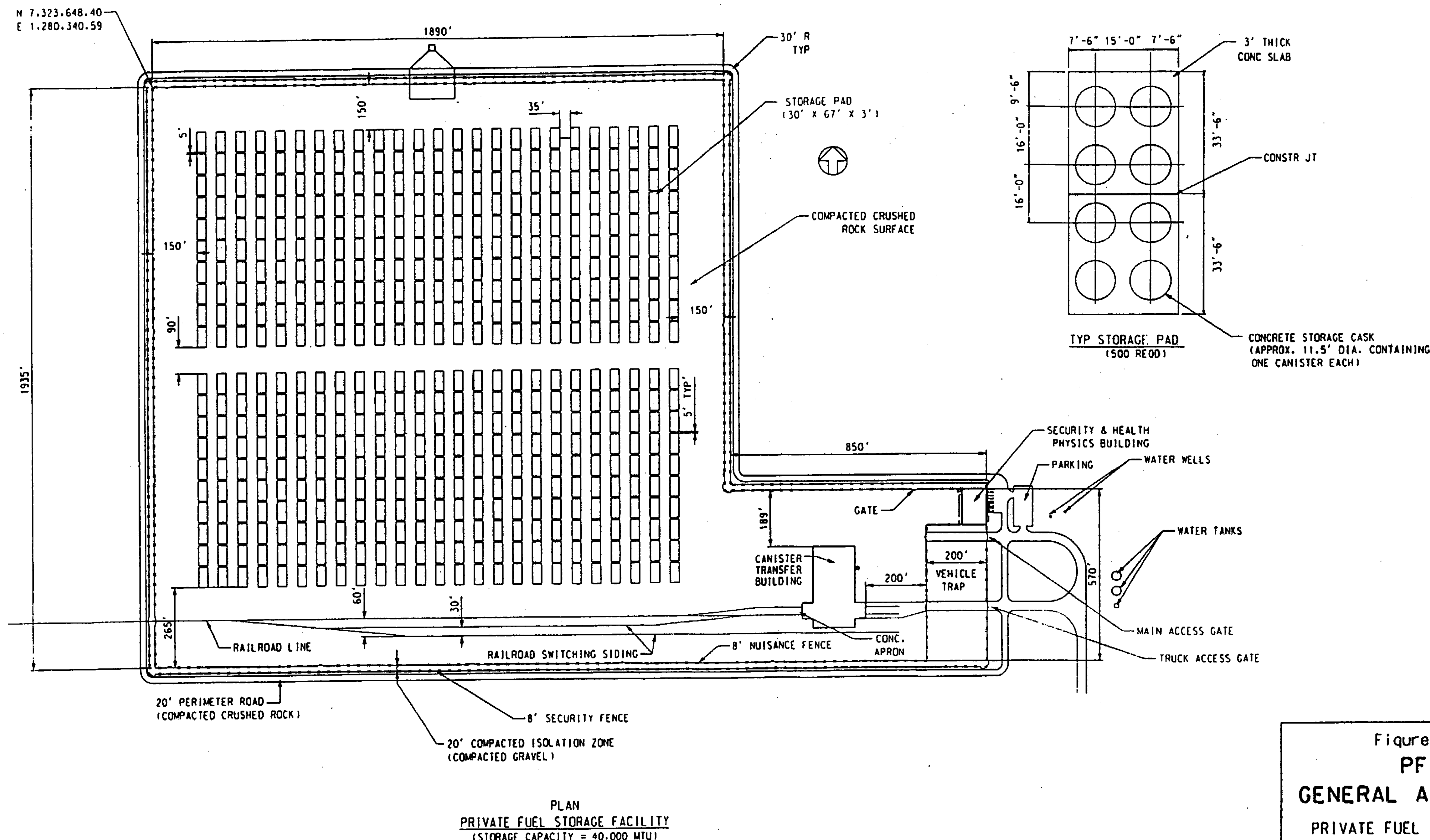


Figure 1.2-1
PFSF
GENERAL ARRANGEMENT
PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

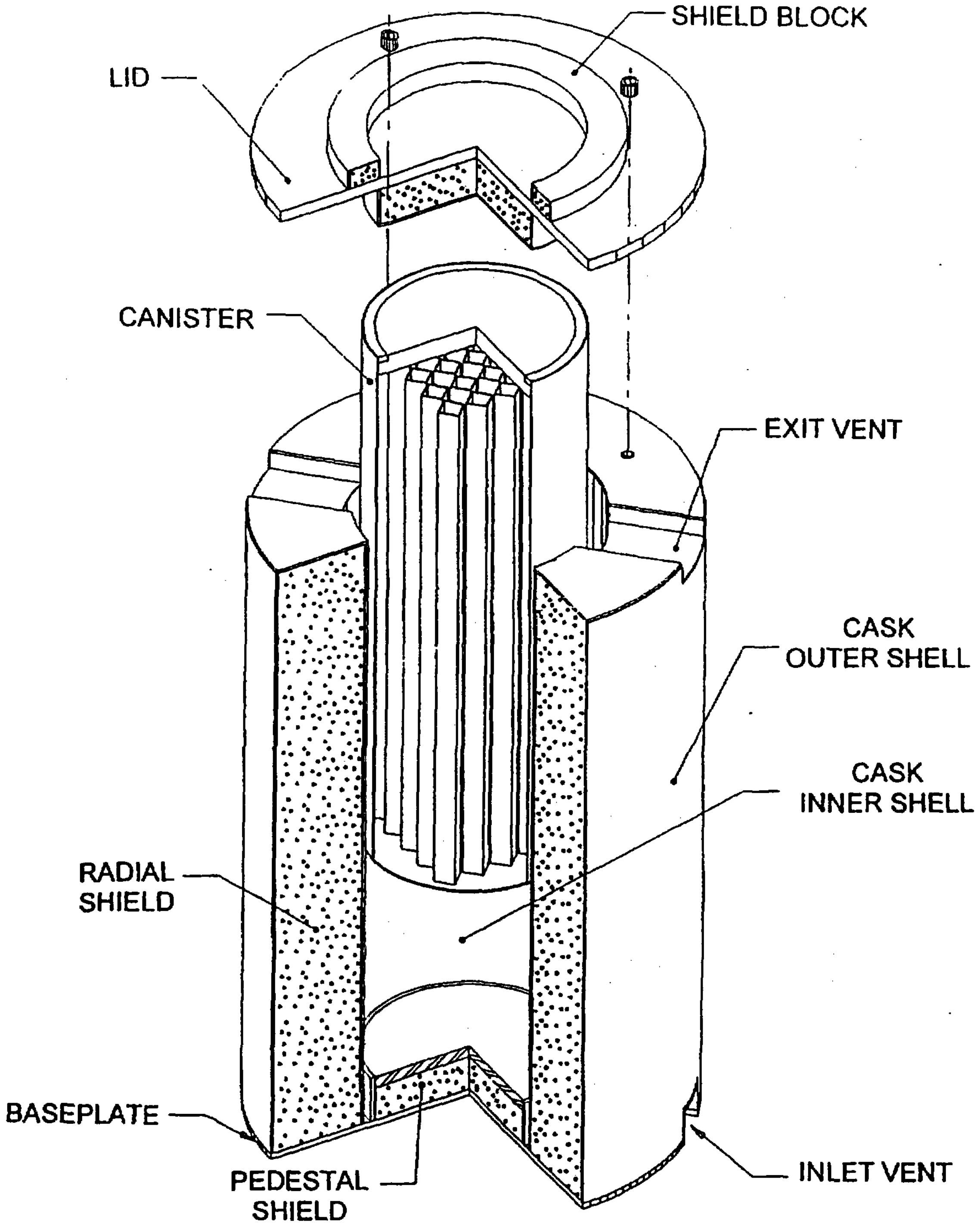


Figure 1.3-1

HI-STORM 100 CASK SYSTEM COMPONENTS

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

(deleted)

Figure 1.3-2

(deleted)

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

CHAPTER 2

SITE CHARACTERISTICS

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

TABLE OF CONTENTS (cont.)

TABLE OF CONTENTS (cont.)

TABLE OF CONTENTS (cont.)

LIST OF APPENDICES

TABLE OF CONTENTS (cont.)

LIST OF TABLES

TABLE OF CONTENTS (cont.)

LIST OF TABLES

TABLE OF CONTENTS (cont.)

LIST OF FIGURES

TABLE OF CONTENTS (cont.)

LIST OF FIGURES

TABLE OF CONTENTS (cont.)

LIST OF FIGURES

TABLE OF CONTENTS (cont.)

LIST OF FIGURES

THIS PAGE INTENTIONALLY LEFT BLANK

CHAPTER 2

SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location

The proposed site for the Private Fuel Storage Facility (PFSF) is located on the Skull Valley Indian Reservation in Tooele County, Utah. This county is in the northwestern portion of the state bordered on the north by Box Elder County; to the east by Davis, Salt Lake, and Utah Counties; to the south by Juab County; and to the west by the State of Nevada (Elko County). Tooele County is a combination of environments including the Great Salt Lake, western deserts, fertile valleys, and rugged mountains. Many areas of the county are undeveloped and isolated. Most land in Tooele County is under the administration of the Bureau of Land Management (BLM) and the military where large portions of the county are used for federal and military land uses (at Dugway Proving Ground, the Utah Test and Training Range, Tooele Army Depot North and South Areas) and for hazardous waste incineration and storage (at USPCI, Aptus, and Envirocare facilities) (Tooele, 1995).

The proposed PFSF site is located in a valley floor typical of the local basin-range topography. The Stansbury Mountains (maximum elevation 11,031 ft) separate the PFSF from Tooele City, which is located approximately 27 miles east northeast (Figure 1.1-1). Skull Valley is sparsely populated with limited agricultural or other activity. Land owners and administrators of the Skull Valley area include the BLM, privately owned ranches, the Skull Valley Indian Reservation, the Wasatch National Forest, and the

Dugway Proving Ground (Tooele, 1995). There are no existing industrial, recreational, or residential uses within the boundaries of the proposed site.

2.1.2 Site Description

The proposed facility would be located on property leased from the Skull Valley Band of Goshute Indians (Band), within Township 5 South, Range 8 West, Section 6, with appurtenant facilities located along the project access road through sections 4, 5, 7, 8, and 9. The northwest corner of the PFSF is located at 40° 24' 50"N, 112° 47' 37" W. The area immediately surrounding the site consists of undeveloped range land owned by the Band, BLM, and private landowners. The PFSF has a restricted area enclosing an area of approximately 99 acres for cask storage. In addition, an owner-controlled area (OCA) encompassing 820 acres will be bounded by typical range fencing to identify the limits of site activity. Figure 2.1-2 shows details of the plant perimeter and the proposed configuration of facilities. Skull Valley Road (designated as Federal Aid Secondary Road (FAS) 108) is located to the east of the site and traverses Skull Valley from Interstate 80 south to the intersection with State Route 199.

2.1.2.1 Other Activities Within the Site Boundaries

No activities are currently conducted within the area proposed for development of the PFSF.

2.1.2.2 Boundaries for Establishing Effluent Release Limits

There are no radioactive or other effluent releases associated with the proposed facility.

2.1.3 Population Distribution and Trends

Population within a 5-mile radius centered on the proposed PFSF consists of tribal residents on the Skull Valley Indian Reservation and two isolated ranches on Skull Valley Road north of the Reservation boundary. The closest residents to the PFSF are two tribal homes located approximately 2 miles southeast of the project and those residences in the Skull Valley Indian Reservation village, approximately 3.5 miles east-southeast of the site. There are about 30 residents currently living on the Reservation.

Two private residences are located northeast of the proposed site along Skull Valley Road, approximately 2.75 and 4.0 miles away. Therefore, the estimated population within a 5-mile radius is 36 persons (30 Goshutes and 2 households of approximately 3 persons each) (Figure 2.1-1). Because of the remoteness of the Skull Valley and because a majority of the land within 5 miles is owned by either the BLM or the Reservation, it is unlikely that the permanent population within a 5-mile radius of the proposed PFSF would change significantly during the proposed license period.

No transient or institutional populations are present within 5 miles of the proposed PFSF. The Skull Valley Road passes through the Reservation approximately 2.5 miles from the site. Traffic on this roadway is primarily related to local resident travel and travel between Interstate 80 and Dugway Proving Ground. During October 1996, a survey was conducted to identify existing and planned public facilities and institutions within a 5-mile radius of the facility. Due to the remoteness and extreme low population density of the area (36 persons within 5-mile radius), no facilities such as hospitals, prisons, and recreational areas are located or planned within the 5-mile study area.

2.1.4 Uses of Nearby Land and Waters

Land use within the Reservation boundary consists of residential uses by tribal members (approximately 30 persons living on the Reservation) and the Tekoi Rocket Engine Test Facility operated by Alliant Techsystems on leased Reservation lands. This facility, located approximately 2.5 miles south-southeast of the PFSF on the south side of Hickman Knolls, has been operated at this location from 1975 until recently.

In the 5-mile radius around the site there are approximately 28,000 acres of BLM land, 9,000 acres of privately-owned land, and 13,000 acres of land that are part of the Skull Valley Indian Reservation. The section is nearly flat, sloping gently downward to the north with small, local elevation changes of about 1 ft.

The principal land use in Skull Valley is range land for livestock grazing. Cattle and sheep are grazed, especially in winter when the livestock is brought down from the higher mountain elevations. The majority of land (55 percent) within a 5-mile radius of the site is owned and managed by the BLM as part of the Pony Express Resource Area (PERA). The remainder of the land is split almost evenly between Reservation property and private ownership.

BLM land within the 5-mile radius is part of the Skull Valley and South Skull Valley grazing allotments.[1] Most of the range land within the Skull Valley allotment (85 percent) is considered to be of fair to poor condition with the overall conditions in decline (BLM, 1988). The allotment is divided into three pastures: West Cedar, Eightmile, and Black Knoll. The southeast corner of the Black Knoll Pasture is within

---

[1] An allotment is an area of land where one or more permittees may graze livestock.

the 5-mile radius. Two operators are authorized to graze up to 5,000 sheep and 2,300 cattle within the Skull Valley allotment from November 1 to April 30. Sheep graze in alternate years. Cattle graze following a 3-year cycle: from November 1 to April 30th one year, November 1 to February 28th the following year, and November 1 to February 28 and April 1 to April 30th the third year (BLM, 1985).[2] Portions of two pastures in the South Skull Valley allotment are within the 5-mile radius: the east end of the Cochrane Pasture (Pasture 1) and the northern edge of the Post Hollow Pasture (Pasture 2). The permitee for these pastures is authorized to graze a maximum of 700 cattle and 3,800 sheep from November 1 to April 30th in alternating years (BLM, 1986).[3]

In addition to grazing, recreation use is also allowed on BLM land within the PERA. Off-highway vehicle (OHV) use, dispersed camping, and hunting are principal uses of BLM property within the PERA (BLM, 1988). There are no designated camping areas or OHV trails or roads within the 5-mile radius, though the BLM land within the radius is given an OHV designation category "A," meaning that it is open to all types of motor vehicle use (BLM, 1992).

---

2 The maximum authorized for the three pastures in the 271,000-acre Skull Valley allotment are 5,000 sheep and 2,300 cattle. Considerably fewer sheep and cattle would be expected within the corner of the allotment inside the 5-mile radius.

3 The permittee is allowed to graze livestock at two other pastures within the South Skull Valley allotment outside the 5-mile radius so we would expect considerably fewer sheep or cattle grazing within the 5-mile radius.

THIS PAGE INTENTIONALLY BLANK

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

The PFSF site is situated in the northwest corner of the Skull Valley Indian Reservation in Tooele County, Utah. The Reservation consists of approximately 18,000 acres, of which the PFSF site area is approximately 820 acres, or less than 5% of the reservation area. The PFSF site location was selected by the Skull Valley Band of Goshute Indians in order to avoid disruption of tribal roads, housing or cultural facilities. Figure 1.1-1 shows the facilities and locations addressed in this section.

The area surrounding the PFSF site is very sparsely populated, with the nearest residence 2 miles southeast of the site. The Skull Valley Band of the Goshute Village, with a population of about 30, is 3.5 miles east-southeast of the PFSF site. Terra, a small residential community with a population of 120 (Tooele County Commission, 1995), is located 10 miles east-southeast of the PFSF.

2.2.1 Hazards from Facilities and Ground Transportation

Tekoi Rocket Engine Test Facility

The only industrial, transportation or military facility within 5 miles of the PFSF is the Tekoi Rocket Engine Test facility, located about 2.5 miles south-southeast of the PFSF. This facility, located on leased reservation lands, was used in the past by Alliant Techsystems Inc. to periodically test rocket engines mounted on stationary bases and explosive charges. Hickman Knolls, with an elevation of approximately 4873 ft, is situated directly between the PFSF (approximate elevation 4465 ft) and the Tekoi Test facility (elevation 4600 ft). The relative location of Hickman Knolls between the PFSF and Tekoi Test facility, and the distance of 2.5 miles would substantially deflect and disperse overpressures from an explosion at the Tekoi Test facility, precluding any hazard to the PFSF. In order for this facility to be used by Alliant Techsystems Inc. in

the future, a new lease agreement between Alliant and the Goshute Band would need to be negotiated. There are no other facilities that could present the threat of an explosion or other hazard within 5 miles of the PFSF.

Major Transportation Corridors

Interstate Highway 80 and the Union Pacific Railroad main line are located 24 miles north of the PFSF site. Any events associated with either the interstate highway or the railroad will not present a hazard to the PFSF due to the relatively large distance involved. The Skull Valley Road runs essentially north-south between Interstate 80 and the town of Dugway, population 1,700, 12 miles south of the PFSF. Dugway is a residential community supporting the nearby Dugway Proving Ground and has no facilities which could present a hazard to the PFSF.

Dugway Proving Ground

The U.S. Army's Dugway Proving Ground is a 1,315 square mile range and test facility located west of the town of Dugway. The Dugway Proving Ground performs testing of all types of military equipment in chemical and biological environments, as well as smoke, obscurant and incendiary testing, and munitions testing. Open air testing is not permitted by law, and there have been no accidents or releases of toxic gas from the facility or associated transportation activities. The Proving Ground has a mean elevation of 4,350 ft above sea level and is surrounded on three sides by mountain ranges. The Cedar Mountains, with an elevation of 5,300 ft or greater, lie between the Proving Ground and the PFSF. The following potential threats posed by the Dugway Proving Grounds were assessed: 1) the firing of conventional ground weapons in military testing and training; 2) the testing, storage, and disposal of chemical munitions and agents; 3) the testing of biological materials; 4) the transportation of biological, chemical and hazardous materials to and from Dugway Proving Ground; and 5) unexploded ordnance. The PFSF will be located over 8 miles north from the northeastern boundary of Dugway Proving Ground and will be approximately 20 miles

from the locations where most of the activities involving chemical agent and biological materials take place at of Dugway Proving Ground. By virtue of the distance between the PFSF and the locations on of Dugway Proving Ground where the ostensibly hazardous activities take place, the nature of the activities, and the safety precautions that are taken with respect to all potentially dangerous activities at of Dugway Proving Ground, those activities would not pose a significant hazard to the PFSF, as discussed in the following paragraphs.

Military training exercises and the firing and testing of conventional weapons will not pose a hazard to the PFSF because 1) the firing of weapons is covered by rigid procedures, 2) the closest firing position to the PFSF is more than 15 miles away, 3) the ranges of most of the weapons are insufficient to reach the PFSF from those distances, and 4) the weapons are fired toward the south and northwest, away from the PFSF. It is not credible that a conventional munition fired from Dugway would strike the PFSF.

Chemical munitions and agent at Dugway will pose no significant hazard to the PFSF. Open air testing of chemical munitions and agents was prohibited by law in 1969 (50 U.S.C. § 1512), and has not been conducted since 1969. Thus, activities at Dugway Proving Ground involving chemical agent and munitions are limited to indoor testing of chemical agent, storage of agent and unexploded chemical munitions recovered from the firing ranges, and disposal of chemical agent. None of these activities pose a hazard to the PFSF because of their distance from the PFSF and the limited quantities of agent whose release would be credible. The in-door testing of chemical agent is done in facilities – located close to 20 miles from the PFSF – designed to preclude the release of chemical agent, and thus would pose no credible hazard to the PFSF. Similarly, the locations at which chemical munitions and agent are stored on Dugway Proving Ground are located more than 17 miles from the PFSF and are stored under a strict set of rules governing their storage, including State regulations under RCRA for

the storage of chemical munitions and related agent. By virtue of the distance to the PFSF and the many controls designed to protect public health and safety, the release of chemical agent from chemical munitions or agent stored at Dugway does not pose a credible hazard to the PFSF. The worst credible threat posed by chemical agent at Dugway would arise from the accidental detonation of a previously unexploded 8-inch projectile filled with chemical agent GB (which is an extremely unlikely event). The distance at which such an event would pose a threat is on the order of several miles, far less than the distance to the PFSF. Likewise, the disposal of chemical munitions and agents is done under rigorous control, including regulation by the State under RCRA, and would pose no credible hazard to the PFSF.

Biological materials present at Dugway Proving Ground would likewise not pose a credible hazard to the PFSF because all use of biological materials at Dugway Proving Ground is conducted in the Life Sciences Test facility – more than 20 miles from the PFSF – under engineering and procedural controls designed to prevent the release of material to the environment. The United States destroyed its biological agents and munitions after a presidential decree in 1969 and Dugway Proving Ground does not test biological agents for use in warfare. All testing done at the Life Sciences Test Facility is for defensive purposes. Further, even if biological material at the test facility were to escape, it would pose no significant hazard to the PFSF, in that it would have almost no chance of surviving in the environment long enough to be carried the 20 miles from the facility to the PFSF. Thus, the use of biological materials at Dugway Proving Ground poses no credible hazard to the PFSF.

The transportation of chemical agent or biological materials to or from Dugway does not pose a significant hazard to the PFSF. Larger shipments of such material are performed with extraordinary safety precautions and, moreover, do not travel along Skull Valley Road. Small, laboratory quantities of material could potentially be shipped

by common carrier along Skull Valley Road, but the safe packaging of those shipments is strictly regulated by the Department of Transportation so as to prevent a release even in the event of an accident. Hazardous wastes shipped from Dugway Proving Ground do not include chemical agent but rather only chemically neutralized agent, which is far less hazardous and would not threaten the PFSF even if spilled on Skull Valley Road.

Unexploded ordnance would not pose a significant hazard to the PFSF in that 1) it is extremely unlikely that such ordnance would explode spontaneously or accidentally and 2) even if it did, the PFSF is far enough away that the material in the round would not pose a significant hazard. Unexploded ordnance is not likely to be found off Dugway Proving Ground close enough to pose a risk to the PFSF, in that the firing ranges at Dugway are all at least 15 miles away and Army records of where munitions were fired at Dugway give no indication that munitions were fired elsewhere.

The Dugway Proving Ground receives and ships conventional Army weapons approximately 95 times a year. Some of these shipments could travel the Skull Valley Road, which present the only credible potential for an explosion near the PFSF. An accident associated with the transportation of explosives along the Skull Valley Road would be a minimum of 1.9 miles from the Canister Transfer Building and 2 miles from the nearest cask storage pad. Based on the methodology of Regulatory Guide 1.91, the Skull Valley Road is located much further from the PFSF than the distances required to exceed 1 psi overpressure for detonation of explosives transported by highway, as discussed in Section 8.2.4.

The Tooele Army Depot facilities, where toxic gas munitions are stored and incinerated, are located west and south, respectively, of Tooele City. The North Tooele Army Depot is 17 miles east-northeast of the PFSF and the South Tooele Army Depot is 21 miles east-southeast of the PFSF. The Stansbury Mountains, with an elevation of approximately 8,000 feet, lie between the PFSF and the Tooele Army Depots. The activities and materials at the Tooele Army Depots will therefore present no credible

hazard to the PFSF, because of their relative distance and the intervening Stansbury Mountains.

2.2.2 Hazards from Air Crashes

Aircraft flights in the vicinity of the PFSF take place to and from Michael Army Airfield on Dugway Proving Ground, on and around the Utah Test and Training Range (UTTR), and on federal airways J-56 and V-257. While there are no civilian airports within 25 miles of the PFSF, minimal general aviation traffic may also transit the region. PFS performed conservative upper bound calculations, upheld by the Atomic Safety and Licensing Board (Board), which establish that the likelihood of an aircraft crash impacting the PFSF and breaching a spent fuel storage cask is less than 7.37 E-7 per year; furthermore qualitative factors indicate that the true probability of such an event is much lower than this upper bound calculation, perhaps by an order of magnitude or more. (Cornell January 2004; Board Decision February 2005) The upper bound calculated probability value of 7.37 E-7 is an extremely low probability, well below the 1 E-6 regulatory standard the NRC has promulgated for above ground facilities at geologic repositories (which are similar to ISFSIs) (61 Fed. Reg. 64,257, 64,261-62, 64,265-66 (1996)), and which the Commission has adopted for ISFSIs, such as the PFSF (Commission Decision November 2001). Therefore, aircraft crashes do not present a credible hazard to the PFSF and the facility does not need to be designed to withstand the impact of an aircraft crash.

2.2.2.1 Michael Army Airfield and Airway IR-420

Michael Army Airfield is located on the Dugway Proving Ground, 17 miles south-southwest of the PFSF. This military airfield has a 13,125 foot runway, and can accommodate all operative aircraft in the Department of Defense inventory, although the majority of the aircraft flying to and from Michael AAF, other than F-16 aircraft from Hill Air Force Base (AFB) which are accounted for in Section 2.2.2.2.1 below, are large

cargo aircraft. The airspace over the Dugway Proving Ground is restricted. Military airway IR-420 passes near the PFSF site area. The methods of NUREG-0800 Section 3.5.1.6 were used to estimate the probability of an aircraft impacting the PFSF from this airway, using the equation:

$P = C \times N \times A / w$, where

P = probability per year of an aircraft crashing into the PFSF
C = in-flight crash rate per mile
N = number of flights per year along the airway
A = effective area of the PFSF in square miles
w = width of airway in miles

NUREG-0800 states the in-flight crash rate as 4 E-10 per mile, which is appropriate to apply to the types of aircraft flying to and from Michael AAF addressed here. (PFS August 2000) Information provided by the Dugway Proving Ground indicates that approximately 414 flights annually can be taken as an upper bound for this airfield. The effective area of the PFSF is 0.2116 mi^2, calculated using Department of Energy (DOE) formulas. (DOE 1996) The width of the airway is 10 nautical miles (nm), or 10nm x 1.15 mile/nm = 11.5 miles. The probability of an aircraft impacting the PFSF is therefore 3.0 E-9 per year. Because of the distance from the PFSF to Michael Army Airfield, takeoff and landing operations at Michael pose a negligible hazard to the PFSF.

2.2.2.2 Utah Test and Training Range

The UTTR is an Air Force training and testing range over which the airspace is restricted to military operations. It is divided into a North Area, located on the western shore of the Great Salt Lake, north of Interstate 80, and a South Area, located to the west of the Cedar Mountains, south of Interstate 80 and northwest of Dugway Proving

Ground. (PFS August 2000) The airspace over the UTTR extends somewhat beyond the range's land boundaries and is divided into military operating areas (MOAs) and restricted areas. The MOAs on the UTTR are located on the edges of the range, adjacent to the restricted areas. The PFSF site is located over 18 statute miles east of the eastern land boundary of the UTTR South Area and 8.5 statute miles northeast of the northeastern boundary of Dugway Proving Ground. The site lies within the Sevier B MOA, two statute miles to the east of the edge of restricted airspace. (PFS August 2000)

Military aircraft flying in or around the UTTR South Area comprise three groups: 1) F-16 fighter aircraft flying from Hill Air Force Base (AFB), near Ogden, Utah, down Skull Valley en route to the UTTR range (Section 2.2.2.2.1.1); 2) aircraft departing the UTTR range via the Moser Recovery to return to Hill AFB (Section 2.2.2.2..1.2); and 3) aircraft conducting training in the restricted airspace on the range (Section 2.2.2.2.3). Aircraft flying in or around the UTTR North Area pose no credible hazard to the PFSF because of the distance from the facility.

2.2.2.2.1 F-16s Transiting Skull Valley

2.2.2.2.1.1 F-16s Transiting Skull Valley En Route to the UTTR

F-16 fighter aircraft fly north to south down Skull Valley, within Sevier B and Sevier D MOAs, en route from Hill AFB to the UTTR South Area. The F-16s use the eastern side of Skull Valley as their predominant route of travel and typically pass approximately five miles to the east of the PFSF site. (PFS August 2000) The U.S. Air Force has indicated that the F-16s typically fly between 3,000 and 4,000 ft. above ground level (AGL), with a minimum altitude of 1,000 ft AGL. Data from Hill Air Force Base (AFB) indicate that the number of F-16 flights transiting Skull Valley within the Sevier B MOA were 3,871, 4,250, and 5,757 in Fiscal Years (FY) 1998, 1999, and 2000, respectively and that F-16 flights transiting Sevier D (which lies above Sevier B) were 215, 336, and

240 in in Fiscal Years (FY) 1998, 1999, and 2000, respectively. (PFS August 2000) On the basis of its assessment of Air Force operational policies, PFS projected that the annual number of F-16 sorties transiting Skull Valley flown by aircraft now stationed at Hill AFB would be an average of the FY99 and FY00 Sevier B sorties, or approximately 5,000. Because the Air Force had also stated that 12 additional F-16s would be authorized at Hill AFB in FY2001, which would increase the number of authorized F-16s at the base from 69 to 81, i.e., by 17.4 percent, PFS projected , that thee future annual number of F-16 sorties from Hill AFB through Skull Valley would increase by 17.4%, from 5,000 to 5,870. (PFS August 2000)

The Board, however, determined that the number of F-16 flights transiting Skull Valley for purposes of calculating an aircraft crash probability for the PFSF should conservatively be based on the sum of the FY 2000 flights through Seviers B and D. (Board Decision March 2003) Additionally, in order to account for the 12 additional F-16s authorized to be stationed at Hill AFB beginning in FY 2001, this sum (5,997) was increased by 17.4 percent to 7,040, which was the number of F-16 sorties that the Board used in calculating the annual probability of an F-16 aircraft crash at the PFSF utilizing the NUREG-0800 methodology for calculating aircraft crash impact probabilities. The impact probabilities as calculated by the Board are the basis for the probabilities set forth in this section of the FSAR.

Although the predominant route of travel for the F-16s is down the eastern side of Skull Valley, away from the PFSF; an impact probability was calculated, using the methodology of NUREG-0800, in which it was conservatively assumed that the F-16 flights are uniformly distributed within the MOA airspace in the vicinity of the PFSF. (PFS August 2000) Although PFS had used an airway width of 10 miles to calculate the Skull Valley F-16 impact probability, the Board determined that the MOA airspace in the vicinity of the PFSF should be treated as an airway with a width of 6 miles. (Board Decision March 2003) Given the flight characteristics of the F-16, the PFSF has an effective area of 0.1337 mi^2, assuming a facility at full capacity with 4,000 spent fuel

storage casks on site and the crash rate for the F-16 was calculated from Air Force data to be 2.736 E-8 per mile. (PFS August 2000; Board Decision March 2003)

PFS determined, from an extensive review of Air Force F-16 accident investigation reports, that over 90 percent of the F-16 crashes that would result from accident-initiating events that could occur in Skull Valley would leave the pilot in control of the aircraft after the event. Furthermore, because of the training Air Force pilots receive in responding to such in-flight events, the flight characteristics of the F-16, the absence of other built up areas in Skull Valley, and the small effort required for the pilot to avoid the PFSF site in the event of a crash caused by an accident-initiating event leaving him in control of the aircraft, PFS's experts (three former U.S. Air Force pilots) determined that the pilot would be able to direct the aircraft away from the PFSF at least 95 percent of the time in which such an event caused a crash in Skull Valley. Accordingly, 85.5 percent (90% x 95%) of the crashing F-16s would be able to avoid the PFSF and hence the calculated crash impact hazard to the PFSF using the NUREG-800 methodology should be reduced by this fraction. (PFS August 2000) The Board agreed with PFS that over 90 percent of the F-16 crashes that would result from accident-initiating events that could occur in Skull Valley would leave the pilot in control of the aircraft after the event but found insufficient evidence to conclude that the pilots would direct the aircraft away from the PFSF in at least 95 percent of such accident occurrences. (Board Decision March 2003). Accordingly, no quantitative credit is taken in the probabilities presented here for the likelihood that the pilot of an F-16 crashing in Skull Valley would direct the aircraft away from the PFSF, but the likelihood that pilots would do so is recognized as a "material conservatism" in the "Unanalyzed Event Probability" calculation discussed Section 2.2.2.2.1.3 below. (Board Decision February 2005).

2.2.2.2.1.2 F-16s Transiting Skull Valley Using the Moser Recovery Route

Most of the F-16s returning to Hill AFB from the UTTR South Area exit the northern edge of the range (away from the PFSF) in coordination with air traffic control.

However, some aircraft returning to Hill from the UTTR South Area may use the Moser recovery route, which runs from the southwest to the northeast, approximately two miles from the PFSF site. The Moser route is only used during marginal weather conditions or at night under specific wind conditions which require the use of Runway 32 at Hill AFB. Based on information from local air traffic controllers, conservatively estimated, the Moser recovery is used by less than five percent of the aircraft returning to Hill. (PFS August 2000) According to the Air Force, 5,726 F-16 sorties were flown on the UTTR South Area in 1998, almost all of which flew from Hill AFB (not all aircraft transit Skull Valley en route to the South Area). Thus, fewer than 286 aircraft per year (5% x 5,726) were estimated to have used the Moser recovery on their return flights in 1998.

The annual crash impact probability for aircraft flying the Moser recovery route was calculated using the NUREG-0800 methodology. The Moser recovery route was defined as an airway with a width, w, of 10 nautical miles (11.5 statute miles) (equal to the width of military airway IR-420). The same F-16 crash probability, 2.736 E-8 per mile; and site effective area, 0.1337 mi^2, was used as for the calculation as for calculating the crash impact probability for F-16s transiting Skull Valley en route to the UTTR. Using these parameters, and updating the number of flights to reflect the additional sorties expected to be flown by F-16s from Hill AFB based on FY99 and FY00 data and the stationing of the 12 additional F-16s at Hill AFB (Section 2.2.2.2.1.1), PFS calculated (taking into account pilot avoidance) the probability of an F-16 crashing and impacting the PFSF while returning to Hill AFB via the Moser recovery route to be 2.00 E-8. (PFS August 2000) Using slightly different input parameters for the number of flights and pilot avoidance, the NRC Staff calculated a slightly higher crash impact probability of 2.5 E-8. (Board Decision March 2003)

The Board found that, excluding from the calculation any credit for pilot avoidance, the Staff's estimate of crash impact probability of 1.6 E-7 and PFS's slightly lower

estimate(of 1.38 E-7) were both reasonable. PFS used the higher impact probability of 1.6 E-7 in its evaluation of the Unanalyzed Event Probability for F-16 impacts discussed Section 2.2.2.2.1.3 below. (Cornell January 2004)

2.2.2.2.1.3 Unanalyzed Event Probability for F-16s Transiting Skull Valley

PFS performed extensive structural analyses of an F-16 impacting the spent fuel storage casks and the Canister Transfer Building at the PFSF site in order to determine those categories of impacts (i.e., speed, angle, and location of impact) that could be shown would not breach a spent fuel storage cask and hence would cause no radiological release at the site. (Cornell January 2004; Holtec 2004; Stone & Webster July 2003). The probability of those impacts for which the structural analyses show that no breach would occur can be "screened out" of the aircraft crash impact probability for purposes of determining whether a radiological release caused by an aircraft crash is a credible event. (Cornell January 2004; Board Decision February 2005) Thus, if the probability of occurrence of the remaining impacts that are not screened out by the structural analyses – known as the "Unanalyzed Event Probability" (UEP) – is less than 1 E-6, the likelihood of an aircraft crash causing a radiological release at the PFSF site would not be a credible event that would need to be designed against under applicable NRC standards.

Based on the structural analyses performed for the spent fuel storage casks and the CTB, PFS calculated the combined UEP (assuming a fully loaded facility) for F-16s transiting Skull Valley en route to the UTTR and for F-16s transiting Skull Valley via the Moser recovery route on their return to Hill AFB. This combined UEP for for F-16s transiting Skull Valley was 6.35 E-7. (Cornell January 2004).

2.2.2.2.2 Jettisoned Ordnance

PFS also calculated the probability that ordnance jettisoned from a crashing F-16 in Skull Valley would impact the PFSF. (PFS August 2000) Some of the F-16 flights through Skull Valley carry ordnance (live or inert) and in the event of an incident leading to a crash in which the pilot would have time to respond before ejecting from the aircraft (e.g., an engine failure), one of the pilot's first actions would be to jettison any ordnance carried by the aircraft. PFS conservatively assumed that the F-16s would be uniformly distributed across the valley, despite the fact that their predominant route of flight is down the eastern side of the valley and that, according to the Air Force, aircraft carrying live ordnance avoid flying over populated areas to the maximum extent possible To calculate the probability that jettisoned ordnance would impact the PFSF site, PFS followed an approach similar to that of NUREG-0800, using the formula P = N x C x e x A/w where N is the number of F-16 aircraft carrying ordnance while transiting Skull Valley en route to the UTTR, C is the F-16 crash, e is the percentage of crashes in which the pilot would be expected to jettison ordnance, A is the area of the PFSF site, and w, is the width of the flyway. (PFS August 2000)

The Board adopted this formula for calculating ordnance impact probability but used different input parameters than those used by PFS for N and w. (Board Decision March 2003) Based on the input parameters adopted by the Board, the probability that the ordnance would impact the PFSF is

$$P = 587 \times 2.736\text{ E-8} \times 0.90 \times 0.08763 / 6 = 2.11\text{ E-7}$$

PFS performed structural analyses of a jettisoned ordnance impacting the spent fuel storage casks to determine those categories of jettisoned ordnance impacts that could be shown would not breach a spent fuel storage cask and hence would cause no radiological release at the site. (Lancaster July 2003; Soler August 2004) As with F-16 impacts discussed in Section 2.2.2.2.1.3, .the probability of those impacts for which the

structural analyses show that no breach would occur can be "screened out" of the jettisoned ordnance impact probability for purposes of determining whether a radiological release caused by a jettisoned ordnance impact is a credible event. (Cornell January 2004; Board Decision February 2005) After performing these analyses, PFS determined that the remaining Unanalyzed Event Probability for those jettisoned ordnance impacts at the PFSF not screened out by these structural analyses was 0.48 E-7. (Cornell January 2004).

In addition to the potential hazard posed by direct impacts of crashing aircraft and jettisoned ordnance, PFS also calculated the hazard to the PFSF posed by jettisoned live ordnance that might land near the facility and explode on impact, as well as the hazard posed by a potential explosion of live ordnance carried aboard a crashing aircraft that might impact the ground near the PFSF. (PFS August 2000; Board Decision March 2003) At the outset, aircraft transiting Skull Valley near the PFSF do not carry armed live ordnance. Furthermore, the U.S. Air Force has indicated that the likelihood that unarmed live ordnance would explode when impacting the ground after being jettisoned is "remote" and the Air Force has no records of such incidents in the last 10 years. Thus, it is highly unlikely that jettisoned live ordnance or live ordnance carried aboard a crashing aircraft that did not directly impact the PFSF would damage the facility. Nevertheless, to calculate a numerical hazard to the facility, PFS assumed that such ordnance would have a 1 percent chance of exploding and assessed that damage to the PFSF would result if an explosion occurred close enough that the blast overpressure would damage a storage cask or the Canister Transfer Building, without hitting either one. The explosive overpressure limit for a storage cask was taken to be 10 psi. The limit for the Canister Transfer Building was taken to be 1.5 psi. PFS assumed that the ordnance in question was a 2,000 lb. bomb, the largest single piece of ordnance carried by the F-16s that transit Skull Valley. The Air Force indicated that 193 F-16s transited Skull Valley in 1998 with live ordnance. PFS calculated the probability that an F-16 carrying live ordnance would crash and jettison the ordnance so as to impact near the PFSF, or crash near the PFSF without jettisoning the ordnance,

following the same method it used to calculate the probability that an F-16 would crash and impact the facility. The results of PFS's final calculation showed that the annual probability that a storage cask or the Canister Transfer Building would be damaged by an explosion of live ordnance jettisoned from a crashing aircraft or carried aboard a crashing aircraft that impacted the ground near the PFSF was equal to 2.43 E-10. If this probability is adjusted to reflect 1) the additional sorties expected to be flown by F-16s from Hill AFB, based on FY99 and FY00 data, and the stationing of the 12 additional F-16s at Hill AFB, and 2) the decreased usage of ordnance in FY99 and FY00, the probability would fall below 1 E-10. This is exceedingly low and is insignificant relative to the other aircraft crash and jettisoned ordnance impact hazards calculated for the PFSF.

2.2.2.2.3 Aircraft Training on the UTTR

According to the Air Force, 8,284 sorties were flown over the UTTR South Area in 1998. (PFS August 2000) Those aircraft conducted a variety of activities, including air-to-air combat training, air-to-ground attack training, air-refueling training, and transportation to and from Michael Army Airfield (which is located beneath UTTR airspace). Hazards posed by aircraft flying to and from Michael Army Airfield are addressed in Section 2.2.2.1 above. Of the remaining aircraft, only fighter aircraft conducting air-to-air training represent a potential hazard to the PFSF, in that aircraft conducting air-to-ground attack training do so over targets that are located more than 20 miles from the PFSF site and aircraft conducting air refueling training do so on the far western side of the UTTR, over 50 miles from the site. The Air Force indicated 6,360 fighter sorties were flown on the UTTR South Area in 1998 and one-third, or approximately 2,120, involved fighter aircraft conducting air-to-air training.

PFS calculated the crash impact probability for fighter aircraft conducting air-to-air training on the UTTR as follows:

$P = C_a \times A_c \times A/A_p \times R$, where

P = annual crash impact probability

C_a = total air-to-air training crash rate per square mile on the UTTR

A_c = the area of the UTTR from which aircraft could credibly impact the PFSF in the event of a crash

A = effective area of the PFSF in square miles

A_p = the footprint area, in which a disabled aircraft could possibly hit the ground in the event of a crash

R = the probability that the pilot of a crashing aircraft would be able to take action to avoid hitting the PFSF

The controlling factor in this calculation was A_c,the area from which an aircraft could credibly impact the PFSF in the event of a crash. (PFS August 2000; Board Decision March 2003) In PFS's revised calculation in its July 2001 Addendum to the August 2000 Aircraft Crash Report based on new information, A_c, was taken to be the portion of the UTTR within 5 miles of the PFSF. Based on Air Force F-16 mishap data, a crashing aircraft more than 5 miles from the PFSF would have to be under control of the pilot in order to glide and reach the site, and the pilot would guide any such aircraft away from the site, which is outside the land boundaries and the restricted airspace of the UTTR. Additionally, because pilots conducting training on the UTTR maintain a buffer area between their training operations and the UTTR boundary, PFS used a three-mile buffer zone on the edge of the UTTR restricted areas in its calculations to represent the fact that aircraft do not fly within three miles of the edges of the restricted areas while conducting training on the UTTR. Because the PFSF is located two miles from the edge of the closest restricted area, aircraft conducting air-to-air training on the UTTR would therefore be five or more miles from the PFSF and have practically no chance of hitting the PFSF in the event of a crash.

Accordingly, the maximum annual air crash impact probability for aircraft conducting air-to-air training on the UTTR South Area was determined to be significantly less than 1 E-8, based primarily on the fact that practically no aircraft on the UTTR experiencing an in-flight mishap leading to a crash would be close enough to the PFSF site to impact it. (PFS August 2000; Board Decision March 2003)

2.2.2.3 Aircraft Flying Federal Airways

Federal airway J-56 runs east-northeast to west-southwest at a distance (from the airway centerline) of 11.5 miles north of the PFSF. (PFS August 2000) Local air traffic controllers have indicated that fewer than 12 aircraft per day use the airway. The crash impact probability for aircraft on the airway was calculated for the PFSF using the method of NUREG-0800. Using the standard width for federal airways, J-56 is 8 nautical miles (9.2 statute miles) wide and the closest edge of J-56 is 6.9 miles from the PFSF. For facilities outside an airway, the effective width of the airway, w, is equal to the actual width plus twice the distance from the facility to the closest edge. Thus, J-56 has an effective width of 23 miles. The number of aircraft, N, is conservatively taken to be 12 per day, the crash rate, C, from NUREG-0800 is 4 E-10 per mile, and the effective area of the PFSF for commercial airliners (the most common aircraft on the airway) is 0.2615 mi^2, assuming a full facility with 4,000 casks. Accordingly, the maximum annual crash impact probability is 1.9 E-8. (PFS August 2000).

Federal airway V-257 runs north and south at a distance (from the airway centerline) of 19.5 miles east of the PFSF. (PFS August 2000) Local air traffic controllers have indicated that fewer than 12 aircraft per day use the airway. The crash impact probability for aircraft on the airway was calculated for the PFSF using the method of NUREG-0800. V-257 is 12 nautical miles (13.2 statute miles) wide and its closest edge is 12.6 miles from the PFSF. Thus, V-257 has an effective width of 39 miles. The number of aircraft, N, is conservatively taken to be 12 per day, the crash rate, C, is

4 E-10 per mile, and the effective area of the PFSF is 0.2615 mi^2. Accordingly, the annual crash impact probability is 1.2 E-8. (PFS August 2000).

2.2.2.4 General Aviation

There are no civilian airports within 25 miles of the PFSF, the PFSF is located in a sparsely populated area, and the PFSF is located inside a military operating area (MOA) in which flight by civilian aircraft is limited while the MOA is being used by the Air Force (and which is avoided by general aviation pilots because of the military flight activity that takes place there). Thus, the general aviation traffic over Skull Valley is negligible; in fact F-16 pilots who have flown from Hill AFB through Skull Valley indicate never having seen any, or having seen only minimal, general aviation traffic there. Therefore, it is highly unlikely that a general aviation aircraft would crash into the PFSF. (PFS August 2000) PFS estimates that the probability would be less than 1 E-8 per year.

First, a conservative upper bound for the crash impact probability for general aviation aircraft was calculated using National Transportation Safety Board (NTSB) crash data and the population of general aviation aircraft in the state of Utah. (PFS August 2000) The crash impact probability is equal to C_a x A, where C_a is the crash rate per square mile and A is the effective area of the PFSF. In 1995, the 182,600 general aviation aircraft in the United States suffered 412 fatal accidents. There are 1,218 general

aviation aircraft in the state of Utah, which covers an area of 84,094 mi^2. FAA crash data indicate, however, that only 15 percent of all general aviation crashes occur during the cruise mode of flight, which, because there are no airports nearby, is the mode in which general aviation aircraft would be flying near the PFSF. Furthermore, business jets experience 7.85 percent of all general aviation fatal crashes and they can be excluded from this calculation, in that they fly mostly on federal airways. The effective area of the PFSF with respect to general aviation aircraft crashes is 0.1173 mi^2

(assuming a fully loaded facility with 4,000 casks). Accordingly, the annual crash impact probability for general aviation aircraft is 5.25 E-7. (PFS August 2000).

The crash impact hazard to the PFSF, however, would be reduced below the calculated impact probability, in that the spent fuel storage casks would be able to withstand the crash impact of most general aviation aircraft. Fifty-five percent of all general aviation aircraft are single-engine piston types weighing less than 3,500 lbs. (PFS August 2000) Such aircraft typically fly at speeds under 100 knots (114 mph). Therefore, the impact of such aircraft at the PFSF would be bounded by the design basis tornado missile impact for the HI-STORM spent fuel storage casks, an automobile weighing 1800 kg (3,968 lbs.) moving at a speed of 126 mph. (p. 8.2-17) Thus, the impact of such light general aviation aircraft would not cause a radioactive release from a storage cask. Therefore, the calculated upper bound general aviation crash impact hazard to the PFSF can be reduced by 55 percent to 2.36 E-7 per year.

Based on the observations of F-16 pilots in Skull Valley, however, the general aviation traffic level there is significantly below the statewide average traffic level. Thus, the Skull Valley crash rate is significantly less than the statewide rate. The methodology of NUREG-0800, Section 3.5.1.6, can be used to correlate these observations with an associated hazard probability. (PFS August 2000).

The crash impact hazard posed by general aviation aircraft flying through Skull Valley is given, according to NUREG-0800, by P = N x C x A/W (see Section 2.2.2.1). The fatal crash rate, C, for fixed wing, powered general aviation aircraft in the cruise mode of flight, is 3.82 E-8 per mile. For Skull Valley, the corridor width, W, was set at 10 miles, which is conservative for purposes of this evaluation, and the PFSF effective area, A, was set at 0.1173 mi^2, the value computed above for general aviation. Therefore, if P is equal to 1 E-8, then N must be 22 general aviation aircraft per year transiting Skull Valley in the neighborhood of the PFSF. This number, however, includes single-engine

piston general aviation aircraft weighing less than 3,500 lbs., which account for at least 55 percent of all general aviation aircraft and which pose no hazard to the PFSF. Refining this calculation to exclude those aircraft, the required number of aircraft transiting Skull Valley becomes 49 per year, or approximately one flight every 7 days. It is reasonable to assume that if general aviation aircraft were transiting Skull Valley on a weekly basis, they would be seen by the F-16 pilots (e.g, in FY98 there were approximately 11 F-16 flights per day), but in fact the F-16 pilots have never, or seldom, seen them. (PFS August 2000) Thus, PFS concludes that the general aviation crash impact hazard to the PFSF is less than 1 E-8.

Furthermore, the effective general aviation crash impact hazard posed by those aircraft whose impacts would not be bounded by the design basis tornado missile (i.e., the 45 percent of all general aviation impacts not excluded above) is shown to be even lower than 1 E-8 by PFS calculations demonstrating that such aircraft would not penetrate a spent fuel storage cask even in the remote event they were to impact the PFSF. PFS employed the methodology used by the Department of Energy to assess aircraft crash risks to hazardous facilities (DOE 1996; Davis et al. 1998) and determined that the engine of a general aviation aircraft, which would be the bounding component of a crashing aircraft, could not penetrate a storage cask. (PFS August 2000). General aviation aircraft other than jets may be modeled as having engines weighing between 230 and 800 lbs. and impacting at speeds between 67 and 280 miles per hour. (Kimura and Budnitz 1987) PFS calculations showed that an 800-lb. engine impacting at a speed of 280 miles per hour would have substantially less energy than that required to penetrate a spent fuel storage cask. Therefore, PFS's assessment of the crash impact hazard posed by general aviation aircraft whose impacts would not be bounded by the design basis tornado missile is highly conservative, particularly in view of the structural analyses subsequently performed by PFS for F-16s impacting the PFSF.

2.2.2.5 Cumulative Air Crash Unanalyzed Event Probability

Aircraft crash probability calculations for the PFSF were thus performed for all potential aircraft hazards in the Skull Valley environs. Further, as set forth in Sections 2.2.2.2.1.3 and 2.2.2.2,2 PFS performed structural analyses of F-16 and jettisoned ordnance impacts at the the PFSF site in order to determine those categories of impacts that could be shown would not breach a spent fuel storage cask and hence could be "screened out" of the crash impact probability for purposes of determining whether a radiological release caused by an aircraft crash is a credible event. If the probability of occurrence of the remaining impacts that are not screened out by such structural analyses – defined as the "Unanalyzed Event Probability" (UEP) in Section 2.2.2.2.1.3 – is less than 1 E-6, the likelihood of an aircraft crash causing a radiological release at the PFSF site would not be a credible event that would need to be designed against under applicable NRC standards. (Cornell January 2004; Board Decision February 2005) The cumulative maximum UEP for the PFSF (treating air crash impact probabilities as an UEP where no UEP was calculated) for the various potential aircraft hazards in the Skull Valley environs is given in the table below.

Aircraft Crash Unanalyzed Event Probabilities	
Aircraft	**Maximum Annual Probability**
Skull Valley F-16s (both transiting en route to the UTTR and returning via the Moser recovery route)	6.35 E-7
UTTR Aircraft	<1 E-8
Aircraft on Airway J-56	1.9 E-8
Aircraft on Airway V-257	1.2 E-8
General Aviation Aircraft	<1 E-8
Aircraft on Airway IR-420	3.0 E-9
Cumulative Aircraft Crash Hazard	<6.89 E-7
Jettisoned Military Ordnance	4.8 E-8
Cumulative Hazard	<7.37 E-7

The table shows that the cumulative air crash UEP hazard probability is less than 1 E-6 for the PFSF. Qualitative factors discussed below show further that the true hazard probability is significantly less than the calculated probability. Thus, aircraft crashes do not pose a credible hazard to the PFSF and the PFSF does not need to be designed to withstand the effects of air crash impacts.

2.2.2.6 Projected Growth in Air Traffic

The Federal Aviation Administration projects that the number of commercial aviation flights in the United States will increase by approximately 66 percent between 1998 and 2025, that the number of general aviation flights will increase by approximately 14 percent over the same period, and that the number of military flights will not increase during this period. (FAA 1999) Furthermore, based on PFS assessment of Air Force operational policies, it is not expected that F-16s at Hill AFB will fly significantly more sorties per aircraft per year than were flown in FY00. (PFS August 2000) Because most of the air traffic near the PFSF site is military, the growth in commercial and general aviation projected by the FAA will have no material effect on the air crash impact probability calculated for the facility.

2.2.2.7 Conservatisms in the Calcualted PFSF Air Crash Hazard Probability

While the calculated cumulative aircraft and jettisoned ordnance hazard for the PFSF is less than 7.37 E-7, qualitative factors indicate that the true hazard is much lower. (PFS August 2000; Cornell January 2004; Board Decision February 2005) Three major conservatisms exist in the analyses described above.

First, the above F-16 hazard probability calculated for the PFSF takes no credit for the likelihood that a pilot in control of a crashing F-16 in Skull Valley would direct it away from the PFSF site. PFS's experts, three former U.S. Air Force pilots, determined that, in the circumstances of an F-16 crashing in Skull Valley, a pilot in control of a crashing F-16 would, at least 95% of the time, successfully direct the crashing F-16 away from the PFSF site. (PFS August 2000) The State of Utah's opposing expert witness (also a former Air Force pilot) agreed that for a large body of accidents, a pilot of a F-16 crashing in Skull Valley would be able to avoid the PFSF. (PFS Hearing Tr.2002 at 8432, 8502-03) Accordingly, the fact that a pilot in a large number of instances would be able to direct the crashing aircraft away from and avoid crashing into the PFSF is a

"material conservatism" in the calculated F-16 hazard probabilities for the PFSF. (Board Decision February 2005).

Second, the UEP calculated by PFS for the F-16 impacts is highly conservative because all impacts are assumed to be direct, radial impacts onto the casks near the top or the middle of the cask with the centerline of the F-16 impacting the centerline of the cask. The probability of such worst-case impacts is low as it is much more likely for the impact to occur off-center causing both a reduced effect on the cask itself as well as a reorientation of the aircraft resulting in the dissipation of energy and the reduction of force transmitted to the cask. (Holtec 2004; Cornell January 2004; Board Decision February 2005). Moreover, treating all F-16 impacts as radial impacts directly onto to the cask effectively – and very conservatively – assumes that F-16s which hit the skid area in front of the casks (included in and constituting approximately 15% of the cask storage effective area) will continue, with no damage or decrease in speed, to impact the cask as if it had done so directly. (Board Decision February 2005).

Third, the UEP calculations for the F-16 and ordnance impacts are bounding calcultions shown by analyses <u>not</u> to cause radiation release and says nothing as to whether scenarios above the bounding case will in fact cause a radioactive release. (Cornell January 2004; Board Decision February 2005). Indeed, PFS performed calculations above the bounding speed used as the basis for computing the UEP for F-16 impacts for which it was shown that no breach of the cask and canister would occur. (Soler July 2004; Board Decision February 2005)

Beyond these conservastims, many other conservative assumptions were made by PFS in calculating the upper bound hazard of 7.37 E-7 for the PFSF. These include, for example, calculating the hazard probability for a fully loaded, 4,000 cask facility which would be the case for only a short period in the life of the PFSF; using conservative force time histories for modeling the impact of an F-16 onto a cask; conservatively assuming that canisters transfer operations would take place in the Canister Transfer

Building continually around the clock for 365 days; and many more. (PFS August 2000, Stone & Webster July 2003; Holtec 2004; Cornell January 2004).

2.2.3 The Use of Ordnance on the UTTR

As discussed in Section 2.2.2.2, military aircraft conduct air-to-ground attack training using air-delivered ordnance on the UTTR South Area. Military aircraft also conduct weapons testing, including the testing of cruise missiles. (PFS August 2000) As shown in the following paragraphs, the use of air-delivered ordnance on the UTTR does not pose a significant hazard to the PFSF (the hazard posed by jettisoned ordnance in Skull Valley was calculated in Section 2.2.2.2.1 above). The PFSF site is located 18 miles to the east of the easternmost land boundary of the range. Based on the following paragraphs it is concluded that weapons use on the UTTR does not pose a credible hazard to the PFSF and the facility does not need to be designed to withstand a weapon impact.

Weapons use on the UTTR is strictly controlled and the UTTR has never experienced an unanticipated munitions release outside of designated launch/release areas. Aircraft flying over Skull Valley are not permitted to have their armament switches in a release capable mode, and all switches are "safe" until the aircraft are inside DOD land boundaries. Master Arm switches are not actually armed until the aircraft are on the ranges within the UTTR where the bombs are to be dropped. Furthermore, the targets on the UTTR are all over 20 miles from the PFSF site and there are no run-in headings for weapons delivery over the Skull Valley area.

Hung Ordnance

The probability of "hung ordnance" (i.e., the failure of ordnance to release from an aircraft when delivery is attempted) and an unintentional release of the ordnance in Skull Valley are exceedingly low. First, most aircraft do not even carry live ordnance

but instead carry training ordnance such as Bomb Dummy Units (BDU) or inert filled or empty MK82 500 lb bombs. According to the U.S. Air Force, only approximately 15% of the 8,711 UTTR sorties flown in Fiscal Year 1998 actually carried live ordnance. Training bombs, by contrast, pose no explosive hazard to the PFSF and the dead weight of the BDUs pose no risk to the facility as well. BDU-33's have ballistic characteristics similar to MK 82 bombs but carry only a small smoke charge for marking purposes. They weigh only 25 pounds and are often the weapon of choice for training missions. Second the probability that any ordnance will "hang" is very low. Michael AAF is the designated primary airfield for aircraft landing with live hung ordnance that has failed to release. There were only five hung ordnance aircraft diversions/recoveries into Michael AAF during 1998. Since only approximately 15% of the aircraft sorties carry live ordnance, a total of only five hung ordnance recoveries in 1998 for a total of about 2,000 sorties (approximately 15% of the 13,367 over the UTTR) produces a probability for failing to release of approximately one in 400. Moreover, a failure to release does not mean there will be an inadvertent release or an inadvertent release and explosion. As indicated above, the Air Force has never had an unintentional release of ordnance outside the launch/drop/shoot boxes on the UTTR. All of these are obviously within the UTTR and in fact are over 20 statute miles from the PFSF site.

Finally, the probability of "hung ordnance" striking the PFSF is not credible because aircraft carrying hung ordnance do not fly over Skull Valley. In the event of hung ordnance, the first priority is to maintain aircraft control and then assess the situation and take appropriate action. Pilots contact Clover Control Air Traffic Control Facility and advise them of the situation. When hung ordnance is encountered, the pilot has the option of either jettisoning the rack and munitions on the range, if able, or recovering to base. Michael AAF is the designated primary recovery base for hung ordnance, although Hill AFB is available as well. Pilots request clearance to Michael AAF for hung ordnance recovery/landing. Pilots maintain a stable flight path and remain in Visual Meteorological Conditions by avoiding clouds. Clover Control provides assistance as required and ensures Michael AAF is prepared to receive the aircraft to

include fire fighting equipment and medical personnel standing by. The pilot maneuvers the aircraft to the northwest, approximately 20 statute miles from the proposed PFSF site, and proceeds to Michael AAF, avoiding rapid or steep turns and abrupt climbs or descents. Test facilities or any populated areas are avoided. A long straight-in approach with a shallow rate of descent is established to a full stop landing on runway 12 (to the southeast). Runway 12 is 13,125 ft long and 200 ft wide with a barrier cable at the end. After landing, Dugway Proving Ground Explosive Ordnance Disposal personnel inspect and safe the ordnance.

The UTTR record of no unintended release of live ordnance outside of designated launch/release areas and the procedure for landing aircraft with hung ordnance, which avoids populated areas and approaches Michael Army Airfield from the northwest, away from the PFSF, assures that hung ordnance will not impact the PFSF. Consequently, hung ordnance striking the PFSF is not a credible event.

Cruise Missiles

PFS has assessed the hazard posed to the PFSF by cruise missile testing. (PFS January 2001) Missile launches (where the missiles are released from aircraft carrying them) are generally confined to the northern and western portions of the UTTR and are at least 30 statute miles away from the PFSF site. Run-ins, drops, and launches are normally done from north to south or east to west and are thus directed away from the PFSF site. Cruise missile targets on the UTTR are located at least 18 miles from the PFSF. Cruise missile flight paths on the UTTR are plotted to approach no closer than within 10 nautical miles of the PFSF site. Cruise missiles and other weapon systems that have a capability of exceeding range boundaries are required to have a Flight Termination System (FTS) installed prior to testing on the UTTR. The FTSs are designed to promptly terminate the weapons' flight paths in the event of an anomaly. Before a bomber launches a test cruise missile, the Mission Control Center verifies that the missile's remote control systems are working properly. At all times throughout the flight the cruise missile FTS must detect a signal that in effect permits the missile to

keep flying (FTS discussed in USAF Accident Investigation Board Report, 12/10/97). If the missile does not detect the signal for a preset time, the FTS activates. Safety Officers can also activate the FTS, if required, at any time. The Range Safety Officer at Mission Control and the Airborne Range Instrumentation Aircraft are also both capable of terminating missile flight almost immediately. The UTTR has never experienced an FTS failure and the Air Force is aware of no instance where a cruise missile impacted the ground more than one mile laterally from the missile's plotted flight path. Consequently, a cruise missile striking the PFSF is not a credible event.

2.3 METEOROLOGY

2.3.1 Regional Climatology

2.3.1.1 Data Sources

The description of the regional climatology of Skull Valley and the characterization of the PFSF site climate are based on "Climatography of the United States No. 60, Climate of Utah" published by the National Climatic Data Center (NOAA, 1960), long-term meteorological data collected by the National Weather Service at the Salt Lake City International Airport (SLCIA) as summarized by the National Climatic Data Center (NOAA, 1992), and "Utah Climate" published by the Utah Climate Center, Utah State University (Ashcroft et al., 1992). Normals, means, and extremes of temperature, precipitation, relative humidity, and wind speeds are taken from NOAA (1992). The SLCIA is located approximately 50 miles northeast of the site at an elevation of approximately 4,220 ft; the PFSF site is at an elevation of approximately 4,465 ft. Meteorological data collected at SLCIA, within 50 miles of the site, can be considered representative of the general climate of the site.

Extreme wind data are also obtained from Simiu et al. (1979), and extreme precipitation data are supplemented by the U.S. Department of Commerce (1955). Data on tornado occurrences and probabilities are derived from NOAA (1975-1995), Ramsdell and Andrews (1986), and Grazulis (1993). Occurrences of severe storms, hail, ice storms, and unusual events are taken from NOAA (1975-1995). Information on the frequency of stagnation conditions (poor dispersion) are obtained from Hosler (1961) and Holzworth (1974).

2.3.1.2 General Climate

The climate of the PFSF site in Skull Valley, Tooele County, Utah, can best be described as "semi-arid continental" marked with four well-defined seasons. Summers are characterized by hot, dry weather but the high temperatures are usually not oppressive because the relative humidity is generally low and the nights usually cool. July is the hottest month with temperature readings above 90°F. The mean diurnal temperature range is about 30°F in the summer and 18°F during the winter. Temperatures above 100°F in the summer or colder than 0°F in the winter occur occasionally. Winters are cold, but usually not severe. Mountains to the north and east act as a barrier to frequent invasions of cold continental air.

Heavy fog can develop under temperature inversions in the winter and persist for several days. Precipitation is generally light with the driest months being in summer and early fall and the wetter months in the spring when storms from the Pacific Ocean are moving through the area more frequently than at any other season of the year. Winds are usually light to moderate, although occasional high winds have occurred in every month of the year, particularly in March.

Utah's climate is the result of several factors. These factors include its latitude, elevation above sea level, location with respect to the average storm track over the Intermountain Region, and its distance from the principal moisture sources, the Pacific Ocean and Gulf of Mexico. The mountain ranges in the western United States also have a significant impact on the climate of the region, particularly the Cascade and Sierra Nevada Ranges and the Rocky Mountains. Pacific storms must cross the Cascades and Sierras before reaching Utah, resulting in much of the moisture being removed by precipitation as the moist air rises over the high mountains. Therefore, the prevailing westerly air flow reaching Utah is relatively dry, which results in light precipitation.

Besides the mountain ranges, the most influential natural feature affecting the climate of the area is the Great Salt Lake. This large inland body of water, which never freezes over because of its high salt content, can moderate the temperatures of cold winter winds blowing from the northwest and north and helps drive a lake/valley wind system. This system is characterized by on-shore breezes flowing southward off the lake during warm sunny days when the lake temperature is colder than the land. This wind system reverses to off-shore breezes during evening and nighttime hours when the land cools below the temperature of the lake. The warmer lake water during the winter and spring also contributes to increased precipitation in the valley downwind from the lake.

The range of temperatures in the area is rather large from winter to summer. Summers are generally hot and dry with temperatures reaching 90°F or higher approximately 56.5 days per year on average at Salt Lake City. The mean monthly temperature in July is 77.5°F with an average maximum temperature of 93.2°F. Winters are cold but usually not severe with an average monthly temperature of 28.6°F in January along with a daily minimum temperature of 19.7°F. The average number of days with temperatures reaching 32°F or below at Salt Lake City is 124.6 days with the first freeze normally occurring in mid-October and the last freeze occurring in late April. The growing season is normally over 5 months in length.

Precipitation tends to be heaviest in the spring months with the larger amounts occurring between March and May and the least amounts in the summer. The annual average rainfall amount at Salt Lake City is 15.3 inches with the largest amounts occurring in April with 2.21 inches and least amounts occurring in July at 0.72 inches. Precipitation occurs an average of 90.3 days per year (0.01 inches or more) at Salt Lake City. The average annual snowfall (1963 - 1992) is 57.6 inches per year occurring mostly between November and April and ranging from a low of 30.2 inches to a high of 110.8 inches.

On an annual average basis, relative humidities at Salt Lake City range from a high of 67 percent in the early morning hours to 43 percent in the afternoon. On a seasonal basis, the highest relative humidities occur in the fall and winter while summer relative humidities are generally the lowest. A better measure of humidity is dew point, which indicates the actual amount of moisture in the air as it is the temperature at which saturation occurs. Monthly average dew point temperatures in this area generally range from a high of mid-40°F in the summer to lows of low-to-mid 20°F in winter. Heavy fog with visibility below 0.25 mile at Salt Lake City is not a frequently occurring phenomenon, with an average annual frequency of 11.6 days per year, but does normally occur about 2 to 4 times per month during winter.

Winds at Salt Lake City are generally light to moderate with the highest speeds occurring during spring and summer with an average of approximately 10 mph for those months. The lightest winds occur in late fall and winter with the lowest monthly average wind speed of 7.4 mph occurring in December. The highest monthly wind speed of 9.7 mph occurs in August, and the long-term mean wind speed for the year is 8.8 mph. The prevailing wind direction at Salt Lake City is from the south-southeast or southeast throughout the year. The overall prevailing wind direction is from the south-southeast.

The winds observed at Salt Lake City for the years 1988 through 1992 are depicted on a seasonal and annual basis in wind roses presented in Figures 2.3-1 through 2.3-5. The wind roses show the percent of the time (rings) that the wind blows from each of 16 directions (N, NNE, NE,...NNW) by the length of the bars. The shading of the bars also indicates the frequency of occurrence of wind speeds within the wind speed classes shown on the figures. For example, Figure 2.3-5 indicates that for the prevailing winds from the south-southeast, approximately 3 percent are in the 0 to 3 miles per hour (mph) wind speed class while about 16 percent of the winds are in the 3 to 7 mph class.

2.3.1.3 Severe Weather

2.3.1.3.1 Maximum and Minimum Temperatures

Based on observations taken by the National Weather Service at SLCIA, temperatures reaching 90°F or higher occur 56.5 days per year on average. The record high temperature at Salt Lake City based on 63 years of observations is 107°F occurring in July 1960, with record high temperatures ranging from 105°F (1951 - 1958 Iosepa South Ranch data) to 109°F (1950 - 1992 Dugway data) in Skull Valley (Ashcroft et al., 1992). The lowest recorded temperature at Salt Lake City is -30°F occurring in February 1933, with record low temperatures from -11°F (Iosepa South Ranch) to -29°F (Dugway) in Skull Valley (Ashcroft et al., 1992).

2.3.1.3.2 Extreme Winds

The highest observed fastest-mile wind speed at Salt Lake City is 71 mph from the northwest (March 1954) based on 56 years of observations while the peak gust is 69 mph from the southwest (May 1989) based on 8 years of observations. The fastest mile wind speed is the fastest speed of any "mile" of wind (Huschke, 1959). These wind gusts observed at Salt Lake City are consistent with observations of peak gusts observed in Skull Valley and Tooele County with Dugway Proving Grounds recording a peak gust of 68 mph from a thunderstorm and Tooele reporting a gust of 72 mph in a severe thunderstorm (NOAA, 1975-1995).

The 50- and 100-yr return period fastest-mile wind speeds at a height of 10 meters for Salt Lake City are 70.4 and 74.5 mph based on an extreme value Type I probability distribution (Simiu et al., 1979). These fastest-mile winds should be multiplied by a "gust response factor" to account for the fluctuating nature of wind and its interaction with

buildings and other structures (ANSI, 1982). For exposures in open terrain with scattered obstructions having heights generally less than 30 feet (includes flat, open country and grasslands), the gust response factor at a height of 30 feet is 1.26. Therefore, the 50- and 100-yr design wind speeds at an elevation of approximately 30 feet are 88.7 and 93.9 mph, respectively.

2.3.1.3.3 Tornadoes

The state of Utah experiences relatively few tornadoes. A total of 77 tornadoes were reported for the period from 1953 to 1995 (NOAA, 1975-1995), and only four were considered "significant" based on an examination of tornadoes from 1880 to 1993 (Grazulis, 1993). None of the "significant" tornadoes occurred in Tooele County. The state averages one tornado per year with a density of 0.12 tornadoes per 10,000 square miles (NOAA, 1975-1995), ranking 43rd out of the 50 states plus the District of Columbia. By contrast, the state of Texas has reported 5,674 tornadoes over the same period of time with a density of 4.90 tornadoes per 10,000 square miles.

Table 2.3-1 summarizes occurrences of tornadoes within a 1-degree latitude-longitude square centered on the PFSF site. This list of four tornadoes covering the period from 1975 to 1995 is based on the publication "Storm Data" (NOAA, 1975-1995). For each tornado, Table 2.3-1 identifies the date of occurrence, the county in which the tornado occurred, path length in miles, path width in yards, and the Fujita Scale classification (explained in Table 2.3-2).
Using the information provided in Table 2.3-1, the probability of a tornado striking a point within the 1-degree latitude-longitude square centered on the PFSF site is estimated as follows (Thom, 1963):

$$P = z \times t / A,$$

where:

P = mean probability of a tornado strike per year

z = geometric mean tornado path area (mi^2)

t = mean number of tornadoes per year

A = area of 1-degree square (mi^2)

The area (A) of the 1-degree box centered on the site can be estimated in square miles according to 4774.3 cos(L) (Ramsdell and Andrews, 1986) where L is the latitude of the middle of the box. With an estimated latitude of 40.3 degrees north, A is approximately 3,641 square miles. The geometric mean of the tornado path lengths and widths given in Table 2.3-1 are 0.93 mile and 66.9 yards, respectively, yielding a geometric mean tornado path area of 0.035 mi^2. The mean number of tornadoes per year is 0.14 based on three observed tornadoes within the area over a 21-year period of record. Therefore, the probability of a tornado striking the PFSF site is estimated to be 1.37×10^{-6} per year or a recurrence interval of 728,200 years. This value compares with an arithmetic average tornado strike probability of 9.72×10^{-7} per year and an expected probability of 3.06×10^{-6} per year (based on a log normal distribution) for the entire state as computed in NUREG/CR-4461 (Ramsdell and Andrews, 1986) based on tornadoes for the period 1954 to 1983.

2.3.1.3.4 Hurricanes and Tropical Storms

Because of the site's location, approximately 800 miles from the Pacific Coast and more than 1,200 miles from the Gulf of Mexico, hurricanes and tropical storms do not affect this area.

2.3.1.3.5 Precipitation Extremes

The maximum observed 24-hour rainfall amount at Salt Lake City is 2.4 inches occurring in April 1957 along with a maximum monthly amount of 7.0 inches. The highest recorded daily and monthly precipitation amounts at Dugway are 1.46 and 3.16 inches, respectively (Ashcroft et al., 1992). The highest short-term rainfall rate observed in Skull Valley was 0.95 inch in 30 minutes at Dugway during a thunderstorm in August 1984 (NOAA, 1975-1995). The extreme hourly rainfall rate for Salt Lake City with a 50-year return period is approximately 1.2 inches per hour (U.S. Dept. of Commerce, 1955).

The maximum recorded monthly snowfall at Salt Lake City is 41.9 inches in March, 1977 along with a maximum 24-hour snowfall of 18.4 inches in October, 1984. At Dugway, the maximum monthly and daily snowfall amounts for the period 1950 to 1992 are 21.2 and 9.0 inches, respectively.

2.3.1.3.6 Thunderstorms and Lightning Strikes

Salt Lake City averages 36.7 thunderstorm days per year with the highest number of days (7-8 days per month) occurring in July and August (NOAA, 1992). An examination of "Storm Data" from 1975 through 1995 (NOAA, 1975-1995) indicates that there were approximately 20 occurrences of severe thunderstorms in Tooele County during this time and two instances of lightning strikes causing injuries reported in Tooele County. According to "Storm Data", there were a total of 82 lightning injuries in the entire state during the period 1959 to 1995.

2.3.1.3.7 Snowstorms

Heavy snowfalls with blizzard conditions (snow, temperature < 20°F, wind speed > 35 mph) can affect the state but are relatively infrequent. In the Tooele County area, only a few unusually heavy snowfalls (> 12 inches) were reported between 1975 and 1995 (NOAA, 1975-1995).

2.3.1.3.8 Hail and Ice Storms

Hail in Utah is relatively infrequent but it may damage fruit and vegetables in limited areas during the spring and summer (NOAA, 1960). In Tooele County specifically, only four instances of sizable hail were noted during the period from 1975 to 1995 (NOAA, 1975-1995) with some hail stones reaching 1.25 inches in size in Tooele.

Freezing rain and glazing events are not numerous in Utah but do occur occasionally. There were only four reported ice storm events in Tooele County and the northwest valleys over the 1975-1995 period (NOAA, 1975-1995). These ice storms have caused power outages, traffic accidents, and downed tree limbs.

2.3.1.3.9 Poor Dispersion Conditions

The most commonly occurring meteorological condition leading to poor dispersion is the low-level (< 500 feet) or ground-based inversion. Because temperature normally decreases with altitude, an inversion is a condition in which temperature increases with height or is inverted from the normal condition. This vertical temperature profile leads to very stable atmospheric conditions in which very little motion or turbulence takes place. Thus, a pollutant emitted into such an atmosphere experiences very little dilution due to atmospheric turbulence. Inversions tend to be diurnal in nature, generally occurring

during nighttime and early morning hours and breaking up during the heating of the day. As a result of this behavior, inversions are usually not a very persistent phenomenon.

The frequency of occurrence of low-level inversions in the PFSF site area can be estimated from isopleths of inversion frequency developed by Hosler (1961). The PFSF site falls on the 45 percent of total hours per year isopleth on an annual basis for inversion frequency. Seasonally, winter and fall experience the highest inversion frequencies at approximately 50 percent of total hours. Summer and spring inversion frequencies have been estimated to be approximately 35 to 40 percent.

A more persistent condition of poor dispersion leading to air pollution episodes is the stagnating anticyclone, which is an area of high atmospheric pressure that essentially remains stationary for a period of several days. The stagnating anticyclone is characterized by light wind speeds (<9.0 mph), no precipitation, and shallow mixing depths (< 1,600 feet). These conditions lead to very little dilution or ventilation of pollutants emitted into this air mass, which tend to recirculate within the anticyclone.

The degree to which these poor dispersion conditions occur at a given location can be estimated from a study of episodes of limited dilution conducted by Holzworth (1974). In this study, a ventilation or dilution factor was calculated as the product of mixing height (meters) and layer average wind speed (meters/second) for 62 upper air stations throughout the United States for the period 1960-1964. A low value of this factor indicates slower dilution. The Salt Lake City upper air station was found to have 1-, 2-, 3-, 4-, and 5-day episode ventilation factors of 99.9, 236.3, 322.1, 357.2, and 463.5 m^2/sec compared to those of the worst case station in Lander, Wyoming, of 14, 17, 34, 57, and 55 m^2/sec. Salt Lake City ranked 9th, 18th, 13th, 12th, and 16th out of 62 stations in 1-, 2-, 3-, 4-, and 5-day episode ventilation factors.

2.3.2 Local Meteorology

2.3.2.1 Data Sources

The meteorology of the Skull Valley site can be partially characterized using long-term meteorological data collected by the National Weather Service at the SLCIA (NOAA, 1992). This climatological data set is the most comprehensive available for this area. The SLCIA is located approximately 50 miles northeast of the site at an elevation of approximately 4,220 ft. With the PFSF site being located at an elevation of approximately 4,465 ft, meteorological data collected at SLCIA can be considered representative of the general climate of the site but need to be supplemented with data more representative of local conditions.

The valley location of the PFSF site has an influence on the local meteorology relative to that of SLCIA with the Stansbury and Oquirrh Mountains rising to elevations of above 10,000 ft between the two locations. The location of the Great Salt Lake to the north of Skull Valley as opposed to west and northwest of SLCIA also probably causes some meteorological differences between the two locations. Therefore, meteorological data collected in Skull Valley are also needed to characterize the local conditions. Monthly average temperature and precipitation data collected at various locations in Skull Valley are available from a book published by the Utah Climate Center (Ashcroft et al., 1992). The data collected at Dugway, located approximately 12 miles south of the PFSF site at an elevation of 4,340 ft, have the longest period of record (1950 - 1992) and appear to be the most reliable. Other useful data were collected at Iosepa South Ranch, which is located about 12 miles north of the PFSF site at an elevation of 4,415 ft, during the period from 1951 - 1958.

The Onsite Meteorological Monitoring Program, described in detail in Section 2.3.3, will provide hourly average data on wind speed, wind direction, temperature, relative humidity, precipitation, barometric pressure, and solar radiation for characterization of the local meteorology because many of these parameters are not available from other sources. The tower is located approximately 3 miles southeast of the PFSF site at the closest point where power and a telephone line are available. This location is judged to be a suitable representative for "onsite" meteorological data collection. The tower location is in the same topographic setting as the proposed site with the Stansbury Mountains to the east and northeast being sufficiently distant from both locations as to cause insignificant differences in meteorological observations between the two locations. Both sites are essentially the same distance from the Great Salt Lake and the Wasatch Mountains to the east. Given that the intent of the meteorological data collection program is to characterize the local meteorology and not for radiological dispersion calculations, this location provides representative data.

2.3.2.1.1 Precipitation

Normal monthly precipitation tends to be concentrated in the winter and spring months with the larger amounts occurring between December and May and the least amounts in the summer and early fall. The annual average rainfall rate at Salt Lake City is 15.3 inches per year with a record 24-hour rainfall of 2.4 inches. Precipitation occurs an average of 90 days per year (0.01 inch or more). Precipitation data collected in Skull Valley indicate a range of annual precipitation of from 7 to 12 inches per year with increasing amounts at higher elevations in the Stansbury Mountains, maximizing at Deseret Peak with approximately 40 inches per year (Hood and Waddell, 1968). A 43-year record (1950 - 1992) of precipitation data at Dugway indicates a normal annual precipitation rate of 8.2 inches per year. An 8-year record (1951 - 1958) at Iosepa South Ranch indicates an average annual precipitation rate of 9.6 inches per year. The PFSF

site data indicate annual precipitation amounts of 9.5 and 10.8 inches respectively for the years 1997 and 1998. Therefore, the valley location of the PFSF site tends toward the lowest precipitation amounts in the area. Table 2.3-3 summarizes monthly precipitation amounts for Salt Lake City and Skull Valley locations, including the PFSF site.

The long-term average annual snowfall (1963 - 1992) at Salt Lake City is 57.6 inches per year occurring mostly between November and April and ranging from a low of 30.2 inches in 1979 - 1980 to 110.8 inches in 1973 - 1974. The maximum recorded monthly snowfall is 41.9 inches in March 1977 along with a maximum 24-hour snowfall of 18.4 inches in October 1984. Information on snowfall amounts at Dugway and Iosepa South Ranch indicate normal annual snowfalls of 16.0 and 21.3 inches, respectively, with maximum monthly amounts of 21.2 and 17.7 inches. The record daily snowfalls at Dugway and Iosepa South Ranch are 9.0 and 8.0 inches each.

2.3.2.1.2 Temperature

The range of temperatures in the area is rather large from winter to summer. Summers are relatively hot with temperatures reaching 90°F or higher approximately 56 days per year on average at Salt Lake City. The average daily maximum temperature at Salt Lake City in July is 93.2°F and mean maximum temperatures at Dugway and Iosepa South Ranch exceed 90°F during July and August. Winters are moderately cold with an average monthly temperature of 28.6°F in January at Salt Lake City along with a daily minimum temperature of 19.7°F. Similar winter temperatures are experienced in Skull Valley with average monthly values near 30°F in December and January. The average number of days with temperatures reaching 32°F or below at Salt Lake City is 125 days with the first freeze normally occurring in October and the last freeze occurring in April. The annual average temperatures at Salt Lake City is approximately 52°F for the period 1951 - 1980 with Skull Valley average temperatures ranging from 49 to 51°F. Table 2.3-4

provides daily maximum, daily minimum, and average temperatures by month for the period 1951 to 1980 for Salt Lake City, 1950 to 1992 for Dugway, and 1951 to 1958 for Iosepa South Ranch. These data are compared with the temperature data collected during the 2-year PFSF site meteorological monitoring program.

2.3.2.1.3 Wind Direction and Speed

Winds at Salt Lake City are moderate and are fairly uniform over the year with the highest average speed (9.7 mph) occurring in August and the lightest average wind speed (7.4 mph) occurring in December. The long-term mean wind speed for the year is 8.8 mph. The prevailing wind direction at Salt Lake City is from the southeast or south-southeast throughout the year. The winds at the PFSF site based on the 2-year monitoring program are very similar to those of Salt Lake City. They are fairly uniform over the year with the highest monthly average speed (9.6 mph) occurring in April and the lightest monthly average wind speed (7.4 mph) occurring in November and December. The 2-year average wind speed at the PFSF site is 8.7 mph.

Table 2.3-5 provides mean wind speeds by month for a 62-year period of record and prevailing wind directions by month for Salt Lake City, along with the 2-year average values for the PFSF site. Long-term wind information is not available specifically for the Skull Valley.

2.3.2.1.4 Humidity, Fog, Thunderstorms

PFSF site relative humidity values are summarized on a monthly average basis along with those for Salt Lake City in Table 2.3-11. The Salt Lake City data are the averages of four time-of-day values from NOAA, 1992, while the PFSF site values are based on hourly averages for a 2-year period. The table indicates that the relative humidity values,

although for different time periods, are fairly similar to each other with the PFSF site values being somewhat higher during the spring and summer months. Relative humidity values are not available specifically for the other Skull Valley locations.

Heavy fog with visibility below 0.25 mile at Salt Lake City is not a frequently occurring phenomenon, with an average annual frequency of 11.6 days per year, but does normally occur 2 to 4 times per month during winter.

Salt Lake City also has a mean of 36.7 thunderstorm days per year and approximately 5 to 8 thunderstorm days per month from May through August.

2.3.2.1.5 Solar Radiation

The maximum total insolation for a 12-hour period recorded during the onsite meteorological monitoring program at the PFSF is 706.5 g cal/cm^2 or 684.6 watts/m^2 over a 12-hour period. In order to confirm the temporal representativeness of the on-site solar radiation data, this value is compared with long term solar radiation data available for Salt Lake City from the Solar and Meteorological Surface Observational Network (SAMSON) 3-volume CD-ROM set. It contains hourly solar radiation data along with selected meteorological elements for the period 1961-1990 which is the standard climatological period as defined by the World Meteorological Organization. The dataset includes both observational and modeled data. The hourly solar elements are: extraterrestrial horizontal and extraterrestrial direct normal radiation; global, diffuse, and direct normal radiation. The global horizontal radiation values were used in this comparison, as they represent the total amount of direct and diffuse solar radiation on a horizontal surface.

According to the National Climatic Data Center (NCDC) in Asheville, NC, the SAMSON database is the most reliable solar radiation data available for the area. The quality of other sources of solar radiation data in the area of PFSF cannot be confirmed by NCDC. Using the 30-year SAMSON database, the maximum total insolation for a 12-hour period was determined to be 753.8 g cal/cm^2 or 730.4 watts/m^2 over a 12-hour period. Although these values are somewhat higher than those derived from the 2-year PFSF site data, they are comparable to the site data and clearly demonstrate that the design values of 800 g cal/cm^2 or 775 watts/m^2 over a 12-hour period from 10 CFR 71.71 are bounding.

2.3.2.1.6 Atmospheric Stability and Mixing Heights

The dispersion of an air contaminant by atmospheric turbulence and diffusion can be characterized by the stability of the atmosphere. Pasquill (1961) has developed an atmospheric stability classification scheme that divides atmospheric diffusion levels into six classes labeled A through F. Stability class A represents the most "unstable" and diffusive category representative of conditions during warm sunny afternoons while F is the most "stable" and least diffusive class generally occurring during the night and early morning hours under light or calm winds. The intermediate stability class D represents "neutral" atmospheric stability and is typified by cloudy, windy conditions. These stability classes are generally determined from National Weather Service meteorological data using a combination of wind speed, cloud cover, and daytime solar insolation observations.

Table 2.3-6 presents the frequency of occurrence of each Pasquill stability class as determined for Salt Lake City based on 5 years (1988 - 1992) of meteorological data collected at the airport. This table indicates that the dispersion environment of the area is dominated by "neutral" (stability class D) stability (moderate dispersion) with stable

atmospheric conditions (weak dispersion) being approximately 60 percent more frequent than unstable conditions (strong dispersion).

Table 2.3-7 presents seasonal average mixing heights for Salt Lake City (Holzworth, 1972). The morning and afternoon mixing heights in Table 2.3-7 were approximated by the National Climatic Data Center from vertical temperature measurements taken by the National Weather Service twice daily for the period 1960 to 1964. The mixing height is defined as the height above the surface through which relatively vigorous vertical mixing occurs (Holzworth, 1972). As such, the mixing height defines the vertical layer of the atmosphere through which pollutants can be mixed. Low mixing heights, which are characteristic of the nighttime dispersion environment, generally result in higher pollutant concentrations at the surface for low-level sources (below the mixing height). The higher mixing heights occurring during midday are conducive to greater dispersion and lower ground level pollutant concentrations.

2.3.2.1.7 Air Quality

The air quality in the site area is generally very good. The U.S. Environmental Protection Agency (EPA) has adopted National Ambient Air Quality Standards (NAAQS) for six air pollutants known as criteria pollutants. The primary standards are designed to protect public health while the secondary standards are designed to protect public welfare (includes protection of economic interests, vegetation, and visibility). The Utah Department of Environmental Quality (DEQ) has adopted the federal NAAQS as the state ambient air quality standards; Table 2.3-8 shows these standards.

Ambient air monitoring data collected by the DEQ at several monitoring stations throughout the state are used to determine whether or not these NAAQS are being met. Areas where the standards are attained are referred to as "attainment" areas and those

areas not attaining the standards are called "nonattainment" areas. This project is located in the Wasatch Front Intrastate Air Quality Control Region (AQCR) which is in attainment for nitrogen dioxide (NO_2), carbon monoxide (CO), particulate matter with aerodynamic diameter less than 10 microns (PM10), lead (Pb), and ozone based on monitoring data collected in the AQCR. A portion of eastern Tooele County is currently non-attainment for the primary and secondary sulfur dioxide (SO_2) standard.

The attainment status of the Wasatch Front Intrastate AQCR is supported by the three most recent years (1993 - 1995) of available ambient air quality monitoring data collected by the DEQ in the AQCR. Table 2.3-9 summarizes these data showing the highest annual average and second highest short-term (1-, 3-, 8-, 24-hour) monitored values in the county for each pollutant and averaging time. These data demonstrate that ambient criteria pollutant concentrations are well below the NAAQS.

Given that all of Tooele County, except for the highest elevation areas in the far eastern part of the county, is currently in attainment of the NAAQS for all criteria pollutants and the available monitoring data indicate air pollutant concentrations generally well below the NAAQS, air quality is not expected to have any effect on the operation of the PFSF facility.

2.3.2.2 Topography

The PFSF is located in Skull Valley at an elevation of approximately 4,465 ft. The Stansbury Mountains, located 9 miles to the east, rise to a maximum elevation of 11,031 ft. The Cedar Mountains, located approximately 11 miles to the west, rise to a maximum elevation of approximately 7,600 ft. The detailed topographic features in proximity to the site are shown on Figure 1.1-1.

2.3.3 Onsite Meteorological Measurement Program

The On-site Meteorological Monitoring Program provides hourly average data on wind speed, wind direction, air and soil temperature, relative humidity, precipitation, barometric pressure, and solar radiation for characterization of the local meteorology since several of these parameters are not available from other sources. The tower is located approximately 2 miles southeast of the PFSF site at a roadside shop where AC power and a telephone line are available as shown on a section of a 7.5 minute United States Geological Survey (USGS) topographic map in Figure 2.3-6.

This location is suitable for "onsite" data collection from a meteorological representativeness perspective. The tower location is in the same topographic setting as the proposed site with the Stansbury Mountains to the east and northeast being sufficiently distant from both locations as to cause insignificant differences in meteorological observations between the two locations. Both sites are essentially the same distance from the Great Salt Lake and the Wasatch Mountains to the east. Given that the intent of the meteorological data collection program is to characterize the local meteorology and not for radiological dispersion calculations, this location provides representative data.

The meteorological monitoring system consists of a heavy-duty 10-meter tower with wind speed and direction sensors at the 10-meter level and temperature, relative humidity, and solar radiation sensors at the 2-meter level. Barometric pressure and precipitation (rain and snow) are recorded at ground level. Soil temperature is recorded at 1 meter below the surface. The tower is equipped with a lightning protection and grounding system and a signal line surge protector. The monitoring system is protected by a 6-foot-high fence made of galvanized steel that includes a lockable gate. The system meets the intent of

10 CFR 50, Appendix B and the siting and accuracy requirements of Safety Guide 23 and the recommendations of the ANSI standard, ANSI/ANS-2.5.

All measurements are recorded using a Campbell Scientific CR10X digital datalogger which is configured for periodic download of data to laptop PC or data cartridges. The system is also configured with a dial-up modem for remote data interrogation and downloading. All systems are capable of operating and recording data without outside power using a 12-volt battery and solar panel, with the exception of the rain gage heater. Sampling interval for all parameters, except precipitation and sigma theta, is 60 seconds or less with mean values computed using 30 instantaneous values equally spaced over a 15-minute period. Hourly averages of these parameters are calculated from the 15-minute averages. Precipitation is recorded once per hour. Sigma theta is determined from no less than 180 instantaneous values of lateral wind direction during a 15-minute sampling period. The datalogger is located in a weather tight enclosure near the base of the tower. The detailed specifications the vendor was required to meet are provided in Table 2.3-10. The system consists of the following sensors and equipment as provided by Climatronics Corporation:

1. One 10-meter heavy duty folding aluminum tower.
2. Two WM-IIIA wind speed/direction sensors (one as spare).
3. Two platinum temperature sensors (one as spare).
4. Two naturally aspirated radiation shields for temperature (one as spare).
5. One relative humidity sensor and radiation shield.
6. One soil temperature sensor.
7. Two solar radiation sensors (one as spare).
8. One tower boom.
9. One matrix radiation cable.
10. One recording rain gauge (tipping bucket).

11. Precipitation heater cables.
12. One analog barometric pressure sensor.
13. One Campbell Scientific CR10X datalogger with weather tight enclosure.
14. One 12-V 20-amp/hour battery.
15. One 20-watt solar panel.
16. Datalogger software (including sigma theta calculation).
17. Data display software with archiving.
18. One Opto-Isolated RS232 interface.
19. One phone modem (1200 baud).
20. One full height grounding kit for lightning protection.
21. One signal line surge protector.

The system is designed to assure at least a 90 percent data recovery and to minimize extended periods of instrument outage through the use of spare parts and surveillance procedures. The initial calibration of all instruments, sensors, recorders, and test equipment, including spare parts and spare instruments, was performed by Climatronics and audited by Stone & Webster. Future calibrations will be performed at six month intervals. A calibration history is kept for each sensor and recorder and for the test equipment. The measurements for wind speed, wind direction, and temperature are traceable to the National Institute of Standards and Technology (NIST). The instruments used for calibration have a certificate traceable to NIST, where applicable.

All data collected by the meteorological monitoring system are validated by a Certified Consulting Meteorologist (CCM) using validation procedures that compare monitored values with other sources of nearby meteorological data where possible and by checking values against a set of reasonableness criteria for each parameter. Meteorological parameter values that fall outside of prescribed ranges are checked for validity using

other sources of data where possible and by examining the synoptic conditions that caused the readings.

2.3.4 Diffusion Estimates

No atmospheric diffusion estimates have been developed for the PFSF as there are no routine or accidental radiological releases that can be postulated for this facility. Although a postulated release of radionuclides is assumed for the hypothetical loss of confinement accident analysis in Chapter 8, the atmospheric diffusion characteristics are based on Regulatory Guide 1.145 with a wind speed of 1 meter/sec, atmospheric stability class F, with no consideration for plume meander.

As stated above, PFS has evaluated the regional climatology using the data collected at SLCIA. Local conditions were characterized by evaluating data collected at Dugway and Iosepa South Ranch and by performance of the Onsite Meteorological Monitoring Program. PFS believes the data sources evaluated (SLCIA, Dugway, and Iosepa South Ranch) are reliable and provide information representative of the local meteorological conditions. With the inclusion of the data collected by the onsite meteorological monitoring program, PFS believes that accurate and sufficient information has been provided for use in any analyses requiring meteorological values.

2.4 SURFACE HYDROLOGY

2.4.1 Surface Hydrologic Description

The PFSF site is situated near the middle of Skull Valley about 24 miles south of highway I-80 and the Great Salt Lake. Figure 2.4-1 shows the topography of the site and surrounding area. Skull Valley was formerly occupied by Lake Bonneville, an inland sea that covered the area from about 30,000 to 10,000 years before present (B.P.). The valley is nearly 50 miles long and 22 miles wide at its widest point and slopes gently northward to Great Salt Lake at approximately 30 ft per mile.

North-south trending mountain ranges rise abruptly from the valley floor on both sides. On the east side of the valley, the Stansbury Mountains rise to 11,031 ft elevation at Deseret Peak, while on the west side, the Cedar Mountains rise to about 7,600 ft elevation. Both ranges are composed mainly of limestone and dolomite with lesser amounts of quartzite, sandstone, and shale, ranging in age from Early Cambrian to Tertiary.

A thick alluvial apron exists at the base of the ranges, formed by a series of coalescing alluvial fans. This apron (or bajada) is composed mainly of coarse clastic material derived from erosion of the adjacent ranges by high gradient streams. The surface of the bajada in the vicinity of the PFSF site slopes westward at about 165 ft per mile and meets the valley bottom about 1.5 miles east of the PFSF location.

Precipitation in Skull Valley ranges from 7 to 12 inches per year with only about one-third that amount falling during the growing season (Hood and Waddell, 1968). The uplands receive considerably more precipitation, up to 40 inches in the Stansbury Mountains and 16 to 20 inches in the lower Cedar Mountains. Much of this is in the

form of snow that enters the hydrologic system as runoff in the spring. However, very little of this water actually reaches the valley bottom as stream flow.
The pervious unconsolidated deposits of the bajada intercept runoff and serve as the main zone of recharge to the groundwater system. As a consequence, there are few perennial streams in Skull Valley and none in the vicinity of the site. The poorly developed intermittent drainages are mainly dry washes incised a few feet into the valley bottom. All of these washes in the site vicinity drain northward or northwestward and likely carry water for only short periods during spring runoff or during infrequent summer thunderstorms.

At a few locations in Skull Valley, especially along the eastern foothills, ground water intersects the surface in the form of springs. These springs have created surface channels, such as those near Timpie and Delle. In general, most spring flow is lost to the recharge area or is consumed by evapotranspiration. No springs occur within a 5-mile radius of the site. The nearest perennial surface flow downstream from the proposed site occurs about 10 miles to the north near Salt Mountain. This flow is attributed mainly to springs at the base of the hill, west of Skull Valley Road. It eventually joins other spring and stream flow, creating a large wetland/mudflat system, before draining to Great Salt Lake.

There are no perennial lakes or ponds in the site area other than a few stock ponds or small reservoirs built for irrigation purposes. These impoundments are commonly filled by water diverted in ditches or pipelines whose sources are in the canyons of the Stansbury Mountains.

There are no public or private surface drinking-water supplies in the site vicinity. Potable water supplies for the Skull Valley Indian Reservation, and the few scattered

ranches or farms along the east side of the valley, are wells drilled into the unconsolidated or semi-consolidated sediments that form the alluvial fan along the base of the Stansbury Mountains. Consequently, there is no potable surface water supply that could be subject to normal or accidental effluents from the facility.

2.4.1.1 Site and Structures

The land surface at the site is approximately 4,465 ft elevation and is nearly flat, sloping gently to the north. A few shallow, dry washes and former beach or lake bottom features provide slight relief to the site area. Desert shrubs and grasses form a thin vegetative cover.

No streams that would even be considered intermittent cross the facility area. The closest stream with a significant channel crosses the northeast corner of Section 6 and the center of Section 5, about 1,500 ft from the northeast corner of the facility (Figure 2.4-1). The channel is up to 3 ft deep and 6 to 8 ft wide in some areas. It carried no water during the observation period between June 1996 and February 1997.

The facility and its structures are described in Chapter 1. The PFSF storage area is approximately 99 acres. The storage area is graded for surface drainage with slopes from south to north of approximately 0.5%. Site elevations vary from approximately 4,460 ft at the north to 4,470 ft at the south. A stormwater detention basin is located on the north side of the storage area to collect runoff from the storage area.

2.4.1.2 Hydrosphere

Watersheds contributing runoff to the areas of the access road and the 3-mile-long rail road adjacent to the PFSF site are shown in Figure 2.4-1. Watershed runoff contributing

to the access road area is primarily from the east mountain range of the valley, and is designated as Basin A in Figure 2.4-1. Basin A is a watershed comprising an area of approximately 270 square miles. Watershed runoff contributing to the 3-mile-long rail road adjacent to the PFSF site is from the west mountain range on the west of the valley, and is designated as Basin B in Figure 2.4-1. Basin B is a watershed comprising an area of approximately 64 square miles (40,960 acres). The PFSF is separated from Basin A and Basin B by an earthen berm proposed for construction to keep out runoff from the two basins. The topography and approximate sheet flow directions in the site vicinity are also shown in Figure 2.4-1.

A major portion of Basin A runoff originates in the east upland extending from the lower Stansbury Mountains to the Lookout mountain in the south. Runoff is drained into the valley by a number of intermittent streams (see Figure 2.4-1). The flow from the mountain front, after crossing the alluvial fans at the foothills of the mountains is quickly lost to the pervious sublayer and evapotranspiration to become an intermittent stream. Stream flow would be produced only by very heavy rainfall or during snowmelt conditions.

Basin B runoff is primarily from the upland in the Lower Cedar Mountains on the west of PFSF. The runoff converges to the 3-mile-long rail road through many small streams. Similar to Basin A, these small streams are normally dried, stream flow can only be seen after a heavy thunder storm.

During site visits conducted between June 1996 and February 1997, several hydrologic observations were made. No perennial streams were observed to cross Skull Valley road from the uplands to the east, nor were any perennial streams observed west of the site to the base of the Cedar Mountains. There are no upstream or downstream flow control structures whose failure could conceivably affect the site or its access road. The only

structures located in the area are very small reservoirs in the foothills used as stock ponds or for collection of water for irrigation purposes.

Hood and Waddell (1968) indicate that the groundwater table in Skull Valley in the vicinity of the site ranges from elevation 4,300 to 4,350 ft. The groundwater table at the site is at approximate elevation 4,350 ft, a depth of approximately 125 ft based on a monitoring well near the Canister Transfer Building (CTB), installed in early 1999. This value is consistent with the upper bound of the range of depths to the water table reported by Geosphere Midwest (Appendix 2B), who report seismic refraction results indicate that the water table may be located at depths of between 90 and 136 ft (Seismic Lines 1, 2, & 3) below existing grade. Because of the great depth to the groundwater table, it is very unlikely that the groundwater regime could have any influence on the stability of structures at the site.

2.4.2 Floods

There is no evidence the site area has experienced flooding in the past. Storm-induced runoff will provide sheet flow toward the site which will easily be controlled by construction of short diversion berms near the southern portions of the PFSF.

Analyses of the probable maximum precipitation were performed to determine a PMF for stormwater drainage Basins A (SWEC, 1999a) and B (SWEC, 1999c). The analyses demonstrated that the site would not be in the flood plain caused by any flood event.

2.4.2.1 Flood History

The PFSF site is located in an area of western Utah with a semi-arid climate, receiving average annual precipitation of 7 to 12 inches (Hood and Waddell, 1968). There are no

perennial water courses within 4 miles of the site. The nearest streams are high gradient streams that drain the slopes of the Stansbury Mountains through steep-walled canyons. This flow is quickly lost to the unconsolidated sediments comprising the alluvial apron at the foot of the mountains and becomes part of the groundwater system. No perennial surface flow makes its way across Skull Valley road which runs north-south, approximately 1.5 miles east of the PFSF site.

There is no evidence of past flooding in the site area and only minor development of drainage channels created by infrequent thunderstorms (<1 to 2 ft deep). There is no evidence of flash-flooding in the area, such as flood deposits, nor are there channels that could affect the site if they were subject to a flash flood.

The only conceivable scenario for floods would involve a return to climatic conditions of the Late Pleistocene causing a significant rise (~300 ft) in the level of Great Salt Lake. Those conditions generally require millennia to develop, therefore, this scenario is dismissed.

2.4.2.2 Flood Design Considerations

Hypothetical Probable Maximum Precipitation (PMP) events were analyzed to determine maximum flooding elevation at the PFSF site due to flood flows from Basin A and Basin B. The analyses included the general storm and the local storm events, as discussed below in Section 2.4.2.3. Determination of Probable Maximum Flood (PMF) was based on the procedures given in Hydrometeorological Report (HMR) 49 (U.S. Department of Commerce, 1977). In Basin A, the PMF is generated by a general storm, while in Basin B, it is generated by a local storm.

For an extremely conservative PMF (flow = 85,000 cfs for Basin A, 102,000 cfs for Basin B, detailed in Section 2.4.2.3), results of hydraulic analysis indicate that maximum water surface elevation is predicted to be 4,506.4 ft upstream of the access road and the PMF berm in the east floodway with runoff from Basin A, and 4,478.2 ft upstream of the rail spur and the PMF berm in the west floodway from Basin B. Both of the predicted flood elevations are below the designed top elevations of the PMF diversion berms of 4,507.5 ft and 4,480 ft, respectively. At cross sections downstream of the access road, rail spur and the PMF berms, the predicted flood elevations (SWEC 1999d) are below the designed site grade elevations of 4,463 ft to 4,475 ft. Consequently, all Structures, Systems, and Components (SSCs) classified as being Important to Safety are located above the PMF flood plains.

Figures 2.4-2, 2.4-3, 2.4-4 and 2.4-5 show a plan and profile (elevation) views of the PFSF site PMF berm, the PFSF rail line, the PFSF access road, and the PFSF access road PMF berm, respectively. In each of these figures, the maximum PMF water level is also shown in the profile view.

The results of the PMF analyses (SWEC, 1999d) demonstrate that the floodwater from the PMF will not inundate the pad emplacement area. In the discussion that follows, note that the elevations of the tops of the cask storage pads range from a low of Elevation 4,463 ft at the northern end of the pad emplacement area, to a high of Elevation 4,475 ft at the southern end. These are shown in Pad Emplacement Area Foundation Profiles 7-7' through 12-12', presented as Sheets 9 through 14 of Figure 2.6-5. The locations of these profiles are presented on Figure 2.6-19.

Figure 1 of SWEC (1999d) identifies the locations of Drainage Basins A and B, which are also shown on Figure 2.4-1. Figure 8 of this calculation presents a plan view of the PMF flood boundaries, and it indicates that the PMF boundaries for both Basins A and

B do not reach the site, because of the PMF Diversion Berm. Figure 3 presents the PMF water surface profile at the Access Road from Drainage Basin A. It illustrates that, although the flood waters pass over the top of the Access Road approximately 5,000 ft east of the site and floods the area to the northeast of the site, the surface of the flood is much lower in the vicinity of the PFSF.

Figure 8 (SWEC, 1999d) illustrates that in the northeast corner of the pad emplacement area, the maximum flood water level is Elevation 4,456.74 ft. The tops of the closest (and lowest) cask storage pads in this area are at Elevation 4,463 ft. Thus, the elevation of the pads at this point is greater than 6 ft above the PMF elevation. This is also illustrated in Figure 5 of the calculation, which presents a cross-section view of the PMF water level near the northeast corner of the PFSF site. Figure 8 also illustrates that, at the entrance of the Access Road to the PFSF, which is east of the Canister Transfer Building, the maximum elevation of the PMF is only 4,466.39 ft. The final grade in this area of the PFSF is Elevation 4,475, which is also the elevation of the tops of the closest cask storage pads. Thus, the elevations of the closest pads are greater than 8 ft above the PMF elevation in this area of the PFSF. Therefore, the site will not be inundated by the maximum water levels due to the PMF occurring within Drainage Basin A.

Figures 6 and 7 of the calculation present similar information for the PMF occurring within Drainage Basin B, which impacts the railroad west of the PFSF. The boundary of the PMF in Basin B is also shown in plan view on Figure 8. As indicated in these figures, the maximum elevation of the PMF near the PFSF site at the railroad crossing is 4,478.22 ft, which is higher than the tops of the cask storage pads in the south and west corner of the pad emplacement area. However, this floodwater is precluded from reaching the pad emplacement area by the PMF Diversion Berm, which has a top elevation of 4,480 ft. Where the PMF Diversion Berm ends near the middle of the

western edge of the pad emplacement area (Figure 2.6-2, Sheet 1), the PMF water level is only as high as Elevation 4,464.83 ft. The tops of the cask storage pads east of that area are at Elevation 4,469 (shown in Pad Emplacement Area Foundation Profile 7-7', Sheet 9 of Figure 2.6-5), which is greater than 4 ft above the PMF elevation. Therefore, the site will not be inundated by the maximum water levels due to the PMF occurring within Drainage Basin B.

Stormwater runoff from Basin A and Basin B drains as a sheet flow toward the PFSF site. An earthen berm (PMF berm) and drainage ditch system will be constructed on the south and west sides of the PFSF storage site to divert the PMF stormwater flows around the site to the east floodway and the west floodway. Consequently, all Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from the sheet flows associated with the PMF from Basin A and Basin B by the PMF berm.

The PFSF site drainage systems (both offsite and onsite) are designed for the 100-yr storm event. Offsite drainage system design due to Hickman Knolls runoff is conveyed around the south and west sides of the PFSF. This flow is then discharged at a permissible velocity to the Skull Valley natural drainage system. Flows resulting from a storm event more severe than a 100-yr event from Hickman Knolls are also diverted into the Skull Valley drainage system. Onsite drainage system design due to local runoff is conveyed by a surface flow system utilizing swales channeled to a stormwater collection and detention basin where it can evaporate and seep into the soil.

The PFSF access road and the rail road drainage systems are designed to safely convey the surface water under the roadway during a 100-yr storm event. During a PMP, the excess runoff will overtop the access road embankment and the rail road embankments. The flood overflow will be contained with a north-south berm tied into Hickman Knolls to

prevent flows from approaching the PFSF site. Downstream of the access road and the rail road, the PMF returns to the natural flow conditions. Access to the site by normal vehicular traffic, as well as emergency vehicles, will be provided at all times except during a storm which is more severe than a 100-yr storm event.

2.4.2.3 Effects of Local Intense Precipitation

The PMP was estimated based on HMR 49. HMR 49 requires different evaluations for combinations of storm events. Although the PMF generated by a general storm is a major concern in the evaluation of maximum flow, the local thunder storm in the summer time, having the greatest potential rain fall intensity and short duration may produce a larger PMF. Consequently, the PMP for both general storm and local storm must be analyzed. The PMP events are used with the HEC-1 program (U.S. Army Corps of Engineers, 1990) to determine peak discharges. After these evaluations are completed, the largest peak discharge is selected as the PMF. The water surface profile corresponding to the PMF is determined with the HEC-RAS (U.S. Army Corps of Engineers, 1997) backwater program.

According to the procedures given in HMR 49, the total precipitation for the general PMP were estimated to be approximately 10.7 and 10.5 inches during a 24-hour duration for the month of August (SWEC, 1999a, 1999c) at the Basin A and Basin B, respectively.

The total precipitation during the local PMP were estimated to be approximately 14.1 and 12.7 inches during a 6-hr duration at the Basin A and Basin B, respectively.

The 1-hr local PMP induced by a short duration thunderstorm over a 1 square mile basin at the site was determined to be 10.1 and 9.8 inches at the Basin A and Basin B,

respectively. Durational variation and areal reduction described in HMR 49 were employed to estimate the incremental PMP.

The total duration of the PMP was divided into six increments. The sequence position of hourly incremental PMP for the 6-hr thunderstorm was arranged in accordance with HMR No. 5 (U.S. Weather Bureau, 1947) as shown in the following table:

Increment	HMR No. 5 Sequence Position
Largest hourly amount	Third
2nd largest	Fourth
3rd largest	Second
4th largest	Fifth
5th largest	First
least	Last

The local PMP depths at 15-minute intervals for the first hour were determined by the procedures provided in HMR 49. The remaining 15-minute intervals were estimated from the depth-duration curve as documented in the calculation (SWEC, 1999a, 1999c). The sequence of the four 15-minute incremental PMP has the greatest intensity in the first 15-minute interval. The second, the third, and the fourth largest were placed after.

The general PMP depths at 1st, 6th, 12th, 18th, 24th, 48th, and 72nd hours were determined by HMR 49. The remaining 3-hr increments can be found from the depth-duration relationship. Time distribution procedures given by the Corps of Engineers (U.S. Army, 1952) were used for all other storms except the local PMP. The total duration of the storm was divided into four increments. The increments were arranged

in a sequence with the largest one at the middle and decreased progressively to either side of the greatest increment.

The HEC-1 computer program (US Army, COE, 1990) was used to determine the peak discharge. Soil type and its corresponding hydrological group, runoff curve number (CN), were estimated based on USDA's unpublished county soil report. The Utah Division of Water Resources suggested CN=70 for the type of soil surface condition in the Skull Valley. Consequently, CN=70 was assumed in the calculation.

The Kirpich formula (Chow, 1964) was used to estimate the time of concentration (Tc) for the overland flow on grassed surface. Based on the measured watershed length and the slope of the basin, the time of concentration at Basin A was calculated to be 11.24 hours. For Basin B, the time of concentration was estimated to be 4.17 hours, by using the regression equation developed by U.S. Corps of Engineer ("Time of Concentration vs Drainage Area for Mountain Watershed in Utah", no date, provided by Water Resource Division, State of Utah).

Based on the HEC-1 ,model computer output, the results of the PMF are summarized in the following:

	Basin A	Basin B
$Q_{Local\ PMF}$ =	40,237 cfs	68,500 cfs
$Q_{General\ PMF}$=	52,983 cfs	20,972 cfs

The larger peak discharge is selected as the PMF for the basin:

Q_{PMF}=	53,000 cfs	68,500 cfs

An extremely conservative case that is unlikely to happen in the Skull Valley drainage basin was also analyzed. In Basin A, assuming CN = 96 and Tc = 11 hours, the calculated PMF = 85,000 cfs. In Basin B, assuming CN=96, and Tc = 4.17 hours, the calculated PMF = 102,000 cfs. A CN = 96 is equivalent to an impervious surface or a saturated ground condition.

For the design of the access road and the railroad spur, the 100-yr floods for Basin A and B were estimated (SWEC, 1999a, 1999c) from the FHWA method (FHWA, 1977) and USGS method (USGS, 1994). The results of the 100-yr flood are summarized in the following

	Basin A	Basin B
$Q_{100\ (FHWA)}$ =	2,430 cfs	1,391 cfs
$Q_{100\ (USGS)}$ =	2,317 cfs	936 cfs

The larger peak flow is selected as the 100-yr flood for the basin

Q_{100} =	2,430 cfs	1,400 cfs

As described above, the extremely conservative PMF flows were calculated to be 85,000 cfs and 102,000 cfs for Basin A and Basin B, respectively. The PMF would most likely flow to the north along the east and west fringe of the PFSF site to Great Salt Lake. The peak discharges of 85,000 cfs and 102,000 cfs were used for backwater computation. The computer program HEC-RAS (US Army, COE, 1997) was used to compute Basin A and Basin B maximum flood profile elevations at various cross section and flood plains at the vicinity of the site (SWEC, 1999a, 1999c, 1999d).

For Basin A, with existing natural topography the predicted maximum water elevation is 4,501.3 ft at the access road (SWEC, 1999a). The predicted flood levels at the southeast and northeast corners of the site are approximately 4,468.8 ft and 4,456.7 ft, respectively. The site elevation of 4,475 – 4,463 ft is above the calculated water elevations. Consequently, the site is not in Basin A PMF flood plain.

In addition, the PMF flood levels for Basin A and B after the construction of the access road and rail line embankment were calculated. The predicted maximum water elevation at the access road and rail road is 4,506.4 and 4,478.1 ft, respectively.

All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from the sheet flow associated with Basin A and B PMF by an earthen berm to be built with top elevation at 4,507.5 and 4,480.0 ft, respectively.

2.4.3 Potential Maximum Flood on Streams and Rivers

Since there are no perennial streams or rivers in the vicinity of the PFSF, a PMF analysis is not required on streams and rivers. However, an ephemeral stream bed is present; therefore, PMF analyses were performed for this ephemeral stream as described above in Section 2.4.2.3.

2.4.4 Potential Dam Failures (Seismically Induced)

There are no flow control structures on any stream upgradient from the site; therefore, there is no potential for impact on the site from potential dam failures.

2.4.5 Probable Maximum Surge and Seiche Flooding

No surge or seiche flooding is possible, as there is no large water body near the site.

2.4.6 Probable Maximum Tsunami Flooding

The site is not located near a coastal area. As a result no tsunami sea waves are anticipated.

2.4.7 Ice Flooding

There are no water bodies near the site on which ice flooding conditions could arise.

2.4.8 Flooding Protection Requirements

All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from flooding by diversion berms to deflect potential flows generated by PMF from both the east mountain range (Basin A) and the west mountain range (Basin B) watersheds.

2.4.9 Environmental Acceptance of Effluents

With the exception of the sanitary system, there are no liquid releases that result from the normal operation of the PFSF.

THIS PAGE INTENTIONALLY LEFT BLANK

2.5 SUBSURFACE HYDROLOGY

2.5.1 Regional Characteristics

Skull Valley is a north-trending valley extending 50 miles from Lookout Pass in the Onaqui Mountains, to the southwest shore of the Great Salt Lake. It is one of many linear valleys of the Basin and Range bordered by relatively young fault-block mountains. These blocks are composed mainly of limestone and dolomite with a few beds of quartzite, sandstone, and shale, ranging in age from Early Cambrian to Tertiary. Primary permeability of these rocks is generally low; secondary permeability exists as joints, fractures, faults, and bedding plane separations.

A large portion of the precipitation that falls in the uplands runs off the steep hillsides as spring snowmelt in short, high-gradient streams, with little infiltration into the mountain blocks. Another portion drains eastward, becoming part of the hydrologic system of the adjacent Tooele and Rush valleys while some is discharged as springs in the foothills along the edges of the valley.

Another portion enters the valley-fill aquifers through an extensive recharge area consisting of alluvial fans at the base of the ranges. Hood and Waddell (1968) estimated the long-term average annual runoff from the uplands is about 32,000 acre-feet with only a small part of this actually flowing out of Skull Valley. They estimated the average annual groundwater discharge and recharge is between 30,000 to 50,000 acre-feet with evapotranspiration accounting for 80 to 90 percent of the total discharge.

The valley-fill deposits are unconsolidated and semi-consolidated rocks of Tertiary and Quaternary age. They consist of inter-stratified colluvium, alluvium, lacustrine, and fluvial deposits with minor basalt and ash, and some eolian material. These sediments

are derived almost entirely from the surrounding uplands and constitute the main groundwater reservoir.

In general, the coarser deposits are near the perimeter of the valley, grading into well-sorted sand and gravel, and interlayered with lacustrine silt and clay towards the center of the valley. Thick beds of clay exist in some areas and may create local, confined aquifers where they interfinger with sand and gravel along the alluvial fans.

The Salt Lake Group of Tertiary age comprises the majority of the valley fill ranging in thickness from 2,000 ft to over 6,000 ft (Arabasz et al., 1987). The overlying Quaternary rocks are mainly silt and clay, Pleistocene lake sediments, and reworked lake sediments. Sack (1993) has recently mapped and described the various Quaternary and Holocene surficial deposits in Skull Valley.

The Tertiary and older Quaternary deposits are slightly to highly permeable, depending upon grain size and degree of cementation. The deeper, more consolidated deposits contain some volcanic deposits that may reduce the permeability. The Tertiary and Quaternary deposits probably contain most of the groundwater of usable quality in storage in this part of Utah.

The younger Quaternary and Holocene sediments in the valley bottom have generally low permeability except for areas of windblown sand, and old beach and bar deposits. Precipitation on or surface runoff to the valley bottom remains ponded until it evaporates. The precipitation that is absorbed does not reach the water table in the southern and central parts of the valley because of the depth of the water table, the low permeability of the soil materials, and the low amount of precipitation. Most of this water is captured by plants and transpired; a small portion evaporates directly through

capillary action and contributes to the development of a high alkali content in the surface soils.

Groundwater flow is generally northward toward the Great Salt Lake. Hood and Waddell (1968) calculated that with a transmissivity of 2,675 sq ft/day, the annual volume of underflow out of the valley is about 800 acre-feet per year. Pumpage from wells for all purposes was estimated at 5,000 acre-feet per year in Skull Valley and is not believed to have changed significantly in the last 30 years.

Domestic water wells are developed almost exclusively in the unconsolidated alluvial fan deposits along the east side of Skull Valley. This same area serves as the main recharge area for the valley. Water quality is also the highest in this area. Discrete sand and gravel lenses are sufficiently interconnected so that water moves from bed to bed as a single hydrologic unit. Groundwater is commonly between 110 and 160 ft below ground in this area.

Farther out in the valley where lake clays have been deposited between granular layers, some degree of confinement occurs and, as a result, many irrigation and stock wells are under artesian conditions. These wells are commonly drilled to depths between 250 to 500 ft but maintain static water depth of 100 ft or less. Figure 2.5-1 indicates the location of all water wells within 5 miles (8km) of the PFSF. Some well records indicate artesian flow at the ground surface from wells just south of the Reservation. This information dates from the 1940's to 1960's (Arabasz et al., Appendix F, 1987).

Groundwater quality varies significantly in Skull Valley, dependent mainly on proximity to the bordering ranges. The alluvial apron along the base of the Stansbury Mountains contains the lowest total dissolved solids (TDS) in the valley, with concentrations from

100 to 800 mg/l. In the southernmost part of the valley, TDS concentrations range from 700 to about 900 mg/l with a few isolated wells above 1,000 mg/l TDS. A well south of the Reservation yielded a TDS concentration of greater than 2,500 mg/l (Arabasz et al., Appendix F, 1987). Sodium and chloride are the major ions found in these waters.

Toward the center part of the valley, away from the alluvial apron, unconsolidated lacustrine materials are interstratified with clastic material. Wells in this area tend to have lower yields and poorer quality water (TDS > 1,000 mg/l) and are used mainly for irrigation and stock watering. The north end of the valley has generally high TDS concentrations, in the range of 1,600 to 7,900 mg/l with sodium and chloride again being the main constituents (Arabasz et al., Appendix F, 1987).

2.5.2 Site Characteristics

Based on boring data obtained at the site, the uppermost soil layer consists of interbedded silt, silty clay, and clayey silt with a thickness of approximately 30 ft. This layer is underlain by very dense fine sand and silt that extends to a depth of approximately 45 ft. At this depth a carbonate soil (the Promontory soil) formed in subaerially-deposited alluvium was encountered in numerous borings across the site. Quaternary deposits are believed to extend to a depth of about 85 ft where Tertiary Salt Lake Formation was encountered in several borings. The groundwater table was encountered in borings at a depth of 125 feet. Hood and Waddell (1968) indicate that ground water flows from the south to the north in Skull Valley, toward Great Salt Lake. Based on their Plate 1, the hydraulic gradient is estimated to be approximately 9.5×10^{-4}.

Soil interpretations prepared by USDA (undated) indicate that the permeability of a silt soil in Skull Valley ranges from 0.2 to 0.6 inches/hr. The average groundwater rate of flow was estimated to be approximately 2.8×10^{-3} to 8.5×10^{-3} gallons/day/sq ft. The source of groundwater flow at the site is mainly derived from precipitation that falls at the higher elevations of the Stansbury Mountains. As a result of the low permeability deposits and high evapotranspiration at the site, rainfall at the site is unlikely to contribute to groundwater flow.

Water needs during construction (10,000 gallons/day) and operation (1,800 gallons/day) of the PFSF are modest. During operation, it will be similar to a light industrial facility with a 24-hour a day contingent of security personnel. Highest water demand is associated with the larger daytime work-force as well as operation of a concrete batching plant during initial construction. It is anticipated that surface storage tanks would be erected for potable water, emergency fire water, and for the concrete batching plant, as it is unlikely that water wells drilled into the main valley aquifer would yield adequate quantities of water for these purposes on demand. Several wells on the site may be required to meet the demand. Localized drawdown of the valley aquifer would occur in the vicinity of the wells, the extent of which can not be determined until the wells are drilled, developed, and pump-tested, but which would not extend beyond the site boundary.

Based on initial testing of the onsite monitoring well, it has been determined that operation of the PFSF water well will have no measurable off-site effects on existing groundwater quality or levels (SWEC, 1999b).

2.5.3 Contaminant Transport Analysis

The nature and form of the material stored (spent fuel assemblies in sealed metal canisters) and the method of storage (dry casks) preclude the possibility of a liquid contaminant spill. Discussion of potential contamination of groundwater is not applicable since the depth to groundwater at the site is substantially removed from any activity at the site finished grade.

2.6 GEOLOGY AND SEISMOLOGY

2.6.1 Basic Geologic and Seismic Information

Geological and seismological investigations for the PFSF site and area began in 1996 and included review of pertinent published and unpublished literature, consultation with geologists and seismologists familiar with the area, and reconnaissance level geologic mapping. In addition, a test boring program was conducted at the site at that time and along the 2.5-mile-long access road to characterize subsurface soil conditions. The drilling was performed by Earthcore, Inc. of Salt Lake City under the direct supervision of Stone & Webster Engineering Corporation (SWEC). Laboratory testing of soil for engineering properties was performed in SWEC's Soil Testing Laboratory in Boston, Massachusetts. Seismic reflection and refraction (both P- and S-wave) surveys were completed in 1996 in order to determine soil and rock stratigraphy, bedrock surface profiles, depth to the water table, seismic velocities of the underlying rock and soil, and engineering parameters of near-surface soils from shear wave velocities. This work was performed by Geosphere Midwest of Brooklyn Park, Minnesota, also under the direct supervision of Stone & Webster.

Professor Donald Currey of the University of Utah completed an evaluation of surficial linear features near the site at Stone & Webster's request. Drs. William Nash and Michael Perkins of the University of Utah analyzed volcanic ash samples from site borings and trench excavations for age correlation purposes.

Additional work was performed in 1998 in response to NRC Requests for Additional Information (RAI). This work included review of existing literature and data, and discussions with researchers knowledgeable with the structural and stratigraphic setting of Skull Valley. In addition, an extensive program of surface and subsurface

investigations was conducted under the direction of Geomatrix Consultants, Inc. and Stone and Webster. Their investigations included the following activities and are described in detail in Geomatrix Consultants, Inc. (2001a):

- photogeologic interpretation of aerial photos
- surficial and bedrock geologic mapping
- geomorphic mapping and interpretation
- geophysical investigations including interpretation of gravity data, conducting a magnetometer survey, reprocessing and interpretation of proprietary P-wave reflection data, and completing a high-resolution shear wave reflection survey (Bay Geophysical Associates, 1999).
- Trenching and test pit excavation and mapping
- Drilling and logging of boreholes
- Geochronologic analysis

As a result of these investigations and in response to recent changes to 10CFR50, Part 100 (100.23), as well as anticipated changes to Part 72 (SECY-98-126), probabilistic analyses were performed to evaluate the potential for fault displacement and ground shaking hazards at the PFSF. The design basis for vibratory ground motion was assessed at this time. The results of this effort are reported in Geomatrix Consultants, Inc. (2001a and b).

Stone and Webster also conducted a drilling program in 1998 in the area of the Canister Transfer Building (CTB) for engineering purposes. One of these borings reached a total depth of 225 ft. and a groundwater monitoring well was installed. Additional geotechnical investigations included cone penetrations tests, which were performed in April 1999, and test pits, which were excavated in the pad emplacement area in January 2001. Refer to Section 2.6.1.5 for additional details concerning these geotechnical investigations.

The site is situated in western Utah near the eastern boundary of the Basin and Range Physiographic Province with the Middle Rocky Mountain Province (Figure 2.6-1). This

area is characterized by a series of roughly north-south trending, tilted fault block ranges separated by down-faulted linear basins. The PFSF is located near the middle of the Skull Valley basin, at approximate elevation 4,465 ft, between the Stansbury Mountain range on the east and the Cedar Mountains on the west. The top 25 to 35 ft of surficial soils at the site are mainly lacustrine marly silts and clays. Below about 25 to 35 ft is a very dense fine sand with minor gravel and silt layers to at least 100 ft deep (Appendix 2A). The base of the late Pleistocene Bonneville alloformation is believed to be at a depth of about 45 ft. in the site area where the Promontory Soil was identified (Geomatrix Consultants, Inc., 2001a) and the soil blow-counts increase dramatically (Appendix 2A). The base of the Quaternary section is not well-constrained but the Tertiary "Walcott ash" is known in several borings at a depth of about 85 ft.

Bedrock was not encountered in the borings but is believed to occur at a depth of between 520 and 880 ft, based on seismic survey results (Appendix 2B). Bedrock outcroppings, about 1.5 miles south of the site at Hickman Knolls, have been mapped as the Fish Haven Dolomite of Late Ordovician age (Moore and Sorensen, 1979; Geomatrix Consultants, Inc., 2001a). As indicated in Section 2.4.1.2, the groundwater table is about 125 ft below grade at the site, based on the observation well and seismic refraction surveys.

The Stansbury fault, exposed along the base of the western escarpment of the Stansbury Mountains, is about 6 miles east of the site. The fault dips to the west and is projected beneath the PFSF site at a depth of 4.4 miles (55 degree dip assumed). The most recent events on the Stansbury fault displace late Pleistocene shorelines that are estimated to be about 18,000 years old. Arabasz et al. (1987) consider the Stansbury fault capable of a maximum magnitude 7.3 earthquake. Empirical relations by Wells and Coppersmith (1994) that relate surface rupture length to magnitude suggest the maximum earthquake magnitude on the Stansbury fault is 7.0 ± 0.28 (moment mag.). Helm (1995) has calculated that the next seismic event on the fault should be a Ms 6.8-6.9 ± 0.04 , based

on strain accumulation rates of previous events. Geomatrix Consultants, Inc. (2001a) calculate an expected value (mean) of **M** 7.0 for the maximum magnitude on the Stansbury fault.
Two unnamed faults were identified in the PFSF area and are informally named the East and West faults (Geomatrix Consultants Inc., 2001a). Late Pleistocene activity is indicated for both of these faults, based on geophysical and geomorphological studies. The East fault lies 0.9 km east of the site and the West is 2 km to the west. Mean maximum magnitudes for the East and West faults were calculated to be **M** 6.5 and **M** 6.4, respectively. The Stansbury, East, and West faults are the most important structures with respect to the assessment of seismic hazard in the PFSF vicinity. A transition zone or zone of distributive fault offset between the East and West faults was identified. The potential for surface fault displacement beneath the PFSF is evaluated in Geomatrix Consultants, Inc. (2001a).

The maximum "random" earthquake for this region has been defined by Pechmann and Arabasz (1995) as $M_L = 6.5$.

A probabilistic seismic hazard analysis was performed (Geomatrix Consultants, Inc., 2001a) to assess vibratory ground motion and fault displacement at the PFSF site. Peak ground accelerations for the design basis earthquake (2,000-yr return period) were calculated to be 0.711g horizontal and 0.695g vertical (Geomatrix Consultants, Inc., 2001b). Ground surface displacements associated with faults believed to exist beneath the site were determined to be less than 0.1 cm for the same return period.

The closest Quaternary igneous activity is 50 miles south of the PFSF site at Fumarole Butte on Crater Bench. Basaltic volcanic activity here is believed to be between 950,000 and 880,000 years old (Hecker, 1993) and does not present a threat to the integrity of the PFSF at that distance, if it were to become active.

2.6.1.1 Site Geomorphology

Site topography is shown on Figure 2.6-2. Topography in the site vicinity is shown on Figures 1.1-1 and 2.6-4. The PFSF site lies near the center of Skull Valley about midway between the Stansbury Mountains and the Cedar Mountains. Skull Valley is in a part of the Great Basin that was once occupied by Lake Bonneville, during the late Pleistocene (30,000 to 12,000 years B.P.). As the climate became warmer in the latest Pleistocene and outlets for the lake were abandoned, the lake shrank in size and the water became saline. The gently north-sloping floor of Skull Valley is the former bottom of the lake and the unconsolidated deposits at the site are sediments laid down in and by Lake Bonneville. About 2 miles east of the site, the valley bottom meets the toe of an alluvial apron built up from a series of coalescing alluvial fans along the base of the Stansbury Mountains. The apron slopes at about 200 ft/mile in the vicinity of the Skull Valley Indian Reservation village. A wave-cut bench or terrace can be seen near the head of the apron representing the maximum level of Lake Bonneville about 15,000 years B.P. at elevation 5,240 ft. Also present here is a scarp and small graben in Quaternary deposits reflecting Quaternary movement on the Stansbury fault (Barnhard and Dodge, 1988; Geomatrix Consultants, Inc, 2001a).

The apron is only slightly dissected by streams originating in the steep bedrock terrain of the Stansbury Mountains. Stream and spring flow is rapidly absorbed into the coarse granular fan deposits resulting in very little water reaching the valley bottom as surface runoff in this area.

The valley floor is relatively smooth, being interrupted in only a few locations by bedrock outcrops, such as Hickman Knolls rising about 400 ft above the valley bottom near the site. Relief on the valley bottom is slight consisting of a few shallow (1 to 3 ft) north-trending dry washes and low (1 to 3 ft) linear soil ridges. The washes are marked by

more dense desert shrub vegetation, whereas the ridges tend to be grass covered. The washes carry water for very short periods during spring snowmelt and infrequent, local thunderstorms. A few shallow depressions appear to pond water at times until it evaporates. This network of shallow washes eventually leads offsite to the north where it joins the central valley drainage system leading to the Great Salt Lake. Perennial surface water is found about 10 miles north of the site in a large mudflat fed mainly by springs along the toes of alluvial fans emanating from the Stansbury Mountains.

Other features recognized on the valley bottom near the site include beach ridges and shoreline deposits associated with Lake Bonneville and eolian dune deposits in various forms, mainly parabolic or shrub-coppice dunes (Sack, 1993). Sack (1993) interpreted linear and curvilinear tonal features from air photos as possible fault traces in soil near the site. Dr. Donald Currey of the University of Utah examined these features in the field and believes they are not of tectonic origin, but are beach ridges developed during the transgression of Lake Bonneville. His report on these features is included as Appendix 2C. Subsequent trenching and augering of these features have affirmed Currey's original interpretation (Geomatrix Consultants, Inc., 2001a).

There is no evidence of flash-flooding near the site area nor any deposits indicative of mudflows or recent landslides. The great depth to bedrock and the very dense condition of most subsurface soils preclude the development of collapse or uplift features associated with karst terrains or tectonic depressions. There is no history of mineral extraction or injection in the area and little likelihood of future development. Withdrawal of water in the area is widely scattered and consists of a few domestic supply wells, irrigation wells, and stock-watering wells. There is no potential for subsidence from water withdrawal because of the distance from these sources and the present depth to water at the site (greater than 100 ft).

In summary, the geomorphology of the site is typical of a semi-arid to arid desert setting. The adjacent ranges are affected by mass-wasting processes and stream erosion that deliver their load of sediments to a complex of alluvial fans at the edge of the ranges. Most of the sediment load is dropped here as the water infiltrates or evaporates. The central part of the valley is relatively unaffected by fluvial processes. Mechanical and chemical weathering of rock and soil proceeds very slowly in this flat dry environment. Essentially, the only geomorphic processes to affect the site are microprocesses wherein soil moisture from occasional precipitation is drawn upward by capillary action and evaporates near the ground surface. This results in a gradual buildup of calcium carbonate, alkali, and sulfate in the near-surface soils. Soils at the site are described in the County soil report (USDA, unpublished report) as being calcareous and saline.

2.6.1.2 Geologic History of Site and Region

2.6.1.2.1 Bedrock

The Skull Valley PFSF site lies above a sediment-filled, structural basin that is bounded on the east and west by uplifted range blocks, the Stansbury-Onaqui Mountains and the Cedar Mountains, respectively. This pattern is repeated throughout western Utah and Nevada and elsewhere and is so characteristic that the name Basin and Range is applied to the physiographic area containing this structural arrangement (Figure 2.6-1). The eastern border of this province is generally drawn along the north-south trending Wasatch Front about 55 miles east of the site. The western boundary of the Front is known to be a major, active normal fault, the Wasatch fault, along which the Front has been uplifted and the Salt Lake basin is down-dropped. This major structural element is believed to have persisted since at least Late Precambrian time. The Uinta arch, which includes the present Uinta Mountains east of the Wasatch Front, is an east-west trending, anticlinal structure with a similarly long history of uplift. It intersects the Wasatch line at

right angles and is believed to have influenced sedimentation patterns, as well as provided a stable buttress during tectonic episodes. Evidence of the Uinta arch has been traced as far west as central Nevada (Roberts et al., 1965) and is postulated to have affected sedimentation patterns in the rocks of the Stansbury Mountains and patterns of faulting and mineralization (Zoback, 1983; Helm, 1995; Stokes, 1986). The regional bedrock geology is depicted on Figure 2.6-3. A stratigraphic column for the Skull Valley area is shown on Figure 2.6-17. Structural geologic cross sections were drawn north and south of the PFSF site across Skull Valley. These profiles are included in Geomatrix Consultants, Inc. (2001a) as Figures 2-1 and 2-2 with their locations shown on Figure 1-1.

The Wasatch line may have its origin during the Late Precambrian breakup of the North American craton as the resulting rift margin. Clastic deposition off the craton margin eventually became carbonate shelf deposition as the shoreline migrated eastward. The site of this deposition is believed to be the Cordilleran geosyncline, receiving some of the greatest thicknesses of Precambrian and Paleozoic sediments found anywhere (Stewart, 1976). Shallow marine deposition persisted throughout much of the Paleozoic, except along the Uinta arch where periodic uplift caused erosion of previously deposited sediments or non-deposition. Orogenic events during the Mid to Late Paleozoic, such as the Antler orogeny, greatly affected the edge of the continent, then located in Nevada, and detrital sedimentation became predominant.

During the Triassic and Jurassic Periods, shallow marine deposition alternated with long episodes of subaerial erosion and deposition, mainly east of the Wasatch line. No sedimentary rocks from this period are known in the site vicinity (Moore and Sorensen, 1979). Orogenic events continued to affect the Cordilleran geosyncline, compressing and uplifting these sediments progressively from west to east.

A distinct orogenic event has been identified in western Utah, occurring during the Late Cretaceous. It is called the Sevier orogeny and is characterized by extensive tectonic shortening of the geosyncline and eastward folding and thrusting of sediments along numerous low angle thrust faults. Tooker (1983), Tooker and Roberts (1971), and Christie-Blick (1983) discuss evidence for these thrust sheets in several mountain ranges near the PFSF site, including the Stansbury, Cedar, Sheeprock, and Oquirrh Mountains. The Uinta arch and the Wasatch line are believed to have influenced the lateral extent and placement of some of these thrust sheets.

The Laramide orogeny closely followed the Sevier orogeny beginning in the Late Cretaceous and ending in early Eocene time. Its effects were felt mostly east of the Wasatch line; uplift of the Uinta Mountains by block faulting is one of the results of this event. The Laramide orogeny ended in the mid-Eocene with a major change in the regional stress regime. Extensional tectonics replaced compressional and were accompanied by widespread igneous activity. Normal faulting was the common expression of the extension.

The timing of development of basin-and-range tectonics varies considerably in the western U.S. and several theories of origin have been proposed. Stewart (1978) provides a summary of much of the evidence, based mainly on age relationships to volcanism associated with the extension of the crust. He concludes that extensional faulting of the Great Basin, of which the Skull Valley area is a part, probably started about 17 million years ago (m.y.a.). Moore and McKee (1983) conclude basin-and-range faulting is no older than 12 to 13 m.y. in the Stansbury Mountains, based on ages of basalt flows there. Evidence of Quaternary movement is indicated by the fault scarps offsetting Quaternary deposits along the west side of the Stansbury Mountains, as well as numerous other areas in the Great Basin. Seismicity along some of these faults, in situ stress

measurements, and earthquake focal plane solutions (summarized in Smith, 1978) indicate extensional activity continues today.

2.6.1.2.2 Site Area Structural Geology and Geologic History

The Stansbury Mountains are a north-trending range in the eastern Great Basin. They are about 30 miles long and 10 miles wide with a maximum elevation of 11,031 ft at Deseret Peak, just northeast of the PFSF. A smaller range, the Onaqui Mountains, is an extension of the Stansbury range to the south. A generalized profile of the range includes a steep, curvilinear western slope dissected by a series of young canyons. The eastern slope into Tooele and Rush Valleys is much less rugged, perhaps reflecting a typical tilted fault block origin.

The overall structure of the range is a doubly-plunging anticline uplifted along the steep westerly-dipping Stansbury fault (Tooker and Roberts, 1971). The stratigraphy and general geology of the Stansbury Mountains has been studied most completely by Rigby (1958). He divided the history of the range into two episodes of uplift and folding, each followed by long periods of erosion. These activities took place with the rocks at or near the present location of the range.

Tooker and Roberts (1971), Tooker (1983), and Roberts et al. (1965) reinterpreted much of Rigby's work to include a sequence of at least four thrust slices derived mainly from a site an unknown distance to the west. Most of the folding and faulting internal to the mountain block is believed to have occurred prior to or during the deformation that carried these rocks to their present location during the Late Cretaceous. Additional details of these older events and cross sections through the area are found in Geomatrix Consultants, Inc. (2001a), Section 2. Regardless of the provenance of the rocks in the Stansbury Mountains, it is evident the range itself is the result of normal faulting

associated with extensional tectonics developed since the beginning of the Miocene, about 24 m.y.a. This process continues today and is a major focus of seismic safety considerations for the PFSF, as well as other facilities in the region.

The Stansbury Mountains are but one of numerous mountain ranges in the Great Basin with similar origins and characteristics. The ranges are oriented roughly north-south, are commonly 9 to 12 miles wide, and are separated by valleys or basins filled with alluvium and colluvium derived from the ranges. The thickness of sediment in the valleys ranges from 1,000 ft to as much as 12,000 ft. Elevation of the ranges (and subsidence of adjacent basins) occurs by movement along major faults on one or both sides of the blocks. It is generally believed that the faulting is distributed along several range-front faults, many of which are buried beneath the valley-fill deposits. Many of the mountain blocks show significant tilt; in the eastern Great Basin, most blocks are tilted to the east (Stewart, 1978).

Latest movement is known to be Quaternary or younger on many of the range front faults. Offset of Quaternary sediments or Holocene alluvial fans is well documented in numerous studies, particularly along the Wasatch fault. The Stansbury fault has been considered to be active at least since the work of Rigby (1958). More recent analyses suggest the fault may be segmented with movement on the southern segment occurring less than 18,000 years B.P. (latest Pleistocene) (Helm, 1995; Geomatrix Consultants, Inc., 2001a). Detailed discussion of the Stansbury fault and the seismic implications are found in Section 2.6.2.3, and Sections 5 and 6 of Geomatrix Consultants, Inc. (2001a).

Other Tertiary normal faults in Skull Valley have been proposed by various authors (Cook et al., 1989; Helm, 1995; Zoback, 1983). Recent work for the PFSF has identified two additional west-dipping normal faults in the vicinity of the PFSF, based mainly on geophysical data and subtle geomorphic expression (Bay Geophysical

Associates, 1999; Geomatrix Consultants, Inc., 2001a). These faults are informally named the "East" and "West" faults and are discussed in detail in Geomatrix Consultants, Inc. (2001a), Sections 2, 5, and 6. As shown on the cross sections, Figures 2-1 and 2-2 in Geomatrix Consultants, Inc. (2001a), the East fault is interpreted to form the east margin of the Tertiary basin that underlies Skull Valley whereas the West fault lies within the basin, west of the PFSF location. The East and West faults are interpreted to merge together about 9 miles southeast of the site. The PFSF appears to be located in the stepover zone between the East and West faults where the slip is transferred from the East to the West fault. The west boundary of the Tertiary Skull Valley basin is believed to be the East Cedar Mountains fault.

Interpretation of the high resolution seismic reflection survey (Bay Geophysical Associates, 1999) performed across the PFSF site indicates the East fault displaces a subsurface reflector believed to be the unconformity at the base of the Bonneville alloformation on the Promontory soil. The Bonneville sediments are 30,000 years or younger. Therefore, the East fault is considered to be capable as defined in 10CFR50 Appendix A. The West fault is also considered to be capable based on apparent changes in elevation along the late Pleistocene Stansbury gravel bar, southwest of the PFSF. The evidence for the fault and an analysis of its slip rate are discussed in Section 2 and 5 of Geomatrix Consultants, Inc. (2001a).

The zone of distributed faulting, where slip is transferred from the East fault to the West fault, was also interpreted from the high resolution reflection survey. Small normal faults, both east-and west-dipping, were imaged; some were interpreted to offset the base of the Bonneville alloformation whereas others clearly do not (Bay Geophysical Associates, 1999). Displacement on individual faults within the zone of distributed faulting is small (Geomatrix Consultants, Inc., 2001a, Table 5-1). A drilling program conducted across this

zone confirmed the presence and nature of this zone of faulting, as shown on Figure 5-4 in Geomatrix Consultants, Inc. (2001a).

2.6.1.2.3 Surficial (Basin-fill deposits)

Unconsolidated and semi-consolidated rocks of Tertiary and Quaternary age are believed to underlie Skull Valley. The total thickness is unknown but has been inferred to be 6,000 to 7,000 ft near the Cedar Mountains on the west side of the valley (Johnson and Cook, 1957) to 6,000 to 8,000 ft near I-80 (Baer and Benson, 1987). These materials are believed to include complex interlayered conglomerates, alluvium, lake deposits, and a few volcanic ash deposits, some of which must date from the beginning of Basin and Range faulting (12 to 13 m.y.a.) (Moore and McKee, 1983). The oldest of these deposits are included as the Salt Lake Group of Miocene-Pliocene age. This formation outcrops in the southern part of Rush Valley and is in the subsurface of Tooele Valley, east of the Stansbury Mountains (Everitt and Kaliser, 1980). The Salt Lake Group is not mapped at the surface on the Quaternary geologic map of Sack (1993) (Figure 2.6-4). Davis Knolls, about 5 miles southeast of Dugway, is identified on the State Geologic Map (Hintze, 1980) as part of the Salt Lake Formation. Moore and Sorensen (1979) mapped this area as early Tertiary conglomerate, tuffaceous sandstone, and fresh-water limestone. Several borings at the PFSF site encountered volcanic ash at depths between 65 and 90 ft. An ash at a depth of 85 ft was correlated with the Walcott tuff, known to be 6.6 million years old (Appendix 2E).

Everitt and Kaliser (1980) believe there is an unconformity at the top of the Salt Lake Group prior to deposition of the largely unconsolidated sand, silt, gravel, and clay of Quaternary age (<2 m.y.). This appears to be the case in Skull Valley as evidenced by the distinct reflector at a depth of about 85 ft in the high resolution shear wave reflection survey, reflector Q/T from Bay Geophysical Associates (1999) and Geomatrix

Consultants, Inc. (2001a). Fossils are rare from these deposits; as a result, the boundary between the Tertiary and Quaternary is somewhat arbitrarily drawn on the bases of degree of consolidation, relative amount of volcanic material, and degree of deformation evident.

The surficial geology of Skull Valley is predominantly unconsolidated material of Quaternary age deposited in Lake Bonneville (~ 30,000 to 12,000 years B.P.). Pre-Lake Bonneville lacustrine deposits have been found in other valleys in the region indicating numerous lakes occupied the Salt Lake basin prior to Lake Bonneville. These deposits date from at least 600,000 B.P. to 30,000 B.P. (Lund et al., 1990). Pre-Lake Bonneville sediments were encountered in borings, test pits, and trenches in the site vicinity, as discussed in Geomatrix Consultants, Inc. (2001a).

Gilbert (cited in Sack, 1993) believed that the extensive pre-Bonneville alluvial fans were an indication of a long period of hot, dry climate prior to the transgression of Lake Bonneville. Most investigators believe that the Bonneville lake cycle began between 30,000 and 25,000 years B.P., coinciding with the final glacial maximum in the Rocky Mountains (Scott, 1988; Oviatt et al., 1992). Lake levels continued to rise until about 21,000 to 20,000 years B.P. when the level remained somewhat stable for an extended period of time. The Stansbury shoreline developed at this time and has been identified throughout the Bonneville Basin (Oviatt et al., 1990), near elevation 4,468 ft. Sack (1993) has mapped the Stansbury shoreline through the southern part of Section 6, T5S-R8W near the PFSF, based on aerial photographs (Figure 2.6-4). Geomatrix Consultants, Inc. (2001a) mapped numerous additional Stansbury shoreline features in the area as shown on their Figure 1-3.

Continued filling of the basin after 20,000 years B.P. caused the lake to rise to its maximum elevation of about 5,240 ft approximately 15,300 years B.P. At that time, an

outlet for the lake into the Snake River drainage was reached. The Bonneville shoreline was created at this time and can be seen as a bench on the alluvial fan east of the PFSF. At about 14,500 years B.P., unconsolidated deposits in the lake outlet channel were rapidly eroded. The lake dropped more than 300 ft in a matter of a few weeks, and the Bonneville flood resulted. The outlet stabilized at about elevation 4,740 ft, and the Provo level developed (Malde, 1968). Sack (1993) and Geomatrix Consultants, Inc. (2001a) have also mapped this shoreline east of the PFSF site on the alluvial fan (Figure 2.6-4). This shoreline also was identified at an elevation of approximately 4830 ft on Hickman Knolls by Geomatrix Consultants (2001a).

Climatic change beginning about 14,000 years B.P. caused the gradual shrinkage of Lake Bonneville to at least the lowest level of present Great Salt Lake by about 12,000 years B.P. (Currey, 1990). A brief transgression of the lake occurred between about 10,900 and 10,300 years B.P. to about elevation 4,250 ft (Currey, 1990). This level is known as the Gilbert level of the Great Salt Lake and has been mapped about 11 miles north of the PFSF site (Sack, 1993). Since that time the lake has receded and fluctuates within about 20 ft elevation of its historic average (Lund et al., 1990). Only once in the past 10,000 years has the level of the lake been as high as 4,220 ft (Atwood and Mabey, 1995). The PFSF site is at approximate elevation 4,450 ft, well above any probable historic level of the Great Salt Lake.

2.6.1.3 Site Geology

The site geology was investigated in 1996 by a subsurface drilling program totaling 24 borings to a maximum depth of 100 ft and a seismic refraction and reflection program. Logs of these borings are included in Appendix 2A and the results of the seismic surveys are found in Appendix 2B.

Additional investigations were undertaken in 1998 and included surficial and bedrock mapping, excavation and mapping of numerous test pits and trenches, drilling of more than 40 additional boreholes to a maximum depth of 225 ft, and completion of 6 kilometers of high-resolution seismic shear-wave reflection lines. A summary of these efforts is included below. Additional detail and discussion are found in the original reports of this work (Geomatrix Consultants, Inc., 2001a; Bay Geophysical Associates, 1999).

The PFSF site is situated near the center of Skull Valley where Quaternary lacustrine and geomorphic features dominate the topography. The stratigraphy beneath the site consists of approximately 500 to 800 ft of Quaternary and Tertiary basin fill overlying Paleozoic bedrock. The nature of the deepest Tertiary deposits is unknown at this time but is believed to include sediments of the Salt Lake Formation, mainly sand, silt, marl and tuff in varying states of consolidation. The Salt Lake Formation extends up to a depth of about 85 ft in the central part of the PFSF. A volcanic ash at that level has been correlated with the Walcott tuff, know to be late Miocene in age (approximately 6 m.y.; Appendix 2E). This boundary was also identified as a prominent reflector on the high-resolution shear wave profiles (Bay Geophysical Associates, 1999; Geomatrix Consultants, Inc., 2001a, Plate 4).

There is evidence for four major lake cycles in the Bonneville basin during the past 700,000 years (Machette and Scott, 1988; Oviatt et al., 1997). Evidence for the three oldest is not well preserved regionally and was found only sporadically in the PFSF vicinity. The most recent cycle, the Lake Bonneville cycle, occurred between about 30,000 and 12,000 years ago and is well documented in Skull Valley. Several transgressions and recessions of the lake occurred during this time, each leaving an identifiable characteristic in the geomorpholoy of the valley or in the stratigraphic record. This evidence is presented in detail in Geomatrix Consultants, Inc. (2001a, Section 3.2). Near-surface Pleistocene deposits at the PFSF consist mainly of fine sand, silt, clay and

marl. In general, the finer grained materials, such as silt, clay and marl were deposited during the deeper water portions of the lake cycle and the sand represents shallower, near-shore beach or deltaic fan environments. The engineering properties of those materials are discussed in Section 2.6.1.6. Locally, Holocene eolian and fluvial activities have reworked the surface soils to some extent (Sack, 1993). Eastward from the PFSF, along the proposed access road to Skull Valley Road, the influence of the proximity to the range-front alluvial fans is apparent as an increase in gravel content at shallow depths (Appendix 2A).

Bedrock is not exposed at the PFSF site but is found about 1.5 mile to the south at Hickman Knolls, and about 1.5 mile northeast in a series of low hills informally called Castle Rock Knoll. Hickman Knolls is inferred to consist of Fish Haven Dolomite of Ordovician age (Moore and Sorensen, 1979; Geomatrix Consultants, Inc.,2001a). At this location the formation is a medium to dark gray dolomite and limestone breccia (Figure 2.6-17). Bedding is massive to indistinct and breccia pebbles are angular to sub-round and appear to be the same composition as the enclosing matrix. Bedding strikes northerly to northeasterly and dips to the east at moderate to steep angles. Bedrock fracturing consists mainly of two sets of high angle fractures, one trends east-west and the other north-south. These fractures tend to coincide with more silicified zones that form prominent scarps on the Knolls that are strongly expressed in the morphology and are associated with many of the aerial-photo lineaments (See Plate 1, Geomatrix Consultants, Inc., 2001a).

Several faults and ductile shear zones were identified at Hickman Knolls during the recent investigations. Geomatrix Consultants, Inc. (2001a) presents evidence that indicates the faults developed prior to the dolomitization process and the shear zones are likely penecontemporaneous with the process of brecciation. No large, through-going faults are believed to exist on Hickman Knolls.

There has been some enlargement of a few joints resulting from dissolution and a few small caves or openings (1 to 4 ft deep) can be seen on some of the steeper rock faces. Karst conditions do not exist at Hickman Knolls nor are they likely to develop because of the near-desert environment and the depth to groundwater (greater than 100 ft). The outcrop mapped northeast of the PFSF site (Castle Rock Knoll) has been identified as Deseret Limestone of Mississippian age (Moore and Sorensen, 1979).

Faults in the PFSF vicinity important to the understanding of site geology include the East, West, "F", and smaller faults in the "zone of distributed faulting." These faults are discussed briefly in Section 2.6.1.2.2 and in detail in Geomatrix Consultants, Inc. (2001a), Sections 2, 5, and 6.

2.6.1.4 Geologic Map of Site Area

Figure 2.6-4 is the geologic map of the PFSF site, reproduced from Sack (1993). Areas of bedrock outcrop are indicated in addition to the surficial deposits. Figure 1-3 in Geomatrix Consultants, Inc. (2001a) is a photo-geologic map of the site vicinity showing additional details and interpretations of geologic features. Scarps in soil near the site identified on Sack's map have been investigated by Dr. Donald Currey for this project (Appendix 2C). Both Currey and Geomatrix Consultants, Inc. (2001a) concluded the features were related to lacustrine processes of Lake Bonneville and are not of tectonic origin.

2.6.1.5 Facility Plot Plan and Geologic Investigations

Figure 2.6-2 is a plot plan showing the locations of the major structures of the PFSF, the locations of the 1996 geotechnical borings and geophysical survey lines, and the location of foundation profiles through the pad emplacement area. Plate 1 of Geomatrix Consultants, Inc. (2001a), indicates the locations of both the 1996 and 1998 investigations, exclusive of the geotechnical borings for the Canister Transfer Building. Figure 2.6-18 shows the locations of the geotechnical borings that were drilled in the vicinity of the Canister Transfer Building in 1998, along with the locations of the CTB foundation profiles.

Geotechnical boring programs were conducted in 1996 and 1998. The borings drilled in October 1996 were located in the pad emplacement area and along the access road corridor, as shown in Figure 2.6-2. The borings drilled in October and December of 1998 were located in the Canister Transfer Building area, as shown in Figure 2.6-18. The soil samples obtained from these borings were sent to the Stone & Webster Geotechnical Laboratory in Boston for testing. The results of the boring programs and laboratory testing are found in Appendix 2A.

In April 1999, ConeTec, Inc performed cone penetration tests (CPT) and dilatometer tests (DMT) in the pad emplacement area and the Canister Transfer Building area. The locations of these CPTs and DMTs are presented in Figure 2.6-19. The results from this subsurface investigation are presented in ConeTec (1999). The primary goal of this investigation was to develop profiles of the relative strength and compressibility of the soils within the depth interval of 10 ft to ~25 ft in the pad emplacement area. As stated in ConeTec (1999), the other interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design.

This program included performing 37 cone penetration tests (CPT) to develop continuous profiles of the strength of the soils in the upper layer (from the surface down to ~25 ft) within the pad emplacement area and 2 under the CTB. Sixteen of these were performed using a seismic CPT to measure down-hole P and S-wave velocities, and the two CPTs performed in the CTB included resistivity measurements. The cone penetration testing program also included performing dilatometer tests (DMT) to develop profiles of the variation of compressibility of the in situ soils. These were located, primarily, in areas where the tip resistance profiles from the CPT tests indicated that the in situ soils had the lowest strengths and the highest compressibilities.

Phase 1 of this program included performing 36 CPTs, located on a grid pattern of ~300 ft within the entire pad emplacement area. This layout provided nine CPTs in each of the four quadrants of the pad emplacement area. Several of these CPTs were located in close proximity to the borings that were drilled previously at the site, permitting correlations between the previous boring and laboratory data to be utilized in the interpretation of the CPT and DMT data. Additional CPTs and DMTs were performed in the vicinity of Borings CTB-4, CTB-5(OW), and C-1, to obtain data for correlating the CPT data with the laboratory testing that was performed on samples from these borings, as well.

The results of the Phase 1 CPTs included measuring continuous profiles of tip resistance and sleeve friction stress, which were used to identify the extent and thickness of the lower blow count soils within the upper layer. The plots of corrected tip resistance, Q_t, vs depth, presented in Appendix E of ConeTec (1999), document the relative strength and compressibility of the soils within the profile. The results are consistent with the results of the borings that were drilled previously at the site; i.e., Q_t increases from grade to a depth of about 15 to 17 ft. Below this depth, it drops slightly

or remains constant with depth, down to a depth of about 23 ft, at which point it increases markedly, as did the Standard Penetration Test blow counts in most of the borings in the pad emplacement area.

These data were interpreted to provide profiles of the variation of strength, which are plotted in Appendix D and listed in tabular form in Appendix F of ConeTec (1999). A review of the plots of the undrained shear strength, s_u, vs depth indicates that s_u measured in the CPTs increases with depth, and it generally exceeds 1 tsf. Note, this value corresponds with the lower bound of the values of s_u measured in the CU and UU tests. These plots indicate that s_u remains fairly constant in the depth range from ~15 ft to ~23 ft, and normally exceeds 2 tsf. Therefore, the lower blow count zone at approximately 20 ft has undrained shear strengths that are at least twice those used in the analyses of the stability of the cask storage pads.

In Phase 2 of the cone penetration testing program, dilatometer tests were performed to measure, in situ, the variation of compressibility of the soils vs depth at the locations identified in Phase 1 where the softer soils exist. The compressibility is reported as the constrained modulus, M, in the plots and tables included in Appendices G and H of ConeTec (1999).

The plots of M vs depth in Appendix G of ConeTec (1999) show that M generally is lowest near the surface of the site, increases with increasing depth to about 4 to 5m (13 to 16 ft), at which point it decreases, generally remaining fairly constant at a value that is equal to or greater than that near the top of the profile. This trend is evident on the plots of DMT-1, 2, 3, 4, 8, 9 (excluding the high modulus values above 2.5m), 11, 12, 14, 15, 16, 17, and 18. Although DMT-6 found a slight decrease in M from ~5.5m to 7m, the resulting values were higher than in the other DMTs in this depth range. DMT-

5, 7, and 13 show only slight increases in M with depth to ~4 to 5m, followed by slight drop in modulus to ~7.5 to 8m.

In general, DMT-10 had the lowest constrained modulus (i.e., highest compressibility) for the entire profile. DMT-10 is anomalous in that M remains fairly constant throughout the entire depth range of ~2m to 7.8m, with a minimum value of 130 bars (135.7 tsf). This DMT was located about half-way between Borings B-1 and C-1 at the northern edge of the pad emplacement area. Note, the consolidation tests reported in Attachment 2 of Appendix 2A were performed on samples obtained at a depth of 10 ft in Boring C-1 and C-2, which were near this location.

See Section 2.6.1.12.1 for discussion of incorporation of these CPT results in the bearing capacity and settlement analyses.

In January 2001, 16 test pits were excavated in the pad emplacement area to obtain bulk soil samples for tests required for design of the soil cement that will be constructed in the pad emplacement area and in the area surrounding the Canister Transfer Building. These test pits were excavated using a backhoe at the locations shown in Figure 2.6-19 to depths of approximately 6 ft. Logs of these test pits and the results of laboratory tests performed on these samples are included in Attachment 9 of Appendix 2A. The sample descriptions presented on these logs are based on visual observations made during excavation, as well as particle-size analyses and Atterberg limits test results. Solid horizontal lines are used on the test pit logs to designate the locations of the top of ground and the bottom of the test pits, as well as interfaces observed in the field between strata. The dashed horizontal lines indicate strata changes based on the results of the laboratory tests.

Geophysical surveys were conducted at the site and are discussed in Section 2.6.1.10.

2.6.1.6 Relationship of Major Foundations to Subsurface Materials

Figure 2.6-5, Sheets 1 through 14, present foundation profiles in the pad emplacement area, showing the locations of the proposed structures in relationship to the subsurface materials encountered in the borings. Based on the borings and laboratory test data, the generalized subsurface profile consists of three layers. The uppermost layer extends to a depth of between 25 and 35 ft below existing grade and is mainly interlayered silt, silty clay, and clayey silt. Standard Penetration Test (SPT) N-values for this layer vary between 1 and 20 blows per ft. The average N-value of 16 blows per ft and the median N-value of 14 blows per ft indicate that these are "stiff" or "medium dense" materials. The casks and the Canister Transfer Building will be placed on mat foundations, and the other proposed structures will be constructed on strip and spread footings founded on this layer.

Excluding the surficial samples, which need not be considered because they will be mixed with cement to form soil cement, only a few samples obtained from the pad emplacement area have N-values of less than 8 blows/ft. These were all obtained from the depth range of 5 to 7 ft, and they include Samples S-2 of the following:

Samples Obtained Within Pad Emplacement Area That Have SPT N-values Less Than 8 Blows/ft
(Excluding Surficial Samples, Which Will Be Replaced by Soil Cement)

Boring	N-Value (Blows/ft)	LL (Liquid limit)	PI (Plasticity Index)
B-2	5	47.4	21.8
D-2	6	46.4	15.3
C-3	6	43.1	20.7
C-4	7	69.5	25.3
D-4	4	49.3	21.6

As indicated by the index property tests presented in Figure 2.6-20, the soils obtained from this depth interval (5 to 7 ft) typically are clayey silt/silty clay. Liquid limit values for these soils indicate they typically are high plasticity clays. Figure 4 of DM-7.1 (NAVFAC, 1982) indicates that clays of high plasticity with SPT N-values of 4 to 8 typically have unconfined compressive strengths of 1 to 2 tsf.

This result is further confirmed by the cone penetration tests performed in the pad emplacement area. All of these CPTs measured cone tip resistance values, Q_t, of the soils within the depth range of 5 to 7 ft, that exceed 15 tsf, as shown in the plots of Q_t vs depth in Appendix A of ConeTec (1999). Using an empirical cone factor, N_{kt}, of 12.5 to estimate the undrained shear strength, these tip resistance values result in undrained shear strengths that exceed 1 tsf, indicating the unconfined compressive strengths exceed 2 tsf.

Table 7.1 of Terzaghi & Peck (1967) indicates that clayey soils with strengths of 1 to 2 kg/cm^2 (~1 to ~2 tsf) are characterized as having "stiff" consistency. Therefore, existence of a few N-values that are less than 8 blows/ft for the soils within the pad emplacement area do not adversely impact the characterization of these soils as "stiff".

The value of standard penetration resistance, N, was determined to be approximately 15 blows/ft for the top 25 to 30-ft soil layer based on the data obtained in Borings A-1 through A-4, B-1 through B-4, C-1 through C-4, and D-1 through D-4. This set of borings represents all of the borings that were drilled over the entire proposed pad emplacement area, as shown in Figure 2.6-2. The blow count data are summarized in Table 2.6-5. As indicated in this table, the average blow count is 15.7 blows/ft, and the median value is 14.0 blows/ft. These two values were combined to obtain the value of ~15 blows/ft.

These blow count data are plotted versus elevation in Figure 2.6-27. The average blow count for each 5-ft elevation interval is plotted using an open circle, whereas the median value is plotted as an open square. Also shown, by the heavy dash-dot line, is the average value of all SPT blow counts in the upper 25 to 30-ft layer of silt, silty clay, and clayey silt, at N = 15.

A distinct change in material occurs at about 25 to 35 ft, where refusal (N>100 blows per 6 inches) conditions are often encountered. The following 25 to 30 ft consists of very dense, dry, fine sand. Thin layers of fine gravel and coarse sand also are evident. A few clayey zones were encountered, but they had no apparent effect on the blow counts. The borings that were drilled to a depth of 100 ft or more (Borings A-1, D-4, CTB-1, and CTB-5) indicate that this layer is mainly underlain by very dense silt, silty sand, and sandy silt with occasional layers of clayey silt. Several layers of volcanic ash were also encountered in these borings.

A groundwater observation well was installed in Boring CTB-5(OW) in early 1999 in the vicinity of the Canister Transfer Building. Initial readings from this well indicate the groundwater table is about 125 ft below ground surface, approximate elevation 4,350 ft. Seismic refraction results (Appendix 2B) in the vicinity of the Storage Facility (see Figure 2.6-2, Seismic Lines 1 & 2) indicate the compression wave (P-wave) velocity changes from approximately 2,780 ft/sec to approximately 5,525 ft/sec at depths of between 90 and 131 ft below grade, which corroborates the depth to the water table measured in CTB-5(OW).

As indicated in Section 2.6.1.5, cone penetration tests were performed to further investigate the properties of the upper 25 to 30-ft thick layer at the site. There are some differences between the results of the cone penetration testing and the borings in regard to descriptions of the types of soils encountered, mostly in the 10 to 20-ft depth

range. The CPTs indicated that the soils between approximately 10 ft and 20 ft below existing grade at the site behave as though they are silty sands and sandy silts. This finding was not corroborated by the descriptions of the soils obtained from that zone in the borings, many of which are confirmed by laboratory test results.

The soil behavior type data presented in the CPT report (ConeTec, 1999) were determined using empirical correlations derived primarily from data collected from testing saturated and uncemented soils. Experience has shown that, typically, the cone penetration tip resistance is high in sands and low in clays and the friction ratio ($R_f = F_s/Q_t$) is low in sands and high in clays. This observation is incorporated in several soil classification charts. The correlations used to interpret the data measured in the CPTs performed at the site were developed by Robertson and Campanella (1988) and are presented in Figure 5, "Soil Behavior Type Classification Chart," of ConeTec (1999).

The soil descriptions shown in the CPT test results in ConeTec (1999) are based on the soil behavior type zones defined by the cone tip resistance and friction ratio values. The SBTs indicate that the soils within the 10 to 20-ft depth range generally behave like sandy silts or silty sand/sand, while the boring logs show the soils to be primarily a slightly to highly plastic silt. This difference is believed due to the partially saturated and weakly cemented soils encountered at the site. Many of the samples in this depth range had water contents near to or below the plastic limit. It is believed that, because of the low water contents and partial saturation effect, the slightly to highly plastic silts have low or no dynamic pore pressure response, which results in low measured sleeve friction values when penetrated by the cone. The cementation effect results in higher cone tip resistance values when penetrated by the cone and, likely, lower sleeve friction values. The higher tip resistance, in combination with measured sleeve friction values that are lower than they would otherwise be if they were saturated and uncemented, results in the interpretation that these soils behave like sandy soils when the SBT

classification chart developed by Robertson and Campanella (1988) is used. However, as observed in the borings and confirmed by the laboratory test results, most of these soils are clayey silts to silty clays.

As stated on page 51 of Robertson Campanella (1988),

> *"... CPT classification charts cannot be expected to provide accurate predictions of soil type based on grain size distribution but provide a guide to soil behaviour type. The CPT data provide a repeatable index of the aggregate behavior of the in situ soil in the immediate area of the probe."*

As indicated on page 8 of ConeTec (1999),

> *"It should be noted that it is not always possible to clearly identify a soil type based on Qc, Fs and Ud."*

Factors such as changes in stress history, *in situ* stresses, sensitivity, stiffness, macrofabric, mineralogy, and void ratio will also influence the classification. As indicated on page 8 of ConeTec (1998),

> *"... the chart is global in nature and provides only a guide to soil behaviour type (SBT). Overlap in some zones should be expected and the zones should be adjusted somewhat based on local experience.*
>
> *If no prior CPT experience exists in a given geologic environment it is advisable to obtain samples from appropriate locations to verify the classification and soil behaviour type."*

To appropriately use the cone penetration test results for soil classification at this site, the soil behavior type chart developed based on saturated uncemented soils must be recalibrated for the partially saturated and cemented soils encountered at the site. This was done by plotting the results of the soil classifications determined based on the borings and laboratory tests alongside plots of the soil behavior types from the CPT results. The borings and CPTs selected for this comparison were those that were performed within ~50 ft of each other, which included the following:

Boring	CPT	Distance Between (Ft)
CTB-4	CPT-37	4.2
CTB-5(OW)	C{T-38	4.2
C-1	CPT-39	5.5
A-2	CPT-34	15.0
B-3	CPT-20	29.8
A-3	CPT-32	50.9

Figure 2.6-30, Sheets 1 through 6, present these plots. The boring data are plotted vs elevation to indicate the locations of the samples, the Standard Penetration Test N-values, the USC Codes, and the sample descriptions. Also indicated are the SBT values appropriate for the samples obtained in the borings. The soil behavior type values from the CPTs are listed and plotted along the right side of the figures to facilitate comparison. The differences shown under the column labeled □SBT represent the SBT zoning shift required to more correctly characterize these soils as a result of the effects of partial saturation and cementation, as discussed above. Evaluation of these ΔSBT values leads to the conclusion that the soil behavior type values reported in ConeTec (1999) that are greater than 5 (i.e., sandier soils), as well as some of those equal to 5 (i.e., clayey silts), typically should be adjusted downward one or two zones to more accurately reflect the soil classifications that were determined based on the borings and confirmed by the laboratory tests performed specifically for the purpose of classifying the soil types.

To more accurately reflect the actual soil classification at the site, the SBT values reported by ConeTec (1999) were adjusted downward by 1 zone for those whose SBT values were greater than 5. Based on the results of this recalibration, the soil behavior type data from ConeTec (1999) are replotted, as shown in Figure 2.6-5, Sheets 2 through 14, along with the data from the soil borings, to generate the generalized soil profiles at the pad emplacement area.

As shown in Figure 2.6-2, Borings AR-1 through AR-5 were drilled along the proposed corridor for the access road, which extends easterly from the area in the vicinity of the proposed Operations & Maintenance (O&M) Building and the Administration Building to Skull Valley Road. These borings indicate that the near-surface soils are similar to the uppermost layer described above; i.e., silt, silty clay, and clayey silt, although the layer is somewhat thinner. Sands were encountered at depths of 5 and 10 ft in Boring AR-1

and from a depth of 5 ft to 20 ft in Boring AR-2. These sands are likely the subsurface continuation of the surficial beach ridges identified by Currey (Appendix 2C) and shown on Figure 1-3 of Geomatrix Consultants, Inc. (2001a). Silty or sandy gravels were encountered at a depth of 30 ft in Boring AR-3, 20 ft in Boring AR-4, and 6 ft in Boring AR-5.

These borings did not encounter bedrock. Interpretation of the seismic reflection survey data (Appendix 2B) indicates that the depth to bedrock is between 520 ft and 820 ft below the surface at the site in the vicinity of the storage pads and that it drops off towards the east, dipping from an estimated depth of 740 ft at Station 700 on Seismic Line 3 (shown on Figure 2.6-2) to approximately 1,020 ft at the eastern end of this seismic line. This is consistent with the interpretation that the Tertiary half-graben basin beneath Skull Valley is tilted down to the east along the East fault (Geomatrix Consultants, Inc., 2001a).

The original subsurface investigation, performed during the latter part of 1996, determined the suitability of the soil at the site for the proposed facility. The boring data indicated that the subsurface profile (Figure 2.6-5) was fairly consistent across the storage pad area. The results of the seismic survey performed in the storage pad area (Appendix 2B, Figures 4.1 & 4.3, primary wave refraction sections for Seismic Lines 1 & 2), corroborated the generalized subsurface profile developed based on the borings.

Additional subsurface work has been performed in response to PFSF SAR RAI No.1, dated April 1, 1998, including field work that was performed at the site (Geomatrix, 2001a) in response to PFSF SAR RAI No. 1, Question 2-5. This program focused on defining the presence and capability of all faults beneath the site area, and it included borings, trenching, and high resolution seismic reflection surveys. Additional laboratory (Atterberg limits) testing was performed and is reported in Attachment 2 of Appendix 2A. These data are also included on Figure 2.6-20. The detailed stratigraphy

developed from this program corroborated the conclusions drawn from the previous work.

The boring and trench data reported by Geomatrix (2001a) are stratigraphically very consistent across the storage pad area with the original boring data presented in Appendix 2A, both in the north-to-south and the east-to-west directions. The upper 30-ft (approximately) layer of soil is comprised of mixtures of silt, silty clay, clayey silt, and some sandy silt, that can be interpreted to represent stages in the cyclic history of Lake Bonneville (Geomatrix, 2001a). These stages are, from oldest to youngest, a Stansbury deepwater facies, a post-Stansbury transgressive and regressive facies, and the Provo and Bonneville deepwater facies.

Four borings (A-1, D-4, CTB-1, and CTB-5(OW)) were drilled to depths in excess of 100 ft and 25 borings were drilled to depths of approximately 50 to 75 ft (see Figure 2.6-5 sheets 1 through 14 and Figures 2.6-21 through 2.6-23. Logs of these borings are included in Attachment 1 of Appendix 2A. Standard penetration test (SPT) blow counts were obtained, generally, at 5-ft intervals in these borings. The data from these borings show a consistent picture across the site.

These site materials are consistent with what would be expected for deposits of a lacustrine environment, away from the direct influence of range-front alluvial fans. These deposits are overlain at the surface by thin, post-Provo eolian silt and recent playa deposits. They lie upon a uniform, fine sand that forms a nearly horizontal surface across the site at about elevation 4,440 ± 5 ft. This sand is the Stansbury transgressive facies, representing a series of shorelines and deltas that developed as Lake Bonneville initially occupied the area and rose to the deepwater Stansbury level. This sand is dense, having SPT blow counts generally in the range of 70 to over 100 blows/ft. At the base of the sand unit is an unconformity marked by the Promontory

soil, which developed on pre-Bonneville subaerial deposits. This gravelly layer occurs in the borings drilled in the storage pad area at a depth of about 45 to 50 ft (at about elevation 4410 to 4430 ft). Below the sandy gravel, the sediments are very hard silts, with some very dense sands. The SPT blow counts in these materials are generally well in excess of 100 blows/ft. Below a depth of approximately 90 ft, an ash marker horizon is encountered that indicates penetration into the Tertiary sediments of the Salt Lake Group.

As shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), the cask storage pads will be founded in the surficial eolian silt layer. The eolian silt, in its in situ loose state, is not suitable for founding the cask storage pads. Instead of excavating the eolian silt and replacing it with suitable structural fill, the eolian silt will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Soil cement will also be utilized around the Canister Transfer Building. The characteristics of the soil cement will be engineered during detailed design to meet the necessary strength requirements. See Section 2.6.4.11 for additional details about the soil cement.

As indicated in Section 2.6.1.5, in January 2001, 16 test pits were excavated at the PFSF site in the pad emplacement area to obtain soil samples for use in the laboratory analyses necessary to design the soil cement. It was observed from these test pits that the depth of the eolian silt was shallower than previously believed (approximately 2 ft on average, rather than 3 ft). The borings previously performed in this area obtained soil samples at depths from grade to 2 ft and from 5 ft to 7 ft. Therefore, as later observed in the test pits, the interface between the eolian silt and the silty clay/clayey silt fell between the samples collected in the borings. The soil unit descriptions from Trench T-2 in the pad emplacement area (Plate 3, Geomatrix, 2001a) also corroborates the soil-cement test pit observations; i.e., the fine sandy silt (eolian deposit) overlying

the sandy clayey silt (combic B soil horizon developed on Bonneville deep-water sediment) is not expected to extend much deeper than approximately 2 ft from the ground surface. Furthermore, these observations are verified by Atterberg limits tests that have recently been performed on the samples obtained from these test pits, which indicate that the soil samples collected below depths of 2 ft are exclusively cohesive clayey silt/silty clay with high plasticity indices.

The previous interpretation of the eolian silt boundary assumed that this boundary lay where the initial spike in the cone penetration tip resistance bottomed out. This assumption was made in order to obtain a conservative upper-bound estimate of the amount of soil cement required for the soil improvement of the noncohesive eolian silt. This increase in tip resistance was previously assumed to represent a layer of slightly cemented eolian silt. However, as observed in the January 2001 soil-cement test pits and in Trench T-2, the interface between the noncohesive eolian silt and the cohesive clayey silt/silty clay more closely corresponds to the initial increase in the cone tip resistance, along with an accompanying steep increase in the sleeve skin friction resistance. These observations are consistent with the experience of soil classification using electric CPT data, which indicate that sandy soils (noncohesive) tend to produce high cone resistance and low friction ratio, whereas soft clay soils (cohesive) tend to produce low cone resistance and high friction ratio (p.51, Lunne, Robertson and Powell, 1997). Therefore, it is expected that the transition from the noncohesive soil to the cohesive soil will be characterized by a steep increase of the cone skin friction resistance.

Based on the correlations and evaluations discussed above, the transitional boundary between the surficial noncohesive eolian silt and underlying cohesive clayey silt/silty clay presented in Figure 2.6-5, Sheets 1 through 14, was re-interpreted to make it consistent with the soil-cement test pit observations and laboratory classification test

results performed on soil samples from the test pits in the pad emplacement area. The interpretation also considered that the measurement of sleeve friction (f_s) is often less accurate and less reliable than the cone resistance (p.51, Lunne, Robertson and Powell, 1997). The boundary was re-interpreted based on consideration of the consistency between various cone penetration tests to obtain a smoothed boundary, instead of interpreting each cone penetration test discretely. This re-interpretation of the eolian silt boundary results in a reduction of the estimated amount of eolian silt, and a corresponding reduction in the amount of soil cement required under the cask storage pads (see Figure 4.2-7), minimizing environmental impacts associated with construction of the facility (i.e., fewer truck trips for cement deliveries, less water required to mix the soil cement, less dust associated with excavation, mixing, and compaction of soil cement, ...).

Refer to Sections 2.6.1.11 and 2.6.2.1 for a discussion of the engineering characteristics of these soils and to Section 2.6.4.11 for information concerning the soil cement.

2.6.1.7 Excavations and Backfill

The proposed detention basin, which will be excavated approximately 5 ft deep over an area that is approximately 200 ft x 800 ft at the north side of the proposed storage facility, is the only major excavation proposed for the PFSF. Shallow excavations will be required to construct the cask storage pads and the strip and spread footings supporting the other structures at least 30 inches below finished grade (to provide protection against frost heave), as well as to provide drainage ditches along the proposed access road.

Excavations for footings deeper than 3 ft shall be completed to the design grades, maintaining stable slopes of not steeper than 2 horizontal to 1 vertical. After construction of the foundations, the excavations will be backfilled with structural fill or soil cement to minimize potential problems in the future.

It was originally intended that the cask storage pads would be founded on the silty clay/clayey silt layer shown underlying the near-surface layer of eolian silt in Figure 2.6-5, Sheets 1 through 14. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Refer to Section 2.6.4.11, Techniques to Improve Subsurface Conditions, for additional details.

The in situ materials generally are not adequate for use as structural backfill; therefore, it is expected that structural fill materials will be obtained from an offsite source. Structural fill material shall be granular material consisting of well graded sand and gravel, containing no more than 10% of material passing the #200 sieve and a maximum particle size not greater than 6 inches. Samples of the structural fill material

shall be tested for gradation in accordance with ASTM D-422 and for moisture-density relationship in accordance with ASTM D-1557. New gradation and moisture-density tests shall be required whenever a change in material is observed.

Structural fill material shall be placed in thin lifts, not exceeding 8-inch loose thickness, spread evenly, and compacted to 95% of the maximum dry density as determined in accordance with ASTM D-1557. Compacted surfaces shall be protected from freezing and, if found frozen, shall be excavated, wasted, and replaced with new compacted fill. Compacted surfaces shall be pitched to freely drain to eliminate puddling of storm water. Compacted material shall be tested frequently by performing in-place density and moisture tests, as specified in the construction specifications.

2.6.1.8 Engineering-Geology Features Affecting ISFSI Structures

Engineering Geology is discussed in Section 2.6.4.

2.6.1.9 Site Groundwater Conditions

The groundwater table at the site was encountered in Observation Well CTB-5(OW) at a depth of 125 ft (elevation 4,350 ft) in the vicinity of the Canister Transfer Building. Seismic refraction velocities along Seismic Lines 1, 2, & 3 (Appendix 2B) are indicative of saturated conditions at depths ranging from 90 ft to 136 ft below ground surface across the site area (elevation 4,334 ft to 4385 ft), which corroborates the depth to the water table measured in Boring CTB-5(OW). Local groundwater conditions, based on limited water well data in the area, are somewhat variable and dependent upon the subsurface extent of alluvial fan materials. Stock-watering wells four and five miles westerly from the site have water depths of 280 and 295 ft, (elevations 4,350 ft and 4,325 ft, respectively). About 2.5 miles northeast of the site the water table is at 188 ft depth (elevation 4,350 ft),

and 6 miles southeast several wells flow at the surface (elevation 4,605 ft). A well at the Tekoi Rocket Engine Test Facility about 3 miles south of the site was drilled to 400 ft and has static water at 80 ft below ground surface (elevation about 4,480 ft). All the above-mentioned wells were completed in unconsolidated materials without drilling into the bedrock. The locations of all wells within 5 miles of the PFSF are identified in Figure 2.5-1. These data suggest that the main aquifer in the central part of Skull Valley is unconfined or semi-confined and occurs mainly within the fine-grained Tertiary Salt Lake Group deposits. These sediments interfinger with coarse-grained alluvial fan material along the toe of the fan and may create confined conditions where they overlap the fan deposits. The fan deposits are the main recharge zone for the valley aquifers and the main source for domestic water wells in the valley. The aquifer in the fans is unconfined for the most part, but becomes confined and under artesian conditions downslope where the lake and basinal deposits onlap the fan at depth. Water wells drilled near the lower edge of the fan, such as at the Rocket Engine Test Facility, may penetrate several hundred feet of sediments before encountering a coarse alluvial fan layer. Since the coarse layer is under artesian pressure, the level of water in the well will rise upward to the static condition or may flow at the surface, such as occurs just south of the Reservation.

Groundwater levels at the site appear to closely correlate with levels in the main valley aquifer. They do not appear to be affected by proximity to the alluvial fan. At this time it is believed an adequate quantity of suitable quality water can be developed within the site area for the PFSF needs. Specific properties of aquifer materials are unknown at this time. As discussed in ER Sections 4.5.5 - 4.5.7, based on initial testing of the site monitoring well, it is believed that groundwater withdrawals at the PFSF site would have no measurable impact on off-site wells, either up-gradient or down-gradient (SWEC, 1999b). Surface soil at the site has a permeability of 0.2 to 0.6 inch/hr, whereas the soil on the alluvial fan has a permeability of 6 to 20 inches/hr (USDA, unpub. data). As

discussed in ER Section 4.5.5, it is estimated that the average withdrawal rate from the well over a 42 year period will be approximately 2,040 gallons per day (1.4 gpm).

Groundwater quality in the area is variable, with the best quality associated with wells developed in the alluvial fans near the Stansbury Mountains. In general, water quality is lower in the valley bottom, but it is suitable for irrigation or stock watering without treatment. The main dissolved ions are sodium and chloride (Hood and Waddell, 1968). There is also a tendency for the quality to be lower farther north, down-valley, towards the Great Salt Lake, although there are exceptions to this trend. Total dissolved solids range from 1,600 to 7,900 mg/l at the northern end of the valley (Arabasz et al., 1987, App. F). Most sources of water in the valley are high in calcium and would be classified as very hard. Aquifer transmissivities range from 500 to 30,000 sq ft/day with an average for Skull Valley estimated at 5,000 sq ft/day (Arabasz et al., 1987, App. F).

2.6.1.10 Geophysical Surveys

Seismic Refraction and Reflection Surveys

Results of seismic refraction and reflection surveys performed at the site in 1996 are found in Appendix 2B. Engineering properties of site materials based on the geophysical investigations are discussed in Section 2.6.1.11. The results of 1998 geophysical surveys (seismic reflection, gravity, and magnetic) are discussed in Geomatrix Consultants, Inc. (2001a) and Bay Geophysical Associates (1999).

Seismic Cone Penetration Tests

Seismic cone penetration tests were performed at the locations designated as "SEIS CPT" on Figure 2.6-19. The purpose of these tests was to measure down-hole P and S-wave velocities. The results of these tests are presented in Appendix C of ConeTec (1999), and the average velocities vs depth are shown in Figure 2.6-28.

Shear wave velocities of soils are dependent on the effective stress, void ratio, and for clays, the plasticity index and overconsolidation ratio of the soils. If all of these parameters were the same, it would be expected that the shear wave velocities would increase with increasing depth in the profile. The apparent leveling off of the shear wave velocities at a depth of about 10 to 15 ft in the results of the seismic CPTs that were performed at the site (Appendix C of ConeTec, 1999) is an indication that one or more of these parameters have changed. A review of the Q_t plots, which are included on the left-hand side of the same pages that present the shear wave velocities vs depth, indicates that the tip resistance increases greatly in this zone. This increase in tip resistance is most likely associated with a change in soil type, as indicated by the SBT plots on the right-hand side of these same pages, as well as by a decrease in the void ratio of these soils. Therefore, it is not unexpected that the shear wave velocities would change within this zone.

A review of the shear wave velocities vs depth presented in Appendix C of ConeTec (1999) indicates that they do not level off with depth. The general trend in the data is to increase with respect to depth; however this trend is masked by the presence of the marked increase in the shear wave velocities in the "harder" zone that exists generally within the depth range of about 13 feet to about 20 feet. If the shear wave velocities associated with this harder zone are excluded, all of the plots of shear wave velocities show a general increase with respect to depth. This general increase in velocity with increasing depth is more readily observed in Figure 2.6-28.

Downhole Seismic Velocity Tests

Northland Geophysical (2001) conducted confirmatory downhole shear and compression wave velocity measurements in Borings CTB-5(OW) and CTB-5A to a maximum depth of 106.5 feet. The data from Boring CTB-5(OW) were measured in a PVC-cased boring to a maximum depth of 50.8 feet. The shear and compression wave velocity data from this boring are very consistent with the velocities obtained from the

seismic cone penetration tests in the depth range of 0 to 30 feet (Geomatrix, 2001c). The data from Boring CTB-5(OW) show a trend of gradually increasing velocity in the depth range of 30 to 50 feet. Boring CTB-5A was drilled adjacent to CTB-5(OW) using a hollow-stem auger, and shear wave velocities were measured with the geophones clamped within the auger stem. The measurements were initiated near the bottom of the hole at a depth of 106.5 feet and continued upward until the data quality began to deteriorate above 44 feet. As a result, there was a very limited range of overlap between the measurements in Borings CTB-5(OW) and CTB-5A. The velocities in the two borings were similar (see Figures 3 and 4 of Northland Geophysical, 2001). The data from Boring CTB-5A show fairly uniform velocities for the depth range of 55 to 95 feet and an increase in velocity below 95 feet. The results of the measurements taken in these downhole seismic velocity surveys have been incorporated into the analyses discussed in Section 2.6.2, Vibratory Ground Motion.

2.6.1.11 Static and Dynamic Soil and Rock Properties at the Site

Geotechnical laboratory tests were performed on samples obtained from the borings. The results of these tests are included in Appendix 2A and are summarized below. Figure 2.6-20 presents plots of the SPT blow counts vs depth in the pad emplacement area, on a row-by-row basis. This figure also presents the index properties that were measured for these soils, along with the results of triaxial testing. Dry densities of subsurface soils at the site are plotted vs depth in Figure 2.6-31. Comparison of these plots indicates that the soil properties are fairly consistent across the site.

2.6.1.11.1 Pad Emplacement Area

The results of the tests of the silty clay/clayey silts obtained from the upper 25 to 30 ft layer in the pad emplacement area are as follows:

Index Property:	Minimum	Maximum	Average
Water Content, %	8	58	32
Liquid Limit	25	77	44
Plastic Limit	20	46	30
Plasticity Index	0.5	38	14
Moist Unit Weight, pcf	64	91	78
Dry Unit Weight, pcf	40	71	56
Void Ratio	1.4	3.2	2.1
Saturation, %	28	64	53
Specific Gravity = 2.72			

Consolidation parameters:	Low	High	Average
Maximum past pressure, ksf:	5.6	7.2	6.2
Virgin compression ratio, CR:	0.25	0.34	0.29
Recompression ratio, RR:	0.008	0.017	0.012

Rate of secondary compression is shown by the dashed curve in Figure 2.6-6.

Total-stress strength parameters are ϕ = 24.9° and c = 1.22 ksf based on direct shear tests that are included in Attachment 7 of Appendix 2A. Total-stress parameters are ϕ = 21.3° and c = 1.4 ksf based on the unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial tests that are included in Attachments 5 and 8 of Appendix 2A. These strengths were measured in the laboratory by consolidating the samples to various confining pressures, but generally to the confining pressure that will exist prior

to the seismic loading, and then shearing them rapidly to simulate conditions that will exist during the earthquake loading.

The original triaxial tests (results reported in Appendix 2A, Attachments 2, 4, and 5) were performed at confining stresses that represent the static conditions that will exist under the fully loaded pads. To demonstrate the cohesive nature of these soils, an additional consolidated-undrained triaxial compression test was performed at a confining stress of 1 ksf, which is representative of the minimum confining stresses expected to exist under the fully loaded pads when the maximum uplift forces due to the design basis ground motion occurs, and one test was performed at a confining stress of 0, which is essentially an unconfined compression test. The results of these tests are included in the total-stress strength parameters reported above, and details of these tests are included in Attachment 8 of Appendix 2A.

The dotted line shown in the plot of Mohr's circles of the results of CU tests performed on samples from Boring B-1 that is included in Attachment 8 of Appendix 2A is tangent to the Mohr's circle for Sample U-2B of Boring B-1. It indicates that the cohesion of this specimen is slightly less than that of the other specimens tested. This strength was lower because its dry density (γ_d = 46.3 pcf) was lower than that of the other specimens. As indicated by the plots of water content vs depth presented in Figure 2.6-20, most of the *in situ* soils in the upper ~25-ft layer at the site have w_n < 50%, which is more like Samples U-2C and U-2D; hence the recommendation that c = 1.4 ksf for these soils.

For the partially saturated cohesive soils at the site, the strength of the soil is dependent on its apparent cohesion, friction angle, as well as the consolidation pressure. The strain rates are very high during the seismic event; therefore, the partially saturated cohesive soils will be stressed essentially under undrained

conditions. The effect of the pore pressure response, if any, during such tests, either positive or negative, will be manifested directly in the shear strength value measured. Because the strain rate of the laboratory tests is at least one order of magnitude slower than the rate associated with the design basis ground motion, the strength measured in the laboratory is a lower-bound estimate of the strength that will be available to resist the dynamic loadings during the seismic event. See Section 2.6.1.11.5, "Dynamic Strength of Cohesive Soils," for additional details.

The bearing capacity of the structures is dependent primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec(1999) and plotted in Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial tests were performed.

In determining the bearing capacity of shallow foundations, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the cask storage pads, the effective width of the footing is decreased because of the large eccentricity of the load on the pads due to the seismic loading. As indicated in Table 2.6-7, the minimum effective width of the cask storage pads occurs for Load Case II, where B' = 15.6 ft. For the load cases where the three components of the earthquake are combined in accordance with ASCE (1986), the minimum effective width of the cask storage pads occurs for Load Case IIIB, where B' = 15.7 ft. Figure 7 of Calculation 05996.02-G(B)-4 (SWEC, 2001b) illustrates that the anticipated slip surface of the bearing capacity failure for B' ~15 ft is limited to the soils within the upper two-thirds of the upper layer. Therefore, in the bearing

capacity analyses of the cask storage pads, the undrained strength measured in the triaxial tests was not increased to reflect the increase in strength measured for the deeper-lying soils in the cone penetration testing.

Table 6 of Calculation 05996.02-G(B)-5 (SWEC, 2000c) summarizes the results of the triaxial tests that were performed within depths of ~10 ft at the site. The undrained shear strengths measured in these tests are plotted vs confining pressure in Figure 11 of that calculation. This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction. As indicated, the undrained strength of the soils within ~10 ft of grade was assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 11 of Calculation 05996.02-G(B)-5 (SWEC, 2000c) illustrates that the undrained strength of these soils increases as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative, lower-bound value for use in the dynamic bearing capacity analyses of these structures.

Effective-stress strength parameters are estimated to be c = 0 ksf, even though these soils may be somewhat cemented, and $\phi = 30°$. This value of ϕ is based on the average PI value for these soils, which equaled 14%, as shown in the table presented above, and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

Therefore, static bearing capacity analyses of the cask storage pads were performed using the following soil strengths:

Case IA Static using undrained strength: $\phi = 0°$ & c = 2.2 ksf.

Case IB Static using effective-stress strength: $\phi = 30°$ & $c = 0$.

The pads will be constructed on and within soil cement, as illustrated in Figure 4.2-7 and described in Sections 2.6.1.7 and 2.6.4.11. The unit weight of the soil cement is assumed to be 100 pcf in the bearing capacity analyses. The strength of the soil cement was conservatively ignored in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained at a depth of 5.7 ft to 6 ft in Boring C-2. These tests were performed at normal stresses that were essentially equal to the normal stresses expected:

- under the fully loaded pads before the earthquake,
- with all of the vertical forces due to the earthquake acting upward, and
- with all of the vertical forces due to the earthquake acting downward.

The results of these tests are presented in Attachment 7 of the Appendix 2A and they are plotted in Figure 7 of Calculation 05996.02-G(B)-5 (SWEC, 2000c). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses of the cask storage pads constructed directly on the silty clay are performed using the shear strength measured in these direct shear tests for the normal stress that equals the vertical stress under the fully loaded cask storage pads prior to imposition of the dynamic loading due to the earthquake. As shown in Figure 7 of Calculation 05996.02-G(B)-5 (SWEC, 2000c), this shear strength is 2.1 ksf and the friction angle is set equal to 0° for the clayey soils underlying the cask storage pads.

2.6.1.11.2 Canister Transfer Building Area

The results of the tests of the silty clay/clayey silts obtained from the upper 25 to 30 ft layer in the Canister Transfer Building area are as follows:

Index Property:	Minimum	Maximum	Average
Water Content, %	7	86	40
Liquid Limit	28	83	51
Plastic Limit	18	48	30
Plasticity Index	4	38	20
Moist Unit Weight, pcf	73	118	92
Dry Unit Weight, pcf	40	98	65
Void Ratio	0.7	3.3	1.8
Saturation, %	40	88	71
Specific Gravity	2.71	2.73	2.72
Consolidation parameters:	**Low**	**High**	**Average**
Maximum past pressure, ksf:	6	26	13
Virgin compression ratio, CR:	0.13	0.37	0.31
Recompression ratio, RR:	0.014	0.020	0.018

Total-stress strength parameters are ϕ = 21.1° and c = 1.13 ksf, based on the average values of the direct shear test results for samples from Borings CTB-6 & CTB-S, presented in Attachments 7 and 8 of Appendix 2A.

Table 6 of Calculation 05996.02-G(B)-5 (SWEC, 2000c) summarizes the results of the triaxial tests that were performed within depths of ~10 ft at the site. The undrained shear strengths measured in these tests are plotted vs confining pressure in Figure 11 of that calculation. This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction. As indicated, the undrained strength of the soils within ~10 ft of grade was assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical

stress existing near the middle of the upper layer prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 11 of Calculation 05996.02-G(B)-5 (SWEC, 2000c) illustrates that the undrained strength of these soils increases as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative, lower-bound shear strength value for use in the dynamic bearing capacity analyses of these structures.

The bearing capacity of the structures is dependent primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec (1999) and plotted in Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial tests were performed.

In determining the bearing capacity of shallow foundations, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 119.5 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is reasonable to use the average strength of the soils in the upper layer in the bearing capacity analyses of this structure, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

The undrained strength used in the bearing capacity analyses of the Canister Transfer Building is a weighted average strength that is applicable for the soils in the upper layer. This value is determined using the value of undrained shear strength of 2.2 ksf noted above for the soils tested at depths of ~10 ft and the relative strength increase measured for the soils below depths of ~12 ft in the cone penetration tests that were performed within the Canister Transfer Building footprint. As indicated on Figure 2.6-18, these included CPT-37 and CPT-38. Similar increases in undrained strength for the deeper lying soils were also noted in all of the other CPTs performed in the pad emplacement area.

Attachment B of Calculation 05996.02-G(B)-13 (SWEC, 2001c) presents copies of the plots of s_u vs depth for CPT-37 and CPT-38 (from Appendix D of ConeTec, 1999). These plots are annotated to identify the average undrained strength of the cohesive soils measured with respect to depth. As shown by the plot of s_u for CPT-37, the weakest zone exists between depths of ~5 ft and ~12 ft. The results for CPT-38 are similar, but the bottom of the weakest zone is at a depth of ~11 ft. The underlying soils are all much stronger. The average value of s_u of the cohesive soils for the depth range from ~18 ft to ~28 ft is ~2.20 tsf, compared to s_u ~1.34 tsf for the zone between ~5 ft and ~12 ft. Therefore, the undrained strength of the deeper soils in the upper layer was ~64% (Δs_u = 100% x [(2.20 tsf – 1.34 tsf) / 1.34 tsf] higher than the strength measured for the soils within the depth range of ~5 ft to ~12 ft. The relative strength increase was even greater than this in CPT-38.

Using 2.2 ksf, as measured in the UU triaxial tests performed on specimens obtained from depths of ~10 ft, as the undrained strength applicable for the weakest soils (i.e., those in the depth range of ~5 ft to ~12 ft), the average strength for the soils in the entire upper layer is calculated as shown in Figure 4 of that calculation (SWEC, 2001c). The resulting average value, weighted as a function of the depth, is s_u ~3.18 ksf. This

value would be much higher if the results from CPT-38 were used; therefore, this is considered to be a reasonable lower-bound value of the average strength applicable for the soils in the upper layer that underlie the Canister Transfer Building.

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 11 of Calculation 05996.02-G(B)-5 (SWEC, 2000c), which presents a summary of the triaxial test results for soils obtained within ~10 ft of the ground surface at the site. This plot of s_u vs confining pressure illustrates that this weighted average value (3.18 ksf) is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~5 ft below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underling the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses of the Canister Transfer Building.

Effective-stress strength parameters are estimated to be $\phi = 30°$ and c = 0 ksf, even though these soils may be somewhat cemented. This value of ϕ is based on the average PI value for the soils underlying the Canister Transfer Building, which equaled 20%, as shown in the table presented above, and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

Therefore, static bearing capacity analyses are performed using the following soil strengths:

Case IA Static using undrained strength parameters: $\phi = 0°$ & c = 3.18 ksf.

Case IB Static using effective-stress strength parameters: ϕ = 30° & c = 0.

and dynamic bearing capacity analyses are performed using ϕ = 0° & c = 3.18 ksf.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in Figure 2.6-18, within the footprint of the Canister Transfer Building. These specimens were obtained from Elevation ~4469, the elevation near the bottom of the perimeter key will be constructed at the base of Canister Transfer Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is engaged to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A. Copies of these plots are included as Figures 9 and 10 of Calculation 05996.02-G(B)-5 (SWEC, 2000c), with annotations that identify the range of vertical stresses due to the design basis ground motion, as well as the shear strength used in the analysis of the sliding stability of the Canister Transfer Building founded on the in situ silty clay/clayey silt. Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for the normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 9 and 10 of Calculation

05996.02-G(B)-5 (SWEC, 2000c), this average shear strength is 1.8 ksf and the friction angle is set equal to 0°. No credit is taken in this sliding stability analysis for the increased frictional strength due to the increased normal force multiplied by tan ϕ when the earthquake forces act downward, nor is credit taken for the strength increase that is typical for cohesive soils subjected to dynamic loadings.

For the sand or sandy soils layer in the Canister Transfer Building area found in some of the borings located at a depth of 8 to 20 ft:

Index Property	Minimum	Maximum	Average
Water Content, %	3	15	6
Moist Unit Weight, pcf	85	105	98
Dry Unit Weight, pcf	77	102	93
Void Ratio	0.64	1.2	0.83
Saturation, %	11	32	19
% Fines	9	38	23
Specific Gravity = 2.69			

2.6.1.11.3 Pad Emplacement & Canister Transfer Building Areas

Effective-stress strength parameters are estimated to be $\phi = 30°$ and c = 0, based on the plasticity index of the silts and clays. These values are very conservative for the sandy soils, which are characterized as dense based on their SPT N-values and the CPT Q_t data. Note, Appendix D of ConeTec (1999) indicates that ϕ based on the CPTs generally exceeds 35 to 40°.

The recommended coefficients of earth pressure for the silts and clays are as follows:

- At-rest, K_o, is 0.5
- Active, K_a, is 0.33
- Passive, K_p, is 3.0.

The recommended value of the coefficient of vertical subgrade reaction of the silt, silty clay, clayey silt for a 1 ft x 1 ft square is 100 kips/ft^3 for the clayey soils. Where the near-surface soils are cohesionless silts, this value should be 120 kips/ft^3. This value should be reduced for footing widths greater than 1 ft by applying a reduction factor, RF, calculated as follows:

For clayey soils: $RF = 1/B$

For cohesionless soils, $RF = [(B+1) / 2B]^2$

where B is the effective width of the footing.

This value should also be reduced for rectangular footings by (1 + 0.5 x B / L) / 1.5, where L is the effective length of the footing.

The recommended value of the coefficient of vertical subgrade reaction of the in situ clayey soils for use in design of the storage pads is 2.75 kips/ft^3; and for the cohesionless soils is 26 kips/ft^3.

The recommended value of the coefficient of horizontal subgrade reaction of the in situ clayey soils for use in the design of drilled caissons is 67 / B kips/ft^3. For cohesionless soils, the recommended value of the coefficient of horizontal subgrade reaction is 20 • z / B kips/ft^3.

Soil compressibility parameters and values of total-stress shear strength for the partially saturated silty clay/clayey silt were obtained from a number of tests to provide conservative results that were applicable for the upper 25 to 30 feet over the storage pad area because of the consistency of the subsurface conditions encountered in these borings. In addition, these results are considered to be conservative for the soils in the upper layer because they were obtained from testing specimens from the upper 25 to 30 feet where the Standard Penetration Test (SPT) blow count was less than or equal to the average value of all samples obtained in this layer, as indicated in Figure 2.6-27. Note, the SPT blow count is directly related to the density and strength of soils and inversely related to compressibility of soils.

Figure 54.4 of Terzaghi and Peck (1967) illustrates this for cohesionless soils. This figure presents the relationship between SPT blow counts (values of "N" in the figure), density, and compressibility of sands. It indicates that the density increases as the N-value increases. It also illustrates that a footing of a given width has a higher allowable

soil pressure for a given settlement (1" in this chart) as the SPT blow count increases. Therefore, as the blow count increases, the strength of cohesionless soil increases and its compressibility decreases.

Table 45.2 of Terzaghi and Peck (1967) presents the relationship between consistency, SPT blow count, and strength of clay. This table indicates that the consistency increases from very soft to hard for blow counts ranging from less than 2 blows/ft to greater than 30 blows/ft, respectively. Table 2 of Terzaghi (1955) indicates that the coefficient of subgrade reaction, defined as the ratio between the pressure at a given point of the surface of contact and the settlement produced by that load, increases as the consistency of clay increases. Therefore, as the SPT blow count increases, the consistency of clay increases, and the compressibility (and, hence, settlement) decreases.

This has been demonstrated by the laboratory testing that was performed on samples obtained at greater depths in the Canister Transfer Building area. Additional laboratory tests were performed on samples of the soils from deeper within the profile than those that were tested (from depths of about 10 to 11 ft) in 1996. These tests on the deeper soils are reported in Attachment 6 of Appendix 2A. With the exception of the CU test performed on Sample U-14D from CTB-5, which shows evidence that the sample may have been disturbed (i.e., the peak shear stress occurred at an axial strain of about 12% compared to 5% for most of the other CU tests), these tests indicate that the strengths of the deeper soils are higher than those tested in 1996 and their compressibilities are lower.

The depths of the specimens tested for strength and compressibility in Attachment 2 of Appendix 2A were selected to investigate conditions at a depth of about 10 feet below grade, which represents a depth of approximately ½ the width of the loaded area below

the foundation due to the loading from the storage cask. It is generally acknowledged in geotechnical engineering that the zone of influence of loads on foundations spread out below the footing (e.g., Section 8.3 of Lambe and Whitman, 1969). The stress increase is greatest at the base of the footing, and it dissipates to an insignificant value at a depth of twice the width of the foundation. It is common practice to place a greater emphasis on the depth below the foundation equal to the width of the load. Testing the soils at ½ of this depth provides parameters that reflect the average performance of the soils within the depth equal to the width of the loaded area.

As indicated in Figure 2.6-27, the average and median values of the SPT blow counts, plotted for each 5-ft elevation interval versus elevation, illustrate that the blow counts increase with depth from grade. This figure also indicates that the locations of the specimens tested for strength and compressibility fall within the zone where the average and median blow counts for each 5-ft elevation interval were less than or equal to the average value for the entire layer (15 blows/ft). Since the strength of these soils is directly related to the blow count, testing soils whose blow count is less than the average provides a conservative estimate of the strength of the soil. In addition, since the compressibility of these soils is inversely related to their blow count, testing soils whose blow count is less than the average provides a conservative estimate of their compressibility and, hence, result in conservative (i.e., higher) estimates of settlements that the cask storage pads will experience.

Figure 2.6-20 plots all of the N-values vs depth for each of the borings drilled in the proposed emplacement area. The borings are plotted by row in the four sets of plots according to their locations in the field, as shown in Figure 2.6-2. That is, the top row of plots on Sheet 1 of Figure 2.6-20 includes the data from the northernmost row of borings, the next row down the sheet represents the next row of borings, moving south

on the site, etc. The N-value plots in Figure 2.6-20 illustrate that the soils in the upper 25 to 30-ft thick layer of the profile do not vary significantly across the site.

Additional field work, performed at the site in 1998, is described in Geomatrix Consultants, Inc, (2001a) and included detailed lithostratigraphic soils mapping in test pits and trenches, as well as logging of continuous split-barrel samples in closely spaced boreholes. The results of these studies reaffirm the consistency of the upper layer of the subsurface profile across the site.

Subsurface profiles and stratigraphic descriptions are presented in Plates 3 and 4 in Geomatrix Consultants, Inc (2001a), and they illustrate convincingly that the subsurface conditions are very uniform. They identify a thin (<2.5 ft) surface layer of eolian silt and playa deposits with a poorly developed soil structure. This layer corresponds to the first SPT sample in the borings that were drilled in late 1996 (Attachment 1 of Appendix 2A) in the proposed pad emplacement area. This layer is underlain by a sequence of typical lacustrine sediments associated with several stages of Lake Bonneville, an inland sea that covered the area from about 30,000 to 10,000 years before present (B.P.). These sediments are, by and large, the fine-grained end members of a ternary diagram consisting of silt, clay, and sand. Samples are consistently described as silt, silty clay, clayey silt, or sandy silt. Geomatrix used the term marl or marly as an additional component of these descriptions, which refers to a high calcium carbonate content clay or silt deposited in a fresh-water environment (deep-water facies of Bonneville alloformation).

Geomatrix was able to subdivide the lacustrine sequence into several lake stages based on sedimentary relationships and physical characteristics exposed in continuous wall exposures in trenches and test pits. Their subdivisions of the Bonneville alloformation, presented in their Plate 3, "Map of North Wall Trench T-2", are as follows:

- Bonneville Deep-Water Blocky,
- Bonneville Deep-Water Laminated,
- Post-Stansbury Transgressive, and
- Stansbury Regressive.

This sequence extends to a depth of about 25 to 30 ft, where a continuous, nearly horizontal layer of dense, fine sand is encountered. This layer is the "Stansbury Transgressive", and it represents the oldest deposit of the Bonneville Cycle. The base of this unit occurs at a depth of about 45 to 50 ft and is believed to be an unconformity represented by the Promontory soil. This boundary is an apparent seismic velocity contrast that is recognizable on the recent seismic reflection profiles as a continuous, nearly horizontal layer, the Qp reflector (Geomatrix Consultants, Inc, 2001a).

2.6.1.11.4 Collapse Potential of High Void Ratio Soils

Due to the high void ratios of some of the in situ soils and their weakly cemented nature, there is the potential that these soils may be collapsible soils, which could settle dramatically due to wetting caused by the PMF flood or due to vibrations from the design earthquake. The following section demonstrates that these soils are not "collapsible soils". It also demonstrates that these soils will not be subject to wetting due to floodwaters associated with the Probable Maximum Flood (PMF), because, as indicated in Section 2.4.2.2, the tops of the cask storage pads are at least 4 ft above the nearest approach of the PMF to the PFSF pad emplacement area. The collapse potential due to vibrations from the design earthquake is demonstrated to be nonexistent based on the results of the cyclic triaxial tests, as described in Section 2.6.4.7.

The collapse potential of the high void ratio soils was determined in accordance with the requirements ASTM D5333–92, "Standard Test Method for Measurement of Collapse Potential of Soils." As indicated in Section 5.1 of this ASTM, "collapsible soils"

are subject to "... sudden and often large induced settlements when these soils are saturated ...". The test method consists of performing consolidation tests, wherein the tests are initiated with the specimens at the natural water content and, at a predetermined vertical stress, inundating the specimens with distilled water to determine their proclivity to collapse upon wetting.

PFS performed ten consolidation tests. The results of these tests are summarized on Table 2.6-12, sorted in order of decreasing void ratio. As indicated, five of these were inundated with water in accordance with ASTM D5333–92 after the primary consolidation had occurred for pressure increments that were less than the static load expected underneath the cask storage pads and just slightly greater than the static load expected underneath Canister Transfer Building. Four of the five consolidation specimens that were inundated were samples representative of the high void ratio soils, having void ratios that ranged from 1.95 to 2.51.

Following inundation, these specimens were kept inundated throughout the remainder of the consolidation tests. If susceptible to collapse, their collapse would have been manifested at some point during the performance of the consolidation tests. All of the inundated samples acquired degrees of saturation greater than 96%, which is in excess of the typical degree of saturation (~80% per Dudley, 1970) necessary to produce collapse in most collapsible soils. However, none of these specimens exhibited vertical displacements that would be interpreted as collapse in response to wetting.

NAVFAC DM-7.1 (1982) defines "collapse potential" as the additional strain induced by inundation; i.e.,

$$CP = \frac{\Delta H_c}{H_0}\text{, expressed as a percentage.}$$

where ΔH_c is the change in height of the consolidation test specimen upon wetting and H_0 is the initial height of the specimen.

The inundation of the specimens tested typically resulted in less than ~0.1% additional vertical strain for sustained loadings of more than 800 minutes. This additional vertical strain is believed to be due to secondary compression and not soil collapse. However, even if this is considered to be collapse, the collapse potential equals only 0.1%. Figure 6 of Chapter 1 (Page 7.1-41 NAVFAC DM-7.1 (1982), entitled "Typical Collapse Potential Results," indicates that the "Severity of Problem" due to potential for collapse of soils with collapse potential of 0 to 1% is described as "No problem". Thus, these soils are not "collapsible soils".

These specimens did not collapse at any of the stress levels imposed during these tests, including those as high as 16 ksf, which is greatly in excess of the stresses (< 2 ksf) to be imposed due to the foundation loads. Comparison of the stress-strain plots of the specimens that were inundated with those that were not inundated, shows that they are nearly the same. If these soils had a tendency to collapse, this would not be the case. The inundated specimens would show increased vertical displacements if they collapsed. Therefore, based on the industry-accepted method of determining the collapse potential of soils due to wetting, these soils are not "collapsible soils".

All of the inundated specimens were obtained from the upper silty clay/clayey silt layer shown in the foundation profiles, Figure 2.6-5, Sheets 1 through 14, for the pad emplacement area, and Figures 2.6-21 through 2.6-23 for the Canister Transfer Building area. This is the stratigraphic unit at the site that exhibited the high void ratios and, hence, the low unit weights.

Figure 2.6-31 present a plot of dry densities vs depth of the subsurface soils at the site and illustrates that the upper silty clay/clayey silt layer has the lowest unit weights of the three clayey layers that exist beneath the site. Therefore, the soils in the upper layer are more likely to be collapsible soils than those in the underlying layers. As discussed above, the results of the consolidation tests performed on the upper layer indicate that they are not collapsible; therefore, the soils in the underlying layers are not collapsible, as well.

Further, these underlying soil layers are sufficiently removed from the surface (depths are > 10 to 12 ft) of the site that it is extremely unlikely they could ever become wetted due to surface waters. The overlying soils are fine-grained silty clay/clayey silts, which have very low permeabilities. In addition, the upper layer of silty clay/clayey silt will be capped by a layer of engineered soil cement. As shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), the soil cement will nominally extend 2 ft below the bottoms of most of the pads (it will be a minimum of 1 ft thick and shall have a maximum thickness of 2 ft), making it approximately 5 ft thick. The permeability of the compacted soil cement is expected to be lower than that of the underlying silty clay/clayey silt. In addition, the site is pitched to the north and the site drainage is designed to direct rain falling on the site to the Detention Pond at the northern end of the PFSF. Therefore, surface water will flow off the site to the north, along the top of the soil-cement layer, and it will not wet the underlying soils.

The results of the Probable Maximum Flood (PMF) analyses (SWEC, 1999d) demonstrate that the floodwater from the PMF will not inundate the pad emplacement area. As indicated in Section 2.4.2.2, there is no opportunity for water due to the PMF to pond within the pad emplacement area. Therefore, even if the soils were collapsible, they would not be subject to collapse due to wetting caused by the PMF.

The soils at the site have a different depositional history than the collapsible soils that are present in other parts of Utah (Rollins and Williams, 1991, "Collapsible Soil Hazard Mapping for Cedar City, Utah"). As indicated in Section 2.6.1.1, the unconsolidated deposits at the site are sediments laid down in Lake Bonneville. The collapsible soils referred to in Rollins and Williams (1991) are deposits that are formed as alluvial-fan and debris-flow sediments and in some wind-blown silts. These soils can be very susceptible to collapse upon wetting, and sometimes collapse from activities as seemingly benign as lawn watering. Note, Cedar City, Utah is located in the southwestern part of Utah, which is very far removed from the site.

In addition, there is no history or evidence of this phenomenon occurring in Skull Valley. As indicated in Section 2.6.1.1, soils at the site are described in the County Soil Report (USDA, unpublished report). The purpose of such reports is to identify the locations of various soil types and to describe their suitability for construction of septic systems, dwellings, and roads. However, there is no mention of collapsible soils in the County Soil Report applicable for Skull Valley. It is reasonable to expect that if collapsible soils of the type found in Cedar City, Utah were present in the vicinity of the site, they would be mentioned in the County Soil Report. At the minimum, there would be cautionary statements regarding the design and installation of septic systems, which discharge water to subsurface soils, and which would be subject to damage if they were constructed within or above collapsible soils.

2.6.1.11.5 Dynamic Strength of Cohesive Soils

It has been recognized in the past that the strength of cohesive soil increases as the rate of loading increases. For example, Casagrande and Shannon (1948) conducted soil dynamics investigations in 1948 with research efforts directed at finding the effects of rate of loading on soils common to the Panama Canal zone, i.e., clays, muck, shales, and dense dry sand. A "strain-rate" effect, defined as the ratio of maximum dynamic strength to the maximum static strength, was observed in all soils tested,

except for the dry sand. Tests performed on Cambridge Clay (Cambridge, MA), showed that, tested at a rapid rate of loading (0.02 sec), the strength of the clay was approximately 1.9 times greater than that measured at a slow rate of loading (465 sec). This is illustrated in Figure 2.6-24.

As reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\ dynamic} \sim 1.5 \times c_{u\ static}$.

Schimming et al (1966) studied the effects of loading rate on the strength of various soil types and defined the "apparent cohesion (c_a)" ratio to compare the dynamic and static failure envelopes of soil. Two different strain-rate strength tests were used in the study. For "dynamic" tests, the maximum shear force in soil specimens was attained within a period of 1 to 5 milliseconds after imposition of the initial force. Conversely, for "rapid static" tests, times to failure ranged from 30 seconds to nearly 50 seconds.

The c_a ratio is defined as: $c_a = c\ (\text{dynamic}) \div c\ (\text{rapid static})$.

Strength and index properties of the silty clay at the PFSF site are very similar to a soil studied by Schimming et al (i.e., Jordan Buff Clay). Average values of the index properties for both soils are as follows:

	PFSF Silty Clay	Jordan Buff Clay
Dry density (pcf)	65 (35)	86 (2)
Water content (%)	39 (117)	32 (2)
Liquid limit (LL)	51 (42)	54 (2)
Plastic limit (PL)	29 (42)	26 (2)
Plasticity index (PI)	21 (42)	28 (2)
Cohesion (psf)	1,100 (2)	1,124 (2)

Note: numbers in parentheses above indicate number of tests.

They report that the c_a ratio for this clay ranged from approximately 1.8 to 2.0, as shown in Figure 2.6-25.

Direct shear tests were performed on samples of the silty clay obtained from Elevation 4,468.4 to 4,469.4 in the Canister Transfer Building area, which is near the bottom of the foundation mat (Elevation 4,470). The results of these tests are included in Attachments 7 and 8 of Appendix 2A and they indicate that the average cohesion of these soils is ~1.1 ksf. The rate of loading used in these tests is slower than the "rapid static" tests performed by Schimming et al (1966). The rate of loading due to the design basis ground motion approximates those used for the "dynamic" tests performed by Schimming et al. To estimate the cohesion that will be available to resist these dynamic forces, the cohesion measured in the direct shear tests are multiplied by an estimated c_a ratio, which Schimming et al indicated varied between 1.8 and 2.1 for similar soils. Therefore, the cohesion available to resist forces caused by the design basis ground motion is estimated to be at least 1.5 to 2 times those measured in the direct shear tests. Stone & Webster (1995) used a similar approach for determining the dynamic strength of clays available to resist uplift loads on H-piles for Category I structures at the TVA's Sequoyah Nuclear Power Plant, Units 1 and 2.

Analyses of resistance to sliding of the Canister Transfer Building due to dynamic forces from the design basis ground motion, discussed in Section 2.6.1.12.2, do not take credit for the fact that the strength of cohesive soils increases as the rate of loading increases (Casagrande and Shannon, 1948, Das, 1993, and Schimming et al, 1966); therefore, they represent a very conservative lower-bound estimate of the sliding stability.

The dynamic foundation parameters in support of the soil-structure interaction analyses are discussed in Section 2.6.2.1.

2.6.1.12 Stability of Foundations for Structures and Embankments

All exterior footings will be founded at a depth of no less than 30 inches below finished grade to provide protection against frost, in accordance with local code requirements. Interior footings in heated areas may be founded at shallower depths, if desired.

The minimum factor of safety against a bearing capacity failure due to static loads (dead load plus maximum live loads) is 3.0.

In accordance with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and due to static loads plus loads from extreme environmental conditions, such as the design basis ground motion, is 1.1. In addition, it is recommended that a factor of safety of 1.1 be used to design footings against a bearing capacity failure from static loads plus loads due to the design basis ground motion.

If the factor of safety against sliding is less than 1 due to the design basis ground motion, additional analyses of the displacements the structure may experience are performed using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes to demonstrate that such displacements, if they did occur, would not have an adverse impact on the performance of the Important-to-Safety structures.

Recommended design earth pressure distributions are presented in Figure 2.6-7. Lateral earth pressures for determining driving forces shall be based on K_o, the at-rest earth pressure coefficient. These can be reduced to "active" earth pressures if the yield ratio exceeds 0.1%, where yield ratio, S/H, is defined as shown for the active case in Figure 2.6-8. In determining "passive" pressures resisting lateral movement, assume

the lateral earth pressure coefficient varies from K_o at a yield ratio of 0% to a maximum of K_p at a yield ratio of 2%, where yield ratio, S/H, is defined as shown for the passive case in Figure 2.6-8. Compaction-induced lateral stresses are determined as shown in Figure 2.6-9.

2.6.1.12.1 Stability and Settlement Analyses—Cask Storage Pads

Bearing Capacity

The bearing capacity of the cask storage pads was determined using the general bearing capacity equation and associated shape, depth, and inclination factors, as presented in Winterkorn and Fang (1975). Refer to Calculation 05996.02-G(B)-04 (SWEC, 2001b) for details. These analyses are based only on the strength of the silty clay/clayey silts underlying the pads. They conservatively ignore the presence of the soil cement that will be constructed adjacent to and beneath the cask storage pads (Figure 4.2-7) and the dense sand layer at a depth of ~25 to 30 ft. Even with these conservative assumptions, they demonstrate that there is an adequate factor of safety against a bearing capacity failure for both static and dynamic loadings.

As indicated in Revision 7 of Calculation 05996.02-G(B)-04 (SWEC, 2001b), the soil cement could be designed to have sufficient strength to provide, in passive resistance alone, all of the horizontal resistance required to obtain a factor of safety against sliding that exceeds the criterion (FS=1.1) for dynamic loadings. The current version of this calculation demonstrates that the soil cement beneath the pads will have sufficient shear strength to resist all of the horizontal forces due to the earthquake; therefore, the soil cement adjacent to the pads is not required to resist sliding. However, soil cement will be constructed adjacent to the pads to provide an adequate subbase for support of the cask transporter, in lieu of placing and compacting structural fill. The unconfined compressive strength of this soil cement likely will be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 in. from the ground surface). The soil

cement adjacent to the pad will resist some of the horizontal loads due to the earthquake, which will minimize the effects of the angle of inclination of the vertical load on the allowable bearing capacity. The allowable bearing capacity is inversely related to the angle of inclination of the load, and it is markedly reduced for the inclination angles applicable for the dynamic horizontal loads from the design basis ground motion. The presence of the soil cement, therefore, will greatly enhance the bearing capacity of these foundations.

These analyses included determination of factors of safety against a bearing capacity failure of the foundation due to static loads and due to static plus dynamic loads from the design basis ground motion (PSHA 2,000-yr return period earthquake). The dynamic bearing capacity analyses are discussed in detail in the section below titled *"Dynamic Bearing Capacity of the Cask Storage Pads."*

Static Bearing Capacity of the Cask Storage Pads

Table 2.6-6 presents the results of the bearing capacity analyses for the following static load cases. As indicated above, the minimum factor of safety for these static load cases is 3.

Case IA Static using undrained strength parameters (ϕ = 0° & c = 2.2 ksf).

Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).

As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 4 ksf. However, loading the storage pads to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively use ϕ = 0° and c = 2.2 ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A, to model the end of construction. Using the estimated effective-stress strength of ϕ = 30° and c = 0 results in a higher allowable bearing pressure. As shown in Table 2.6-6, the gross allowable bearing capacity of the cask storage pads for static loads for the effective-stress soil strength is greater than 9 ksf.

Effect of Cohesionless Soils Underlying the Cask Storage Pads on Bearing Capacity

Unconsolidated-undrained triaxial test results included in Attachment 1 of Appendix 2A indicate that 2.2 ksf is a reasonable lower-bound value to use for bearing capacity analyses of the undrained conditions to represent the end-of-construction case. These triaxial tests were performed at confining pressures comparable to the in situ vertical stresses existing near the middle of the upper layer prior to the construction of the pads, thus, provide a realistic estimation of the minimum strength that will be available for resisting a bearing capacity failure at the end of construction.

As indicated in Section 2.6.1.6, based on the CPT program, most of the soils underlying the pad emplacement area are mischaracterized as soils that behave as "sandy" soils, rather than as cohesive soils. These soils were found to be mostly cohesive soils in the borings that were drilled in 1996, as indicated in Attachment 1 of Appendix 2A. The soil behavior types reported in ConeTec (1999) were determined based on correlations developed from testing saturated, uncemented soils. The soils at the site are partially saturated and cemented; thus, the soil behavior types determined from the cone penetration test data must be recalibrated to agree with the soil classifications determined based on samples obtained in the borings and tested in the laboratory.

Figure 2.6-30, Sheets 1 through 6, present comparisons of the boring and laboratory soil classifications plotted vs elevation alongside the soil behavior type data from nearby cone penetration tests. The differences shown under the column labeled ΔSBT represent the SBT zoning shift required to more correctly characterize these soils as a result of the effects of partial saturation and cementation, as discussed above. Evaluation of these ΔSBT values leads to the conclusion that the soil behavior type values reported in ConeTec (1999) that are greater than 5 (i.e., sandier soils), as well as some of those equal to 5 (i.e., clayey silts), typically should be adjusted downward one or two zones to more accurately reflect the soil classifications that were determined

based on the borings and confirmed by the laboratory tests performed specifically for the purpose of classifying the soil types. Conservatively adjusting these data by subtracting 1 from the SBTs that were reported to be greater than 5 (i.e., "sandy" soils), as discussed in Section 2.6.1.6, results in the SBTs presented on the pad emplacement area foundation profiles, Sheets 2 through 14 of Figure 2.6-5. As shown on these figures, the subsurface soils that were reported in ConeTec (1999) as being silty sand/sand and sands are more correctly described as silts with some sandy silts. The following discussion is included to demonstrate that even if these soils are cohesionless soils, the factor of safety against a bearing capacity failure is much greater than that reported above for the clayey soils identified in the borings.

Whereas the bearing capacity of cohesive soils is a function of the strength of the soil, that of cohesionless soils is also a function of the width of the foundation. The foundations in question for this project have widths that are 30 ft or greater. Such large foundations, supported by soils having Standard Penetration Test blow counts that were measured for these soils, have much greater bearing capacities if they are founded on cohesionless soils than if supported by undrained cohesive soils. Therefore, characterizing the soils in the upper layer as cohesive even though some of these may be cohesionless provides a conservative estimate of the bearing capacity.

Analyses of bearing capacity were made in Calculation 05996.02-G(B)-4, (SWEC, 2001b), based on the assumption that the entire upper layer, approximately 25 to 30-ft thick, was comprised of cohesive soils similar to those tested at depths of 10 to 12 ft. In these analyses, the strength of the soils in the entire upper layer (~25 to 30-ft thick) was set equal to the minimum value measured in the UU tests (s_u = 2.2 ksf) that were performed on samples obtained from depths of approximately 10 to 12 feet. As indicated on Table 2.6-6 for Case IA, the factor of safety of the cask storage pad foundation is 7.0 using this undrained strength for the cohesive soils. The results for

Case IB Table 2.6-6 illustrates that the factor of safety against a bearing capacity failure increases to greater than 15 when the effective-stress strength of ϕ = 30° is used.

The friction angle used in the effective-stress strength analyses discussed above is less than the friction angle shown for the soils that behave as sandy soils (SBT>5) based on the CPT data presented in Appendix D of ConeTec (1999). These plots illustrate that most of the "Phi" values are between 35° and 40° for these soils, with very few values that are slightly less than 35°. Therefore, assuming that all of the soils underlying the cask storage pads are cohesionless, as represented by the preponderance of soils that behave as "sandy" soils based on the uncorrected CPT SBT data, the factor of safety against a bearing capacity failure will be much greater than 15.

Static Settlements of the Cask Storage Pads

Analyses were performed to estimate the maximum total settlement of the cask storage pads as a result of the weight of the pad and the weight of eight, fully loaded, Holtec HI-STORM casks (356.5 K) in Calculations 05996.02-G(B)-3 (SWEC, 1999e) and 05996.03-G(B)-21 (SWEC, 2001a). The actual bearing pressure for this case was about 1.9 ksf, and the estimated total settlement of the pad was determined to be about 1.7 inches. The maximum total settlement consists of the following three components:

• Elastic settlement	0.5 inches
• Primary consolidation settlement	0.8 inches
• Secondary compression	0.4 inches
• Maximum total settlement	1.7 inches

The maximum differential settlement between the center of the crushed rock aisle and the center of the storage pads is 1-1/2 inches, when the 0.25 inches of immediate settlement of the pad emplacement area is removed. The estimated settlement of the storage pads at the long edge of the pads is approximately half that value, or 3/4 inch.

The maximum differential settlement between the long edge of the storage pads and the center of the crushed rock aisle between the storage pads is therefore less than or equal to 3/4 inch (SWEC, 2001a).

The crushed rock surface materials will be installed flush with the top of the storage pads and removed as required in order to accommodate the actual settlement of the pads. Exposed edges of the pads will be chamfered and the compacted aggregate surface material will be feathered to meet the edges of the raised pads for transporter access, as shown in Figure 4.2-7.

This settlement represents an upper-bound estimate of the total compression, because it was developed assuming that the consolidation characteristics that were measured for the clayey soils at a depth of about 10 ft are applicable for the entire upper layer (~25 to 30 ft). The SPT data from the borings and the CPT results indicate that the soils become stiffer within the 10 to 20 ft depth zone. Additional consolidation tests performed on samples obtained from depths of about 25 ft in the Canister Transfer Building area, reported in Attachment 6 of Appendix 2A, indicate that the soils at that depth are less compressible than those used to estimate the settlements presented above. Further, based on the CPT program, most of the soils underlying the pad emplacement area are characterized as soils that behave as "sandy" soils, rather than as cohesive soils. Such soils are much less compressible than the clayey soils described above. Therefore, assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured at a depth of 10 to 12 ft conservatively overestimates the expected settlements.

<u>Effect of Cohesionless Soils Underlying the Cask Storage Pads on Settlements</u>

As discussed above, the soil behavior types determined from the cone penetration test data and reported in ConeTec (1999) must be recalibrated to agree with the soil classifications determined based on samples obtained in the borings and tested in the

laboratory. Figure 2.6-30, Sheets 1 through 6, present comparisons of the boring and laboratory soil classifications plotted vs elevation alongside the soil behavior type data from nearby cone penetration tests. These figures illustrate that the soil behavior type values reported in ConeTec (1999) that are greater than 5 (i.e., sandier soils), as well as some of those equal to 5 (i.e., clayey silts), typically should be adjusted downward one or two zones to more accurately reflect the soil classifications that were determined based on the borings and confirmed by the laboratory tests performed specifically for the purpose of classifying the soil types. The following discussion is included to demonstrate that even if these soils are cohesionless, the estimated settlements will be much less than those reported above assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured in consolidation tests performed on samples obtained at a depth of 10 to 12 ft.

A review of the CPT data (ConeTec, 1999) indicates that most of the soil behavior type (SBT) values represent soils whose behavior is similar to that of "sandy" soils. As indicated in Figure 5 of ConeTec (1999), these include SBT values that are greater than 5. A map was produced to show the thickness of those soils for which the soil behavior type values are greater than 5. The purpose of this map is to readily identify those areas where the subsurface profile differs from the assumption that the soils in the upper layer (~25 to 30 ft) are predominantly cohesive soils.

This map, titled "Contour Map Showing Thickness of Soils with CPT Soil Behavior Type > 5 (Sandy)", is included as Figure 2.6-29. The thickness of the soils beneath the cask storage pads that behave as "sandy" soils based on the CPT data are posted under the CPT identifiers shown on this plan view of the site. These values were calculated by subtracting the top three feet, to account for the proposed depth of the pads, as well as the total thickness of all zones where the SBT values were found to be less than 6, from the total depth of the CPT. The thicknesses were contoured to facilitate interpretation of the SBT > 5 data obtained in the CPT program. As indicated in the figure, the

thickness of the soils that behave as sandy soils (SBT>5) based on the CPT data ranges from 13.8 feet at CPT-15, near the center of the pad emplacement area, to a

high of 26.4 feet at CPT-33 near the center of the western edge of the pad emplacement area. The thicknesses are generally about 20 to 25 feet.

Calculation 05996.02-G(B)-3 (SWEC, 1999e) incorporated the calculation of settlements for the soils whose behavior is similar to that of "sandy" soils based on the CPT data. In this analysis, settlements are calculated based on Equation 6-17 of Lunne, Robertson, and Powell (1997), which was developed by Schmertmann (1970, 1978). This method is applicable for estimating settlements of foundations over sand using CPT data. The Schmertmann method takes into account the depth of footing, time of loading (40 years was used in the analysis), shape of the footing, and strain influence factor, which varies with depth. The equivalent Young's modulus, which appears in the equation, is related to the cone penetration resistance by a factor, α. This factor is related to the degree of loading, soil density, stress history, cementation, age, grain shape, and mineralogy of the deposit. In this analysis, α was assumed to be 5, which is in the middle of the range recommended in ConeTec (1998) for aged (>1,000 years) normally consolidated sands.

Two sets of estimated settlements were calculated and are summarized in the table presented on Page 44 of the calculation. Because of the preponderance of soils whose behavior is similar to that of "sandy" soils based on the uncorrected CPT soil behavior type data, settlements were calculated assuming that the Schmertmann method is applicable to the entire upper layer. As indicated by the left-hand column of settlements reported on Page 44 of the calculation, the estimated settlements for this case varied from 0.34 inches at CPT-26 to 0.56 inches at CPT-38.

The analyses were repeated, excluding those soils whose behavior is not similar to "sandy" soils, since the Schmertmann method is applicable only for cohesionless soils. In this analysis, cohesionless soils were defined as those with SBT values greater than

5, which includes silts, sandy silts, silty sands, and sands. The estimated settlements for this case are presented in the right-hand column on Page 44 of the calculation and range from 0.24 inches at CPT-31 to 0.50 inches at CPT-10.

These results are posted on the map showing the locations of the CPTs on Page 46 of the calculation. As indicated, the differential settlements between CPT locations average less than 0.1 inches. The maximum difference between two adjacent (diagonally) CPTs is 0.19 inches, CPT-34 to CPT-29. Total and differential settlements of this magnitude are not significant in the design or performance of the cask storage pads. These results confirm that if the soils are actually "sandy" soils, as indicated by the uncorrected SBTs from the cone penetration testing (ConeTec, 1999), then the estimated settlements will be much less than those reported above assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured in consolidation tests performed on samples obtained at a depth of 10 to 12 ft.

Dynamic Bearing Capacity of the Cask Storage Pads

The dynamic bearing capacity of the cask storage pads was analyzed in Calculation 05996.02-G(B)-4 (SWEC, 2001b) using two different sets of dynamic forces. The dynamic forces used in the first set of analyses were the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The second set of analyses used the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. These latter dynamic forces represent the maximum force occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad. As in the structural analyses discussed in Section 4.7.1.5.3, "Structural Analysis," the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations

(ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-7 presents the results of the bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the undrained strength that was measured in unconsolidated- undrained triaxial tests (ϕ = 0° and c = 2.2 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

As indicated in Table 2.6-7, the minimum gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion is 4.8 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads were assumed to act in both horizontal directions at the same time and the vertical earthquake load was assumed to be 0. Combining the three components of the earthquake in accordance with ASCE (1986), the minimum allowable value was obtained for Load Case IIIB, wherein 100% of the earthquake loads act in the E-W direction, 40% acts in the N-S direction, and 40% acts in the vertical direction. The actual factor of safety for this condition is 1.2, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

In these dynamic bearing capacity analyses, the dynamic forces were based on the inertial forces due to the earthquake. The total vertical force shown in Table 2.6-7 includes the static weight of the pad and 8 fully loaded casks ± the vertical inertial forces due to the earthquake. The vertical inertial force is calculated as a_V x [pad + cask dead loads], multiplied by the appropriate factor (±40% or ±100%) for the load case. In these analyses, the minus sign for the percent loading in the vertical direction signifies uplift forces, which tend to unload the pad. Similarly, the horizontal inertial forces are calculated as a_H x [pad + cask dead loads], multiplied by the appropriate factor (40% or 100%) for the load case. The horizontal inertial force from the casks was confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad considered in the HI-STORM cask stability analysis ($\mu = 0.8$, as shown in Section 8.2.1.2, Accident Analysis) x the normal force acting between the casks and the pad.

The lower-bound friction case (discussed in Section 4.2.3.5.1B), wherein μ between the steel bottom of the cask and the top of the concrete storage pad = 0.2, results in lower horizontal forces being applied at the top of the pad. This decreases the inclination of the load applied to the pad, which results in increased bearing capacity. Therefore, bearing capacity analyses are not performed for $\mu = 0.2$ in Calculation 05996.02-G(B)-04 (SWEC, 2001b).

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from

static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is at least 10.5 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value was obtained for the 8-cask loading. The actual factor of safety for this case was 1.6, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

As indicated above, these maximum dynamic cask driving forces represent the upper bound of the dynamic forces that could act at the base of the pad. The horizontal forces from the casks were confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction acting at the base of the cask for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in Section 8.2.1.2) x the normal force acting between the casks and the pad. These maximum dynamic cask driving forces can be transmitted to the pad through friction only when the inertial vertical forces act downward; therefore, these analyses are performed only for Load Case IVA. The width (30 ft) is less in the E-W direction than the length N-S (67 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

Because of the nature of the subsurface materials, dynamic settlements due to the design basis ground motion are not expected to occur. See Section 2.6.4.7 for more details.

<u>Overturning Stability of the Cask Storage Pads</u>

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The resisting moment is calculated as the net effective weight of the pad and casks x the distance from one edge of the pad to the center of the pad in the direction of the minimum width. The net effective weight of the pad and casks is the static weight of pad and casks plus and minus the inertial force due to earthquake. The weight of the pad is calculated as 3 ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 905 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

$$\Sigma M_{Resisting} = (1-0.695) \times [905\text{ K} + 2{,}852\text{K}] \times 15\text{ ft} = 17{,}188\text{ ft-K}.$$

The driving moment includes the moments due to the horizontal inertial force of the pad x ½ the height of the pad and the horizontal force from the casks acting at the top of the pad x the height of the pad. The casks are simply resting on the top of the pads; therefore, this force cannot exceed the friction force acting between the steel bottom of the cask and the top of the concrete storage pad. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in Section 8.2.1.2) x the normal force acting between the casks and the pad. This force is maximum when the vertical inertial force due to the earthquake acts downward. However, when the vertical force from the earthquake acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the pad.

When the vertical inertial force due to the earthquake acts upward, the friction force between the casks and the storage pad = 0.8 x (2,852K - 0.695 x 2,852K) = 696 K. This is less than the maximum dynamic cask horizontal driving force of 2,212 K (Table D-1(c) in CEC, 2001). Therefore, the worst-case horizontal force that can occur when the vertical earthquake force acts upward is limited by the upper-bound value of the

coefficient of friction between the bottom of the casks and the top of the storage pad, and it equals 696K. Recalling that the pad height is 3 ft,

$$\Sigma M_{Driving} = 1.5\text{ ft} \times \underset{a_h}{0.711} \times \underset{Wp}{905\text{ K}} + 3\text{ ft} \times \underset{EQhc}{696\text{ K}} = 3{,}050\text{ ft-K.}$$

Therefore, $FS_{OT} = 17{,}188 \div 3050 = 5.63$

This is greater than the factor of safety criterion of 1.1; therefore, the cask storage pads have an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Sliding Stability of the Cask Storage Pads

The sliding stability analyses of the cask storage pads are presented in Calculation 05996.02-G(B)-4 (SWEC, 2001b). These pads will be constructed on and within soil cement, as illustrated in Figure 4.2-7 and described in Sections 2.6.1.7 and 2.6.4.11. The following section discusses the sliding stability of these pads embedded in soil cement and demonstrates that embedding them in soil cement will greatly enhance their resistance to sliding due to dynamic loads from the design basis ground motion.

Subsequent sections demonstrate that sliding will not occur along deeper surfaces within the profile underlying the cask storage pads. First, the sliding resistance of the *in situ* silty clay/clayey silt layer is addressed to demonstrate that sliding will not occur along the interface between the bottom of the soil cement and those soils. As shown in the pad emplacement area foundation profiles (Figure 2.6-5), a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, the subsequent section addresses the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These analyses demonstrate there is an adequate factor of safety against sliding of the cask storage pads and along deeper surfaces beneath the storage pads due to the maximum loadings of design basis ground motion.

Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement

The cask storage pads will be constructed on and within soil cement, as shown in Figure 4.2-7. The analysis of sliding stability of the cask storage pads embedded in soil cement is included in Calculation 05996.02-G(B)-4 (SWEC, 2001b). This analysis demonstrates that the static, undrained strength of the in situ clayey soils is sufficient to preclude sliding (FS = 1.27 vs minimum required value of 1.1), provided that the full strength of the clayey soils is engaged.

The soil-cement layer beneath the pads provides an "engineered mechanism" to ensure that the full, static, undrained strength of the clayey soils is engaged in resisting sliding forces. This soil cement will be designed to have a minimum unconfined compressive strength of 40 psi. The bond between this soil-cement layer and the base of the concrete pad will be stronger than the static, undrained strength of the in situ clayey soils. The factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement is greater than 1.98, which exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. Therefore, the minimum factor of safety against sliding of the overall cask storage pad design is at least 1.27.

As indicated in Figure 4.2-7, the soil cement will extend at least 1 ft below all of the cask storage pads. As shown in Figure 2.6-5, the pad emplacement area foundation profiles, it typically will nominally extend 2 ft below the bottoms of the pads (it will be a

minimum of 1 ft thick and shall have a maximum thickness of 2 ft). Shear resistance will be transferred through the approximately 2-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; thus, the area available to resist sliding will greatly exceed that of the embedded portion of the pads alone, as was used in the analysis described above. Shear resistance requirements at the soil cement/clayey silt interface, therefore, will be lower than those required to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

The soil cement will have higher shear strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the shear strength of the silty clay/clayey silt. Direct shear tests on samples of the soils from the pad emplacement area (presented in Attachments 7 and 8 of Appendix 2A) indicate the shear strength available to resist sliding from loads due to the design basis ground motion is 2.1 ksf, as shown in Figure 7 of Calc 05996.02-G(B)-5 (SWEC, 2000c). The following section indicates that there is an adequate factor of safety against sliding of the pads, postulating that they are constructed directly on the silty clay/clayey silt and neglecting the passive resistance provided by the soil cement that will be surrounding the pads. The factor of safety against sliding along the soil cement/silty clay interface will be much greater than this, because the shearing resistance will be available over the areas between the pads, as well as under the pads, and additional passive resistance will be provided by the continuous soil cement layer existing below the pads. Therefore, the soil cement will greatly improve the sliding stability of the cask storage pads.

Since the resistance to sliding of the cask storage pads is provided by the strength of the bond at the interface between the concrete pad and the underlying soil cement and by the bond between the soil cement under the pad and the in situ clayey soils, the sliding stability of the pads at the end of each column or row of pads are no

different than that of the other pads. Therefore, the pads along the perimeter of the pad emplacement area also have an adequate factor of safety against sliding. Further, the soil-cement layer is continuous throughout the pad emplacement area; therefore, the area available to resist sliding of an entire column of pads greatly exceeds the sum of the areas of only the pads in the column. The factor of safety against sliding of an entire column of pads will, therefore, exceed that of an individual pad.

Sliding Stability of the Interface Between the Soil Cement and the Silty Clay/Clayey Silt Underlying the Cask Storage Pads

The sliding stability of the interface between the soil cement and the in situ silty clay/clayey silt layer underlying cask storage pads is presented in Calculation 05996.02-G(B)-4 (SWEC, 2001b). The sliding stability of this interface is demonstrated by ignoring the presence of the soil cement and demonstrating that the factor of safety against sliding of the pads supported directly on the in situ clayey soils is 1.27, which provides an adequate margin against sliding. As discussed above, the soil cement will distribute the loads from the earthquake deeper into the profile, spreading them out over an area that is much greater than that of the pads. Thus, the shear resistance requirements at the bottom of the soil cement will be less than would be required if the pads were constructed directly on the clayey soils. Therefore, sliding will not occur along the interface between the soil cement and the in situ clayey soils.

In these analyses, the factor of safety (FS) against sliding is defined as:

$$FS = \text{resisting force} \div \text{driving force}$$

The resisting force, or tangential (T) shear force, below the base of the pad is defined as:

$$T = N \tan \phi + c\,B\,L$$

where, N = normal force

ϕ = 0° (for Silty Clay/Clayey Silt)

c = 2.1 ksf, as indicated on Figure 7 of SWEC (2000c) for normal stress equal to the vertical stress at the bottom of the fully loaded pads

B = 30 feet

L = 67 feet

Material directly under and around the pad will be soil cement. In this analysis, however, the presence of the soil cement adjacent to the sides of the pads is ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads along the interface between in situ clayey soils and bottom of soil cement beneath the pads. The potential failure mode is sliding along the surface at the base of the pad. No credit is taken for the passive resistance acting on the sides of the pad above the base. This analysis is applicable for any of the pads at the site, including those at the ends of the rows or columns of pads, since it relies only on the strength of the material beneath the pads to resist sliding.

This analysis conservatively assumes that 100% of the dynamic forces due to the earthquake act in both the horizontal and vertical directions at the same time. The length of the pad in the N-S direction (67 ft) is greater than twice the width in the E-W direction (30 ft); therefore, the dynamic active earth pressures acting on the length of the pad will be greater than those acting on the width, and the critical direction for sliding will be E-W, since passive resistance is ignored.

The soil cement is assumed to have the following properties in calculation of the dynamic active earth pressure acting on the pad from the soil cement above the base of the pad:

γ = 100-110 pcf Initial results of the soil-cement testing indicate that 110 pcf is a reasonable lower-bound value for the total unit weight of the soil cement adjacent to the pads and that 100 pcf is a reasonable

lower-bound value for the total unit weight of the cement-treated soil to be placed beneath the pads.

ϕ = 40° Tables 5 & 6 of Nussbaum & Colley (1971) indicate that ϕ exceeds 40° for all A-4 soils (CL & ML, similar to the eolian silts at the site) treated with cement; therefore, it is likely that ϕ will be higher than this value. This value is not used, however, in this analysis for calculating sliding resistance. It also is used in this analysis only for determining upper-bound estimates of the active earth pressure acting on the pad due to the design basis ground motion.

H = 3 ft As shown in Figure 4.2-7, the pad is 3 ft thick, and it is constructed such that top of the pad is at the final ground surface (i.e., pads are embedded 3' below grade). Soil cement beneath the pad is 1-ft to 2-ft thick. The dynamic forces (active earth pressure + horizontal inertial forces) are greater for deeper depth of soil cement. Therefore, analyze for 2 ft of soil cement beneath the pad. The depth of the pad is used in this analysis for calculating the maximum dynamic lateral earth pressure.

The value of c is based on the results of the direct shear tests that were performed on specimens obtained from a depth of 5 to 6.3 ft from Sample U-1 in Boring C-2, which was drilled in the pad emplacement area. These test results are consistent with the results obtained from the direct shear tests that were performed on samples obtained from within the Canister Transfer Building area (Borings CTB-6 and CTB-S). All of these direct shear test results are reported in Attachments 7 and 8 of Appendix 2A.

The sliding stability was checked for Load Cases III and IV, conservatively assuming that 100% of the dynamic forces due to the earthquake act in both the N-S and Vertical directions at the same time. In determining the resisting forces in these analyses, no credit is taken for passive resistance acting on the embedded pad. The resulting factor of safety was ~2.3 for Load Case III and 1.27 for Load Case IV, which provides an adequate margin against sliding.

The horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force. For Load Case III, the maximum frictional force was much less than the maximum cask driving force. Therefore, the sliding stability also was checked for Load Case IV, which has the dynamic forces due to the earthquake acting downward. With the earthquake force acting downward, the frictional force that can be transmitted from the casks to the top of the pad is large enough to transmit the maximum cask driving force.

The horizontal driving force in these analyses includes the inertial forces of the pad and the soil cement beneath the pad and the maximum cask driving forces reported in Calculation 05996.02-G(PO17)-2 (CEC, 2001) that are due to the PSHA 2,000-yr return period earthquake. The dynamic loads due to soil pressures acting on the embedded pad were also included, calculated based on the Mononobe-Okabe method, as described in Seed and Whitman (1970). The driving forces were calculated based on the peak vertical and peak horizontal accelerations; i.e., no credit was taken for the fact that these peaks are not expected to occur at different times.

The driving force due to dynamic active earth pressures acting on the pad in the E-W direction is greater than that acting in the N-S direction, because the dimension of the pad in the N-S direction (67 ft) is greater than twice the width in the E-W direction (30 ft). The maximum dynamic cask driving force also acts in the E-W direction; however,

the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force. Therefore, ignoring passive resistance, sliding will be more critical in the E-W direction.

As indicated above, these analyses are very conservative for a number of reasons. They combine the maximum horizontal and vertical forces of the earthquake, rather than using reduced values to account for the fact that the peaks in these motions are not expected to occur at the same time. They also conservatively use the shear strength measured in the static direct shear tests; i.e., no credit is taken for the increase in this strength that is applicable for dynamic loadings, as discussed in Section 2.6.1.11.5, "Dynamic Strength of Cohesive Soils." Therefore, these analyses yield lower-bound factors of safety against sliding where the pads are supported on clayey soils. Even with these conservative assumptions, the factor of safety against sliding exceeds the minimum allowable value of 1.1; therefore the pads overlying 2 ft of soil cement are stable with respect to sliding, assuming the strength of the soil cement underlying the pad is at least as high as the undrained strength of the underlying soils.

These analyses illustrate that if the cask storage pads were constructed directly on the silty clay/clayey silt layer, they would have an adequate factor of safety against sliding due to loads from the design basis ground motion. Because the soil cement is continuous between the pads, its interface with the silty clay will be much larger than that provided by the footprint of the pads and used in the analyses described above. The soil cement will be mixed and compacted into the surface of the silty clay, providing a bond at the interface that will exceed the strength of the silty clay. Therefore, this interface will have more resistance to sliding than is included in these analyses and, thus, there will be adequate resistance at this interface to preclude sliding of the pads due to the loads from the design basis ground motion.

Adhesion between the Base of Pad and Underlying Clayey Soils

The analysis described above demonstrates that the static undrained strength of the soils underlying the pads is sufficient to preclude sliding of the cask storage pads over 2 ft of soil cement for the 2,000-yr return period earthquake with a peak horizontal ground acceleration of 0.711g, conservatively ignoring the passive resistance acting on the sides of the pads. This analysis assumes that the full static undrained strength of the clay is engaged to resist sliding. To obtain the minimum factor of safety required against sliding of 1.1, 76% (= 1.60 ksf (required for FS=1.1) ÷ 2.1 ksf available) of the undrained shear strength must be engaged, or in other words, the adhesion factor between the base of the concrete storage pads plus 2 ft of soil cement and the surface of the underlying clayey soils must be 0.76. This adhesion factor, c_a, is higher than would normally be used, considering disturbance that may occur to the surface of the subgrade during construction. Therefore, an "engineered mechanism" is required to ensure that the full strength of the clayey soils is available to resist sliding of these pads on 2 ft of soil cement.

Ordinarily, a foundation key would be added to extend the shear plane below the disturbed zone and to ensure that the full strength of the clayey soils are available to resist sliding forces. However, adding a key to the base of the storage pads would increase the stiffness of the foundation to such a degree that it would exceed the target hardness limitation of the hypothetical cask tipover analysis. Therefore, PFS decided to construct the cask storage pads on (and within) a layer of soil cement constructed throughout the entire pad emplacement area.

As shown in Figure 4.2-7, the soil cement will extend to the bottom of the eolian silt or a minimum of 1 ft below the base of the storage pads and up the vertical face at least 2 ft. In the sliding stability analysis, it is required that the following interfaces be strong enough to resist the sliding forces due to the design earthquake. Working from the bottom up, these include:

1. The interface between the in situ clayey soils and the bottom of the soil cement, and
2. The top of the soil cement and the bottom of the concrete storage pad.

The purpose of soil cement below the pads is to provide the "engineered mechanism" required to effectively transmit the sliding forces down into the underlying clayey soils. The techniques used to construct soil cement are such that the bond between the soil cement and the underlying clayey soils will exceed the undrained strength of the underlying clayey soils.

The shear strength available at each of the interfaces applicable to resisting sliding of the cask storage pads will exceed the undrained strength of the underlying clayey soils. PFS has committed to performing laboratory tests during the design of the soil cement to demonstrate that the required shear strengths can be achieved at the various interfaces, and PFS has committed to performing field tests during construction to demonstrate that the required shear strengths at these interfaces have been achieved.

The soil cement beneath the pads is used as an "engineered mechanism" to ensure that the full static undrained shear strength of the underlying clayey soils is engaged to resist sliding and, as shown above, the minimum factor of safety against sliding of the pads is very conservatively calculated as 1.27 when the static undrained strength of the clayey soils is fully engaged. This value exceeds the minimum value required for the factor of safety against sliding (=1.1); therefore, the pads constructed on top of a layer of soil cement have an adequate factor of safety against sliding. See Section 2.6.4.11 for additional details concerning the use of soil cement.

Limitation of Strength of Soil Cement Beneath the Pads

As indicated in Figure 4-2-7, the soil cement will extend at least 1 ft below the storage pads over the entire pad emplacement area, and, as shown in Figures 2.6-5, Pad Emplacement Area Foundation Profiles, it will typically extend ~2 ft below most of the

pads. Thus, the area available to resist sliding will greatly exceed that of the pads alone. The hypothetical cask tipover analysis imposes limitations on the modulus of elasticity of the soils underlying the pad. The modulus of elasticity of the soil cement is directly related to its strength; therefore, its strength must be limited to values that will satisfy the modulus requirement, but it must still provide an adequate factor of safety with respect to sliding of the pads embedded within the soil cement.

Table 5-6 of Bowles (1996) indicates E = 1,500 s_u, where s_u = the undrained shear strength. Note, s_u is half of q_u, the unconfined compressive strength.

Based on this relationship, E = 750 q_u,

Where E = Young's modulus

q_u = Unconfined compressive strength

An unconfined compressive strength of 100 psi for the soil cement under the pad will limit the modulus value to 75,000 psi. Thus, designing the soil cement to have an unconfined compressive strength that ranges from 40 psi to 100 psi will provide an adequate factor of safety against sliding and will limit the modulus of the soil cement under the pads to an acceptable level for the hypothetical cask tipover considerations.

<u>Sliding Along Contact Between the Concrete Pad and the Underlying Soil Cement</u>

The soil cement will be designed to have an unconfined compressive strength of at least 40 psi to ensure that it will be stronger than required to provide a factor of safety against sliding that exceeds the required minimum value of 1.1. The shear strength equals half of the unconfined compressive strength, 20 psi, which equals 2.88 ksf. Therefore, the resistance to sliding between the concrete storage pad and the top of the soil cement layer beneath the pad will be greater than:

$$\begin{array}{ccccccc} & N & \phi & c & B & L & T \end{array}$$

$$T = 6{,}368\ K \times \tan 0° + 2.88\ ksf \times 30\ ft \times 67\ ft = 5{,}789\ K$$

As indicated above, the driving force, V, is defined as: V = F_{AE} + EQhp + EQhcThe factor of safety against sliding between the pad and the surface of the underlying soil cement is calculated as the resisting force ÷ the driving force, as follows:

$$\underset{T}{FS_{\text{Pad to Soil Cement}} = 5{,}789\text{ K}} \div (\underset{F_{AE\ E\text{-}W}}{65.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{EQhc_{E\text{-}W}}{2{,}212\text{ K}}) = \mathbf{1.98}$$

$$(2{,}920.3\text{ K})$$

Thus, designing the soil cement to have an unconfined compressive strength of at least 40 psi results in an acceptable factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement that exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. In other words, the soil cement will have higher strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the strength of the silty clay/clayey silt.

Soil cement with strengths higher than this are readily achievable, as illustrated by the lowest curve in Figure 4.2 of ACI 230.1R-90, which applies for fine-grained soils similar to the eolian silt in the pad emplacement area. Note, f_c = 40C where C = percent cement in the soil cement. Therefore, to obtain f_c >40 psi, the percentage of cement required would be ~40/40 = 1%. This is even less cement than would typically be used in constructing soil cement for use as road base. The resulting material will more likely be properly classified as a cement-treated soil, rather than a true soil cement. Because this material is located below the frost zone (which is only 30" below grade at the site), it does not need to comply with the durability requirements of soil cement; i.e., ASTM freeze/thaw and wet/dry tests. The design of the mix for this material will require that the unconfined compressive strength of this layer of material will exceed 40 psi to ensure that the shear strength available to resist sliding of the concrete pads exceeds the shear strength of the in situ clayey soils.

Soil Cement Above the Base of the Pads

Soil cement also will be placed between the cask storage pads, above the base of the pads. Revision 7 of Calculation 05996.02-G(B)-4 (SWEC, 2001b), demonstrated that this soil cement could be designed such that its compressive strength alone would be sufficient to resist all of the sliding forces due to the design earthquake. However, as shown above, this soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter. The eolian silt, otherwise, would be inadequate for this purpose and would require replacement with imported structural fill. The soil cement surrounding the pad may also help to spread the seismic load into the clayey soil outside the pad area to engage additional resistance against sliding of the pad. This effect would result in an increase in the factor of safety against sliding.

The unconfined compressive strength of the soil cement adjacent to the pads needs to be at least 50 psi to provide an adequate subbase for support of the cask transporter, in lieu of placing and compacting structural fill, but it likely will be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 in. from the ground surface).

The beneficial effect of this soil cement on the factor of safety against sliding is demonstrated on page 29 and 30 of Calculation 05996.02-G(B)-4 (SWEC, 2001b). The factor of safety against sliding in the north-south direction increases from 1.5 to 2.3 when the passive resistance of the soil cement above the base of the pads is included, and the factor of safety against sliding in the east-west direction increases to

3.3. Therefore, adding the soil cement adjacent to the pads does enhance the sliding stability of each pad.

Sliding Stability of the Cask Storage Pads on Cohesionless Soils

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion have been obtained for the storage pads founded directly on the silty clay/clayey silt layer, conservatively ignoring the passive resistance of the soil cement that will be placed under and adjacent to the pads. The shearing resistance is provided by the cohesive portion of the shear strength of the silty clay/clayey silt layer, which is not affected by upward earthquake loads. As shown in Figure 2.6-5, Pad Emplacement Area – Foundation Profiles, a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, to be conservative, the sliding stability of the cask storage pads was analyzed assuming that the soils in this layer are cohesionless, ignoring the effects of cementation that were observed on many of the split-spoon and thin-walled tube samples obtained in the drilling programs.

The CPT results (ConeTec, 1999) indicate the presence of a layer of soils that behave like silty sands and sands under the clayey layer at a depth of about 10 ft. Note, however, that recalibrating the SBTs as discussed in Section 2.6.1.6 results in most of these silty sands and sands being more correctly identified as clayey silt/silt with some sandy silt, as shown in Sheets 2 through 14 of Figure 2.6-5. The plots included in Appendix D of ConeTec, 1999) indicate that s_u, the undrained shear strength, or the

cohesion, drops to 0 and that ϕ is generally greater than 35 to 40° for these soils. If the cohesion available to resist sliding drops to 0 and cementation effects are ignored, the shearing resistance of this layer is directly related to the normal stress.

Analyses were performed to address the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces. To simplify the analysis, it was assumed that cohesionless soils extend above the 10 ft depth and, thus, the pads are founded directly on cohesionless materials. In this analysis, a friction angle of 30° was used to define the strength of the soils to conservatively model a loose cohesionless layer, even though the values measured in the CPTs generally were greater than of 35 to 40°. Without cohesion and ignoring passive resistance acting against the side of the pad, the resistance to sliding is calculated as N tan 30°, or 0.58 N, where N is the normal force. Because of the magnitude of the peak ground accelerations (0.695g) due to the design basis ground motion at this site, the frictional resistance available when N is reduced due to the uplift from the inertial forces applicable for the vertical component of the design basis ground motion is not sufficient to resist sliding. However, analyses were performed to estimate the amount of displacement that might occur due to the design basis ground motion for this case. These analyses, based on the method of estimating displacements of dams and embankments during earthquakes developed by Newmark (1965), indicate that even if these soils are cohesionless and even if they are conservatively located directly at the base of the pads, the estimated displacements would be less than 2.2 inches. Whereas there are no connections between the ground and these pads or between the pads and other structures, this minor amount of displacement would not adversely affect the performance of these structures.

The resistance to sliding on cohesionless materials is lowest when the dynamic forces due to the design basis ground motion act in the upward direction. The dynamic vertical force acting upward reduces the normal force and, hence, the shearing resistance, at the base of the foundations. Thus, the analyses were performed for Load Cases IIIA, IIIB, and IIIC, in which the pads are unloaded due to uplift from the earthquake forces. These load cases included:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

Newmark's Method of Estimating Displacements Due to Earthquakes

Newmark (1965) defines N W as the steady force applied at the center of gravity of the sliding mass in the direction in which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. For a block sliding on a horizontal surface, N W = T, where T is the shearing resistance between the base of the block on the sliding surface.

Shearing resistance, $T = \tau \times \text{Area}$

where:

$\tau = \sigma_n \tan \phi$

σ_n = Normal Stress

ϕ = Friction angle of cohesionless layer

$$\sigma_n = (\text{Net Vertical Force}) / \text{Area} = (F_v - F_{v(Eqk)}) / \text{Area}$$

$$T = (F_v - F_{v(Eqk)}) \tan \phi$$

$N W = T$

$N = [(F_v - F_{v(Eqk)}) \tan \phi] / W$

The maximum relative displacement of the pad relative to the ground, u_m, is calculated as

$$u_m = [V^2 (1 - N/A)] / (2gN)$$

The maximum horizontal ground velocities required as input in Newmark's method of analysis of displacements due to earthquakes were estimated for the cask storage pads assuming that the ratio of the maximum ground velocity to the maximum ground acceleration equaled 48 (i.e., 48 in./sec per g). The accelerations used for the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., a_H = 0.711g.

The above expression for the relative displacement is an upper bound for all the data points for N/A less than 0.15 and greater than 0.5, as shown in Figure 2.6-26, which is a copy of Figure 21 of Newmark (1965). Within the range of 0.5 to 0.15 the following expression gives an upper bound for all data.

$$u_m = V^2 / (2gN)$$

The following table presents the results (from Calculation 05996.02-G(B)-4, SWEC, 2001b) of estimating the horizontal displacements that the cask storage pads might experience due to the PSHA 2,000-yr return period earthquake if they were founded directly on cohesionless soils with $\phi = 30°$.

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S,	-100% Vertical,	40% E-W.	2.2 inches
Case IIIB	40% N-S,	-40% Vertical,	100% E-W.	1.9 inches
Case IIIC	100% N-S,	-40% Vertical,	40% E-W.	1.9 inches

For this hypothetical, worst case scenario, the estimated relative displacement of the cask storage pads ranges from ~1.9 inches to 2.2 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

The soils in the layer that are assumed to be cohesionless (the one ~10 ft below the pads) are clayey silts and silts, with some sandy silt. To be conservative in this analysis, these soils are assumed to have a friction angle of 30°. However, the results of the cone penetration testing (Appendices D & F of ConeTec, 1999) indicate that these soils have ϕ values that generally exceed 35 to 40°. These high friction angles likely are the manifestation of the cementation that was observed in many of the specimens obtained in split-barrel sampling and in the undisturbed tubes that were obtained for testing in the laboratory. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

In addition, this analysis postulates that cohesionless soils exist directly at the base of the pads. In reality, the surface of these soils is 10 ft or more below the pads, and it is not likely to be continuous, as the soils in this layer are intermixed. For the pads to

slide, a surface of sliding must be established between the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft below the pads, through the overlying clayey layer, and daylighting at grade. As shown in the analysis preceding this section, the overlying clayey layer is strong enough to resist sliding due to the earthquake forces. The contribution of the shear strength of the soils along this failure plane rising from the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft to the resistance to sliding is ignored in the simplified model used to estimate the relative displacement, further adding to the conservatism.

These analyses also conservatively ignore the presence of the soil cement under and adjacent to the cask storage pads. As shown above, this soil cement can easily be designed to provide all of the sliding resistance necessary to provide an adequate factor of safety, considering only the passive resistance acting on the sides of the pads, without relying on friction or cohesion along the base of the pads. Adding friction and cohesion along the base of the pads will increase the factor of safety against sliding.

<u>Sliding Stability of the Cask Storage Pads Based on Only Frictional Resistance Along the Base Plus Passive Resistance</u>

As part of the stability analysis of the cask storage pads (SWEC 2001b), a calculation was performed to check cask storage pad sliding due to the design basis ground motion for the hypothetical case that assumes that resistance to sliding is provided <u>only</u> by frictional resistance along the base of the pads plus passive resistance. This analysis demonstrates that even if the cohesion of the underlying soils is ignored along the interface between the pads and cement treated soil and the underlying soils, the resulting displacement of the pads would be minimal. Assuming the cask storage pads are founded directly on a layer of cohesionless soils with an obviously conservative value of the friction angle, the resulting factor of safety is less than the criterion of 1.1 for sliding stability due to the design basis ground motion. The relative displacement of

the pads was estimated for this case using Newmark's method of estimating displacements of embankments and dams due to earthquakes.

The results of this calculation indicate that displacement of the cask storage pad would range from approximately 2 inches to 6 inches. Even if the pads were to experience horizontal displacements of this magnitude, there would be no safety consequence to the pads or casks, since the pads and casks do not rely on any external "Important to Safety" connections. The impact of potential movement of the cask storage pads during a seismic event on the stability of the HI-STORM storage casks is addressed in SAR Chapter 8, Section 8.2.

2.6.1.12.2 Stability and Settlement Analyses—Canister Transfer Building

Stability and settlement analyses of the Canister Transfer Building were performed to confirm the adequacy of the structure and its foundation. Calculation 05996.02-G(B)-13 (SWEC, 2001c) evaluated the stability of the Canister Transfer Building and determined it is stable with respect to bearing capacity, overturning, and sliding due to static and dynamic load conditions. Calculation 05996.02-G(C)-14 (SWEC, 2000b) evaluated the soil settlements due to static load conditions and found the resulting building settlements to be uniform and small and to have little effect on the structure. Calculation 05996.02-G(B)-11 (SWEC, 2000a) evaluated the soil settlements due to dynamic load conditions and also found the resulting building settlements to be small and to have little effect on the structure. In summary, the stability and settlement analyses of the Canister Transfer Building indicate that the building is stable and will retain its structural integrity and the performance of the structure will not be adversely affected.

Chapter 4 describes the structural analyses of the Canister Transfer Building. The analyses used a finite element model approach and considered the effects of soil-structure interaction. The structural analyses take into account the flexibility of the

foundation underlying the building by the use of finite elements with the stiffness properties of the soil. Non-uniform elastic deformation of the soil, which results in bending moments and shear forces in the base mat are accounted for. This analysis is performed in Calculation 0599602-SC-6 (SWEC, 1998a). Induced stresses resulting from these non-uniform displacements were accommodated in the design of the structure.

The Canister Transfer Building is a large and massive building consisting of exterior reinforced concrete walls 2'-0" thick, a reinforced concrete roof 1'-0" thick, and a solid

reinforced concrete mat foundation 5'-0" thick. The interior partitions that make up the low level waste holding area will be constructed of concrete or concrete masonry. The equipment and office areas on the east side of the building will utilize steel-framed partition walls covered with gypsum board. The total weight (static load) of the building and foundation is approximately 89,400 kips (Calculation 05996.02-SC-5, SWEC, 2001d) or 44,700 tons.

Bearing Capacity of the Canister Transfer Building

The bearing capacity of the Canister Transfer Building foundation was determined using the general bearing capacity equation and associated shape, depth, and inclination factors, as presented in Winterkorn and Fang (1975). Refer to Calculation 05996.02-G(B)-13 (SWEC, 2001c) for details. These analyses are based on the strength parameters for the silty clay/clayey silt layer directly underlying the mat. Conservatively ignoring the presence of the denser layers, which start at a depth of ~25 to 30 ft, these analyses demonstrate that there is an adequate factor of safety against a bearing capacity failure for both static and dynamic loadings. They included determination of factors of safety against a bearing capacity failure of the foundation due to static loads and due to static plus dynamic loads from the design basis ground motion (PSHA 2,000-yr return period earthquake). The dynamic bearing capacity analyses are discussed in detail in the section below titled *"Dynamic Bearing Capacity of the Canister Transfer Building."*

Static Bearing Capacity of the Canister Transfer Building

Table 2.6-9 presents the results of the bearing capacity analyses for the following static load cases. As indicated above, the minimum factor of safety required for static load cases is 3.

Case IA Static using undrained strength parameters (ϕ = 0° & c =3.18 ksf).
Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6.5 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 3.18 ksf, to model the end of construction. This is the average undrained strength of the soils in the upper ~25 to ~30 ft, as discussed in Section 2.6.1.11.2. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0, results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for this soil strength is greater than 50 ksf.

Settlement of the Canister Transfer Building

Analyses were performed to estimate the settlement of the Canister Transfer Building for the static dead and live loads in Calculation 05996.02-G(C)-14 (SWEC, 2000b). A total building settlement of approximately 3 inches is estimated over the life of the building. The settlement will be generally uniform. Of the total building settlement, approximately 1.9 inches will occur within a few years after construction and an additional 1.1 inches will occur during the life of the building. These analyses were performed using the results of the consolidation tests that are included in Attachment 2 of Appendix 2A.

As indicated in Section 2.6.1.12.1 regarding the settlement analyses of the storage pads, this settlement represents an upper-bound estimate of the settlement, because it was developed assuming that the consolidation characteristics that were measured for the clayey soils at a depth of about 10 ft are applicable for the entire upper layer. The SPT data from the borings and the CPT results indicate that the soils become stiffer within the 10 to 20 ft depth zone. Additional consolidation tests performed on samples

obtained from depths of about 25 ft in the Canister Transfer Building area, reported in Attachment 6 of Appendix 2A, indicate that the soils at that depth are less compressible than those used to estimate these settlements.

Dynamic Bearing Capacity of the Canister Transfer Building

The dynamic bearing capacity was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2001c) using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 2001d). The development of these dynamic loads is described in Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3, the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the average undrained strength of the soils in the upper ~25 to ~30 ft, as discussed in Section 2.6.1.11.2 (ϕ = 0° and c = 3.18 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case II, the load combination of full static, 100% of the seismic forces acting in the N-S direction and the E-W direction and 0% in the upward direction. This load case resulted in an actual soil bearing pressure of 2.4 kips per square foot (ksf), compared with an ultimate bearing capacity of 13.2 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~5.5, which is much greater than the minimum allowable factor of safety for seismic loading cases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

Overturning Stability of the Canister Transfer Building

The overturning stability of the Canister Transfer Building was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2001c) using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 2.6-11, and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 2001d), which is described in Section 4.7.1.5.3. The masses and accelerations of the joints used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (~279.5 ft) than about the E-W axis (~240 ft).

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building

including the weight of the block of clayey soils enclosed within the perimeter key is 97,749 K, as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of ~240 ft, or 120 ft. Therefore,

$$\Sigma M_{Resisting} = (97{,}749 - 79{,}779)\ K \times 120\ ft = 2{,}156{,}400\ ft\text{-}K.$$

As indicated on page 15 of Calculation 05996.02-G(B)-13 (SWEC, 2001c), the driving moments include the 40% of $\Sigma M_{N\text{-}S}$, which is 1,082,784 ft-K, and 40% of the moment due to angular acceleration of the structure about the N-S axis which is 186,292 ft-K. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the maximum dynamic vertical force acts in the upward direction, tending to unload the mat and reduce the resisting moment.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}082{,}785^2 + (186{,}292)^2} = 1{,}098{,}694 \quad ft - K$$

and $$FS_{OT} = 2{,}156{,}400 \div 1{,}098{,}694 = 1.96$$

This is for Load Case IIIA. Calculation 05996.02-G(B)-13 (S&W, 2001c) indicates that the factor of safety against overturning for Load Cases IIIB and IIIC are greater than that for Load Case IIIA. This minimum factor of safety against overturning is greater than the criterion of 1.1; therefore, the Canister Transfer Building has an adequate

factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Sliding Stability of the Canister Transfer Building

The Canister Transfer Building will be founded on clayey soils, as indicated in Figures 2.6-21 through 2.6-23. A 1.5-ft deep key will be constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is engaged to resist sliding of the structure due to loads from the design basis ground motion. The sliding stability was evaluated in Calculation 05996.02-G(B)-13 (SWEC, 2001c) based on the loads that were developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, SWEC, 2001d). The strength of the clayey soils was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB near the elevation proposed for the bottom of the key around the perimeter of the CTB mat. The results of these tests are included in Attachments 7 and 8 of Appendix 2A. As indicated in Section 2.6.1.11.2, this shear strength equaled 1.8 ksf for the normal stress that equals the final vertical stress at the bottom of the key following construction of the building.

This analysis assumed that the backfill to be placed around the Canister Transfer Building mat and 1.5-ft deep key would be the soil cement constructed from the eolian silt that was excavated from the area. For this 5-ft deep layer of soil cement, it is assumed that the lower-bound value of γ is 100 pcf, ϕ = 0° & c = 125 psi. The passive resistance available is calculated as follows:

$p_P=2c$

For 5 ft soil cement with a FS of 2.0 applied for the passive resistance, the available resistance is

$P_p = 0.5 \times 5\text{ft} \times 2\times 125 \text{ psi} \times 0.144 \text{ ksf/psi} = 90 \text{ k/LF}$

Figure 5 of Calculation 05996.02-G(B)-13 (SWEC, 2001c) indicates that the CTB mat is actually 240' wide in the E-W direction and 279.5' long in the N-S direction. Therefore, the total passive force available to resist sliding is at least 240' x 90 k/LF = 21,600 k acting in the N-S direction.

Lambe & Whitman (1969, p 165) indicate that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The soil cement will be compacted to a dense state; therefore, this analysis assumed that one-half of the total passive resistance would be available to resist sliding of the building. Note, 0.5% of the 5 ft height of the mat + 1.5-ft deep key = 0.005 x 6.5 ft x 12 in./ft = 0.39 in. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding.

As discussed in Section 2.6.4.11, Techniques to Improve Subsurface Conditions, PFS is performing industry-standard soil cement testing to design the soil-cement mix (SWEC, 2001e), including wet/dry and freeze/thaw tests. These so-called "durability tests" are being performed specifically to ensure that the soil-cement mix to be placed within the frost zone (30" below grade at the site) will be durable enough to withstand the rigors of wetting, drying, freezing, and thawing. These tests are being performed in accordance with ASTM D559 and D560 and will measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. PFS is designing the soil-cement in accordance with the criteria specified by the Portland Cement Association (1971). Following these procedures will ensure that the resulting soil-cement will be able to withstand exposure to the elements, as indicated in the paragraphs cited from p. 8 of the "Soil-Cement Laboratory Handbook" (Portland Cement Association, 1971) in Section 2.6.4.11, Techniques to Improve Subsurface Conditions.

As indicated on p.10 of PFS Calculation 05996.02-G(B)-13 (SWEC, 2001c), the unconfined compressive strength of the soil cement is assumed to be 250 psi. This is consistent with the soil cement mix proposed for use within the frost zone adjacent to the cask storage pads and is based on the assumption that the strength will be at least this value to obtain a soil cement mix design that will satisfy the durability requirements of the ASTM wet/dry and freeze/thaw tests.

Further, PFS will construct the soil cement using techniques that will ensure that the bond between the layers is stronger than the shear strength of the underlying clayey soils. These standard practices in design and construction of soil cement have resulted in successful application of soil cement to more extreme environmental considerations than are applicable for this project and, thus, they will ensure that tensile cracking or shear failure does not occur along these interfaces. Therefore, PFS is considering the effects of freeze/thaw cycles on the top layers of soil cement, for both the pad emplacement area and the soil cement surrounding the CTB.

Although the sliding stability analysis of the CTB does not specifically refer to the potential for tensile or shear failure along lift interfaces within the soil cement layer, the analysis does indicate that the soil cement will be designed in accordance with the ASTM requirements for durability. There are also a number of references in Section 2.6.4.11 documenting PFS's commitment to construct the lift interfaces using techniques to ensure that the bond between soil cement layers is adequate to resist the sliding forces due to an earthquake.

The stability analysis of the Canister Transfer Building considers the horizontal displacements required to develop passive resistance from the soil cement. Pages 19 through 21 and 51 of PFS Calculation 05996.02-G(B)-13 (SWEC, 2001c) present the results of the sliding stability analysis of the Canister Transfer Building assuming that

only half of the passive resistance due to the soil cement is available to resist sliding. This is standard practice in geotechnical engineering analyses of sliding of structures where there is assurance that the material that is being relied upon to provide passive resistance will not be excavated. In this case, one-half of the passive resistance is used (p. 19), specifically, to incorporate the effects of strain-compatibility between the horizontal strains required to develop shear strength along the base of the structure and those required to develop a portion of the passive resistance along the side of the mat.

The effects of wall movement on wall pressure are defined in DM-7 (NAVFAC, 1986, p. 7.2-60) as the ratio of horizontal displacement to the height of the wall to emulate the horizontal strain as defined in Lambe and Whitman. For stiff cohesive soils, the wall rotation or yield ratio, y/H, required to fully mobilize passive resistance is 0.02, or 2%.

Based on the shear modulus estimated from the shear wave velocity of the surficial silty clay/clayey silt, the horizontal displacement of the CTB subjected to the full horizontal earthquake load is calculated to be about 1.5 inches using the elastic solution of a buried horizontal rectangle subjected to shear in an elastic half-space. This horizontal displacement corresponds to a yield ratio, defined as horizontal displacement 4 height of wall, of 1.9% from translation of the 6.5 ft height of the CTB foundation mat adjacent to the soil cement. This yield ratio is larger than the yield ratio required to mobilize one half of full passive resistance for dense sand or stiff cohesive soils. However, this analysis ignores the build-up of passive resistance as the mat displaces on the elastic half-space. Recognizing that the resistance does gradually build as the foundation displaces horizontally, it is reasonable to expect that the resulting horizontal displacement will be somewhat reduced from the 1.5 inches calculated by the elastic solution. Even if the maximum horizontal displacement were to occur from an earthquake, there would be no safety consequence to the CTB since the building does not rely on any external "Important to Safety" connections.

Page 22 of PFS Calculation 05996.02-G(B)-13 (SWEC, 2001c) discusses further the horizontal displacement to induce the assumed passive resistance. It states:

> *"Before a complete sliding failure can occur, the full passive resistance of the soil cement must be engaged. Because the strains associated with reaching the full passive state typically are large for soils, in the analyses where the full passive resistance of the soil cement adjacent to the mat is used, the shear strength of the clayey soils under the building is reduced to a conservative estimate of the residual shear strength, based on the results of the direct shear tests.*
>
> *The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (annotated copies are included in Attachment C of this calculation), illustrate that the residual strength of these soils is nearly equal to the peak strength for those specimens that were tested at confining stresses of 2 ksf. For example, for Sample U-1C from Boring C-2, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%. The specimens that were tested at confining stresses of 1 ksf all show reductions of ~20% at horizontal displacements of ~0.025" past the peak.*
>
> *The final effective vertical stresses at the base of the Canister Transfer Building, σ'_v, are ~1.5 ksf, now that the mat has been changed to 240 ft x*

279.5 ft. This value is approximately half-way between the confining stresses of 1 and 2 ksf used for several of the direct shear tests. The residual strength of the clayey soils beneath the building are expected to show reductions from the peak strength of ~12.5%, since the final effective stresses under the building are ~1.5 ksf. Therefore, based of these results, conservatively assume that the peak strength of the clayey soils beneath the soil cement layer underlying the pads is reduced by 20% to reach residual strength, to account for horizontal straining required to reach a strain applicable to the full passive resistance of the soil cement adjacent to the pad."

The results of the sliding stability analysis of the Canister Transfer Building are presented in Table 2.6-13 and indicate that the factors of safety were ≥1.1 for all load combinations examined. The lowest factor of safety was 1.13, which applies for Case IIIC, where 100% of the dynamic earthquake forces acts in the north-south direction and 40% acts in the other two directions. These results assume that only one-half of the passive pressures are available to resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases (Casagrande and Shannon, 1948, Das, 1993, and Schimming et al, 1966); therefore, they represent a conservative lower-bound estimate of the sliding stability of the Canister Transfer Building founded on in situ silty clay/clayey silt with 5 ft of soil cement around the building.

<u>Sliding Stability of the Canister Transfer Building on Cohesionless Soils</u>

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of

about 10 to 20 ft, especially near the southern portion of the building. Analyses were performed to address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These simplified analyses are presented in Calculation 05996.02-G(B)-13 (SWEC, 2001c). They were performed only for Load Cases IIIA, IIIB, and IIIC, because the resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. As described above, these load cases were defined as follows:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

As shown in Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft below the mat to about 9 ft, and it generally is at a depth of about 6 ft below the mat. These analyses included the passive resistance of the 5' soil cement and soils acting on a plane extending from grade down to the top of the cohesionless layer, plus the shear strength available at the ends of the silty clay block under the mat, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat was included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer. A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that $\phi = 38°$ was a reasonable minimum value for these soils. The factor of safety against sliding along the top of this layer was found to be >1.1 for all Load Cases IIIA, IIIB and IIIC.

These analyses include several conservative assumptions. They are based on static strengths of the silty clay block under the Canister Transfer Building mat, even though, as reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\,dynamic} \sim 1.5 \times c_{u\,static}$. In addition, the silty sand/sandy silt layer is not continuous under the Canister Transfer Building mat, and this analysis neglects cementation of these soils that was observed in the samples obtained in the borings. Therefore, sliding is not expected to occur along the surface of the cohesionless soils underlying the Canister Transfer Building.

2.6.1.12.3 Allowable Bearing Capacity—Other Structures

Other structures at the PFSF include the Administration Building, Operating and Maintenance Building, and Security and Health Physics Building. These structures will be founded on strip and spread footings. The allowable bearing capacity of these footings is limited by shear failure of the soil underlying the footing and by footing settlement.

Bearing capacity analyses were performed for a variety of footing widths and depths for both strip footings and square footings, for vertical loads, and for loads inclined 10 and 20 degrees from the vertical. These analyses were performed using effective-stress strength parameters to investigate long-term conditions, which are applicable for static loads. For these analyses, the allowable bearing pressure was determined using a factor of safety of 3. Bearing capacity analyses were also performed using the undrained strength parameters ($\phi = 0°$ & $c = 2.2$ ksf), which are applicable for earthquake loads. The static analyses yielded the minimum allowable bearing pressures, primarily due to the higher factor of safety required for static loadings.

To limit the expected differential settlements to tolerable values, wall footings of all structures should be designed such that the maximum estimated settlement at the center of the wall along the minimum width of the building is less than or equal to 2 inches. Spread footings supporting column loads spaced approximately 16 ft to 24 ft should be designed such that the maximum estimated settlement at the center of the footing is less than or equal to 1.5 inches. These criteria are based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969).

The gross allowable bearing pressure of these footings is presented as a function of the minimum effective footing width and depth in Figure 2.6-10 for strip footings and Figure 2.6-11 for square footings. In these figures, the straight lines represent the allowable bearing pressure that will provide the required factor of safety against a shear failure and the curves represent the bearing pressure that will result in a given amount of settlement. As indicated, the bearing pressure based on shear failure increases with increasing depth (and, typically, increasing width) of footing. Footing settlement increases as the load increases; therefore, for a given bearing pressure, as the width of the footing increases, there comes a point at which the amount of settlement exceeds the allowable settlement. Thus, as the footing width increases beyond this point, the allowable bearing pressure must decrease as shown by the curves in Figures 2.6-10 and 2.6-11, in order to limit the settlement to a tolerable value.

The design curves in these figures are for vertical loads applied at the center of the footings. For inclined or eccentrically applied loads, the allowable bearing pressures must be reduced. For loadings inclined at 10 degrees from the vertical, these allowables must be reduced by 25%, and for loadings inclined at 20 degrees from the vertical, these allowables must be reduced by 50%. Eccentric loads are addressed using the concept of "effective footing width", where the effective width (and length, if appropriate) of the footing is determined as shown in Figures 2.6-10 and 2.6-11.

2.6.2 Vibratory Ground Motion

The PFSF site is situated near the eastern margin of the Basin and Range province in an area known as the Great Basin. It has long been recognized that the pattern of north-south trending ranges and valleys in the Basin and Range is the result of periodic movement on normal faults that border the ranges on one or both sides. This activity is believed to be related to east-west horizontal extension starting in the late Cenozoic (Zoback and Zoback, 1989) and continues today, as evidenced by historic seismicity patterns, ground surface ruptures associated with infrequent, large magnitude, historic seismic events (6.5 M to 7.5 M), and deformation of late Quaternary and Holocene sediments across range-bounding faults.

The eastern boundary of the Basin and Range with the Middle Rocky Mountains province is commonly placed along the Wasatch Front, the north-south trending and west-facing escarpment that follows the Wasatch fault zone. This boundary is much less distinct than it appears physiographically, however. A transition zone up to 60 miles wide occurs east of the fault zone, in which block faulting overprints compressional features of the Sevier orogeny. Historic seismicity is actually higher east of the Wasatch fault than along it and geophysical data indicate the crustal boundary between the provinces occurs here as well (Smith, 1978). When examined on a regional scale, this belt of seismicity can be seen to be part of a larger zone that extends in a curvilinear pattern from northern Arizona and southern Nevada to northwestern Montana (Figure 2.6-12). This zone was first recognized in 1970 and is known as the Intermountain Seismic Belt (ISB) (Smith and Sbar, 1970; Sbar and Barazangi, 1970). Since that time, numerous investigators have discussed the origin and history of the ISB and have attempted to define the seismicity in a plate tectonic setting. Notable among these are the following: Smith and Sbar (1974), Anderson (1989), Stickney and Bartholomew (1987), Smith (1978), Smith et al. (1989), and Smith and Arabasz (1991).

The Skull Valley PFSF is interpreted to lie within the ISB near its western boundary (Arabasz et al., 1987) although it should be noted the boundary is somewhat arbitrary because of the diffuse, low level of seismic activity in this area. At least 16 earthquakes of magnitude 6.0 or greater have occurred in the ISB since settlement of the area began in the late 1840s (Figure 2.6-12). Ground surface faulting has been documented for three of these events: 1959 Hebgen Lake, MT (M_s 7.5); 1983 Borah Peak, ID (M_s 7.3); and 1934 Hansel Valley, UT (M_s 6.6). Surface faulting has also occurred elsewhere in the Basin and Range, in central and western Nevada and eastern California (Slemmons, 1980). The largest of these were the 1915 Pleasant Valley, NV (7.75 magnitude) and the 1872 Owens Valley, CA (8.0 magnitude) events. Arabasz et al. (1987) discuss these events in relation to determining a maximum size for Wasatch Front earthquakes. They concur with studies by Youngs et al. (1987) that the maximum probable event is M_s 7.5 and could have up to 6 meters of vertical displacement. (For an explanation of the various magnitude designations, see Stover and Coffman, 1993, page 2-3.)

Other studies, summarized by Arabasz et al. (1987), indicate there is a threshold magnitude value below which surface faulting is not likely in the Basin and Range. This value is approximately magnitude 6.0 to 6.5. More recent studies also suggest an estimated maximum magnitude of M_L about 6.5 (Arabasz et al, 1992; dePolo, 1994). This value represents the hypothetical maximum "background" or "random" earthquake for this area, one of several seismic sources evaluated to determine peak ground accelerations at the PFSF site. Geomatrix Consultants, Inc. (2001a) consider the maximum magnitude for the "random" event to be between M 5.5 and 6.5, with a mean value of 6.0.

Probabilistic analysis of capable faults and seismic zones in the region is summarized in Section 2.6.2.3 and detailed in Geomatrix Consultants, Inc. (2001a). Peak acceleration levels of 0.711g for horizontal ground motion and 0.695g for the vertical

ground motion were determined as the design bases of the PFSF for a 2,000-yr return period (Geomatrix Consultants, Inc, 2001b).

2.6.2.1 Engineering Properties of Materials for Seismic Wave Propagation and Soil-Structure Interaction Analyses

Dynamic soil properties were developed for the subsurface soils at the site in Geomatrix Consultants, Inc (2001c), based on the geotechnical and geophysical investigations and surveys that were performed in 1996, 1998, 1999 and 2001. Refer to Sections 2.6.1.5 and 2.6.1.10 for additional details about these investigations and surveys, and to Section 2.6.1.6 for a description of the generalized subsurface profile at the site. The dynamic soil properties include profile layering, low-strain shear and compression wave velocities, Poisson's ratios, unit weights, and shear modulus reduction and damping relationships. In accordance with US NRC Standard Review Plan, Chapter 3.7, which stipulates that SSI analyses be performed using a range of soil properties, three different sets of shear and compression wave velocity profiles were developed. The best-estimate velocity profile and the high and low velocity profiles are tabulated in Table 2.6-1.

One-dimensional site response analyses were performed using the three different velocity profiles presented in Table 2.6-1 to determine the response based on the best-estimate velocities and the high and low velocities. Figures 2.6-13 and 2.6-14 present the strain-compatible shear-wave velocity and damping ratio profiles for these three cases.

Based on the strain-compatible profiles obtained from the one-dimensional site response analyses, idealized horizontally layered soil profiles were developed for use in the soil-structure interaction analyses based on the SASSI continuum model. The dynamic properties for these idealized layers are presented in Table 2.6-2, and the details of this idealization are presented in Geomatrix Consultants, Inc (2001c).

The equivalent, single-layer shear modulus, Young's modulus, damping ratio, and unit weight of the soil were computed as a weighted average of the values within 30 ft below the surface (the minimum width of the cask storage pads). The weighting factors were assumed to decrease linearly with increasing depth. These equivalent dynamic soil parameters were computed for a rectangular foundation of 30 ft by 67 ft in accordance with Table 3.1 of Newmark and Rosenblueth (1971) for vertical, horizontal, and rocking modes. The resulting parameters are presented in Table 2.6-3.

Refer to Section 2.6.1.11 for discussion of the static and dynamic engineering properties of the soils underlying the site.

2.6.2.2 Earthquake History

The historic record of earthquakes in Utah began in 1850 with the publication of the region's first newspapers in Salt Lake City. Prior to mid-1962 when a scattered, state-wide network of seismographic stations became operational, most records were based upon felt reports. A few larger events were recorded instrumentally at regional stations beginning in the 1950's, including seismograph stations at Salt Lake City and Logan since 1955. Since 1974, a network of modern stations (presently > 85 stations) has provided data to the University of Utah's Seismograph Station (Arabasz et al., 1980). Coverage in the PFSF site area has been provided since 1968 by a station at Dugway, about 14 miles to the south; at Fish Springs, about 50 miles southwest; and on Stansbury Island, about 30 miles north-northeast. Arabasz et al. (1980) estimated the historical catalog for the Wasatch Front region to be complete for Modified Mercalli (MM) intensity greater than VIII since 1850; greater than VII since 1880; greater than VI since 1940; and greater than V since 1950. They judged that instrumental monitoring has provided a complete record down to magnitude (M_L) 2.3 since mid-1962.

Arabasz et al. (1987) provide a comprehensive evaluation of the University of Utah earthquake data base with particular application to an area at the north end of the Cedar Mountains, west of Skull Valley. They conclude that the threshold of earthquake detection is M_L approximately 2.0 or less in an area that includes the PFSF site.

Figure 2.6-15 is a map of all earthquakes within 160 km (100 miles) of the PFSF site of magnitude 3.0 or greater from the University of Utah Seismograph Station catalog. Table 2.6-4 is a chronological listing and description of those events. Only one earthquake greater than magnitude 3.0 has been reported within 50 km of the PFSF site. This event occurred on August 11, 1915 at an assumed location north of Deseret Peak in the Stansbury Mountains. It was reported at Iosepa, a settlement on the western foothill of the Stansbury Mountains. The University of Utah catalog indicates a magnitude 4.3, based on conversion of MM intensity V from the felt report (Arabasz et al., 1987). Stover et al. (1986) list an intensity VI for this event. However, Stover and Coffman (1993) do not list this event in their catalog, which has a threshold magnitude of 4.5. The earthquake was not reported in Tooele, less than 20 miles from Iosepa (Everitt and Kaliser, 1980), nor in Salt Lake City, about 43 miles to the east (Arabasz et al., 1987).

The largest historic earthquakes to occur within 160 km (100 mi.) of the PFSF site occurred in the Hansel Valley at the northern end of Great Salt Lake. A magnitude 6.6 earthquake occurred on March 12, 1934 and produced the only surface offset associated with an historic earthquake in Utah. The event occurred beneath an alluvium-filled valley and resulted in 50 cm of vertical ground surface displacement in a zone 12 km long. Some lateral displacement may also have occurred. Liquefaction and land subsidence occurred locally (Smith, 1978). Slight damage was reported in Grantsville and Tooele with MM intensity V experienced at Tooele (Everitt and Kaliser, 1980). Oaks (1987) reports MM intensity VIII in Salt Lake City caused buildings to sway and a 2-ton clock

mechanism fell from the tower of the Salt Lake County Building. Chimneys were toppled and structures were shifted on their foundations. The location of the earthquake is about 90 miles north of the PFSF site and appears to be associated with northerly-trending faults along the base of the Hansel Mountains (dePolo et al., 1989). Four aftershocks occurred within the following 2 months, ranging in size from magnitude 4.8 to 6.1. It is not known what effects, if any, these events had in the PFSF site area. An isoseismal map indicates the PFSF site would have been subject to MM intensity V effects from the original event (Stover and Coffman, 1993).

The Hansel Valley was the site of a prior moderate event magnitude 6.3 on October 6, 1909. Everitt and Kaliser (1980) indicate an MM intensity VII in the epicentral area; the event received no mention in the Tooele paper. The Salt Lake City paper indicated some buildings at the Saltair Resort on the southern shore of the Great Salt Lake were knocked out of plumb. Waves reportedly rolled over the boathouse pier and windows were cracked in Salt Lake City.

The closest magnitude 5.0 or greater earthquakes to the PFSF site occurred near Magna, UT, about 42 miles to the northeast. A magnitude 5.0 event on February 22, 1943 and a magnitude 5.2 event on September 5, 1962 were felt locally in Tooele but no damage was reported (Everitt and Kaliser, 1980). Other sources (Coffman and von Hake, 1973; Stover and Coffman, 1993) report cracked plaster and windows in Salt Lake City and damage to chimneys at Magna from both of these events. Wong et al. (1995) speculate this activity is occurring on the "Saltair structure" and estimate a maximum magnitude 6 for this feature.

Another historic earthquake worthy of mention occurred on August 1, 1900 near the towns of Eureka and Goshen. This magnitude 5.7 event damaged chimneys and plaster

in the epicentral area and caused a mine shaft nearby to be thrown out of alignment (Stover and Coffman, 1993). The epicenter is about 48 miles southeast of the PFSF site.

There is no evidence of any effects from any historic earthquake in the PFSF site vicinity.

2.6.2.3 Determining the Design Basis Ground Motion

Federal regulations governing the requirements for siting an ISFSI are contained in 10 CFR 72. These regulations require that seismicity at an ISFSI located west of the Rocky Mountain Front, such as the PFSF, be evaluated using the criteria for determining the safe shutdown earthquake at a nuclear power plant (10 CFR 100 Appendix A) in the same area. Vibratory ground motion design bases were determined by using a "deterministic" approach based upon a single set of earthquake sources. The regulations for siting nuclear power plants (10 CFR 100.23) were amended in 1997 in order to recognize the inherent uncertainties in geologic and seismologic parameters that must be addressed in determining the seismic hazard at a nuclear power plant site. One of the ways to address these uncertainties is through a probabilistic seismic hazard analysis (PSHA). In response to the Part 100 changes and anticipated changes to Part 72 (SECY-98-126), a probabilistic seismic hazard assessment has been performed for the PFSF for vibratory ground motions and surface fault displacement. Methodologies used and the results thereof are detailed in Sections 6 and 7 and Appendix F of Geomatrix Consultants, Inc. (2001a). The hazards results are presented as mean hazard curves that incorporate the uncertainty in input data and interpretations. The seismic source model used 16 capable fault sources and 4 seismic source zones within 100 km. Clarification of the PSHA formulation is provided in SAR Appendix 2F. In addition, sensitivity analyses were performed at the request of the NRC to provide further justification of the design basis ground motions (PFS letter dated May 31, 2001). An evaluation of recent information that indicates a higher slip rate (1 mm/yr) on the

East Great Salt Lake fault, the potential linkage of the East Great Salt Lake fault with other faults, and the possibility that the Stansbury fault could rupture co-seismically with the East fault, West fault, or East-West combined fault is discussed in Appendix 2G.

The NRC staff has recommended a risk-informed graded approach in their proposed changes to 10 CFR 72 when determining the appropriate hazard frequency or return

period. It was determined that an appropriate design probability level for the PFSF is 5 x 10^{-4} per year or a 2,000-yr return period (PFS letters of April and August 1999).

2.6.2.3.1 Capable Faults

The historical record of earthquakes does not provide a complete assessment of seismic potential in the Basin and Range province. There is considerable evidence of late Quaternary and Holocene surface faulting throughout the Basin and Range of Utah. Hecker (1993) has compiled all known or suspected Quaternary fault locations in Utah and provides a description and summary of the evidence for each feature. Goter (1990) provides a 1:500,000 scale map of Hecker's faults with historic seismicity plotted as well. A portion of Goter's map is reproduced as Figure 2.6-16. Figure 2.6-15 also includes Quaternary faults from Hecker (1993). Geomatrix Consultants, Inc. (2001a) provides a detailed discussion of capable faults and seismic source zones within 100 km, as shown on their Plate 7 and listed in Table 6-1. As can be seen on these maps, it is evident there are numerous Quaternary age faults within 100 miles (160 km) of the PFSF site.

Seismic sources include all structures that have some potential for causing strong ground motion at the PFSF (≥ magnitude 5). Seismic sources modeled in the probabilistic seismic hazard analysis are of two types: fault-specific sources and seismic source zones. Fault-specific sources include mapped late Quaternary faults. Seismic source zones are areas that have similar geological or seismologic characteristics that are assumed to have uniform earthquake potential. Seismic source zones are used to model the occurrence of seismicity that cannot be attributed to mapped late Quaternary faults.

A total of sixteen fault-specific sources were analyzed and included in the PSHA as well as four separate seismic source zones. Fault sources are listed in Table 6-1,

Geomatrix Consultants, Inc. (2001a). The key parameters used to characterize these sources are as follows:

- Total fault length and plan-view geometry
- Probability of activity
- Maximum earthquake magnitude
- Slip rate
- Recurrence

The values for these key parameters and the weighting factors assigned to each parameter for all seismic sources used in the PSHA are given in Table 6-2, Geomatrix Consultants, Inc. (2001a).

Figure 6-12 in Geomatrix Consultants, Inc. (2001a) shows the contributions of the various fault sources to the total hazard for horizontal motion at the Canister Transfer Building (CTB) location. The largest contributors to the hazard are the Stansbury and East-Springline faults. For long period ground motions the contribution due to the Stansbury fault increases due to the potential for larger earthquakes on the Stansbury than on the mid-Valley faults. The contribution of various earthquake magnitude intervals to the mean hazard for horizontal motion at the CTB location is shown on Figure 6-13 (Geomatrix Consultants, Inc., 2001a). It is evident the hazard is dominated by ground motions from nearby M 6 to 7 events, consistent with the proximity of the Stansbury and East-Springline faults to the CTB. Figure 6-22 (Geomatrix Consultants, Inc., 2001a) shows the contributions of the various fault sources to the total hazard for vertical motions. Again, the Stansbury and East-Springline faults are the dominant sources. The effects of using various models of attenuation, fault segmentation, and fault independence are documented in the report.

2.6.2.3.2 Maximum Earthquake

Several estimates have been made of maximum earthquake magnitude on the Stansbury fault. Arabasz et al. (1987), in their evaluation of seismic parameters for the Superconducting Supercollider facility proposed for a location just west of Skull Valley, calculate a maximum magnitude of M_S 7.3. This value is based on a measured maximum displacement on the fault of 12.6 ft (3.86 m) for a single event and regression relationships derived by Youngs et al. (1987).

Helm (1994, 1995) recently studied the Stansbury fault and identified evidence for segmentation of the fault, as mentioned above. Helm calculated a maximum magnitude of $M = 7.0 \pm 0.28$, based on Wells and Coppersmith's (1994) regression and a surface rupture length of 45 km. This length is for the entire Stansbury fault as if both segments ruptured together. If the north segment (20 km) ruptures next, as Helm (1995) suggests is more likely, a moment magnitude 6.6 ± 0.28 event would be generated.

Pechmann and Arabasz (1995) accept Helm's (1995) subdivision of the Stansbury fault and calculate a maximum magnitude (M_w) of 6½ for each segment. They also utilize the empirical relations of Wells and Coppersmith (1994) but their segment lengths are 17 km and 21 km (straightline length).

Wong et al. (1995) estimate a maximum earthquake of M_w = 6¾ for the Stansbury fault, again based on Wells and Coppersmith (1994), but their possible rupture length is 34 km.

Geomatrix Consultants, Inc. (2001a) divided the Stansbury fault into four segments and analyzed five rupture combination scenarios. Based on empirical relationships between magnitude and rupture length, magnitude and rupture area, magnitude and single event displacement, and a relationship between magnitude, rupture length, and slip rate,

Geomatrix Consultants, Inc. determined the maximum magnitude distribution for the Stansbury fault is **M** 6.5 to 7.5 with a mean of 7.0.

Similarly, they also determined mean maximum magnitudes for the recently identified East fault (**M** 6.5) and the West fault (**M** 6.4). These values for the individual faults were utilized in the probabilistic seismic hazard assessment of the PFSF site.

2.6.3 Surface Faulting

The site investigations document the presence of capable faults in the immediate PFSF vicinity. In order to determine the potential hazard of coseismic displacement on these faults, a probabilistic fault displacement hazard analysis was also performed and is described in Geomatrix Consultants, Inc., 2001a, Section 7. Fault displacement hazard analysis is based on methodology developed for the Yucca Mountain repository. Three separate categories of faults that appear to underlie the site were evaluated for displacement hazard: faults that appear to displace the Promontory/Bonneville unconformity (Faults D and F), faults that appear to displace the Tertiary/Quaternary unconformity but not the Promontory/Bonneville (Fault C), and, the zone of distributive faulting between the East and West faults.

Two separate approaches were utilized, an "earthquake approach" and a "displacement approach". Figure 7-8 in Geomatrix Consultants, Inc. (2001a) shows the contribution of the various seismic sources to the displacement hazard using the earthquake approach. The East fault dominates the hazard due to the potential for distributive faulting from a large event near the site. Figure 7-9 compares the mean hazard results for both approaches at the three fault locations beneath the site. The earthquake approach produces similar hazard as the displacement approach at Fault C and lower hazards at the other two locations.

As the consequences of failure of the cask storage system due to fault displacement are comparable to those due to ground motions, the probability level of interest for displacement is also judged to be 5×10^{-4} per year, or a 2,000-yr return period. At these probability levels, the displacements associated with faulting on Faults C, D, and F were determined to be less than 0.1 cm (Geomatrix Consultants, Inc. 2001a, Figure 7-7).

2.6.4 Stability of Subsurface Materials

2.6.4.1 Geologic Features that Could Affect Foundations

Dolomite or limestone bedrock is believed to underlie the site at depths between 520 to 880 ft. Examination of outcrops in the area indicates no evidence of cavernous or karst conditions in these rocks and there is no history of karst development in the region. The near-desert conditions make the development of karst very unlikely and the great depth to bedrock precludes effects at the ground surface. There is no evidence of any significant soluble mineral deposits in the unconsolidated materials beneath the site to at least a depth of 225 ft, and no record from water wells in the valley indicates the presence of similar material at greater depths. Evaporites associated with the waning stages of Lake Bonneville and the Great Salt Lake were not deposited here as the area remained above the extent of saline stages of these lakes.

There is no history of oil or gas development or subsurface mining in the Skull Valley and little potential for development in the future. There are no injection wells in the area and no evidence of past activities affecting the ground surface. Groundwater is withdrawn at a few scattered locations in the valley bottom for irrigation and stock watering but not to such an extent to cause surface subsidence or ground cracking. The nearest wells of this

type are located 2.5 miles northeast of the PFSF and 3 miles southeast. (See Figure 2.5-1).

Bedrock is not exposed at the PFSF site and will not be encountered by excavation or foundations. As a result, problems associated with alteration, deformation, or weathering of bedrock or anomalous in situ stresses are not a consideration for the foundations.

2.6.4.2 Properties of Underlying Materials

Static and dynamic engineering properties of the soils underlying the site are discussed in Sections 2.6.1.6, 2.6.1.11, and 2.6.2.1.

2.6.4.3 Plot Plan

The plot plan is shown in Figure 2.6-2 and discussed in Section 2.6.1.5. Refer to Section 2.6.1.6 for a description of the subsurface profile.

2.6.4.4 Soil and Rock Characteristics

Soil characteristics are described in detail in Sections 2.6.1.6 and 2.6.2.1. No rock will be encountered by excavations or foundations.

2.6.4.5 Excavations and Backfill

Refer to Section 2.6.1.7 for a discussion of excavations and backfill.

2.6.4.6 Groundwater Conditions

Groundwater conditions at and near the PFSF are discussed in Sections 2.5 and 2.6.1.9.

2.6.4.7 Response of Soil and Rock to Dynamic Loading

The engineering properties of the soils underlying the site for use in seismic wave propagation and soil-structure interaction analyses are discussed in Section 2.6.2.1.

Dynamic settlements due to the design basis ground motion are not expected to occur at the PFSF site because of the nature of the subsurface materials. Dynamic settlements, as reported in the geotechnical literature, are based on two different mechanisms, depending on whether the soils are above the groundwater table or below the groundwater table. Silver and Seed (1971) developed a technique for estimating dynamic settlements of dry cohesionless sands above the groundwater table. For such soils, the dynamic settlement mechanism is compaction due to soil grain slip, and it is a function of the magnitude of the cyclic shear strain developed due to the earthquake, the applied number of cycles of this shear strain, and the relative density of the soils.

As indicated in Section 2.6.1.9, the groundwater table is about 125 ft deep at the site. The top 30 ft of the profile consists of silt, silty clay, and clayey silt. The median blow count for this material is 14 blows per ft, indicating that it is "stiff". It appears to be weakly cemented, and unconsolidated-undrained triaxial tests on this material indicate that it has an apparent cohesion that is greater than 2,000 psf. Therefore, the technique for estimating dynamic settlements of soils above the groundwater table is not applicable for these materials, since they are not expected to compact as a result of soil grain slip.

In addition, cyclic triaxial tests were conducted on undisturbed thin-walled tube samples of the soils obtained from the upper ~25-ft thick layer at the site to assess the potential that they might collapse due to shaking caused by the design basis ground motion. These test results are included in Attachment 6 of Appendix 2A, and they demonstrate that the high void ratio soils will not collapse due to shaking caused by the design earthquake.

Five tests were performed on samples from borings in the Canister Transfer Building area. Three of the samples were from the 6 to 10-ft depth range, and the other two were from the 20 to 25-ft depth range. The shallower samples were highly plastic and had void ratios of 1.90, 2.04, and 2.22. The two deeper samples were moderately plastic and had void ratios of 1.26 and 1.55.

These soils will not be saturated during the life of the facility; therefore, in accordance with Section 21.7 of Lambe and Whitman, Soil Mechanics, John Wiley & Sons, New York, NY, 1969, "Partially Saturated Soils," which states:

> "*The best procedure to estimate the strength (of partially saturated soils) is to run tests that duplicate the field conditions as closely as possible: same degree of saturation, same total stress and, if possible, the same pressure in the liquid phase.*"

these tests were performed on samples at their natural water content, using confining stresses that emulate conditions expected under the structures prior to the earthquake.

Under a confining stress of 2.0 ksf, which approximates the final stresses under the storage pads and the Canister Transfer Building in the upper 25 ft layer, an axial cyclic stress of 1.9 ksf was applied at a rate of 1 Hz for at least 500 cycles. This cyclic stress was determined as indicated on Page 18 of Attachment 6 of Appendix 2A, conservatively

ignoring the presence of negative pore pressures. These unsaturated, fine-grained soils are expected to have some negative pore pressures due to the internal menisci of the pore water. Because of this, the vertical effective stresses between the soil grains will always be greater than the calculated vertical total stresses for the partially saturated fine-grained soils. The vertical effective stress, which appears in the denominator of the equation used to calculate the cyclic stress ratio, would be larger if the negative pore pressures were included. If the denominator was increased to include the negative pore pressures, the cyclic stress ratio for the laboratory samples would be lower. Therefore, the higher cyclic stress ratio used for the cyclic triaxial tests presented in Attachment 6 of Appendix 2A overestimates the cyclic deviator stress to apply during the test to emulate the shear stresses to be imposed on these soils due to shaking caused by the design earthquake.

Further, the shear stresses due to the design earthquake were determined for the PFSF deterministic earthquake, which had a peak horizontal acceleration, a_H, of 0.67g. The design earthquake for the PFSF was subsequently changed to the PSHA 2,000-yr return period earthquake, for which $a_H = 0.711g$. Therefore, conservatively ignoring the negative pore pressures in these partially saturated fine-grained soils and estimating the shear stresses based on the PFSF deterministic earthquake, the cyclic axial load applied to these specimens is compatible with that which is expected due to the design earthquake.

The range of double-amplitude strains measured during the test was 0.3% to 1.2%, with an average of 0.7%. All of the samples showed little or no increase of cyclic strain with an increase in the number of stress cycles. The axial cyclic displacement appeared to be elastic in nature, as indicated in the strip chart plots included in Attachment 6 of Appendix 2A, which present the axial displacements measured during the cyclic triaxial tests. These results demonstrate that even with this conservatively high cyclic axial load, the high void ratio soils showed no tendency to collapse. Therefore, these soils will not collapse due to shaking caused by the design earthquake.

The upper soil layer is underlain by very dense, fine sands that have uncorrected blow counts that commonly exceed 100 blows per ft. This material is underlain by silts that have even higher blow counts. Because of their very dense nature, these materials are not susceptible to settlement due to the dynamic settlement mechanism applicable for soils above the groundwater table; i.e., compaction due to grain slip.

The underlying soils that are below the groundwater table are greater than 125 ft below grade. The penetration resistances of these soils, as measured down to a depth of 226 ft in Boring CTB-1 and as indicated by the P-wave velocities (5,100 ft/sec to 5,900 ft/sec) reported by Geosphere Midwest, Inc. (Appendix 2B), demonstrate that these soils are also very dense. Because of their very dense nature, these materials are not susceptible to dynamic settlements, even though they may be saturated.

The in situ void ratio of 1.9 reported in Section 2.6.1.11 for the upper layer of soils in the subsurface profile was determined based on data obtained in performing the consolidation tests that are presented in Attachment 2 of Appendix 2A. These tests were performed on samples of the clayey silt. The void ratio of the nonplastic silts was not determined, but based on the standard penetration test (SPT) N-values of the soils, these nonplastic silts would not be characterized as loose.

A review of test results indicated that nonplastic silts were observed in the split-spoon samples obtained above and below Sample U2 in Boring A-2. Therefore, this Shelby tube was opened to see if it contained nonplastic silts that could be tested to determine the void ratio. However, as indicated by the Atterberg limits test results shown on Table 1 of Attachment 3 of Appendix 2A, this tube contained highly plastic clayey silt. Torvane tests performed on these soils demonstrated that the undrained shear strength ranged from 0.65 to 1.8 tons/ft^2, with an average value of 1.25 tons/ft^2, and the void ratio

averaged 2.1. These results are consistent with the test results reported in Attachment 2 of Appendix 2A for the clayey silt.

Additional Atterberg limits tests were performed on split-spoon samples obtained in Borings A-2, B-3, C-4, and D-4. These results, shown in Table 1 of Attachment 3 of Appendix 2A, confirmed that Samples S3 in Borings A-2 and C-4, and Sample S3A in Boring D-4 were essentially nonplastic. However, these Atterberg limits indicate that Samples S1 in Borings A-2 and B-3 and Sample S2 in Boring D-4, which were described as nonplastic in the boring logs, are actually slightly or moderately plastic. The descriptions on the boring logs were revised to reflect these laboratory results, as well as those included in Attachments 4 through 7 of Appendix 2A.

A review of the sample descriptions included in the boring logs indicates that only two samples of nonplastic silt are characterized as "loose". These two samples, Samples S-1 in Borings AR-2 and AR-3, were both obtained at the ground surface along the access road. Soils at the ground surface are not of interest since they will be removed during construction. All other nonplastic silt samples for which density is included in the description are characterized as being dense, very dense, or compact.

The following discussion applies to the SPT samples obtained in the upper layer of silt, silty clay, and clayey silt in the areas of the site proposed for the cask storage pads, the Canister Transfer Building, and the Security and Health Physics Building. It excludes the samples obtained at the ground surface, which represent soils that will be excavated for construction of the facilities.

The borings in the vicinity of the proposed locations of the cask storage pads, the Canister Transfer Building, and the Security and Health Physics Building (Borings A-1 through A-4, B-1 through B-4, C-1 through C-4, D-1 through D-4, E-3, and E-4) indicate

that the upper layer (~30 ft) consists mostly of soils with some plasticity, especially in the cask storage pad area. The average thickness of nonplastic soils in these borings is ~10 ft. Borings A-2 through A-4, B-1 through B-3, C-1 through C-3, and D-3 have less than or equal to 10 ft of nonplastic soils. Borings A-1 in the northwest, D-1 and D-2 in the northeast, and B-4, C-4, D-4, and E-4 along the south have ~20 ft of nonplastic soils. Note that these nonplastic soils often include occasional thin layers of clay or slightly plastic silt, which will minimize the potential for dynamically induced settlement.

A total of 64 SPT samples of silt (ML) were obtained. Of these, 31 were nonplastic and 33 exhibited some plasticity, ranging from slightly plastic to highly plastic. The N-values for the nonplastic silts in this layer ranged from 11 blows/ft to 40 blows/ft. The median N-value was 18 blows/ft, and the average was 20 blows/ft. This median N-value corresponds to a corrected blow count, N_1, of ~23 blows/ft, based on the relationship between penetration resistance and relative density developed by Gibbs and Holtz (1957) for granular soils.

If the nonplastic silts were cohesionless, they would behave more like fine sands rather than cohesive soils, and based on their N-values, would be classified as very dense rather than loose. Figure 7.5 of Lambe and Whitman (1969) presents the relationship between penetration resistance and relative density developed by Gibbs and Holtz (1957) for granular soils. Using this relationship to estimate the relative density of the non-plastic silts is very conservative, since a decrease in mean grain size tends to cause a decrease in SPT N-value for the same relative density, and the nonplastic silts at the site have a much smaller mean grain-size than the sand and fine sand used by Gibbs and Holtz. Using the 10 psi curve in this figure, or slightly below it, which is the approximate overburden stress for the mid-depth of this layer, fine sands having the median blow count of the nonplastic silts in this layer would be characterized as "very dense", not "loose".

The dynamic settlements of the nonplastic silts in this layer were estimated based on the method presented in Tokimatsu and Seed (1987). As they indicate, for soils above the groundwater table, dynamic settlements are calculated based on procedures originally developed by Silver and Seed (1971), and the effects of multidirectional shaking are estimated based on studies reported by Pyke, Seed, and Chan (1975). The dynamic settlement mechanism is compaction due to grain slip, and it is a function of the magnitude of the cyclic shear strain developed due to the earthquake, the applied number of cycles of this shear strain, and the relative density of the soils.

Figure 13 of Tokimatsu and Seed (1987) presents the relationship between volumetric strain due to compaction, cyclic shear strain, and corrected penetration resistance (N_1) of dry sands for 15 equivalent uniform strain cycles. The cyclic shear strain is estimated based on the average cyclic shear stress due to shaking caused by the design basis ground motion and the shear modulus of the soil. Figure 13 is used to estimate the volumetric strain due to compaction for 15 equivalent uniform strain cycles. Table 4 of Tokimatsu and Seed (1987) is then used to adjust for differences in the number of representative cycles of applied shear stress due to the design basis ground motion (~12 for Magnitude 7) and the 15 cycles used in Tokimatsu and Seed's studies. The dynamic settlement is calculated as the volumetric strain multiplied by the thickness of the nonplastic silts in the layer. Multidirectional effects of the earthquake are addressed by multiplying this result by 2, based on studies reported by Pyke, Seed, and Chan (1975).

The average cyclic shear stress developed in the field due to earthquake shaking is calculated as:

$$\tau_{avg} = 0.65 \bullet a_{max} \bullet \sigma_v \bullet r_d/g = 606 \text{ psf},$$

where: a_{max} = 0.71 g for the design basis ground motion

$\sigma_v = \gamma_{total} \bullet z$ above the groundwater table

γ_{total} = 92 pcf

z = depth below grade

r_d = stress reduction factor, which varies from 1.0 at z=0 to 0.9 at z=30'.

An iterative technique is used to determine the cyclic strain in the field due to the earthquake, □$_{field}$. For an assumed value of the cyclic strain, G is calculated as $G_{max} \bullet G / G_{max}$, where G / G_{max} for the nonplastic silt is estimated using the curve for PI=0 presented in Figure 6 of Vucetic and Dobry (1991). G_{max} equals 2,027 ksf, based on V_s 842 fps and γ_{total} ~ 92 pcf, as indicated in Table 7 of Calc 05996.02-G(PO18)-2, (Geomatrix, 2001c) for the upper 25 to 30-ft layer. The following table presents the results of these iterations.

Determination of Cyclic Shear Strain Due to the Design Basis Ground Motion

Iteration No.	$\gamma_{assumed}$ x10^{-4} in./in.	G / G_{max}	G ksf	γ_{field} x10^{-4} in./in.	$\Delta\gamma$ %
1	10	0.250	507	12.0	16
2	15	0.200	405	14.9	0

The cyclic strain in the field, γ_{field}, is calculated as τ_{avg} / G. Note, it is approximately equal to the assumed cyclic strain for Iteration No. 2; therefore, additional iterations are not required, and γ_{field} is 14.9 x 10^{-4} in./in., or 0.149%.

The volumetric strain due to compaction from 15 cycles is estimated as a function of this cyclic shear strain and N_1 of ~28 blows/ft, based on Figure 13 of Tokimatsu & Seed (1987). This results in a volumetric strain, $\varepsilon_{c,N=15}$, of 0.093%.

The design basis ground motion is magnitude 7 (Section 2.6.2.3). Table 4 of Tokimatsu & Seed (1987) indicates this corresponds to ~12 cycles of loading and that the volumetric strain ratio, $\varepsilon_{c,N=12} / \varepsilon_{c,N=15}$, should be ~0.9. Therefore, the volumetric strain corresponding to the design basis ground motion is $\varepsilon_{c,N=12}$, which is 0.9 x 0.093%, or 0.084%.

$$\varepsilon_c = \frac{\Delta\rho_{dyn}}{\Delta H}$$

where $\Delta\rho_{dyn}$ is the dynamic settlement of the layer,
and ΔH is the thickness of the layer.

The thickness of the nonplastic silts in the upper layer is conservatively estimated to be 15 ft, based on the discussion presented above. Therefore, for unidirectional shaking,

$\Delta\rho_{dyn,1}$ = 0.15 inches = 15 ft x 12 in./ft x $\varepsilon_{c,N=12}$ / 100%.

The dynamic settlement is multiplied by 2 to account for multidirectional shaking due to the earthquake. This results in an estimated dynamic settlement of the nonplastic silts in the upper layer of 0.3 inches.

Examination of these soils, which are deposits from ancient Lake Bonneville, indicates the presence of numerous tiny shells (Ostracodes). Considerable void space was present under some of these shells, and it is believed that these voids are contributing to the high, in situ void ratio measured for the clayey silt.

Calcium carbonate is present in these soils, as evidenced by a vigorous reaction upon application of hydrochloric acid to these soils. Therefore, these soils are believed to be cemented, the result of carbonate cement bonding of the silt and clay-size particles, imparting cohesion to these soils.

The void ratio of 1.9 reported in Section 2.6.1.11 was determined on samples of the clayey silts from the upper layer, not the nonplastic silts. As evidenced by the SPT data,

these nonplastic silts are not loose. The dense nature of these soils, which is most likely the result of carbonate cement bonding of the silt particles, minimizes the potential for dynamically induced settlements due to the design basis ground motion. Ignoring this cementing, the total dynamic settlement is conservatively estimated to be less than ½ of an inch.

This estimated dynamic settlement was determined based on the thickness of nonplastic silts in areas where the nonplastic silts are thickest, not on an average or median thickness. This conservatively overestimates the settlement. In addition, it conservatively neglects the fact that these nonplastic silts are stratified with layers of clay and clayey silt, which will minimize the potential for dynamically induced settlements. Thus, this estimated dynamic settlement is very conservative.

Dynamic settlements will be much less than this over most of the cask storage pad area, since most of the soils in this area are not nonplastic. Rather, these soils are sufficiently stiff and cohesive that they will not experience dynamic compaction due to the shaking caused by the design basis ground motion.

Dynamic settlements of this magnitude are not expected to adversely affect the performance of the facilities.

2.6.4.8 Liquefaction Potential

The soils underlying the proposed PFSF site are not susceptible to liquefaction as a result of the design basis ground motion because they are only partially saturated from grade down to the groundwater level at a depth of 125 ft. The upper ~30-ft thick layer of soils are typically cohesive or cemented and, being essentially dry or only partially saturated, are not subject to liquefaction. The soils from that depth down to the groundwater table at

a depth of 125 ft are similarly only partially saturated and they are very dense. The standard penetration test N-values for these soils typically exceed 100 blows per ft, and they increase with depth. The presence of this greater than 90-ft thick, very dense layer overlying the saturated soils is expected to preclude any surface manifestation of liquefaction (e.g., sand boils) of the saturated soils below the groundwater table, if it were possible for them to liquefy. Below the groundwater table, liquefaction is considered unlikely, however, because the density of the soils encountered in the borings increases with depth, as evidenced by the SPT N-values down to a depth of 226 ft in Boring CTB-1 and the high P-wave velocities (5,100 ft/sec to 5,900 ft/sec) measured for the soils below the groundwater table, reported by Geosphere Midwest, Inc. (Appendix 2B).

2.6.4.9 Design Basis Ground Motion

The design basis ground motion was determined by a probabilistic seismic hazard analysis and is defined as having a peak horizontal ground acceleration of 0.711g and a peak vertical ground acceleration of 0.695g. The development of the design basis ground motion is described in Geomatrix Consultants, Inc. (2001a and 2001b). The site specific response spectra are presented in Table 1 and Figure 4 of Geomatrix Consultants, Inc. (2001b).

2.6.4.10 Static Analyses

Refer to Section 2.6.1.12 for a detailed discussion of static analyses in the stability of foundations for structures.

2.6.4.11 Techniques to Improve Subsurface Conditions

Soil Cement

Discussions presented in Section 2.6.1.12, above, indicate that the soils underlying the eolian silt layer at the surface of the PFSF site are suitable for support of the proposed structures; therefore, no special construction techniques are required for improving the subsurface conditions below the eolian silt. The eolian silt, in its *in situ* loose state, is not suitable for founding the structures at the site. The basemat of the Canister Transfer Building will be founded on the silty clay/clayey silt layer beneath the eolian silt. It was originally intended that the cask storage pads also would be founded on the silty clay/clayey silt layer. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Soil cement will also be utilized around the Canister Transfer Building. The required characteristics of the soil cement will be engineered during detailed design and constructed to meet the necessary strength requirements.

During construction of the storage pads, all of the eolian silt in the quadrant under construction will be excavated. The eolian silt will be mixed with sufficient cement and water and compacted to produce soil cement across the pad area, up to the design elevations of the bottoms of the storage pads. The layer of soil cement beneath the storage pads will have a minimum thickness of 12 inches and a maximum thickness of 24 inches. In the event that the eolian silt layer extends to a depth greater than 2 ft below the elevations of the bottoms of the storage pads, compacted clayey soils will be used to raise the elevation of the subgrade that will support the soil cement layer to an elevation of 2 ft or less below the design elevations of the bottoms of the pads. This will ensure that the layer of soil cement does not exceed a thickness of 2 ft. This is the

maximum permissible thickness of the soil cement layer, since the storage cask hypothetical tipover and drop analyses were performed assuming a 2.0-ft thick layer of soil cement underlying the storage pads.

Strength of Soil Cement and Minimum/Maximum Thickness Requirements

The soil cement underlying the pads shall have a minimum unconfined compressive strength of 40 psi to ensure that there is an adequate factor of safety against sliding of an entire column of pads (S&W Calculation 05996.02-G(B)-4, SWEC, 2001b). This layer of soil cement is required to be no greater than 2-ft thick and have a static modulus of elasticity less than or equal to 75,000 psi to ensure that the decelerations from a hypothetical storage cask tipover event or vertical end drop accident do not exceed HI-STORM design criteria (Section 3.2.11.3).

Following construction of the storage pads on top of this layer of soil cement, additional soil cement will be placed around and between the cask storage pads, extending from the bottoms of the pads to a level that is 28 inches above the bottoms of the storage pads. The remaining 8 inches, from the top of the soil cement up to grade, will be filled with coarse aggregate, placed and compacted to be flush with the tops of the pads to permit easy access by the cask transporter. The soil cement placed around the sides of the storage pads is expected to have a minimum unconfined compressive strength of at least 250 psi to satisfy durability requirements within the depth of frost penetration (based on S&W Calculation 05996.02-G(B)-4 (SWEC, 2001b), as discussed in Section 2.6.1.12.1).

The Canister Transfer Building basemat will be founded on the silty clay/clayey silt layer that is below the eolian silt. The design calls for soil cement to be placed around the Canister Transfer Building base mat to make the free-field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the higher design basis ground motions. Soil cement will surround the

foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft out from the mat in the east and west directions and approximately 280 ft out in the north and south directions. Existing soils (eolian silt and silty clay/clayey silt) will be excavated to a depth of approximately 5 ft 8 inches below grade, mixed with cement, and placed and compacted around the foundation mat.

The soil cement placed around the Canister Transfer Building foundation mat will be 5 ft thick and have a minimum unconfined compressive strength of 250 psi to ensure that there is an adequate factor of safety against sliding of the Canister Transfer Building (based on Calculation 05996.02-G(B)-13 (SWEC, 2001c), as discussed in Section 2.6.1.12.2). The top 8 inches will be filled with compacted coarse aggregate, similar to that used in the pad emplacement area.

PFS is developing the soil-cement mix design using standard industry practice. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010 (SWEC, 2001e). This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-4 (SWEC, 2001b):

> *"The unconfined compressive strength of the soil cement adjacent to the pads needs to be at least 50 psi to provide an adequate subbase for support of the cask transporter, in lieu of placing and compacting structural fill, but it likely will be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 in. from the ground surface)."*

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of Portland Cement Association (1971):

"The freeze-thaw and wet-dry tests were designed to determine whether the soil-cement would stay hard or whether expansion and contraction on alternate freezing-and-thawing and moisture changes would cause the soil-cement to soften."

And on p. 32:

"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.

The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.

1. *Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*
   Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;
   Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;
   Soil Groups A-6 and A-7, not over 7 percent.
2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between concrete and soil-cement, soil-cement and soil-cement, and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot in his testing of bond along soil-cement interfaces.

Based on the above, PFS has adequately defined the measures that will be followed in the design and construction of the soil cement to assure that the assumed bonds can be sustained through the period of interest. PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake. As indicated above, this testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved and during construction to demonstrate that the required interface strengths are achieved. In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

The most recent analyses of the PFSF design basis ground motions assumed the incorporation of a 5 ft thick soil cement layer over the entire pad emplacement area and also surrounding the Canister Transfer Building. The 5 ft soil cement layer around the Canister Transfer Building extends to the free field boundary from the edge of the building basemat. This soil cement layer is assumed to have a minimum shear wave velocity greater than 1,500 fps (Geomatrix 2001a and 2001b). As indicated in Section 2.6.1.2.2, soil cement around the Canister Transfer Building should have a minimum unconfined compressive strength of 250 psi to ensure a factor of safety greater than 1.1 for seismic sliding stability. The design requirements for the 5 ft thick soil cement layer

around the Canister Transfer Building will be based on the results of laboratory and field testing to be conducted during the final design stage.

The surficial layer of eolian silt, existing across the entire site as shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), is a major factor in the earthwork required for construction of the facility. This layer consists of a nonplastic to slightly plastic silt, and it has an average thickness of approximately 2 feet across the pad emplacement area. This layer was expected to be removed prior to construction of the storage pads. However, based on evaluation of the earthwork associated with site grading requirements for flood protection and the environmental impacts of truck trips required to import fill to replace this material, PFS will stabilize this soil with cement and use it as base material beneath the storage pads and adjacent driveways.

Section 2.6.1.12 indicates that there is ample margin in the factor of safety against a bearing capacity failure of the silty clay/clayey silt underlying the site and that the settlements are acceptable for these structures. They indicate that the critical design factor with respect to stability of these structures is the resistance to sliding due to loadings from the design basis ground motion. As discussed in that section, the silty clay/clayey silt layer has sufficient strength to resist these dynamic loadings; therefore, adequate sliding resistance can be provided by constructing the structures directly on the silty clay/clayey silt layer. The soil cement around the storage pads and Canister Transfer Building will be designed and constructed to have a minimum unconfined compressive strength of 250 psi and quality assurance testing will be performed during construction to demonstrate that this minimum strength is achieved. The soil cement directly beneath the storage pads will be designed and constructed to have an unconfined compressive strength of at least 40 psi with static elastic modulus of less than ~75,000 psi. Therefore, the resistance to sliding due to loadings from the design

basis ground motion will be enhanced by constructing the cask storage pads on a properly designed and constructed soil-cement subgrade. See the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in 2.6.1.12.1 for additional details.

Using soil cement to stabilize the eolian silt will reduce the amount of spoil materials generated, create a stable and level base for pad construction, and substantially improve the sliding resistance of the storage pads. The soil cement will be placed above the *in situ* silty clay/clayey silt layer and will be designed to improve the strength of the eolian silt so that it will be stronger than the clayey soils that were originally intended for use as the founding medium for the pads. The soil cement will also be used to replace the compacted structural fill that the original plan included between the rows of pads. This continuous layer of soil cement, existing under and between the pads, will spread the loads from the pads beyond the footprint of the pads, resulting in decreased total and differential settlements of the pads. The layer of soil cement above the base of the pads and the bond and friction of the pad foundation with the underlying soil-cement layer will greatly increase the sliding resistance of the pad.

Soil cement has been used extensively in the United States and around the world since the 1940's. It was first used in the United States in 1915 for constructing roads. It also has been used at nuclear power plants in the United States and in South Africa. The largest soil-cement project worldwide involved construction of soil-cement slope protection for a 7,000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston, TX. Soil cement also was used to replace an ~18-ft thick layer of potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979). The strength of soils can be improved markedly by the addition of cement. The eolian silt at the site is similar to the soils identified as Soil A-4 in Nussbaum and Colley (1971), Soils 7 and 8 in Balmer

(1958), and Soil 4 in Felt and Abrams (1957). As indicated for Soil A-4 in Table 5 of Nussbaum and Colley (1971), the addition of just 2.5% cement by weight to the silt increased the cohesion from 5 psi (720 psf) to 30 psi (4,320 psf). The cohesion for Soils 7 and 8 also were increased significantly by the addition of low percentages of cement, as shown on Tables VI and VII of Balmer (1958). Figure 10 in Felt and Abrams (1957) illustrates the continued strength increase over time for these soil-cement mixtures. Other examples of soil-cement strength increases over time are presented in Figure 4.3 of ACI (1998), Table 6 of Nussbaum and Colley (1971), and Figures 6 and 7 of Dupas and Pecker (1979). Therefore, the soil cement will be much stronger than the underlying silty clay/clayey silt and the strength will increase with time, providing an improved foundation material. This will provide additional margin against sliding compared to the original plan to construct the pads directly on the silty clay/clayey silt layer.

As shown in the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in Section 2.6.1.12.1 above, the shear resistance required at the base of the pads can be provided easily by the passive resistance of the soil cement acting against the vertical side of the foundation and by bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement. Shear resistance will be transferred through the approximately 2-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; therefore, shear resistance requirements within the silty clay/clayey silt layer will be less with the soil-cement layer compared to the original plan to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the

effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and various surface treatments and additives.

His results demonstrated that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength always exceeded 6.6 psi, the minimum required value of cohesion if the passive resistance acting on the sides of the pads is ignored. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 8.7 psi. This value applied for two tests that were performed on samples that had time delays of 24 hours and did not have a cement surface treatment along the lift line. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that had 24-hr delays between lift placements and which did not use the cement surface treatment, the minimum bond strength was 10.7 psi and there were only two others that had bond strengths that were less than 20 psi. Even these minimum values for the group of specimens that did not use a cement surface treatment exceeded the cohesive strength (6.6 psi) required to obtain an adequate factor of safety against sliding without including the passive resistance acting on the sides of the pads, and all of the rest were much greater, generally more than an order of magnitude greater.

DeGroot reached the following conclusions:

1. Increasing the time delay between lifts decreases bond.
2. High frequency of watering the lift line decreases the bond.

3. Moist curing conditions between lift placements increases the bond.
4. Removing the smooth compaction plane increases the bond.
5. Set retardants decreased the bond at 4-hr time delay.
6. Asphalt and chlorinated rubber curing compounds decreased the bond.
7. Small amounts of cement placed on the lift line bonded the layers together, such that failure occurred along planes other than the lift line, indicating that the bond exceeded the shear strength of the soil cement.

DeGroot (1976) noted that increasing the time delay between placement of subsequent lifts decreases the bond strength. The nature of construction of soil cement is such that there will be occasions when the time delay will be greater than the time required for the soil cement to set. This will clearly be the case for construction of the concrete storage pads on top of the soil-cement surface, because it will take some period of time to form the pad, build the steel reinforcement, and pour the concrete. He noted that several techniques can be used to enhance the bond between these lifts to overcome this decrease in bond due to time delay. In these cases, more than sufficient bond can be obtained between layers of soil cement and between the set soil-cement surface and the underside of the cask storage pads by simply using a cement surface treatment.

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the average bond strength of 132.5 psi. Even the minimum value of this range is nearly an order of magnitude greater than the cohesion (6.6 psi) required to obtain a factor of safety against sliding of 1.1, conservatively ignoring the passive resistance available on the sides of the pads. Therefore, when required due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These

techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

A fundamental assumption in the PFS approach is that sufficient bonding and shear transfer between clay and soil cement interfaces can be achieved using various construction techniques. As indicated above, DeGroot has demonstrated that techniques are available that will enhance the bond between lifts of soil cement. These techniques should be equally effective when applied to the soils at the PFSF site. PFS has committed to perform direct shear tests of the interface strengths during the design phase of the soil cement to demonstrate that the required interface strength can be achieved, as well as during construction, to demonstrate that they are achieved.

PFS has discussed the change to use soil cement beneath the storage pads with the project consultants who have analyses in-place that are based on the storage pads resting on the silty clay/clayey silt. The consultants contacted were Geomatrix (development of seismic criteria and soil dynamic properties), Holtec International (cask stability analysis), and International Civil engineering Consultants (pad design). Each has indicated their analyses would not be adversely affected by this proposed change.

The design, placement, testing, and performance of soil cement is a well-established technology. The "State-of-the-Art Report on Soil Cement" (ACI, 1998) provides information about soil cement, including applications, materials, properties, mix proportioning, design, construction, and quality-control inspection and testing techniques. PFS will develop site-specific procedures to implement the recommendations presented in ACI (1998) regarding mix proportioning, testing, construction, and quality control. The following describes the processes that will be used to develop a proper soil-cement mix design and establish adequate sliding resistance at each material interface in the storage pad and soil system:

- Soil-Cement Mix and Procedure Development – The sliding forces due to the design basis ground motion will be resisted by bond between the base and sides of the foundation and the soil cement and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil-cement mix will be designed and constructed to exceed the minimum shear resistance requirements. During the soil-cement design phase, direct shear testing will be conducted along manufactured soil-cement lift contacts and concrete contacts that represent anticipated field conditions. The direct shear testing, along with other standard soil-cement testing, will be used to confirm that adequate shear resistance and other strength requirements will be provided by the final soil-cement mix design. Procedures required for placement and treatment of the soil cement, lift surfaces, and foundation contact will be established in accordance with the recommendations of ACI (1998) during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project.

- Soil-Cement Lift and Concrete Interface – The soil cement will be constructed in lifts approximately 6-in. thick (compacted thickness) as described in ACI (1998). Construction techniques will be used to ensure that the interface between the soil-cement layers will be adequately bonded to transmit shear stresses. As described in Section 6.2.2.5 of ACI (1998), these techniques will include, but will not be limited to: minimizing the time between placement of successive layers of soil cement, moisture conditioning required for proper curing of the soil cement, producing a roughened surface on the soil cement prior to placement of additional lifts or concrete foundations, and using a dry cement or cement slurry to enhance the bonding of concrete or new soil cement layers to underlying layers that have already set. In addition to conventional quality control testing performed for soil-cement

projects, direct shear testing will be performed on representative samples obtained from placed lift contacts to confirm design requirements are obtained. Sacrificial soil-cement lifts may be used to protect the soil-cement subgrade in the pad foundation areas.

- Soil Cement and *In Situ* Clay Interface – The soil cement and *in situ* clay interface will be constructed such that a good bond will be established between the two materials. Construction techniques will be utilized that will ensure that the integrity of the upper surface of the clay is maintained and that a good interface bond between the two materials is obtained. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during the detailed design phase of the project.

An additional benefit of incorporating the soil cement into the design is that it will minimize the environmental impacts of constructing the facility. Using on-site materials to construct the soil cement, rather than excavating and spoiling those materials, will reduce environmental impacts of the project. In addition, replacement of some of the structural fill layer between the rows of pads with soil cement, as shown in Figure 4.2-7, will result in reduced trucking requirements associated with transporting those materials to the site.

Adequacy of the Soil Cement Design

The adequacy of the design of the soil cement surrounding and underlying the pads to ensure the sliding stability of the pads under seismic conditions is demonstrated by S&W Calculation 05996.02-G(B)-04 (SWEC, 2001b). This calculation determined that there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement layer and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value, with

no credit for the soil cement placed between storage pads above the bottom of the pads. The underlying layer of soil cement is also required to have a static modulus of elasticity less than or equal to 75,000 psi to ensure that decelerations of a cask resulting from a hypothetical storage cask tipover event or vertical end drop accident do not exceed design criteria (Sections 4.2.1.5.1.E and 8.2.6).

The large extent of soil cement in the storage pad emplacement area allows the soil cement layer to be considered as part of the free field soil profile for the site response analyses. The properties of the soil cement, higher shear wave velocity and higher density than the existing soils in the area, help to minimize the response at the surface of the site caused by the design basis ground motions. Soil cement was added around the Canister Transfer Building foundation mat to make the free field soil profile for the building consistent with that for the storage pad emplacement area (as discussed in Section 2.6.4.11), and to help resist sliding forces, in conjunction with the building's perimeter key, due to the revised design basis ground motions. The adequacy of this design feature is demonstrated in Calculation No. 05996.02–G(B)-13 (SWEC, 2001c), which determined that the design of the soil cement surrounding the Canister Transfer Building (in conjunction with the building's perimeter key) is adequate to ensure the stability of the Canister Transfer Building under seismic conditions.

2.6.4.12 Criteria and Design Methods

The allowable bearing capacity of footings is limited by shear failure of the underlying soil and by footing settlement. The minimum factor of safety against a bearing capacity failure from static loads (dead load plus maximum live loads) is 3.0 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Allowable settlements are determined based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969) and assume that the differential settlement will be 3/4 of the maximum settlement. Section 2.6.1.12 provides more details.

In order to comply with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Where the factor of safety against sliding is less than 1 due to the design basis ground motion, the displacements the structure may experience are calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. The magnitude of these displacements are evaluated to assess the impact on the performance of the structure. See Section 2.6.1.12 for details about these analyses.

2.6.5 Slope Stability

There are no slopes close enough to the proposed Important to Safety facilities that their failure could adversely affect the operation of these facilities.

THIS PAGE INTENTIONALLY LEFT BLANK

2.7 SUMMARY OF SITE CONDITIONS AFFECTING CONSTRUCTION AND OPERATING REQUIREMENTS

The PFSF site is located near the middle of Skull Valley. The valley is approximately 50 miles long, 22 miles wide, and slopes gently to the north at about a 0.6% grade. The finished grade of the storage facility is sloped gently from elevation 4,475 ft at the southeastern corner to elevation 4,457 ft at the north, where a stormwater detention basin is located. The stormwater detention basin is at elevation 4,450 ft. The Canister Transfer Building, located within the restricted area, has a floor elevation of 4,475 ft. The access road is graded to match the Skull Valley road at the east end (elevation 4,487 ft) and the storage facility at the west end (elevation 4,475 ft) with a maximum slope of approximately 3%.

The entire site, including the access road, is above the 100-yr flood elevation. However, diversion berms are required to deflect the anticipated flows from the PMF due to both the Stansbury Mountains and the Hickman Knolls watersheds.

A diversion berm is provided perpendicular to the access road to channel the Stansbury Mountain PMF to flow away from the PFSF. The PMF berm elevation is 4,507.5 ft where it intersects the access road. The berm is a maximum of 9 ft high at the access road and tapers down in height as it joins the Hickman Knolls to the south. The berm is located approximately 200 ft east of the storage facility along the access road and extends approximately 1,600 ft south and 280 ft north of the access road. The berm will divert PMF-generated flow emanating from the Stansbury Mountains, east of the site, to the north and east of the storage facility. Box culverts are provided under the access road to allow normal and 100-yr floodwater to pass beneath the access road toward the north. PMF flows will flow over the top of the access road.

The storage facility and buildings within the restricted area are protected from the smaller Hickman Knolls PMF by a partial perimeter berm. The berm is located along the entire southern side and halfway along the western side of the storage facility. The berm is approximately 5 ft high and is designed to divert the PMF sheet flow of less than one foot deep to the west and north of the storage facility.

A probabilistic seismic hazard analysis was performed to determine the design basis ground motion at the PFSF. The peak horizontal ground acceleration was determined to be 0.711g in both directions, and the peak vertical ground acceleration was determined to be 0.695g for the 2,000-yr return period mean ground motion.

Subsurface soils at the site are suitable for supporting conventional foundations under both the static and dynamic loading conditions. There is no potential for liquefaction, collapse, or excessive settlement of these soils. There are no slopes, natural or manmade, close enough to the proposed Important to Safety facilities that their failure could adversely affect these facilities.

Dry casks will be used to store canisters containing spent fuel. The canisters will be drained of all liquid prior to being shipped to the facility. Therefore, liquid releases cannot result from operation of the facility.

Groundwater is about 125 ft deep at the site. The method of storage (dry cask), the nature of the storage casks, and the depth to groundwater beneath the site preclude the possibility of groundwater contamination from operation of the facility.

2.8 REFERENCES

ACI, 1998, "State-of-the Art Report on Soil Cement," reported by ACI Committee 230, ACI 230-1R-90 (Reapproved 1997), American Concrete Institute, Detroit, MI.

American National Standards Institute, 1982, American national standard minimum design loads for buildings and other structures: ANSI A58.1-1982, published by the American National Standards Institute, Inc., New York, New York.

Anderson, J.G., Wesnousky, S.G., and Stirling, M.W., 1996, Earthquake size as a function of fault slip rate: Bulletin of the Seismological Society of America, v. 86, No. 3, p. 683-690.

Anderson, R.E., 1989, Tectonic evolution of the Intermontane system; Basin and Range, Colorado Plateau, and High Lava Plains, in Pakiser, L.C., and Mooney, W.D., eds.; Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 163-176.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1987, Evaluation of seismicity relevant to the proposed siting of a Superconducting Supercollider (SSC) in Tooele County, Utah: Technical report for the Dames and Moore Utah SSC Proposal Team, June 1987, 107 pp.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch Front area, Utah, in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500-A-J, pp. D1-D36.

Arabasz, W.J., Smith, R.B., and Richins, W.D., 1980, Earthquake studies along the Wasatch Front, Utah: Network monitoring, seismicity, and seismic hazards: Bulletin of Seismological Society of America, vol. 70, pp. 1479-1499.

Ashcroft, G.L., D.T. Jensen and J. L. Brown, 1992, Utah climate: Logan, UT, Utah Climate Center, Utah State University, 127 p.

ASTM D5333-92, Standard Test Method for Measurement of Collapse Potential of Soils, American Society for Testing and Materials, Philadelphia, PA, 1992.

Atwood, G., and Mabey, D.R., 1995, Flooding hazards associated with Great Salt Lake, in Lund, W.R., ed., Environmental and engineering geology of the Wasatch Front Region: Utah Geological Assoc. Pub. 24, pp. 483-493.

Baer, J.L. and Bensen, A.K., 1987, Results of gravity survey, Skull Valley - Ripple Valley, Tooele County, Utah, in Dames and Moore, The Ralph M. Parsons Company, and Roger Foott Associates, Inc. (preparers), Site Proposal for the Superconducting Super Collider, Geotechnical Report, v. 2, pages E1-E8.

Balmer, G.G., 1958, "Shear Strength and Elastic Properties of Soil-cement Mixtures Under Triaxial Loading", Portland Cement Association, Bulletin No. D 32.

Barnhard, T.P. and Dodge, R.L., 1988, Map of fault scarps formed on unconsolidated sediments, Tooele 1° x 2° quadrangle, northwestern Utah: U.S. Geological Survey Miscellaneous Field Studies Map MF-1990, scale 1:250,000.

Bay Geophysical Associates, Inc., 1999, High-resolution seismic shear-wave reflection profiling for the identification of faults at the Private Fuel Storage Facility, Skull Valley, Utah-final report, prepared for Stone and Webster Engineering Corp., Denver, CO, 16 pp.

BLM, 1985, Skull Valley Allotment Management Plan. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. August, 1985.

BLM, 1986, South Skull Valley Allotment Management Plan. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. January, 1986.

BLM, 1988, Bureau of Land Management, Draft Pony Express Resource Management Plan and Environmental Impact Statement. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. May 1988.

BLM, 1992, Horseshoe Springs Habitat Management Plan. UT-020-WHA-T-7. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. February 26, 1992

Board Decision March 2003, Private Fuel Storage Facility, LLC (Independent Spent Fuel Storage Installation), LBP-03-04, 57 NRC 69 (2003).

Board Decision February 2005, (Independent Spent Fuel Storage Installation), LBP-05-29, 62 NRC 642 (February 24 [as redacted October 28], 2005).

Casagrande, A. and W.L. Shannon, 1948. Strength of Soils under Dynamic Loads. Proceedings, ASCE, Vol. 74, No.4, April, pp. 591-608.

CEC, 2001, PFSF Calculation 05996.02-G(PO17)-2, Rev 3, Storage Pad Analysis and Design, prepared by International Civil Engineering Consultants, Inc, for Stone and Webster Engineering Corp, Denver, CO.

Christie-Blick, N., 1983, Structural geology of the southern Sheeprock Mountains, Utah: regional significance, in Miller, D.M., Todd, V.R., and Howard, K.A., editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 101-124.

Coffman, J.L. and von Hake, C.A., 1973, Earthquake history of the United States, revised edition (through 1970): U.S. Dept. of Commerce - NOAA Publication 41-1, 208 pp.

Commission Decision November 2001, Private Fuel Storage, LLC (Independent Spent Fuel Storage, Installation), CLI-01-22, 54 NRC 255 (2001).

ConeTec, 1998, "Cone Penetration Testing - Geotechnical Applications Guide", October, 1998.

ConeTec, 1999, Cone penetration testing report, Private Fuel Storage Facility, prepared for Stone and Webster Engineering Corp., Denver, CO, 2 volumes.

Cook, K.L., Bankey, V., Mabey, D.R., and DePangher, M., 1989, Complete Bougher gravity anomaly map of Utah: Utah Geological and Mineral Survey Map 122, scale 1:500,000.

Cornell January 2004, C. Allin Cornell, Probability Assessment of the Aircraft Crash Impact Hazard for the Private Fuel Storage Facility Based on Engineering Evaluations of Storage Cask and Canister Transfer Building Structural Integrity (Rev. 1) (Jan. 2004).

Currey, D.R., 1990, Quaternary paleolakes in the evolution of semi-desert basins, with special emphasis on Lake Bonneville and the Great Basin, U.S.A.: Paleogeography, Paleoclimatology, and Paleoecology, vol. 76, pp. 189-214.

Das, B. M., Principles of Soil Dynamics, PWS Publishing Co., Boston, MA, 1993, 570 pp.

Davis, P.R., et al., 1998, Accident Analysis for Continued Storage, Oct. 1998, cited in Department of Energy, Draft Environmental Impact Statement for a Geologic Repository for the Disposal of Spent Nuclear Fuel and High-Level Radioactive Waste at Yucca Mountain, Nye County, Nevada, July 1999, p. 7-31.

DeGroot, G., 1976, "Bonding Study on Layered Soil Cement", REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976.

Department of Energy (DOE), 1996, DOE Standard, Accident Analysis for Aircraft Crash into Hazardous Facilities, DOE-STD-3014-96, Oct. 1996, Appendix B.

dePolo, C.M., 1994, The maximum background earthquake for the Basin and Range Province, western North America: Bulletin of the Seismological Society of America, v. 84, pp. 466-472.

dePolo, C.M., Clark, D.G., Slemmons, D.B., and Aymard, W.G., 1989, Historical Basin and Range Province surface faulting and fault segmentation, in Schwartz, D.P., and Sibson, R.H., editors, Fault segmentation and controls of rupture initiation and termination--proceedings of conference XLV: U.S. Geological Survey Open-file Report 89-315, pp. 131-162.

Dudley, J. H., 1970, Review of Collapsing Soils, Proceedings of the ASCE, Journal of the Soil Mechanics and Foundation Division, Vol 96, No. SM3, March 1970, pp 925-947.

Dupas, J., and Pecker, A., 1979, "Static and Dynamic Properties of Sand-Cement," *Proceedings ASCE*, JGED, Vol. 105, GT 3, March 1979, pp 419-436.

Everitt, B.L. and Kaliser, B.N., 1980, Geology for assessment of seismic risk in the Tooele and Rush Valleys, Tooele County, Utah: Utah Geological and Mineral Survey Special Study 51, 33 pp.

Federal Aviation Administration, Office of Aviation Policy and Plans (FAA), 1999, FAA Long-Range Forecasts, Fiscal Years 2015, 2020 and 2025, FAA-APO-99-5, June 1999.

Federal Highway Administration (FHWA), 1977, Runoff Estimate for Small Rural Watersheds and Development of a Sound Design Method, 248 pp.

Felt, E.J. and Abrams, M.S., 1957, "Strength and Elastic Properties of Compacted Soil-Cement Mixtures, Portland Cement Association, Bulletin No. D 16.

Everitt, B.L. and Kaliser, B.N., 1980, Geology for assessment of seismic risk in the Tooele and Rush Valleys, Tooele County, Utah: Utah Geological and Mineral Survey Special Study 51, 33 pp.

Geomatrix Consultants, Inc, 1997, PFSF Calculation 05996.01-G(PO5)-1, "Development of Soil and Foundation Parameters in Support of Dynamic Soil-Structure Interaction Analyses," San Francisco, CA, March (73 pp).

Geomatrix Consultants, Inc., 2001a, Fault evaluation study and seismic hazard assessment study-final report, Revision 1, prepared for Stone and Webster Engineering Corp., Denver, CO, 3 volumes, March 2001.

Geomatrix Consultants, Inc., 2001b, Development of design ground motions for the Private Fuel Storage Facility, Revision 1, prepared for Stone and Webster Engineering Corp., Denver, CO, 19 pp, March 2001.

Geomatrix Consultants, Inc., 2001c, PFSF Calculation 05996.02-G(PO18)-2, Rev 1, Soil and Foundation Parameters for Dynamic Soil-Structure Interaction Analyses, 2,000-Year Return Period Design Ground Motions, prepared for Stone and Webster Engineering Corp., Denver, CO, 53 pp, March 2001.

Gibbs, H. J., and W. G. Holtz, 1957, "Research on Determining the Density of Sands by Spoon Penetration Testing," *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering* (London), Vol I, 35-39.

Gilbert, G.K., 1890, Lake Bonneville, U.S. Geological Survey Monograph 1, 428 pp.

Goter, S.K., compiler, 1990, Earthquakes in Utah, 1884-1989: U.S. Geological Survey, National Earthquake Information Center, scale 1:500,000.

Grazulis, Thomas P., 1993, Significant tornadoes 1680 - 1991: Published by The Tornado Project of Environmental Films, St. Johnsbury, Vermont.

Hecker, Suzanne, 1993, Quaternary tectonics of Utah with emphasis on earthquake-hazard characterization: Utah Geological Survey Bulletin 127, Salt Lake City, UT, 156 pp.

Helm, J.M., 1994, Structure and tectonic geomorphology of the Stansbury fault zone, Tooele County, Utah, and the effect of crustal structure on Cenozoic faulting patterns, M.S. thesis, Univ. of Utah, Salt Lake City, Utah, 128 pp.

Helm, J.M., 1995, Quaternary faulting in the Stansbury fault zone, Tooele County, Utah, in Lund, W.R., editor, Environmental and engineering geology of the Wasatch Front Region: Utah Geological Association Publication 24, pp. 31-44.

Hintze, L.H. (compiler), 1980, Geologic Map of Utah, Utah Geological and Mineral Survey, Salt Lake City, UT, scale 1:500,000.

Holtec 2004, Updated Structural Evaluation of An F16 Aircraft Impact on Hi-Storm Overpacks at the PFS Facility for Private Fuel Storage, Holtec Report No: HI-2033134 (Rev. 2) (2004).

Holzworth, G.C., 1972, Mixing heights, wind speeds, and potential for urban air pollution throughout the contiguous United States: Environmental Protection Agency, Office of Air Programs, Research Triangle Park, North Carolina.

Holzworth, G.C., 1974, Meteorological episodes of slowest dilution in contiguous United States: National Environmental Research Center, Research Triangle Park, North Carolina, Report No. EPA-650/4-74-002.

Hood,J.W., and Waddell,K.M., 1968, Hydrologic reconnaissance of Skull Valley, Tooele County, Utah, DNR Tech Pub. No. 18, 57 pp.

Hosler, Charles R., 1961, Low-level inversion frequency in the contiguous United States: Monthly Weather Review, pp. 319-339.

Howard, K.A. editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 61-73.

Huschke, R. E., ed., 1959, Glossary of meteorology: Published by the American Meteorological Society, Boston, Massachusetts.

Johnson, J.B., and Cook, K.L., 1957, Regional gravity survey of parts of Tooele, Juab, and Millard Counties, Utah: Geophysics, vol. 22, pp. 48-61.

Kimura, C.Y. and Budnitz, R.J., 1987, Evaluation of External Hazards to Nuclear Power Plants in the United States, Lawrence Livermore National Laboratory, NUREG/CR-5042, p. 6-25.

Lambe, T.W., and R.V. Whitman, 1969, Soil Mechanics, John Wiley & Sons, Inc., N.Y., 553 pp.

Lancaster July 2003, Lanny T.G. Lancaster, Colonel, USAF (Ret.) Associate, Burdeshaw Associates, LTD., Assessment of Jettisoned Ordnance Impact Effects for the Private Fuel Storage Facility (July 11, 2003).

Lund, W.R., Christenson, G.E., Harty, K.M., Hecker, S., Atwood, G., Case, W.F., Gill, H.E., Gwynn, J.W., Klauk, R.H., Mabey, D.R., Mulvey, W.E., Sprinkel, D.A., Tripp, B.T., Black, W.D., and Nelson, C.V., 1990, Geology of Salt Lake City, Utah, U.S.A.: Assoc. of Engineering Geologists Bull., vol. XXVII, pp. 391-478.

Lunne, T., Robertson, P. K., and Powell, J. J. M., (1997), Cone Penetration Testing in Geotechnical Practice, Blackie Academeic & Professional, London, 1997.

Machette, M.N., and Scott, W.E., 1988, Field trip introduction-A brief review of research on lake cycles and neotectonics of the eastern Basin and Range province: Utah Geological and Mineral Survey Misc. Publ. 88-1, p. 7-14.

Malde, H.E., 1968, The catastrophic late Pleistocene Bonneville flood in the Snake River plain, Idaho: U.S. Geological Survey Professional Paper 596, 52 pp.

Moore, W.J., and McKee, E.H., 1983, Phanerozoic magmatism and mineralization in the Tooele 1° x 2° quadrangle, Utah, in Miller, D.M., Todd, V.R., and Howard, K.A., eds., Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 183-190.

Moore, W.J., and Sorensen, M.L., 1979, Geologic map of the Tooele 1° x 2° quadrangle: U.S. Geological Survey Miscellaneous Investigations Series Map I-1132, scale 1:250,000.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1960, Climatography of the United States No. 60, Climate of Utah.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1992, Local climatological data, annual summary with comparative data for 1991: Salt Lake City, Utah.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1975-1995, Storm data and unusual weather phenomena with late reports and corrections.

National Oceanic and Atmospheric Administration, National Climatic Data Center, U.S. Department of Commerce, "Solar and Meteorological Surface Observation Network, 1961 – 1990", Volume III, Western U.S., Version 1.0, September 1993.

NAVFAC, 1982, DM 7.1, Soil Mechanics, Dept of the Navy, Naval Facilities Eng'g, Command, Alexandria, VA.

NAVFAC, 1986, DM 7.2, Foundations and Earth Structures, Dept of the Navy, Naval Facilities Eng'g, Command, Alexandria, VA

Newmark, N. M., 1965, Effects of earthquakes on dams and embankments, Fifth Rankine Lecture, Geotechnique, Institution of Civil Engineers, London, 15(2), pp139-60.

Newmark, N. M., and Rosenblueth, E., 1971, Fundamentals of Earthquake Engineering, Prentice-Hall, Englewood Cliffs, NJ.

Northland Geophysical, LLC, 2001, Report No. 05996.02-G(PO37)-1, Report on Donwhole Seismic Geophysical Testing, Revision 1, March 2001.

Nussbaum, P. J., and Colley, B. E., 1971, *Dam Construction and Facing with Soil-Cement*, Portland Cement Association Publication RD010.01W.

Oaks, S.D., 1987, Effects of six damaging earthquakes in Salt Lake City, Utah, in Gori. P.L., and Hays, W.W., editors, Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Open-file Report 87-585, vol. 2, pp. P-1-95.

Oviatt, C.G., Currey, D.R., and Miller, D.M., 1990, Age and paleoclimatic significance of the Stansbury shoreline of Lake Bonneville, northwestern Great Basin: Quaternary Research, vol. 33, pp. 291-305.

Pacific Gas and Electric Company, 1988, Final Report of the Diablo Canyon Long Term Seismic Program, Docket Nos. 50-275 and 50-323, July 31, 1988.

Pasquill, F., 1961, The estimation of the dispersion of windborne material: Meteorol. Mag., 90, 1063, 33-49.

Pechmann, J.C. and Arabasz, W.J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, in Lund, W.R., editor, Environmental and engineering geology of the Wasatch Front region: Utah Geological Association Publication 24, pp. 77-93.

PFS Letter, Parkyn to Delligatti (NRC), Request for Exemption to 10 CFR 72.102(f)(1), dated April 2, 1999.
PFS Letter, Parkyn to U.S. NRC Document Control Desk, Request for Exemption to 10 CFR 72.102(f)(1), dated August 24, 1999.

PFS August 2000, "Report to Nuclear Regulatory Commission, Aircraft Crash Impact Hazard at the Private Fuel Storage Facility," Revision 4, August 11, 2000; with Revised Addendum dated July 20, 2001.

PFS January 2001, "Risk Assessment of Cruise Missile Accidents Impacting Private Fuel Storage LLC Independent Spent Fuel Storage Installation", Revision 1, dated January 25, 2001.

PFS Letter, Parkyn to U.S. NRC Document Control Desk, Data Needed for NRC Review of License Amendment #22, dated May 31, 2001.

PFS Hearing Tr. 2002, Private Fuel Storage, LLC (Independent Spent Fuel Storage Installation), NRC Hearing Transcript, (May 14,2002).

Portland Cement Association, 1971, "Soil-Cement Laboratory Handbook," Skokie, IL.

Pyke, R., H. B. Seed, and C. K. Chan, 1975, "Settlement of Sands Under Multidirectional Shaking," *Journal of the Geotechnical Engineering Division*, ASCE, 101(4), 379-398.

Ramsdell, J. V. and G. L. Andrews, 1986, Tornado climatology of the contiguous United States: Prepared by Pacific Northwest Laboratory for the U.S. Nuclear Regulatory Commission, NUREG/CR-4461, PNL-5697.

Rigby, J.K., 1958, Geology of the Stansbury Mountains, Tooele County, Utah: Utah Geological Society Guidebook 13, 168 pp.

Roberts, R.J., Crittenden, M.D., Jr., Tooker, E.W., Morris, H.T., Hose, R.K., and Cheney, T.M., 1965, Pennsylvanian and Permian basins in northwestern Utah, northeastern Nevada and south-central Idaho: Amer. Assoc. Petrol. Geologists Bulletin, vol. 49, pp. 1926-1956.

Robertson, P. K., and Campanella, R. G., 1988, Guidelines for Use, Interpretations, and Application of the Cone Penetration and Piezocone Penetration Test, *Soil Mechanics Series No. 105*, Department of Civil Engineering, University of British Columbia, Vancouver, Canada.

Rollins, K. M., and Williams, T., 1991, Collapsible Soil Hazard Mapping for Cedar City, Utah," Proceedings of the 1991 Annual Symposium on Engineering Geology & Geotechnical Engineering, No. 27, Pocatello, Idaho State University, 31 1.

Sack, Dorothy, 1993, Quaternary geologic map of Skull Valley, Tooele County, Utah: Utah Geological Survey Map 150, Scale 1:100,000, 16 p.

Sbar, M.L., and Barazangi, M., 1970, Tectonics of the intermountain seismic belt, western United States, Part I, microearthquake seismicity and composite fault plane solutions: Geological Society of America Abst. with Programs, vol. 2, p. 675.

Schimming, B.B., H.J. Haas, and H.C. Saxe, 1966. Study and Dynamic and Static Envelopes. Journal of Soil Mechanics and Foundation Division, ASCE Vol. 92, No. SM2 (March), pp. 105-24.

Schmertmann, J. H., 1970, "Static cone to compute static settlement over sand," Journal of the Soil Mechanics and Foundations Division, ASCE, 96(SM3), 1011-43.

Schmertmann, J. H., 1978, "Guidelines for cone penetration test, performance and design," US Federal Highway Administration, Washington, D.C., Report FHWA TS-78-209, 145.

Scott, W.E., 1988, Temporal relations of lacustrine and glacial events at Little Cottonwood Canyon and Bells Canyon, Utah, in Machette, M.N. and Currey, D.E., editors: In the footsteps of G.K. Gilbert - Lake Bonneville and neotectonics of the eastern Basin and Range Province, guidebook for field trip twelve, Utah Geological and Mineral Survey Misc. Publ. 88-1, pp. 78-82.

Seed, H. B., and Whitman, R. V., 1970, "Design of Earth Retaining Structures for Dynamic Loads," ASCE Specialty Conference on Lateral Stresses in the Ground and the Design of Earth Retaining Structures, pp 103-147.

Silver, M. and Seed, H. B., 1971, "Volume Changes in Sands During Cyclic Loading," Proceedings of the American Society of Civil Engineers, Journal of the Soil Mechanics and Foundations Division, Vol 97, SM9, September.

Simiu, E., M. J. Changery, and J. J. Filliben, 1979, Extreme wind speeds at 129 stations in the contiguous United States, NBS building science series 118: U.S. Department of Commerce, National Bureau of Standards.

Slemmons, D.B., 1980, Design earthquake magnitudes for the western Great Basin, in Proc. of Conference X, Earthquake hazards along the Wasatch-Sierra Nevada frontal fault zones: U.S. Geological Survey Open-file Report 80-801, pp. 62-85.

Smith, R.B., 1978, Seismicity, crustal structure, and intraplate tectonics of the interior of the western Cordillera, in Smith, R.B., and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, pp. 111-144.

Smith, R.B., and Arabasz, W.J., 1991, Seismicity of the intermountain seismic belt, in Slemmons, D.B., Engdahl, E.R., Zoback, M.D., and Blackwell, D.C., eds., Neotectonics of North America: Geological Society of America, Decade Map Volume 1, pp. 185-228.

Smith, R.B., and Sbar, M.L., 1970, Seismicity and tectonics of the intermountain seismic belt, western United States, Part II, Focal mechanism of major earthquakes: Geological Society of America Abst. with Programs, vol. 2, p. 657.

Smith, R.B., and Sbar, M.L., 1974, Contemporary tectonics and seismicity of the western United States with emphasis on the intermountain seismic belt: Geological Society of America Bulletin, vol. 85, pp. 1205-1218.

Smith, R.B., Nagy, W.C., Julander, D.R., Viveiros, J.J., Baker, C.A., and Gants, D.G., 1989, Geophysical and tectonic framework of the eastern Basin and Range-Colorado Plateau-Rocky Mountain transition, in Pakiser, L.C., and Mooney, W.D., eds., Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 205-233.

Soler July 2004, Testimony of Alan I. Soler on the Structrual Effects of a Potential F-16 Impact on a Spent Fuel Cask Contention Utah K/Confederated Tribes B (July 12, 2004).

Soler August 2004, Surrebuttal Testimony of Alan I. Soler on Jettisoned Ordnance Impact Effects at the PFSF – Contention Utah K/Confederated Tribes B (August 9, 2004).

Stewart, J.H., 1976, Late Precambrian evolution of North America: plate tectonic implication: Geology, vol. 4, pp. 11-15.

Stewart, J.H., 1978, Basin-range structure in western North America: A review, in Smith, R.B. and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, pp. 1-31.

Stickney, M.C., and Bartholomew, M.J., 1987, Seismicity and late Quaternary faulting of the northern Basin and Range province, Montana and Idaho: Seismological Society of America Bulletin, vol. 77, pp. 1602-1625.

Stokes, W.L., 1986, Geology of Utah, Utah Museum of Natural History and Utah Geological and Mineral Survey, Salt Lake City, UT, 280 pp.

Stone & Webster Engineering Corporation (SWEC), 1995. Evaluation of H-Piles, Waste Packaging Area (WPA), and Condensate Demineralizer Waste Evaporator Building (CDWEB), Sequoyah Nuclear Power Plant – Units 1 and 2 (FSAR issues). Tennessee Valley Authority, SE-CEB-SWEC. Calculation No. SCG1S505, Revision 0 (April).

Stone & Webster Engineering Corporation (SWEC), 1998a, Calculation No. 05996.02-SC-6, Revision 0, Finite Element Analysis of Canister Transfer Building.

Stone & Webster Engineering Corporation (SWEC), 1999a, Calculation No. 05996.02-G(B)-12, Revision 1, Flood Analysis with a Larger Drainage Basin.

Stone & Webster Engineering Corporation (SWEC), 1999b, Calculation No. 05996.02-G(B)-15, Revision 2, Determination of Aquifer Permeability from Constant Head Test and Estimation of Radius of Influence for the Proposed Water Well.

Stone & Webster Engineering Corporation (SWEC), 1999c, Calculation No. 05996.02-G(B)-16, Revision 1, Flood Analysis at 3-mile Long Portion of Rail Spur.

Stone & Webster Engineering Corporation (SWEC), 1999d, Calculation No. 05996.02-G(B)-17, Revision 1, PMF Flood Analysis with Proposed Access Road and Rail Road.

Stone & Webster Engineering Corporation (SWEC), 1999e, Calculation No. 05996.02-G(B)-3, Revision 3, Estimate Static Settlement of Storage Pads.

Stone & Webster Engineering Corporation (SWEC), 2000a, Calculation No. 05996.02-G(B)-11, Revision 3, Dynamic Settlements of the Soils Underlying the Site.

Stone & Webster Engineering Corporation (SWEC), 2000b, Calculation No. 05996.02-G(C)-14, Revision 1, Static Settlement of the Canister Transfer Building Supported on a Mat Foundation.

Stone & Webster Engineering Corporation (SWEC), 2000c, Calculation No. 05996.02-G(B)-5, Revision 2, Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria.

Stone & Webster Engineering Corporation (SWEC), 2001a, Calculation No. 05996.02-G(B)-21, Revision 0, Supplement to Estimated Static Settlement of Cask Storage Pads.

Stone & Webster Engineering Corporation (SWEC), 2001b, Calculation No. 05996.02-G(B)-4, Revision 9, Stability Analyses of Storage Pad.

Stone & Webster Engineering Corporation (SWEC), 2001c, Calculation No. 05996.02-G(B)-13, Revision 6, Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation.

Stone & Webster Engineering Corporation (SWEC), 2001d, Calculation No. 05996.02-SC-5, Revision 2, Seismic Analysis of Canister Transfer Building.

Stone & Webster Engineering Corporation (SWEC), 2001e, Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW No. 05996.02-G010, Revision 0.

Stone & Webster Engineering Corporation (SWEC), July 2003, Report No. 59056 (Task 0100)-SC-1 Rev.), Evaluation of Aircraft Impact on the Canister Transfer Building (July 7, 2003).

Stover, C.W. and Coffman, J.L., 1993, Seismicity of the United States, 1568-1989 (Revised): U.S. Geological Survey Professional Paper 1527, 418 pp.

Stover, C.W., Reagor, B.G., and Algermissen, S.T., 1986, Seismicity map of the State of Utah, U.S. Geological Survey Miscellaneous Field Studies Map MF-1856, scale 1:1,000,000.

Terzaghi, K, "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, Vol 5, 1955, pp 297-325.

Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, NY, 1967, pp 347 and 491.

Thom, H. C. S., 1963, Tornado probabilities: Monthly Weather Review 91, pp. 730-736.

Tokimatsu, A. M., and H. B. Seed, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," *Journal of the Geotechnical Engineering Division*, ASCE, 113(8), 861-878.

Tooele County Commission, 1995, Brochure entitled "Tooele County, Utah, Where Land And Sky Embrace."

Tooele, 1995. Tooele County General Plan, November 1995.

Tooker, E.W., 1983, Variations in structural style and correlation of thrust plates in the Sevier foreland thrust belt, Great Salt Lake area, Utah, in Miller, D.M., Todd, V.R., and Howard, K.A. editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 61-73.

Tooker, E.W., and Roberts, R.J., 1971, Structures related to thrust faults in the Stansbury Mountains, Utah: U.S. Geological Survey Professional Paper 750-B, pp. B1-B12.

U.S. Air Force Accident Investigation Board Report, AGM-129 Advanced Cruise Missile, Serial # 90-0061, U.S. Air Force, December 10, 1997, Volume 1.

U.S. Army Corps of Engineers, 1952, Standard Project Flood Determinations, Civil Engineer Bulletin, No. 52-8, Washington, D.C., 19 pp.

U.S. Army Corps of Engineers, 1990, Office of the Chief of Engineers, Flood hydrograph package, HEC-1, Hydrologic Engineering Center, 283 pp.

U.S. Army Corps of Engineers, 1997, Hydrologic Engineering Center, River analysis system, HEC-RAS, Davis, CA.

U.S. Department of Agriculture, undated, Soil survey of Tooele County, Utah, unpublished maps and data, National Resources Conservation Service, Tooele, UT.

U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1977, Probable maximum precipitation estimates, Colorado River and Great Basin drainage, Hydrometeorological Report No. 49 (HMR 49), 161 pp.

U.S. Geological Survey, 1994, Methods for estimating magnitude and frequency of floods in the southwestern United States, Open-File Report 93-419, 211 pp.

U.S. Nuclear Regulatory Commission, 1991, Safety Evaluation Report related to the operation of Diablo Canyon Nuclear Power Plant Units 1 and 2, NUREG-0675, Supplement No. 34, June 1991.

U.S. Weather Bureau, 1947, Thunderstorm rainfall. Hydrometeorological Report No. 5, Department of Commerce, Washington, D.C., 330 pp.

Vucetic, M., and R. Dobry, 1991, "Effect of Soil Plasticity on Cyclic Response," Journal of the Geotechnical Engineering Division, ASCE, 117(1), 89-107.

Wells, D.L., and Coppersmith, K.J., 1994, Analysis of empirical relationships among magnitude, rupture length, rupture area, and surface displacement: Seismological Society of America Bulletin, vol. 84, pp. 974-1002.

Winterkorn, H. F., and Fang, H., 1975, Foundation Engineering Handbook, Van Nostrand Reinhold Co., New York, NY, 751 pp.

Wong, I., Olig, S., Green, R., Moriwaki, Y., Abrahamson, N., Baures, D., Silva, W., Somerville, P., Davidson, D., Pilz, J., and Dunne, B., 1995, Seismic hazard analysis of the Magna tailings impoundment, in Lund, W.R., ed., Environmental and engineering geology of the Wasatch Front Region: 1995 Symposium and Field Conference, Utah Geological Association Publication 24, pp. 95-110.

Youngs, R.R., Swan, F.H., III, Power, M.S., Schwartz, D.P., and Green, R.K., 1987, Probabilistic analysis of earthquake ground shaking hazard along the Wasatch Front, Utah, in Gori. P.L. and Hays, W.W., editors, Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Open-File Report 87-585, Vol. 2, pp. M-1-110.

Zoback, M.L., 1983, Structure and Cenozoic tectonism along the Wasatch fault zone, in Miller, D.M., Todd, V.R., and Howard, K.A., editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 3-27.

Zoback, M.L., and Zoback, M.D., 1989, Tectonic stress field of the continental United States, in Pakiser, L.C., and Mooney, W.D., eds., Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 523-539.

TABLE 2.3-1

SUMMARY OF TORNADO DATA FOR PFSF SITE 1° BOX
(1880 - 1995)*

COUNTY	DATE	PATH LENGTH (MILES)	PATH WIDTH (YARDS)	F-SCALE
Tooele	05/03/1993	0.1	15	F1
Tooele	09/23/1992	N/A	N/A	F0
Tooele	08/30/1992	0.8	200	F0
Tooele	07/25/1991	10	100	F1

* Period of record is 1880 to December 1995.

TABLE 2.3-2

FUJITA TORNADO INTENSITY SCALE

	Classification	Wind Speed (mph)	Description of Damage
F0	Gale Tornado	40 - 72	Light damage. Some damage to chimneys; breaks branches off trees; pushes over shallow-rooted trees; damages sign boards.
F1	Moderate Tornado	73 - 112	Moderate damage. The lower limits is the beginning of hurricane wind speed; peels surface off roofs; mobile homes pushed off foundations or overturned; moving autos pushed off the roads; attached garages may be destroyed.
F2	Significant Tornado	113 - 157	Considerable damage. Roofs torn off frame houses; mobile homes demolished; boxcars pushed over; large trees snapped or uprooted; light-object missiles generated.
F3	Severe Tornado	158 - 206	Severe damage. Roof and some walls torn off well-constructed houses; trains overturned; most trees in forest uprooted; heavy cars lifted off ground and thrown.
F4	Devastating Tornado	207 - 260	Devastating damage. Well-constructed houses leveled; structures with weak foundations blown off some distance; cars thrown and large missiles generated.
F5	Incredible Tornado	261 - 318	Incredible damage. Strong frame houses lifted off foundation and carried considerable distances to disintegrate; automobile-sized missiles fly through the air in excess of 100 meters; trees debarked; steel-reinforced concrete structures badly damaged.
F6	Inconceivable Tornado	319 - 379	These winds are very unlikely. The small area of damage they might produce would probably not be recognizable along with the mess produced by the F4 and F5 wind that would surround the F6 winds. Missiles, such as cars and refrigerators, would do serious secondary damage that could not be directly identified as F6 damage. If this level is ever achieved, evidence for it might only be found in some manner of ground swirl pattern, for it may never be identifiable through engineering studies.

TABLE 2.3-3

NORMAL MONTHLY PRECIPITATION FOR SALT LAKE CITY, DUGWAY, IOSEPA SOUTH RANCH, AND PFSF

	PRECIPITATION (inches)			
MONTH	SALT LAKE CITY[1]	DUGWAY[2]	IOSEPA RANCH[3]	PSFS Site[4]
January	1.35	0.46	0.97	0.42
February	1.33	0.57	0.59	0.48
March	1.72	0.84	1.05	0.37
April	2.21	0.81	1.44	0.93
May	1.47	1.06	1.26	0.72
June	0.97	0.53	0.64	3.16
July	0.72	0.57	0.47	1.23
August	0.92	0.61	0.63	0.60
September	0.89	0.72	0.15	0.96
October	1.14	0.81	0.65	0.74
November	1.22	0.58	0.82	0.20
December	1.37	0.59	0.98	0.38
Annual	15.31	8.15	9.64	10.16

1. Period of record for Salt Lake City is 1951 - 1980.
2. Period of record for Dugway is 1950 - 1992.
3. Period of record for Iosepa South Ranch is 1951 - 1958.
4. Period of record for PFSF Site is 12/96 – 12-98.

TABLE 2.3-4

NORMAL MONTHLY TEMPERATURES (°F) FOR SALT LAKE CITY[1], DUGWAY[2], IOSEPA SOUTH RANCH,[3] AND PFSF[4]

MONTH	DAILY MAXIMUM				DAILY MINIMUM				AVERAGE			
	SLC	DUG	IOSEP	PFSF	SLC	DUG	IOSEP	PFSF	SLC	DUG	IOSEP	PFSF
January	**37**	**37**	**42**	**41**	**20**	**15**	**17**	**20**	**29**	**26**	**29**	**31**
February	44	45	46	42	24	23	20	22	34	34	33	32
March	52	53	53	54	30	29	25	23	41	41	39	39
April	61	63	64	57	37	35	31	29	49	49	48	43
May	72	73	76	72	45	44	38	38	59	59	57	56
June	83	85	86	78	53	53	45	46	68	69	66	63
July	93	94	95	90	62	62	52	54	78	78	74	74
August	90	91	93	90	60	59	53	57	75	75	73	75
September	80	80	86	79	50	48	41	48	65	64	64	63
October	67	66	71	63	39	36	32	32	53	51	52	47
November	50	51	52	52	29	27	22	25	40	39	37	38
December	39	38	43	36	22	17	17	10	31	28	30	23

1. Period of record for Salt Lake City is 1951 - 1980.
2. Period of record for Dugway is 1950 - 1992.
3. Period of record for Iosepa South Ranch is 1951 - 1958.
4. Period of record for PFSF is 12/96 – 12/98.

TABLE 2.3-5

MEAN WIND SPEEDS AND PREVAILING DIRECTIONS FOR
SALT LAKE CITY[1] AND PFSF[2]

MONTH	WIND SPEED (MPH)		PREVAILING DIRECTION	
	Salt Lake City	PFSF Site	Salt Lake City	PFSF Site
January	7.6	8.8	SSE	SE
February	8.2	9.1	SE	ESE
March	9.4	8.9	SSE	SE
April	9.6	9.6	SE	ESE
May	9.5	9.2	SE	SE
June	9.4	9.3	SSE	SE
July	9.6	8.5	SSE	SSE
August	9.7	9.1	SSE	SSE
September	9.1	8.2	SE	SSE
October	8.5	8.6	SE	SE
November	8.0	7.4	SSE	SE
December	7.4	7.4	SSE	SE

1. Period of record is 1951 - 1980.
2. Period of record is 12/96 – 12/98

TABLE 2.3-6

FREQUENCY OF OCCURRENCE OF
ATMOSPHERIC STABILITY CLASSES FOR SALT LAKE CITY

STABILITY CLASS	FREQUENCY OF OCCURRENCE (percent)
A	0.70
B	6.34
C	14.94
D	43.05
E	17.93
F	17.04

1. Period of record is 1988 - 1992.

TABLE 2.3-7

MEAN SEASONAL MORNING AND AFTERNOON MIXING HEIGHTS FOR SALT LAKE CITY[1]

SEASON	MEAN MIXING HEIGHT (meters)	
	MORNING	AFTERNOON
Winter	329	944
Spring	419	2,675
Summer	216	3,737
Fall	238	1,933
Annual	300	2,322

1. Period of record is 1960 - 1964.

TABLE 2.3-8

NATIONAL AMBIENT AIR QUALITY STANDARDS

POLLUTANT	AVERAGING INTERVAL	PRIMARY STANDARD		SECONDARY STANDARD	
		µg/m^3	ppmv	µg/m^3	ppmv
SO_2	Annual	80	0.03	-	-
	24-hr	365	0.14	-	-
	3-hr	-	-	1,300	0.50
PM10	Annual	50	-	50	-
	24-hr	150	-	150	-
CO	8-hr	10[1]	9	10[1]	9
	1-hr	40[1]	35	40[1]	35
O_3	1-hr	235	0.12	235	0.12
NO_2	Annual	100	0.053	100	0.053
Pb	3 months	1.5-	1.5	-	

1. mg/m^3 (milligrams per cubic meter).

TABLE 2.3-9

AMBIENT AIR QUALITY MONITORING DATA FOR
WASATCH FRONT INTRASTATE AQCR

POLLUTANT	AVERAGING INTERVAL	SECOND HIGHEST OBSERVED VALUE (ppmv)			
		1993	1994	1995	AAQS
SO_2[1]	Annual	0.001	0.001	0.001	0.03
	24-hr	0.003	0.003	0.003	0.14
	3-hr	0.010	0.008	0.008	0.50
PM-10[2]	Annual	26.0	26.0	23.0	50
	24-hr	75.0	98.0	49.0	150
CO[3]	8-hr	6.0	6.0	5.0	9.0
	1-hr	12.0	11.0	NA	35.0
O_3[4]	1-hr	0.100	0.109	0.097	0.12
NO_2[5]	Annual	0.033	0.026	0.024	0.053

1. SO_2 data are from Grantsville, Tooele County.

2. PM-10 data are from Grantsville, Tooele County. Concentrations are in units of $\mu g/m^3$.

3. CO monitoring data from Cottonwood, Salt Lake County.

4. Ozone monitoring data from Herriman, Salt Lake County for 1994 and 1995, from Salt Lake for 1993.

5. NO_2 monitoring data from Salt Lake, Salt Lake County.

TABLE 2.3-10

METEOROLOGICAL MONITORING SYSTEM SPECIFICATIONS

<u>Wind Speed Sensor - Low Speed Operating Threshold</u>		
a.	Operating principle -	three cup anemometer utilizing a photochopper
b.	Range -	0.45 - 55 m/s
c.	Threshold -	< 0.45 m/s
d.	Accuracy -	±0.22 m/s for speeds < 2.2 m/s ±10% of measured value otherwise
e.	Operating temperature range -	-40°F to 120°F
f.	Response distance constant -	5 ft of flow for 63 percent recovery
<u>Wind Direction Sensor - Low Speed Operating Threshold</u>		
a.	Operating principle -	lightweight counterbalanced vane
b.	Range -	0° - 540° azimuth
c.	Threshold -	< 0.45 m/s
d.	Accuracy -	±5° azimuth
e.	Operating temperature range -	-4°F to 120°F
f.	Response distance constant -	< 2 meters of flow for 63 percent recovery
g.	Damping ratio -	0.4 - 0.6
<u>Temperature Sensors</u>		
a.	Range -	-40°C - 50°C
b.	Accuracy -	±0.5°C
c.	Sensitivity -	< 0.5°F
d.	Response time -	30 sec
Relative Humidity Sensor		
a.	Range -	0 - 100%
b.	Accuracy -	±1.5°C dew point equivalent (-30°C to +30°C)
c.	Operating temperature range -	-40°F to 120°F
Barometric Pressure Sensor		
a.	Range -	17.72 - 32.48 in Hg
b.	Accuracy -	±0.04 in Hg
Precipitation Gage		
a.	Operating principle -	tipping bucket type (all precipitation types)
b.	Range -	0 - 76 mm/hr
c.	Accuracy -	±10% of total accumulated catch for amts. >5 mm.
d.	Resolution -	0.25 mm
e.	Operating temperature range -	-40°F to 120°F
Solar Radiation Sensor		
a.	Sensitivity -	50 mV per cal/cm2/min
b.	Linearity -	±2%
c.	Response time -	< 1 millisecond

Table 2.3-11

AVERAGE RELATIVE HUMIDITY FOR SALT LAKE CITY[1] AND PFSF[2]

Month	Relative Humidity (percent)	
	Salt Lake City	PFSF Site
January	74.3	74.2
February	69.3	74.3
March	59.0	61.3
April	52.8	61.5
May	48.5	52.4
June	41.3	51.7
July	35.8	40.0
August	38.0	39.5
September	44.8	56.7
October	54.0	60.1
November	66.0	67.5
December	74.5	75.5

1. Average of the four time-of-day relative humidity values for a 32-year period of record
2. Hourly average for the period of record of 12/96 – 12/98

TABLE 2.6-1
LOW-STRAIN DYNAMIC SOIL PROPERTIES INPUT TO SHAKE

Best Estimate Velocity Profile

Constant Tertiary Velocity

Layer Base (ft)	H (ft)	Wave Velocity Vs (fps)	Wave Velocity Vp (fps)	Computed Poisson's Ratio	Density (pcf)
5	5	1500	2390	0.175	100
10	5	528	1131	0.361	80
12	2	727	1260	0.250	80
18	6	854	1472	0.246	100
26	8	872	1440	0.210	94
35	9	1021	1667	0.200	115
50	15	1190	2085	0.258	115
90	40	1800	3400	0.305	120
125	35	2900	5023	0.250	135
300	175	2900	5023	0.250	145
500	200	2900	5023	0.250	145
700	200	2900	5023	0.250	145
Halfspace		6398	11155	0.255	170

Increasing Tertiary Velocity

Layer Base (ft)	H (ft)	Wave Velocity Vs (fps)	Wave Velocity Vp (fps)	Computed Poisson's Ratio	Density (pcf)
5	5	1500	2390	0.175	100
10	5	528	1131	0.361	80
12	2	727	1260	0.250	80
18	6	854	1472	0.246	100
26	8	872	1440	0.210	94
35	9	1021	1667	0.200	115
50	15	1190	2085	0.258	115
90	40	1800	3400	0.305	120
125	35	2900	5023	0.250	135
300	175	2900	5023	0.250	145
500	200	4000	6928	0.250	145
700	200	5000	8660	0.250	145
Halfspace		6398	11155	0.255	170

High and Low Velocity Profiles

Layer Base (ft)	H (ft)	High Range Vs (fps)	High Range Vp (fps)	Low Range Vs (fps)	Low Range Vp (fps)
5	5	2121	3380	1061	1690
10	5	647	1385	431	923
12	2	890	1543	594	1029
18	6	1046	1803	697	1202
26	8	1068	1764	712	1176
35	9	1250	2042	834	1361
50	15	1683	2949	841	1474
90	40	2546	4808	1273	2404
125	35	4101	7104	2051	3552
300	175	4101	7104	2051	3552
500	200	5657	9798	2051	3552
700	200	6398	11155	2051	3552
Halfspace		6398	11155	6398	11155

Source: Geomatrix Consultants, Inc, 2001c

TABLE 2.6-2
DYNAMIC SOIL PARAMETERS FOR SASSI MODEL

High Range Properties							
Depth of Top (ft)	Depth of Bottom (ft)	Density (pcf)	Wave Velocity		Damping Ratio		Poisson's Ratio
			Vs (fps)	Vp (fps)	Shear (%)	Compression (%)	
0	5	100	2120	3380	0.91	0.91	0.176
5	10	80	557	1385	3.48	3.48	0.403
10	12	80	807	1543	2.69	2.69	0.312
12	18	100	983	1803	1.82	1.82	0.289
18	26	94	973	1764	2.31	2.31	0.281
26	35	115	1053	2042	5.07	5.07	0.319
35	50	115	1488	2949	4.04	4.04	0.329
50	90	120	2481	4808	1.21	1.21	0.318
90	125	135	4101	7104	4.28	4.28	0.250
125	300	145	4101	7104	4.28	4.28	0.250
300	500	145	5657	9798	3.10	3.10	0.250
500	700	145	6398	11155	2.53	2.53	0.255
700		170	6398	11155	2.16	2.16	0.255
Best Estimate Properties							
Depth of Top (ft)	Depth of Bottom (ft)	Density (pcf)	Wave Velocity		Damping Ratio		Poisson's Ratio
			Vs (fps)	Vp (fps)	Shear (%)	Compression (%)	
0	5	100	1497	2390	0.94	0.94	0.177
5	10	80	415	1131	4.78	4.78	0.422
10	12	80	622	1260	3.60	3.60	0.339
12	18	100	779	1472	2.29	2.29	0.306
18	26	94	760	1440	3.01	3.01	0.307
26	35	115	818	1667	6.21	6.21	0.341
35	50	115	956	2085	6.13	6.13	0.367
50	90	120	1716	3400	1.74	1.74	0.329
90	125	135	2900	5023	4.32	4.32	0.250
125	300	145	2900	5023	4.32	4.32	0.250
300	500	145	3450	5976	3.67	3.67	0.250
500	700	145	3950	6842	3.33	3.33	0.250
700		170	6398	11155	1.76	1.76	0.255
Low Range Properties							
Depth of Top (ft)	Depth of Bottom (ft)	Density (pcf)	Wave Velocity		Damping Ratio		Poisson's Ratio
			Vs (fps)	Vp (fps)	Shear (%)	Compression (%)	
0	5	100	1053	1690	1.08	1.08	0.183
5	10	80	298	923	6.57	6.57	0.442
10	12	80	622	1260	3.60	3.60	0.339
12	18	100	610	1202	2.97	2.97	0.327
18	26	94	593	1176	3.73	3.73	0.330
26	35	115	614	1361	8.09	8.09	0.372
35	50	115	565	1474	9.82	9.82	0.414
50	90	120	1191	2404	2.18	2.18	0.337
90	125	135	2051	3552	3.97	3.97	0.250
125	300	145	2051	3552	3.97	3.97	0.250
300	500	145	2051	3552	3.97	3.97	0.250
500	700	145	2051	3552	3.97	3.97	0.250
700		170	6398	11155	2.16	2.16	0.255

Source: Geomatrix Consultants, Inc, 2001c

TABLE 2.6-3
DYNAMIC SOIL PARAMETERS FOR SPRING, DASHPOT, AND MASS MODEL

	Upper Range	Best Estimate	Lower Range	
Vp	2205	1527	1157	
Vs	1322	842	579	
G (ksf)	5015	2027	955	
beta S (%)	2.3	3.3	4.6	
E (ksf)	12234	5194	2546	
beta P (%)	2.3	3.3	4.6	
Poisson's Ratio	0.220	0.281	0.333	
Unit Wt. (pcf)	92.4	92.0	91.8	
A (30x67) sqft	2010	2010	2010	
Aspect Ratio	2.233	2.233	2.233	
Vertical Mode				
h	12.10	12.10	12.10	
m (pcf-sec^2)	**34.75**	**34.58**	**34.52**	mass/area (pcf-sec^2)
kv (kcf)	**315.20**	**138.29**	**70.23**	spring constant/area (kcf)
c (kcf-sec)	**4.84**	3.20	**2.28**	dashpot constant/area (kcf-sec)
Horizontal Mode				
h	2.24	2.24	2.24	
Kappa T	0.937	0.892	0.760	
m (pcf-sec^2)	**6.43**	**6.40**	**6.39**	mass/area (pcf-sec^2)
kh (kcf)	**268.79**	**112.24**	**48.52**	spring constant/area (kcf)
c (kcf-sec)	**2.70**	**1.74**	**1.14**	dashpot constant/area (kcf-sec)
Rocking Mode				
h	15.69	15.69	15.69	
Kr	112978035.57	49565892.37	25172167.30	
C	538785.878	356027.756	253487.104	
m (pcf-sec^2)	**45.04**	**44.83**	**44.75**	mass/area (pcf-sec^2)
kr (kcf)	**736.87**	**323.28**	**164.18**	spring constant/area (kcf)
c (kcf-sec)	**3.57**	2.36	**1.68**	dashpot constant/area (kcf-sec)

Source: Geomatrix Consultants, Inc, 2001c

TABLE 2.6-4
(Page 1 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

Year	Date	Origin Time	Latitude	Longitude	Mag	Int	Comments
1850	222	2200	40° 44.94'	111° 50.95'	3.7 I	4	(18)
1853	1201	1815	39° 42.36'	111° 49.90'	4.3 I	5	(1)
1853	1201	1845	40° 14.35'	111° 39.33'	4.3 I	5	(1)
1868	1017	1030	39° 21.67'	111° 35.26'	3.0 I	3	THREE SHOCKS (1)
1873	1227	0300	40° 58.75'	111° 53.08'	3.7 I	4	
1874	618	0600	40° 44.94'	111° 50.95'	3.7 I	4	
1874	618	0700	40° 44.94'	111° 50.95'	3.7 I	4	(1)
1876	322		39° 31.64'	111° 34.89'	5.0 I	6	THREE SHOCKS (1)
1878	821	1200	40° 44.94'	111° 50.95'	3.0 I	3	
1878	907	1900	40° 44.94'	111° 50.95'	3.0 I	3	
1880	917	0627	40° 44.94'	111° 50.95'	3.7 I	4	INT=4-5
1880	1227		41° 42.00'	113° 6.60'	3.0 I	3	TWO SHOCKS,LATE PM (1)
1881	1016	0700	39° 32.54'	111° 27.35'	3.0 I	3	
1883	928	1100	39° 54.60'	112° 7.80'	3.7 I	4	
1884	1208		41° 13.45'	111° 57.55'	3.0 I	3	
1889	1207	1100	39° 15.83'	111° 38.23'	3.7 I	4	
1894	108	1800	39° 45.60'	113° 23.40'	4.3 I	5	(4) HAS WRONG DATE
1894	718	2250	41° 13.45'	111° 57.55'	5.0 I	6	INT=5-7
1895	727	2225	39° 32.54'	111° 27.35'	3.7 I	4	
1896	607	0530	39° 9.19'	111° 49.10'	3.0 I	3	
1896	913	0130	39° 42.36'	111° 49.90'	3.7 I	4	
1896	1003	1550	41° 44.26'	111° 49.85'	3.0 I	3	
1899	1213	1350	40° 44.94'	111° 50.95'	3.7 I	4	
1900	801	0745	39° 57.15'	112° 6.84'	5.7 I	7	
1901	811	1600	40° 44.94'	111° 50.95'	3.0 I	3	
1901	811	1800	40° 14.35'	111° 39.33'	3.0 I	3	
1903	723	0834	41° 5.00'	111° 55.00'	3.0 I	3	LOC ASSUMED (1)
1906	524	2110	41° 13.45'	111° 57.55'	4.3 I	5	THREE SHOCKS (1)
1909	1006	0250	41° 46.00'	112° 40.00'	6.3 I	8	INT=7-9
1909	1117	0630	41° 44.66'	112° 9.72'	4.3 I	5	
1910	522	1428	40° 44.94'	111° 50.95'	5.7 I	7	
1910	523	1545	40° 44.94'	111° 50.95'	3.0 I	3	A'SHOCK (1)
1914	408	1606	40° 59.00'	111° 55.00'	4.3 I	5	LOC ASSUMED (1)
1914	513	1715	41° 13.45'	111° 57.55'	5.7 I	7	INT=6-7
1915	715	2200	40° 14.35'	111° 39.33'	5.0 I	6	

TABLE 2.6-4
(Page 2 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

Year	Date	Origin Time	Latitude		Longitude		Mag	Int	Comments
1915	730	1850	41°	44.66 '	112°	9.72 '	4.3 I	5	ASSGN GARLAND,UT (1)
1915	811	1020	40°	30.00 '	112°	39.00 '	4.3 I	5	INT=5-8,LOC ASSUMED (1)
1915	920	0128	39°	59.58 '	111°	29.40 '	3.0 I	3	TWO SHOCKS (1)
1915	1003	0150	40°	44.94 '	111°	50.95 '	3.0 I	3	(1)
1915	1005	0800	40°	6.00 '	114°	0.00 '	4.3 I	5	INT=5-7,LOC FROM (4)
1916	205	0625	39°	58.37 '	111°	46.87 '	4.3 I	5	ASSGN SANTAQUIN,UT (1,3)
1919	507	2230	39°	31.64 '	111°	34.89 '	3.7 I	4	
1920	918	2010	41°	30.61 '	112°	0.95 '	4.3 I	5	INT=5-6
1920	919	1350	41°	30.61 '	112°	0.95 '	4.3 I	5	INT=5-6
1920	1120	0435	41°	30.61 '	112°	0.95 '	4.3 I	5	INT=5-6
1920	1217	0955	41°	30.61 '	112°	0.95 '	3.7 I	4	A'SHOCK? (1)
1923	607	0415	41°	44.26 '	111°	49.85 '	4.3 I	5	
1925	1201	0730	40°	44.94 '	111°	50.95 '	3.0 I	3	
1926	1219	0330	39°	57.00 '	111°	57.60 '	3.7 I	4	
1932	1111	1000	40°	31.04 '	111°	28.27 '	3.7 I	4	
1932	1221	0613	40°	44.94 '	111°	50.95 '	3.0 I	3	
1934	130	2021	40°	44.94 '	111°	50.95 '	3.0 I	3	
1934	312	1505	41°	42.00 '	112°	48.00 '	6.6 M	9	INT=8-9,PAS (5,8,9)
1934	312	1820	41°	42.00 '	112°	48.00 '	6.1 N	7	A'SHOCK,PAS (5,8,9)
1934	315	1202	41°	42.00 '	112°	48.00 '	5.1 N	6	INT ASSUMED,A'SHOCK (5)
1934	315	1347	41°	42.00 '	112°	48.00 '	4.8 N	5	INT ASSUMED,A'SHOCK (5)
1934	317	2240	41°	46.50 '	112°	5.70 '	3.0 I	3	
1934	414	2126	41°	30.00 '	112°	30.00 '	5.6 N	7	A'SHOCK (1,5,6,8,9)
1934	506	0809	41°	42.00 '	112°	48.00 '	5.6 N	6	A'SHOCK,PAS (5,8,9)
1935	709	1059	40°	44.94 '	111°	50.95 '	3.7 I	4	INT=4-5
1938	318		39°	59.58 '	111°	29.40 '	3.0 I	3	
1938	630	1337	40°	44.94 '	111°	50.95 '	4.3 I	5	SALT LK VALLEY? (1)
1939	331	0640	40°	44.94 '	111°	50.95 '	3.7 I	4	
1940	1123	1300	39°	15.83 '	111°	38.23 '	3.7 I	4	
1940	1125	1425	39°	15.83 '	111°	38.23 '	3.7 I	4	A'SHOCK?
1941	620	1520	41°	44.26 '	111°	49.85 '	3.0 I	3	
1942	418	0545	41°	30.00 '	112°	18.00 '	4.3 I	5	HANSEL VALLEY? (1,6,8)
1942	604	2204	39°	34.80 '	111°	39.00 '	4.3 I	5	TWO SHOCKS (1)
1943	222	1420	40°	42.00 '	112°	4.80 '	5.0 I	6	W. SALT LK VALLEY (1,6,8)
1943	312	1245	39°	21.67 '	111°	35.26 '	3.7 I	4	

TABLE 2.6-4
(Page 3 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

Year	*Date*	*Origin Time*	*Latitude*		*Longitude*		*Mag*	*Int*	*Comments*
1943	410	2242	40°	42.00 '	112°	4.80 '	4.3 I	5	W. SALT LK VALLEY (1,8)
1943	411	1932	40°	42.00 '	112°	4.80 '	3.0 I	3	A'SHOCK
1946	506	0230	41°	43.80 '	112°	7.80 '	4.3 I	5	LOC ASSUMED (1)
1946	1025	1653	40°	42.60 '	112°	6.30 '	3.0 I	3	
1947	307	1414	40°	44.94 '	111°	50.95 '	3.0 I	3	
1947	328	1102	40°	39.90 '	111°	53.40 '	3.7 I	4	INT=4-5
1948	1104	1318	39°	15.83 '	111°	38.23 '	4.3 I	5	
1949	307	0650	40°	44.94 '	111°	50.95 '	5.0 I	6	
1949	307	0709	40°	44.94 '	111°	50.95 '	3.7 I	4	INT=4-6,A'SHOCK (1,4)
1950	102	1953	41°	30.00 '	112°	0.00 '	4.3 N	4	(5,6,8)
1950	220	1459	40°	2.32 '	111°	43.76 '	3.7 I	4	
1950	225	1337	40°	0.00 '	112°	0.00 '	3.0 X	0	(6)
1950	508	2235	40°	2.32 '	111°	43.76 '	4.3 I	5	
1950	721	1923	41°	44.26 '	111°	49.85 '	3.7 I	4	TWO SHOCKS (8)
1951	123	1333	39°	42.36 '	111°	49.90 '	3.7 I	4	
1951	812	0026	40°	14.35 '	111°	39.33 '	4.3 I	5	
1952	721	0100	39°	58.37 '	111°	46.87 '	3.7 I	4	
1952	723	1928	40°	44.94 '	111°	50.95 '	3.7 I	4	
1952	928	2000	40°	23.81 '	111°	51.64 '	4.3 I	5	
1953	524	0254	40°	30.00 '	111°	30.00 '	4.3 I	5	INT=4-6 (4,6,8,11)
1953	816	1600	40°	46.80 '	111°	57.00 '	3.7 I	4	
1954	1101	0745	41°	44.26 '	111°	49.85 '	3.7 I	4	
1955	202	1923	40°	47.00 '	111°	56.00 '	4.3 I	5	INT=4-5,FEB 4? (4,8,12)
1955	512	2257	40°	54.82 '	111°	52.64 '	4.3 I	5	LOC ASSUMED (8)
1955	625	0500	41°	2.51 '	111°	40.49 '	3.7 I	4	
1957	721	1730	41°	30.00 '	113°	0.00 '	3.0 X	0	(6)
1957	1025	1626	40°	0.00 '	111°	0.00 '	3.0 X	0	(6)
1957	1026	0146	40°	0.00 '	111°	0.00 '	3.0 X	0	A'SHOCK? (6)
1958	105	1700	41°	0.00 '	112°	30.00 '	3.0 X	0	(6)
1958	213	2252	40°	20.50 '	111°	26.40 '	5.0 I	6	(6,8,10)
1958	217	1157	39°	30.00 '	113°	0.00 '	3.0 X	0	(6)
1958	1128	1330	39°	42.70 '	111°	50.00 '	4.3 I	5	INT=4-5 (4,8,13)
1958	1201	2050	39°	42.70 '	111°	50.00 '	4.3 I	5	INT=4-5 (4,8,13)**
1958	1201	2230	39°	42.70 '	111°	50.00 '	3.7 I	4	INT=3-4,A'SHOCK(4,6,8,13)
1958	1202	0323	39°	42.70 '	111°	50.00 '	4.3 I	5	INT=4-5,A'SHOCK(4,6,8,13)

TABLE 2.6-4
(Page 4 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

Year	Date	Origin Time	Latitude	Longitude	Mag	Int	Comments
1958	1211	0930	39° 31.80'	111° 1.20'	3.7 I	4	(8)
1960	506	2028	39° 30.00'	111° 0.00'	3.0 X	0	(6)
1960	709	2136	41° 30.00'	112° 0.00'	3.0 X	0	(6)
1961	416	0502	39° 20.40'	111° 39.60'	5.0 I	6	INT=4-6 (4,6,8,14)
1961	1015	2105	39° 12.00'	111° 24.00'	3.0 X	0	(6)
1961	1016	1913	39° 12.00'	111° 30.00'	3.0 X	0	A'SHOCK? (6)
1961	1017	0059	39° 12.00'	111° 30.00'	3.0 X	0	A'SHOCK? (6)
1961	1017	0354	40° 0.00'	112° 30.00'	3.0 X	0	(6)

Number of Earthquakes = 113

TABLE 2.6-4
(Page 5 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

DATA EXPLANATION FOR PRE-JULY 1962 EARTHQUAKE LISTING

(From Pages 119-121 of Earthquake Studies in Utah, 1850 to 1978, edited by W.J. Arabasz, R.B. Smith, and W.D. Richins)

The University of Utah historic catalog contains earthquakes from 1850, the year of publication of the first newspaper in Utah, through June 1962.

The following data are listed for each event:

1. Year (YR), date, and origin time (ORIG TIME) in Universal or Greenwich Mean Time (GMT). "Local time" is 7 hours earlier than GMT (i.e., local time = GMT - 7 hours) except for three time periods when it was 6 hours earlier:
   (1) 02:00 Mar. 31, 1918 - 02:00 Oct. 27, 1918);
   (2) 02:00 Mar. 30, 1919 - 02:00 Oct. 26, 1919; and
   (3) 02:00 Feb. 9, 1942 - 02:00 Sept. 30, 1945.
   Origin time given in hours and minutes for non-instrumental locations, and in hours, minutes, and seconds for instrumental locations.

2. Earthquake location coordinates in degrees and minutes of north latitude (LAT-N) and west longitude (LONG-W). For non-instrumental locations, epicenter is assumed; in most cases, assigned coordinates correspond to location of town or city where felt effects were strongest. Epicentral accuracy ~±25-50 km.

3. MAG, estimated Richter magnitude determined in one of four ways, as indicated by a suffix: (1) I implies estimate from maximum Modified Mercalli Intensity (INT) assuming the Gutenberg-Richter relation (Gutenberg and Richter, 1956, Bull. Seism. Soc. Am. 46, 105-145): MAG = 1 + 2/3 (INT); (2) M implies magnitude determined by Seismological Laboratory in Pasadena, (3) N implies magnitude estimated by University of Nevada (Reference 5, see below); and (4) X implies value arbitrarily assumed for event of unidentified size; X = 2.3 for non-instrumental locations, and 3.0 for instrumental locations.

4. INT, maximum Modified Mercalli Intensity. Unless otherwise noted, intensity is from Reference 1 (see below) for earthquakes through 1949, and from Reference 8 (see below) thereafter. Where sources disagree on maximum intensity, range is indicated as a comment and a maximum value has been interpreted. For events of unidentified size (X suffix in MAG column), intensity II arbitrarily assumed for non-instrumental locations—and no intensity assigned for instrumental locations.

TABLE 2.6-4
(Page 6 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

5. Comments: Compilation of the 1850-1962 catalog has involved the careful checking and correlation of numerous sources—and extensive annotation. For convenience, several abbreviations and numbered references have been used, as outlined below. Earthquakes without comments generally are from Reference 1 for 1850-1949, and from either Reference 6 (instrumental) or Reference 8 (non-instrumental) for 1950-1962.

Abbreviations

LOC:	location
ASSGN:	assigned
INT:	intensity (Modified Mercalli)
MAG:	magnitude
A'SHOCK:	aftershock
F'SHOCK:	foreshock
PAS:	Pasadena, Seismological Laboratory
NEV:	University of Nevada, Reno
ID:	Idaho
UT:	Utah
SALT LK:	Salt Lake

References and Footnotes

(1) Williams, J. S. and M. L. Tapper (1953). Earthquake history of Utah, 1850-1949, Bulletin of the Seismological Society of America, 43, 191-218.

(2) U.S. Geological Survey (1976). A study of earthquake losses in the Salt Lake City, Utah area, U.S. Geological Survey Open-File Report 76-89, 357 p.

(3) Townley, S. D. and M. W. Allen (1939). Descriptive catalog of earthquakes of the Pacific Coast of the United States, 1769 to 1928, Bulletin of the Seismological Society of America, 29, (1), Ch. 4,5,6.

(4) Coffman, J. L. and C. A. von Hake, editors (1973). Earthquake History of the United States, Publ. 41-1, U.S. Dept. Commerce, 208 p.

(5) Jones, A. E. (1975). Recording of earthquakes at Reno, 1916-1951, Bulletin of the Seismological Laboratory, University of Nevada, Reno, 199 p.

TABLE 2.6-4
(Page 7 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
1850—July 1962

(6) NOAA earthquake data file, National Geophysical and Solar-Terrestrial Data Center, Boulder, Colorado.

(7) Pack, F. J. (1921). The Elsinore earthquakes in central Utah, September 29 and October 1, 1921, Bulletin of the Seismological Society of America, 11, 155-165.

(8) U.S. Department of Commerce. United States Earthquakes, annual publication, U.S. Government Printing Office, Washington, D.C.

(9) Gutenberg, B. and C. F. Richter (1954). Seismicity of the Earth, 2nd ed., Princeton Univ. Press, Princeton, N.J.

(10) Berg, J. W. Jr. and R. C. Resler (1958). Investigation of local earthquakes February 13, 1958, near Wallsburg, Utah, Utah Acad. Sci., Arts, Lett. Proc. 35, 113-117.

(11) Hardy, C. T. and G. Gaeth (1959). Field investigation of Utah earthquake, May 23, 1953, Utah Acad. Sci., Arts, Lett. Proc. 36, 137-140.

(12) Hardy, C. T. (1959). Field investigation of Utah earthquake, February 4, 1955, Utah Acad. Sci. Arts, Lett. Proc. 36, 141-143.

(13) Berg, J. W. Jr. (1960). Earthquakes near Nephi, Utah, on November 28, 1958, and December 1, 1958, Utah Acad. Sci. Arts, Lett. Proc. 37, 77-79.

(14) Algermissen, S. T. and K. L. Cook (1962). The Ephraim, Utah, earthquake of April 15, 1961, Utah Acad. Sci. Arts, Lett. Proc. 39, 106-110.

(15) Cook, K. L. and R. B. Smith (1967). Seismicity in Utah, 1850 through June 1965, Bulletin of the Seismological Society of America, 57, 689-718.

(16) Carr, S. L. (1972). The Historical Guide to Utah Ghost Towns, Western Epics, Salt Lake City, Utah, 166 p.

(17) University of Utah (1952). Gazeteer of Utah Localities and Altitudes, Division of Biology, University of Utah, Salt Lake City, 216 p.

(18) Stansbury, H. (1852). An Expedition to the Valley of the Great Salt Lake of Utah, Lipincott, Grambo and Co., 487 pp (see pp. 149-150).

TABLE 2.6-4
(Page 8 of 14)

**EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996**

Year	*Date*	*Origin Time*		*Latitude*	*Longitude*		*Depth*	*MAG*	*NO*	*GAP*	*DMN*	*RMS*
1962	905	1604	27.78	40° 42.92 '	112°	5.33 '	7.0 *	5.2 W	9	188	21	0.41
1962	909	1438	8.92	41° 50.80 '	111°	46.17 '	7.0 *	3.1	8	214	120	0.44
1963	707	1920	39.59	39° 31.96 '	111°	54.51 '	7.0 *	4.4 W	9	89	95	0.33
1963	709	1520	40.85	39° 31.71 '	111°	54.29 '	7.0 *	3.1	5	229	95	0.11
1963	709	2025	25.80	40° 1.70 '	111°	11.41 '	7.0 *	4.0 W	6	92	57	0.78
1963	710	1832	49.76	40° 1.20 '	111°	14.95 '	7.0 *	3.7 W	7	93	60	0.39
1963	814	1230	2.44	41° 37.30 '	112°	4.24 '	7.0 *	3.2	7	267	97	0.15
1963	816	700	58.87	41° 39.66 '	112°	9.82 '	7.0 *	3.0	6	236	103	0.49
1963	1229	415	0.20	39° 8.39 '	114°	17.44 '	7.0 *	3.9 W	6	219	173	0.20
1964	220	2019	48.20	39° 24.79 '	114°	13.06 '	7.0 *	3.2 W	7	141	148	0.44
1964	906	1903	33.75	39° 10.93 '	111°	27.81 '	7.0 *	3.1	10	223	74	0.74
1964	1018	1833	20.80	41° 43.55 '	111°	43.77 '	7.0 *	4.1 W	7	207	7	0.36
1965	1029	1652	50.28	41° 19.15 '	113°	23.26 '	7.0 *	3.7 W	9	203	134	0.51
1966	317	1147	47.41	41° 39.66 '	111°	33.63 '	7.0 *	4.6 W	8	199	23	0.46
1966	1114	1430	49.88	41° 44.70 '	112°	43.85 '	7.0 *	3.2	8	144	76	0.46
1967	216	1921	35.19	41° 16.40 '	113°	20.03 '	7.0 *	4.0 W	10	199	128	0.60
1967	721	1527	57.49	41° 15.85 '	113°	17.91 '	7.0 *	3.6 W	8	198	126	0.28
1967	922	739	53.92	41° 20.75 '	113°	21.97 '	7.0 *	3.1	8	204	136	0.59
1967	924	500	28.58	40° 42.46 '	112°	5.90 '	7.0 *	3.0	8	145	22	0.17
1967	1207	1333	22.49	41° 17.17 '	111°	44.26 '	7.0 *	3.7 W	15	127	51	0.53
1968	116	858	41.53	39° 18.00 '	112°	3.72 '	4.1	3.5 W	7	156	113	0.25
1968	116	917	50.54	39° 17.40 '	112°	2.69 '	7.0 *	3.4 W	13	102	112	0.45
1968	116	920	10.26	39° 18.78 '	112°	2.69 '	7.0 *	3.3 W	9	155	112	0.38
1968	116	941	44.38	39° 16.78 '	112°	1.52 '	7.0 *	3.2 W	13	131	111	0.44
1968	116	942	52.13	39° 15.93 '	112°	2.28 '	7.0 *	3.9 W	7	156	113	0.27
1970	329	1240	40.34	41° 39.74 '	113°	50.39 '	7.0 *	4.7 W	10	203	169	0.57
1970	1025	748	21.94	39° 10.28 '	111°	24.72 '	7.0 *	3.1 W	6	174	71	0.49
1971	422	2301	2.81	39° 24.63 '	111°	56.46 '	7.0 *	3.1 W	8	96	100	0.34
1972	1001	1942	29.52	40° 30.36 '	111°	20.91 '	7.0 *	4.3 W	13	90	36	0.53
1972	1016	2149	31.19	40° 25.27 '	111°	0.97 '	7.0 *	3.4 W	9	129	66	0.53
1976	1105	115	7.06	41° 49.35 '	112°	41.65 '	7.0 *	3.3 W	21	199	39	0.22
1976	1105	248	55.59	41° 48.59 '	112°	41.88 '	7.0 *	4.0 W	15	199	40	0.19
1976	1105	554	0.92	41° 48.94 '	112°	41.77 '	7.0 *	3.4 W	21	199	40	0.25
1976	1126	2226	29.43	39° 30.79 '	111°	15.72 '	7.0 *	3.1 W	15	248	41	0.40
1977	209	42	16.13	39° 17.55 '	111°	6.69 '	7.0 *	3.2 W	16	187	44	0.52

TABLE 2.6-4
(Page 9 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

Year	*Date*	*Origin Time*	*Latitude*	*Longitude*	*Depth*	*MAG*	*NO*	*GAP*	*DMN*	*RMS*
1978	309	630 51.88	40° 45.82'	112° 5.27'	8.8	3.2 W	18	70	14	0.26
1978	729	1404 3.36	41° 50.92'	112° 7.84'	4.2	3.1 W	27	63	18	0.23
1979	224	1243 41.17	41° 43.02'	111° 8.90'	90.5	3.8 W	29	129	41	0.30
1979	325	2141 55.74	41° 20.59'	113° 17.07'	7.0 *	3.2 W	24	218	65	0.30
1980	404	45 4.50	41° 20.19'	113° 17.17'	7.0 *	3.1 W	23	213	65	0.30
1980	406	1045 4.03	39° 56.86'	111° 58.46'	4.4	3.5 W	28	70	17	0.25
1980	517	903 38.64	39° 42.55'	112° 1.59'	7.0 *	3.0	9	121	67	0.29
1980	524	1003 36.47	39° 56.21'	111° 57.59'	7.0 *	4.4 W	17	113	61	0.37
1980	815	625 23.72	41° 39.74'	111° 41.10'	7.0 *	3.1 W	20	106	35	0.35
1981	220	913 1.19	40° 19.33'	111° 44.11'	0.7	3.9 W	32	110	20	0.29
1981	331	2040 45.51	41° 41.42'	111° 2.60'	0.1	3.1 W	37	143	33	0.29
1981	514	511 4.34	39° 28.86'	111° 4.72'	0.7	3.5 W	27	133	59	0.51
1983	829	1253 11.45	41° 4.99'	111° 25.60'	9.8	3.0 W	12	165	50	0.24
1983	1008	1157 53.83	40° 44.88'	111° 59.56'	5.5	4.3 W	30	66	16	0.33
1984	816	1419 21.71	39° 23.50'	111° 56.16'	6.1	3.7 W	12	95	34	0.33
1984	1015	2323 56.53	41° 48.27'	112° 24.10'	4.1	3.4 W	25	81	15	0.17
1985	611	721 45.12	39° 9.93'	111° 28.21'	0.1	3.0	8	169	15	0.42
1986	113	1232 4.63	41° 42.91'	111° 39.88'	7.5	3.3 W	20	64	13	0.24
1986	221	2320 12.51	41° 44.69'	112° 49.11'	7.4	3.6 W	19	225	5	0.23
1986	324	2233 41.23	39° 13.34'	111° 59.90'	0.1	3.3 W	16	105	23	0.25
1986	324	2240 23.41	39° 14.04'	112° 0.37'	0.9	4.4 W	17	87	21	0.27
1986	325	253 1.34	39° 13.53'	112° 0.68'	1.5	3.9 W	17	90	22	0.28
1986	605	805 41.73	41° 15.99'	111° 41.03'	9.6	3.6 W	17	153	18	0.25
1986	919	1041 28.20	41° 27.99'	111° 42.11'	7.5	3.4 W	27	70	16	0.27
1986	1026	1431 56.67	41° 49.47'	112° 18.97'	4.1	3.0 W	27	73	12	0.26
1986	1029	2213 14.48	41° 49.27'	112° 19.09'	4.7	3.6 W	19	75	12	0.15
1986	1031	1158 28.16	41° 49.38'	112° 18.97'	3.6	3.5 W	22	73	12	0.15
1986	1231	1121 56.48	41° 49.31'	112° 18.98'	4.7	3.3 W	18	73	12	0.16
1987	225	1230 33.50	41° 49.10'	112° 19.09'	3.1	3.7 W	21	74	12	0.18
1987	311	156 7.80	40° 7.40'	114° 20.11'	5.4	3.4 W	25	207	92	0.41
1987	323	406 59.94	40° 6.81'	114° 23.27'	1.4	3.4 W	34	200	95	0.36
1987	323	559 12.48	40° 6.63'	114° 19.09'	3.7	3.0 W	22	199	90	0.26
1987	401	1640 41.08	41° 49.23'	112° 19.60'	5.6	3.6 W	18	75	12	0.17
1987	401	2144 46.00	41° 49.34'	112° 19.67'	4.4	3.3 W	18	74	12	0.17
1987	917	831 26.85	41° 12.52'	113° 6.94'	10.2	3.8 W	27	217	62	0.15

TABLE 2.6-4
(Page 10 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

Year	Date	Origin Time		Latitude		Longitude		Depth	MAG	NO	GAP	DMN	RMS
1987	925	409	54.54	41°	12.41'	113°	8.14'	10.8	4.1 W	20	197	64	0.32
1987	925	427	58.36	41°	12.81'	113°	7.91'	10.7	4.8 W	23	219	64	0.25
1987	925	518	13.58	41°	11.74'	113°	12.82'	6.2	4.3 W	24	239	68	0.26
1987	926	28	2.11	41°	12.54'	113°	9.00'	10.1	4.0 W	31	221	6	0.27
1987	926	1447	49.38	41°	12.01'	113°	10.70'	10.0	3.1 W	21	88	14	0.18
1987	928	606	52.25	41°	13.60'	113°	10.85'	9.2	4.0 W	23	93	8	0.23
1987	1001	916	31.25	41°	12.70'	113°	10.83'	7.8	3.6 W	25	86	9	0.21
1987	1002	1435	48.79	41°	33.80'	112°	25.77'	7.7	3.4 W	25	124	16	0.18
1987	1004	1738	26.44	41°	33.52'	112°	25.49'	11.9	3.3 W	20	125	15	0.21
1987	1005	1031	3.09	41°	12.13'	113°	10.52'	8.0	3.3 W	26	86	9	0.20
1987	1019	717	9.92	39°	40.11'	111°	25.52'	1.0	3.6 W	19	72	40	0.18
1987	1023	1944	50.45	41°	11.78'	113°	10.16'	9.5	4.2 W	22	120	8	0.17
1987	1026	416	0.94	41°	12.05'	113°	10.66'	9.4	4.7 W	23	123	9	0.22
1987	1216	1743	7.50	39°	18.70'	111°	12.92'	0.5	3.3 W	26	78	28	0.44
1988	130	537	13.27	41°	12.31'	113°	10.46'	9.1	3.1 W	22	125	4	0.17
1988	522	1910	47.95	39°	53.38'	114°	10.82'	4.6	3.6 W	30	187	70	0.31
1988	710	2045	59.41	41°	13.49'	111°	37.73'	7.0	3.6 W	8	154	51	0.23
1988	711	1146	55.99	39°	11.51'	111°	59.25'	1.7	3.1 W	17	63	26	0.26
1988	921	1758	25.89	39°	18.48'	111°	9.88'	9.1	3.1	20	80	32	0.18
1988	1106	1530	58.83	40°	43.30'	111°	25.07'	11.0	3.3 W	23	131	14	0.31
1989	211	2037	57.28	39°	20.65'	111°	9.67'	7.3	3.2	17	83	33	0.32
1989	327	1141	54.03	41°	37.53'	112°	50.20'	6.5	3.0 W	15	223	44	0.14
1989	621	2154	18.59	41°	42.46'	112°	22.40'	7.8	4.1 W	23	96	17	0.15
1989	627	1551	49.68	41°	47.67'	112°	44.04'	5.6	3.0 W	17	129	4	0.18
1989	703	2244	28.64	41°	42.39'	112°	22.40'	7.4	4.8 W	21	96	17	0.13
1989	705	2251	56.35	41°	42.40'	112°	22.28'	10.0	4.6 W	18	96	3	0.17
1989	719	24	47.60	41°	42.86'	112°	23.05'	9.4	3.2 W	14	99	4	0.13
1989	820	2123	15.61	41°	42.81'	112°	23.70'	6.9	3.0 W	25	79	5	0.11
1989	823	721	19.94	41°	42.64'	112°	23.73'	8.0	3.2 W	22	82	5	0.15
1990	124	903	30.97	41°	45.77'	112°	37.69'	10.0	3.6 W	14	105	12	0.15
1990	205	1023	25.23	39°	30.23'	111°	31.02'	10.2	3.1 W	11	85	21	0.15
1990	223	2240	12.71	41°	11.92'	113°	10.75'	6.9	3.0 W	20	223	66	0.19
1990	504	403	8.09	39°	31.63'	111°	6.05'	1.3	3.0	13	96	40	0.26
1990	901	1812	29.38	39°	17.95'	111°	8.12'	7.4	3.3	10	81	35	0.22
1990	927	1505	55.47	39°	30.01'	111°	1.86'	15.1	3.2	11	105	40	0.20

TABLE 2.6-4
(Page 11 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

Year	*Date*	*Origin Time*		*Latitude*	*Longitude*	*Depth*	*MAG*	*NO*	*GAP*	*DMN*	*RMS*
1990	1121	1216	54.67	39° 29.60'	111° 4.27'	3.5	3.0 W	16	101	42	0.17
1991	206	1346	46.66	39° 29.99'	111° 4.61'	4.3	3.1 W	11	100	42	0.27
1991	315	2033	14.72	39° 21.05'	111° 10.34'	8.5	3.0 W	15	85	32	0.22
1991	821	1347	6.25	39° 21.83'	111° 52.66'	3.7	3.0 W	18	55	25	0.23
1991	1123	1625	6.45	39° 17.56'	111° 8.97'	8.8	3.1	12	84	34	0.18
1992	316	1442	49.64	40° 28.21'	112° 2.69'	10.5	4.2 W	35	79	6	0.29
1992	603	508	30.95	39° 19.04'	111° 9.80'	0.6	3.3 W	24	85	44	0.55
1992	609	2330	18.57	39° 18.12'	111° 9.56'	6.1	3.4	16	84	33	0.29
1992	626	1107	50.17	39° 18.64'	111° 9.52'	4.4	3.0	21	84	33	0.40
1992	628	715	13.77	39° 19.07'	111° 9.64'	9.2	3.0	12	88	33	0.27
1992	705	1222	22.76	39° 18.81'	111° 9.60'	5.6	3.7 W	17	84	33	0.37
1992	711	1323	7.62	39° 18.52'	111° 8.94'	9.3	3.0 W	19	85	34	0.33
1992	1104	1822	10.10	41° 30.59'	113° 23.27'	7.1	4.8 W	30	208	59	0.27
1992	1104	1825	3.53	41° 32.42'	113° 22.16'	7.6	3.8 W	18	254	56	0.19
1992	1109	1811	25.56	39° 16.17'	111° 48.20'	8.9	3.0	26	51	17	0.34
1992	1110	736	6.89	39° 20.36'	111° 9.73'	6.3	3.0	18	86	7	0.24
1992	1205	701	25.50	39° 20.13'	111° 9.57'	3.3	3.0	15	84	6	0.27
1993	117	1926	40.33	39° 41.36'	111° 14.39'	4.5	3.0	18	71	39	0.31
1993	927	1121	0.87	39° 19.95'	111° 9.55'	1.1	3.3	22	86	33	0.37
1994	128	2247	39.37	39° 19.70'	111° 8.75'	9.2	3.1	8	99	34	0.12
1994	216	57	7.24	39° 42.13'	111° 15.19'	4.9	3.1	9	74	40	0.14
1994	402	545	25.85	39° 41.62'	111° 15.18'	7.0	3.0	11	74	40	0.12
1994	505	1804	35.04	41° 40.91'	111° 42.72'	16.4	3.2	18	59	14	0.11
1994	506	137	54.19	41° 47.19'	112° 22.10'	0.3	3.6	32	74	11	0.31
1994	506	2242	45.91	40° 4.02'	111° 23.91'	2.8	3.2	28	103	37	0.21
1994	601	48	17.73	39° 42.05'	111° 14.77'	0.8	3.3	7	109	39	0.49
1994	909	2006	42.14	39° 27.67'	111° 30.70'	9.9	3.3	14	79	16	0.19
1994	909	2013	29.83	39° 27.73'	111° 30.60'	7.7	3.3	12	84	16	0.23
1994	910	633	42.42	39° 27.92'	111° 31.04'	7.7	3.9	16	58	16	0.18
1994	910	923	33.30	39° 28.05'	111° 30.65'	6.6	3.6	15	79	17	0.17
1994	1108	157	56.76	39° 19.81'	111° 10.70'	9.2	3.0	12	155	31	0.11
1994	1110	540	6.65	39° 19.87'	111° 10.33'	5.1	3.1	19	84	32	0.25
1994	1123	1630	49.24	39° 27.47'	111° 31.46'	1.8	3.5	22	58	15	0.37
1994	1203	2039	35.28	39° 42.12'	111° 15.73'	8.7	3.1	14	73	40	0.16
1995	105	2123	28.93	39° 42.15'	111° 15.09'	4.1	3.0	12	74	39	0.16

TABLE 2.6-4
(Page 12 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

Year	Date	Origin Time		Latitude	Longitude	Depth	MAG	NO	GAP	DMN	RMS
1995	204	1048	57.87	41° 29.13'	112° 8.86'	13.0	3.3	14	64	14	0.11
1995	303	1458	37.21	39° 19.36'	111° 10.10'	7.5	3.6	9	87	32	0.15
1995	315	2307	0.24	39° 42.06'	111° 15.20'	1.4	3.1	14	79	40	0.19
1995	428	1139	1.72	39° 41.78'	111° 14.89'	1.2	3.0	21	71	39	0.22
1995	430	1623	12.54	41° 43.42'	112° 17.90'	1.7	3.2	34	69	13	0.23
1995	525	512	58.12	39° 41.20'	111° 14.37'	1.1	3.3	9	133	39	0.05
1995	706	22	23.54	39° 54.74'	111° 38.48'	7.8	3.5	12	142	13	0.27
1995	727	1704	36.22	41° 38.44'	111° 58.88'	3.9	3.7	14	75	4	0.09
1995	803	802	47.10	39° 41.27'	111° 14.27'	0.2	3.0	15	70	2	0.18
1995	804	204	54.50	39° 41.31'	111° 14.49'	0.2	3.1	13	70	3	0.37
1995	930	925	58.54	41° 38.75'	111° 1.32'	1.2	3.2	25	215	34	0.23
1995	1008	625	2.97	40° 55.15'	111° 39.69'	5.8	3.4	22	114	19	0.22
1995	1009	2151	15.51	39° 36.87'	111° 6.04'	7.4	3.3	20	90	13	0.14
1995	1011	1336	0.50	39° 37.68'	111° 6.10'	1.3	3.0	20	89	12	0.28
1995	1102	1409	57.68	41° 41.92'	111° 41.01'	11.2	3.2	18	62	14	0.25
1995	1105	1020	12.01	41° 13.46'	113° 12.12'	5.1	3.1	22	225	69	0.30
1995	1108	732	52.75	39° 31.16'	111° 7.57'	2.9	3.4	22	92	20	0.20
1995	1206	425	28.24	40° 44.23'	111° 32.40'	10.9	3.5	36	89	12	0.22
1995	1206	741	6.41	39° 41.70'	111° 14.45'	1.2	3.0	13	75	3	0.36
1995	1215	1243	22.11	39° 42.36'	111° 12.75'	0.2	3.0	17	80	2	0.38
1995	1231	1823	13.66	39° 41.88'	111° 14.47'	1.3	3.0	23	71	3	0.34
1996	202	211	14.62	39° 28.00'	111° 13.81'	1.5	3.5	27	80	25	0.30
1996	318	724	15.43	39° 41.31'	111° 14.61'	1.6	3.0	15	76	3	0.28
1996	429	252	19.24	39° 42.11'	111° 14.50'	0.3	3.3	25	70	3	0.25
1996	602	809	10.01	39° 37.54'	111° 14.46'	5.6	3.5	24	73	8	0.18
1996	705	300	29.98	41° 42.53'	112° 22.68'	1.4	3.8	35	78	6	0.26

Number of earthquakes = 166

* indicates poor depth control
W indicates Wood-Anderson data used for magnitude calculation

TABLE 2.6-4
(Page 13 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

DATA EXPLANATION FOR POST-JULY 1962 EARTHQUAKE LISTING

The following data are listed for each event:

1. Year, Date and Origin Time in Universal Coordinated Time (UTC). Subtract seven hours to convert to Mountain Standard Time (MST).
2. Earthquake location coordinates in degrees and minutes of north latitude (LAT-N) and west longitude (LONG-W), and Depth in kilometers. "*" indicates poor depth resolution: no recording stations within 10km or twice the depth.
3. MAG, computed local magnitude for each earthquake. "W" indicates Wood-Anderson records were used.
4. NO, number of P and S readings used in solution.
5. GAP, largest azimuthal separation in degrees between recording stations used in the solution.
6. DMN, epicentral distance in kilometers to the closest station.
7. RMS, root-mean-square error in seconds of the travel-time residuals:

$$RMS = [\Sigma_i(W_iR_i)^2) / \Sigma_i(W_i)^2]^{1/2}$$

where:

R_i is the observed minus the computed arrival time for the i^{th} P or S reading,

W_i is the relative weight given to the i^{th} P or S arrival time (0.0 for no weight through 1.0 for full weight).

TABLE 2.6-4
(Page 14 of 14)

EARTHQUAKES: MAGNITUDE 3.0 AND GREATER, 1850—1996
160KM RADIUS AROUND 40° 24.50'N AND 112°47.50'W
July 1962—September 1996

The Utah region includes the state of Utah and extends approximately 15 miles from the state line in the east, south and west directions, and 30 miles to the north of the state line (36de 45min - 42de 30min North latitude and 108de 45min - 114de 15min West longitude).

The University of Utah's instrumental earthquake catalog begins in July, 1962. The July 1, 1962 to September 30, 1974 catalog is based on instrumental earthquake locations from a skeletal regional seismic network (< 26 stations statewide). Beginning in October, 1974, data is available from a dense network of high-gain telemetered stations with significantly better locations and magnitude determinations than those for the previous period. The network expanded from 26 stations in late 1974, though southern and central Utah (1975-1977), into southeastern Idaho and western Wyoming (1976-1977), and in the Wasatch Front (1975-1979), until reaching a stable network of 50-60 stations in the late 1970's. From 1974 though 1980, earthquake data was recorded primarily on 16mm analog film recorders (Develocorders). Beginning in January, 1981, earthquake data was recorded digitally on a DEC PDP 11/34 computer system. Digital recording was switched from the PDP 11/34 computer to a Concurrent 7200 computer in September 1992. Digital seismograms are available for all located earthquakes occurring since January 1, 1981.

TABLE 2.6-5
SUMMARY OF BLOW COUNTS IN LAYER 1 IN STORAGE PAD AREA

ELEVATION		BORING							
TOP	BOTTOM	A-1	A-2	A-3	A-4	B-1	B-2	B-3	B-4
4475	4470				14				22
4470	4465			4	18			9	9
4465	4460		1	9	13		4	U	U
4460	4455	23	U	15	18	13	5	U	15
4455	4450	13	11	15	12	U	13	18	21
4450	4445	22	14	20	20	15	16	12	21
4445	4440	19	17	30	50	20	12	24	34
4440	4435	13	16	34		12	15	28	
4435	4430	36							

ELEVATION		BORING							
TOP	BOTTOM	C-1	C-2	C-3	C-4	D-1	D-2	D-3	D-4
4475	4470				15				8
4470	4465			11	7		6	6	4
4465	4460	3	18	6	11		6	14	24
4460	4455	8	U	8	14	40	11	11	22
4455	4450	U	U	10	15	12	15	9	9
4450	4445	16	13	9	20	14	18	11	16
4445	4440	8	11	22	21	13	17	39	
4440	4435		34			16			
4435	4430								

ELEVATION		N_{AVG}	N_{MEDIAN}
TOP	BOTTOM	BLOWS/FT	
4475	4470	15	15
4470	4465	8	7
4465	4460	10	9
4460	4455	16	14
4455	4450	13	13
4450	4445	16	16
4445	4440	22	20
4440	4435	21	16
4435	4430	36	36

FOR ENTIRE LAYER:

N_{AVG} = 15.7 BLOWS/FT
N_{MEDIAN} = 14.0 BLOWS/FT

U = UNDISTURBED SAMPLE

TABLE 2.6-6

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Static Loads

Case	F_V	$EQ_{H\ N-S}$	$EQ_{H\ E-W}$	$\Sigma M_{@N-S}$	$\Sigma M_{@E-W}$	β_B $EQ_{H\ E-W}$	β_L $EQ_{H\ N-S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	3,757	0	0	0	0	0.0	0.0	13.08	4.36	0.0	0.0	30.0	67.0	1.87	7.0
IB - Static Effective Strength	3,757	0	0	0	0	0.0	0.0	29.22	9.73	0.0	0.0	30.0	67.0	1.87	15.6

$c =$ **2,200** Undrained strength (psf), $\phi=0$.

$\phi =$ **30** Effective stress friction angle (deg), c=0.

$\gamma =$ **80** Unit weight of soil (pcf)

$B =$ **30** Footing width (ft)

$L =$ **67** Footing length (ft)

$D_f =$ **3.0** Depth of footing (ft)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

$FS =$ **3** Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E-W}$ or $EQ_{H\ N-S}$

$\beta_B = \tan^{-1} [(EQ_{H\ E-W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1} [(EQ_{H\ N-S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N-S} / F_V$ $\quad$ $e_L = \Sigma M_{@E-W} / F_V$

$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-7

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B	β_L	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
						$EQ_{H\,E\text{-}W}$	$EQ_{H\,N\text{-}S}$	q_{ult}	q_{all}			B'	L'	q_{actual}	
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	3,757	2,671	2,671	26,982	26,982	35.4	35.4	5.34	4.85	7.2	7.2	15.6	52.6	4.56	1.2
IIIA	1,146	953	953	10,793	10,793	39.8	39.8	9.03	8.21	9.4	9.4	11.2	48.2	2.13	4.2
IIIB	2,712	1,068	2,291	26,982	10,793	40.2	21.5	5.26	4.78	9.9	4.0	10.1	59.0	4.55	1.2
IIIC	2,712	2,291	1,068	10,793	26,982	21.5	40.2	9.45	8.59	4.0	9.9	22.0	47.1	2.61	3.6
IVA	6,368	1,068	1,068	10,793	10,793	9.5	9.5	11.57	10.51	1.7	1.7	26.6	63.6	3.76	3.1
IVB	4,801	1,068	2,671	26,982	10,793	29.1	12.5	8.51	7.73	5.6	2.2	18.8	62.5	4.09	2.1
IVC	4,801	2,671	1,068	10,793	26,982	12.5	29.1	10.05	9.13	2.2	5.6	25.5	55.8	3.38	3.0

$c =$	**2,200**	Undrained strength (psf)
$\phi =$	**0.0**	Friction angle (deg)
$\gamma =$	**80**	Unit weight of soil (pcf)
B =	**30**	Footing width (ft)
L =	**67**	Footing length (ft)
$D_f =$	**3.0**	Depth of footing (ft)
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)
FS =	**1.1**	Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-8

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Maximum Cask Driving Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period for Loading Case IV: 40% N-S, 100% Vertical (Downward), and 40% E-W

Case IV	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B	β_L	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
						$EQ_{H\ E\text{-}W}$	$EQ_{H\ N\text{-}S}$	q_{ult}	q_{all}			B'	L'	q_{actual}	
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
2 Casks	3,790	429	506	6,443	16,183	7.6	6.5	12.42	11.28	1.70	4.27	25.0	26.6	5.71	2.2
4 Casks	6,380	688	791	10,526	33,620	7.1	6.2	11.88	10.79	1.65	5.27	26.7	39.7	6.02	2.0
8 Casks	11,888	1,078	1,142	12,720	36,140	5.5	5.2	11.56	10.50	1.07	3.04	27.9	60.9	7.00	1.6

c =	**2,200**	Undrained strength (psf)
ϕ =	**0.0**	Friction angle (deg)
γ =	**80**	Unit weight of soil (pcf)
B =	**30**	Footing width (ft)
L =	**Varies**	Footing length (ft)
D_f =	**3.0**	Depth of footing (ft)
γ_{surch} =	**100**	Unit weight of surcharge (pcf)
FS =	**1.1**	Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-9

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Static Loads

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	q_{ult} ksf	q_{all} ksf	ft	ft	B' ft	L' ft	q_{actual} ksf	
IA - Static Undrained Strength	97,749	0	0	0	0	0.0	0.0	19.63	6.54	0.0	0.0	240.0	279.5	1.46	13.47
IB - Static Effective Strength	97,749	0	0	0	0	0.0	0.0	169.92	56.64	0.0	0.0	240.0	279.5	1.46	116.61

c = **3,180** Undrained strength (psf), ϕ=0.

ϕ = **30** Effective stress friction angle (deg), c=0.

γ = **90** Unit weight of soil (pcf)

B = **240** Footing width (ft)

L = **279.5** Footing length (ft)

D_f = **5.0** Depth of footing (ft)

γ_{surch} = **80** Unit weight of surcharge (pcf)

FS = **3** Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\qquad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\qquad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-10

SUMMARY – ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING
Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B	β_L	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
						$EQ_{H\ E\text{-}W}$	$EQ_{H\ N\text{-}S}$	q_{ult}	q_{all}			B'	L'	q_{actual}	
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	97,749	111,108	99,997	2,706,961	2,849,703	45.7	48.7	13.17	11.97	27.7	29.2	184.6	221.2	2.39	5.50
IIIA	17,970	44,443	39,999	1,082,784	1,139,881	65.8	68.0	13.80	12.54	60.3	63.4	119.5	152.6	0.99	14.01
IIIB	65,837	44,443	99,997	2,706,961	1,139,881	56.6	34.0	14.10	12.82	41.1	17.3	157.8	244.9	1.70	8.28
IIIC	65,837	111,108	39,999	1,082,784	2,849,703	31.3	59.4	15.04	13.67	16.4	43.3	207.1	192.9	1.65	9.13
IVA	177,528	44,443	39,999	1,082,784	1,139,881	12.7	14.1	17.90	16.26	6.1	6.4	227.8	266.7	2.92	6.12
IVB	129,661	44,443	99,997	2,706,961	1,139,881	37.6	18.9	15.62	14.19	20.9	8.8	198.2	261.9	2.50	6.25
IVC	129,661	111,108	39,999	1,082,784	2,849,703	17.1	40.6	15.99	14.53	8.4	22.0	223.3	235.5	2.47	6.49

c =	**3,180**	Undrained strength (psf)
ϕ =	**0.0**	Friction angle (deg)
γ =	**90**	Unit weight of soil (pcf)
B =	**240**	Footing width (ft)
L =	**279.5**	Footing length (ft)
D_f =	**5.0**	Depth of footing (ft)
γ_{surch} =	**80**	Unit weight of surcharge (pcf)
FS =	**1.1**	Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\qquad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\ e_B$ $\qquad$ $L' = L - 2\ e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-11

FOUNDATION LOADINGS FOR THE CANISTER TRANSFER BUILDING

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

								SHEAR X	SHEAR Y	SHEAR Z	$\Sigma M_{Base\ @\ El\ 93.5}$	
JOINT	ELEV ft	MASS X $k\text{-}sec^2/ft$	MASS Y $k\text{-}sec^2/ft$	MASS Z $k\text{-}sec^2/ft$	Ax g	Ay g	Az g	$F_{H\ N\text{-}S}$ K	$F_{V\ Dyn}$ k	$F_{H\ E\text{-}W}$ k	$M_{@X}=M_{@N\text{-}S}$ ft-k	$M_{@Z}=M_{@E\text{-}W}$ ft-k
0	94.25	260.1	260.1	260.1	1.047	0.78	0.92	8,761	6,551	7,699	5,774	6,571
1	95	1,908.0	1,908.0	1,908.0	1.047	0.78	0.92	64,265	48,055	56,470	367,055	417,724
2	130	420.4	420.4	420.4	1.111	0.82	0.99	15,023	11,106	13,446	490,773	548,331
3	170	304.3	304.3	170.3	1.778	0.91	1.19	17,402	8,939	6,493	496,728	1,331,291
4	190	144.7	117.1	144.7	1.215	0.93	1.41	5,656	3,495	6,554	632,439	545,787
5	190	1.0	27.6	1.0	0	1.84	0.00	0	1,634	0	0	0
6	170	1.0	1.0	134.0	0	0	2.17	0	0	9,336	714,193	0
							TOTALS	**111,108**	**79,779**	**99,997**	**2,706,961**	**2,849,703**

B = 240.0 ft

L = 279.5 ft **WEIGHT = 97,749 k** **FS_{UPLIFT} = 1.23**

Depth = 5.0 ft + 1.5 ft deep key with base at Elev 93.5 ft.

Note: Elevations are referenced to assumed final grade of Elev 100.
Joint 0 equals clayey soils enclosed by perimeter key with γ = 90 pcf and width of key = 6.5 ft.

TABLE 2.6-12

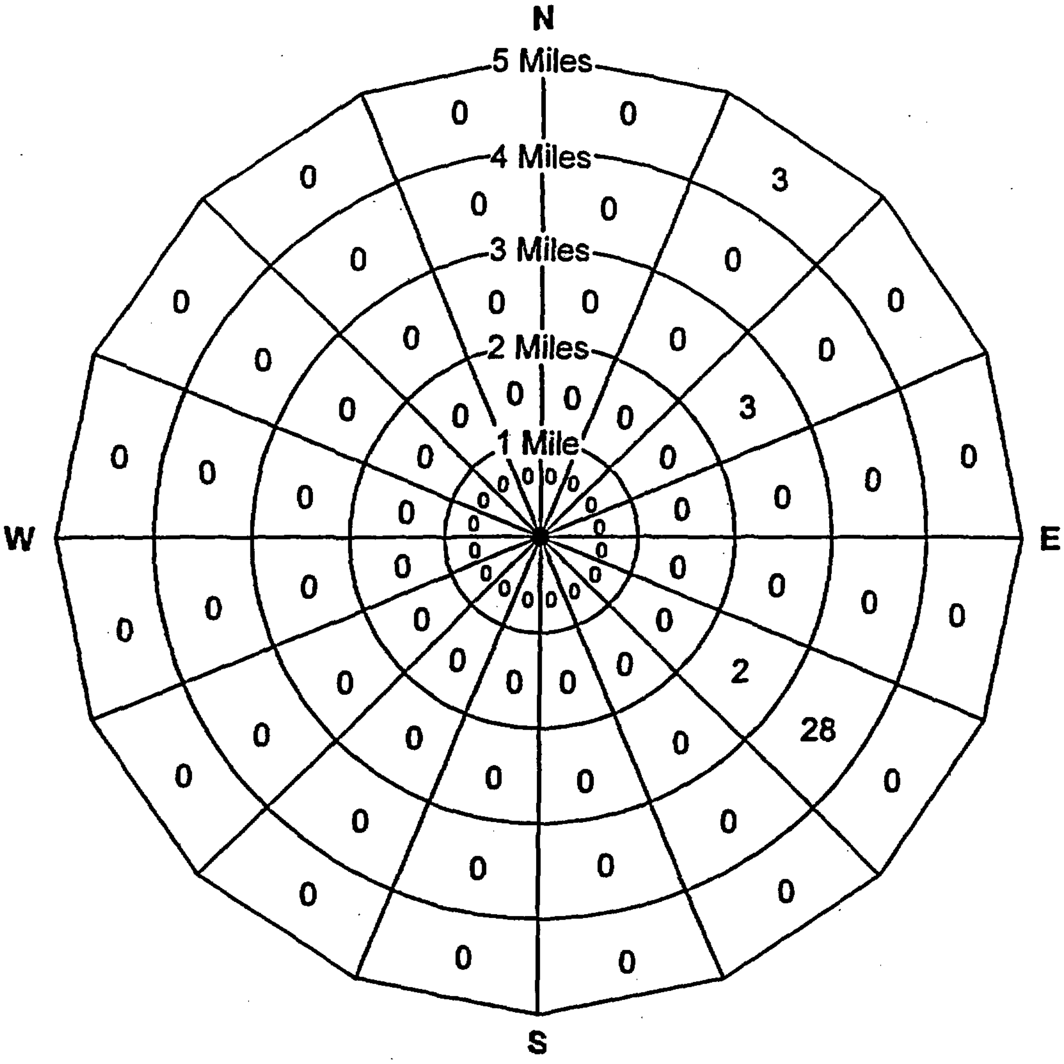
RESULTS OF CONSOLIDATION TESTS IN ORDER OF DECREASING VOID RATIO

Boring	Sample	Average Depth (ft)	USC Code	INITIAL				FINAL				Inundated?	Comment
				Water Content (%)	Dry Density (pcf)	Void Ratio	Sat'n (%)	Water Content (%)	Dry Density (pcf)	Void Ratio	Sat'n (%)		
CTB-N	U-2D	8.6	MH	63.0	48.4	2.511	68.2	60.6	64.0	1.655	99.5	@ 1 TSF	Inundated 41 minutes after application of vertical stress of 2 ksf.
C-1	U-3D	11.4	ML	46.7	51.7	2.285	55.6	62.4	64.1	1.649	103.0	@ 0.5 TSF	
CTB-S	U-3C	10.0	MH	72.2	51.9	2.269	86.6	54.4	67.4	1.519	97.4	@ 1 TSF	Inundated 34 minutes after application of vertical stress of 2 ksf.
C-1	U-3C	11.2	ML	38.9	55.8	2.041	51.8	51.9	68.4	1.484	95.2	No	Porous stones moist.
C-2	U-2E	11.7	ML	39.7	57.5	1.952	55.3	65.0	59.8	1.840	96.0	@ 0.5 TSF	Test stopped @ 2 TSF
CTB-4	U-2E	9.8	CH	48.9	63.2	1.687	78.8	42.1	75.8	1.240	92.3	No	
CTB-5	U-12C	23.5	MH	52.4	63.3	1.683	84.6	43.6	75.0	1.265	93.8	No	
C-1	U-3B	10.8	ML	30.3	64.7	1.625	50.7	28.7	73.4	1.315	59.3	No	Porous stones dry.
C-2	U-2C	10.9	ML	27.6	64.9	1.615	46.4	44.2	76.2	1.230	97.7	@ 0.5 TSF	
CTB-5	U-14E	27.3	CL	26.2	90.9	0.868	82.1	24.9	97.9	0.735	92.2	No	

TABLE 2.6-13
SLIDING STABILITY OF CANISTER TRANSFER BUILDING USING SHEAR STRENGTH ALONG BOTTOM OF PLANE FORMED BY 1.5-FT DEEP PERIMETER KEY AND RESISTANCE FROM SOIL CEMENT

Joint	MASS X	MASS Y	MASS Z	N-S a_x	Vert a_y	E-W a_z	Static F_v	Earthquake $Shear_{N-S}$	Earthquake F_v	Earthquake $Shear_{E-W}$
	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	g	g	g	k	k	k	k
0	260.1	260.1	260.1	1.047	0.783	0.920	8,368	8,761	6,551	7,699
1	1,908.0	1,908.0	1,908.0	1.047	0.783	0.920	61,380	64,265	48,055	56,470
2	420.4	420.4	420.4	1.111	0.821	0.994	13,524	15,023	11,106	13,446
3	304.3	304.3	170.3	1.778	0.913	1.185	9,789	17,402	8,939	6,493
4	144.7	117.1	144.7	1.215	0.928	1.408	3,767	5,656	3,495	6,554
5	1.0	27.6	1.0	0.000	1.840	0.000	888	0	1,634	0
6	1.0	1.0	134.0	0.000	0.000	2.166	32	0	0	9,336
CTB Mat Dimensions: 240' (E-W) x 279.5' (N-S) x 5'						*Totals =*	97,749	111,108	79,779	99,997

For c =		1.70	ksf	ϕ = 0.0	degrees	N (*k*)	*Resisting* T (*k*)	*Driving* V (*k*)	FS
Earthquake Vertical Forces Acting Up	IIIA	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ -79,779	40% $F_{H(EW)}$ 39,999	17,970	135,999	59,792	2.27
	IIIB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ -31,912	100% $F_{H(EW)}$ 99,997	65,837	135,999	109,429	1.24
	IIIC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ -31,912	40% $F_{H(EW)}$ 39,999	65,837	135,999	118,088	1.15
Earthquake Vertical Forces Acting Down	IVA	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ 79,779	40% $F_{H(EW)}$ 39,999	177,529	135,999	59,792	2.27
	IVB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ 31,912	100% $F_{H(EW)}$ 99,997	129,661	135,999	109,429	1.24
	IVC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ 31,912	40% $F_{H(EW)}$ 39,999	129,661	135,999	118,088	1.15
Soil Cement ΔF_H for q_u (psi) =		250	21,600	N/A	25,155	for FS = 2.0			



Total Population: 36

Source: 1990 Census, adapted by SWEC.

Figure 2.1-1

POPULATION DISTRIBUTION WITHIN 5 MILES OF PFSF

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

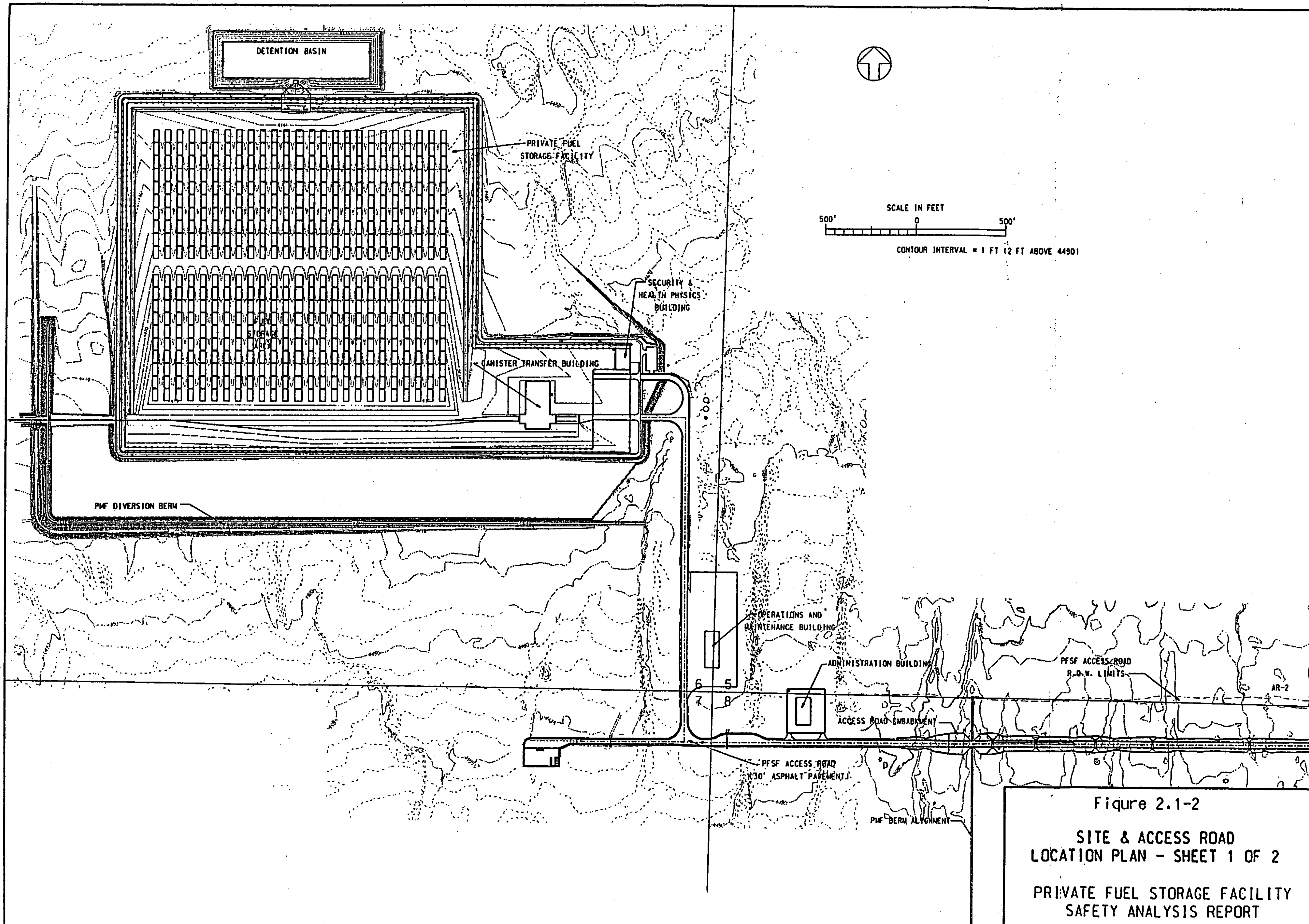


Figure 2.1-2

SITE & ACCESS ROAD LOCATION PLAN - SHEET 1 OF 2

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

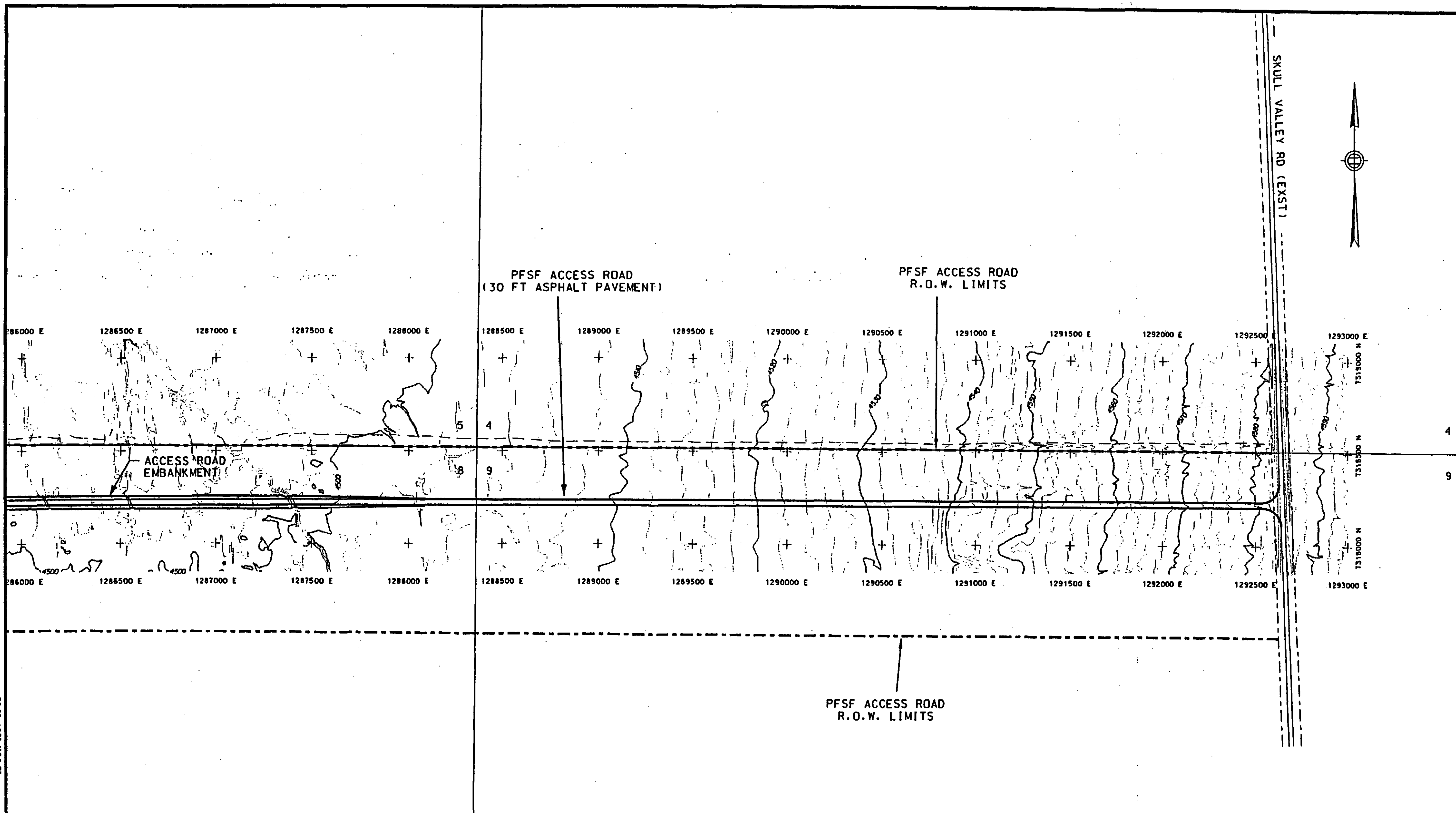


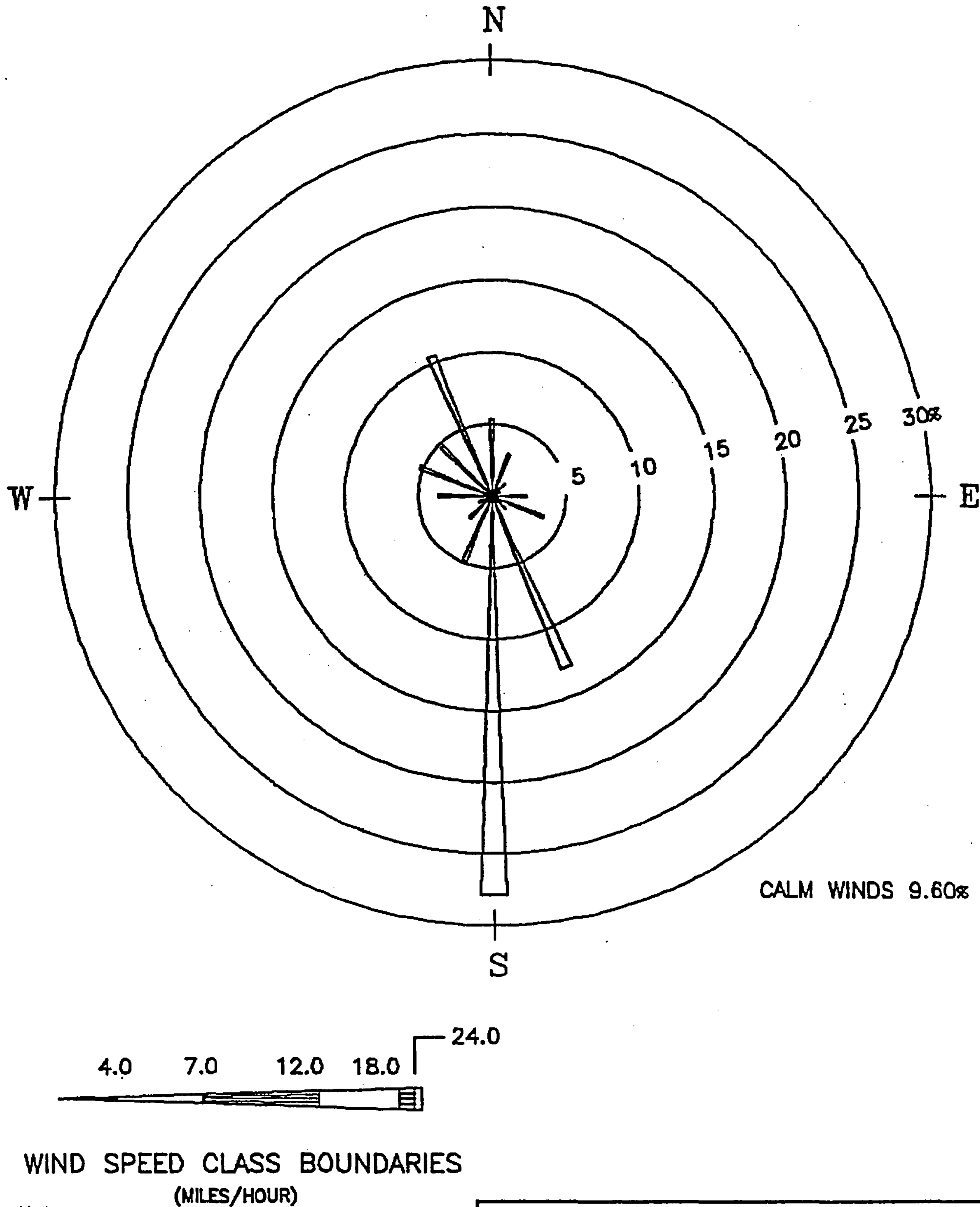
Figure 2.1-2

SITE & ACCESS ROAD LOCATION PLAN - SHEET 2 OF 2

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

13-JUN-1997 16:03

G:\j05996\gsk\b\28\b-0.dgn

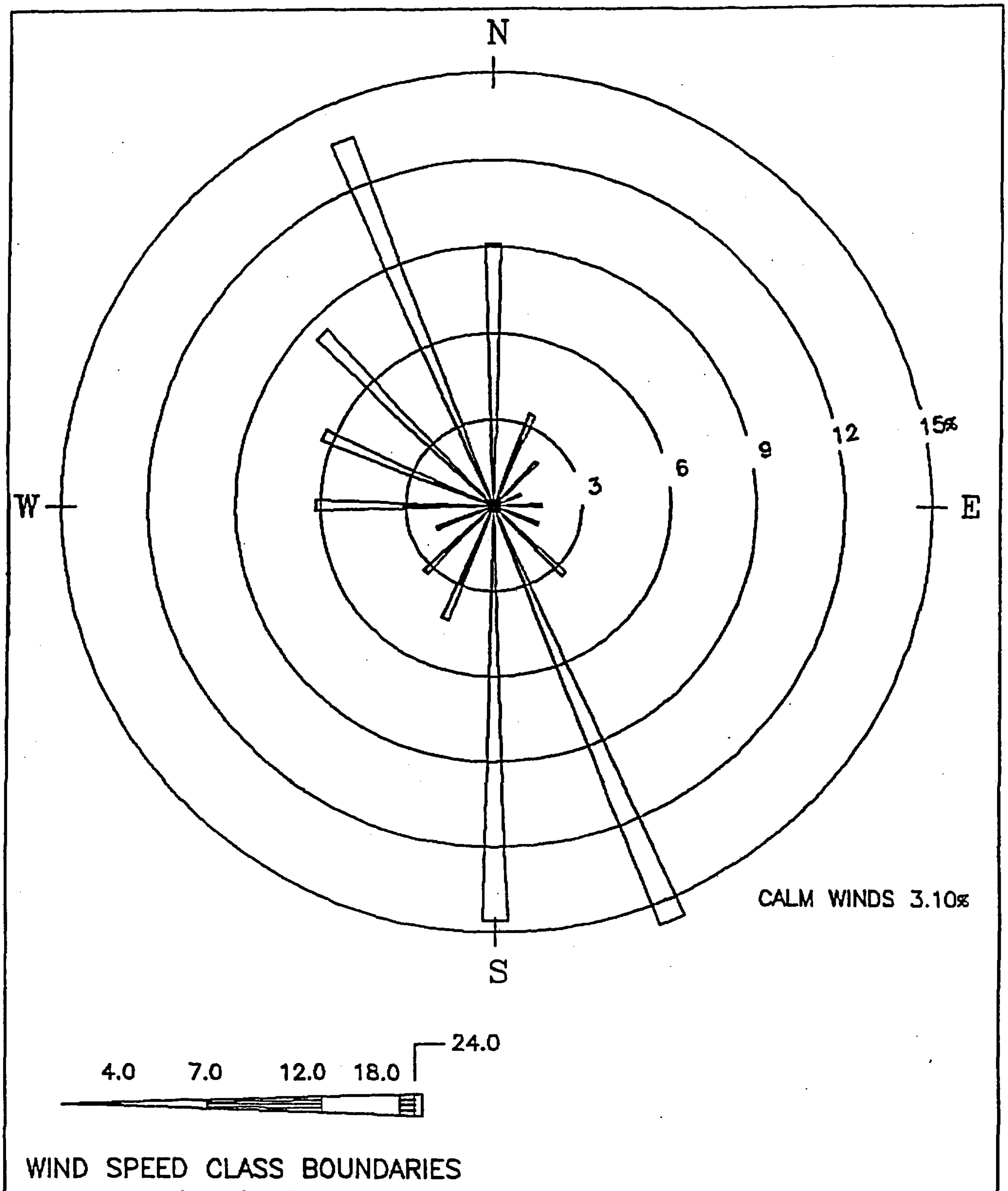


Notes:
Diagram of the frequency of occurrence for each wind direction. Wind direction is the direction from which the wind is blowing. Example - wind is blowing from the north 5.3 percent of the time.

Figure 2.3-1

SALT LAKE CITY WINDROSE: 1988-1992 WINTER

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

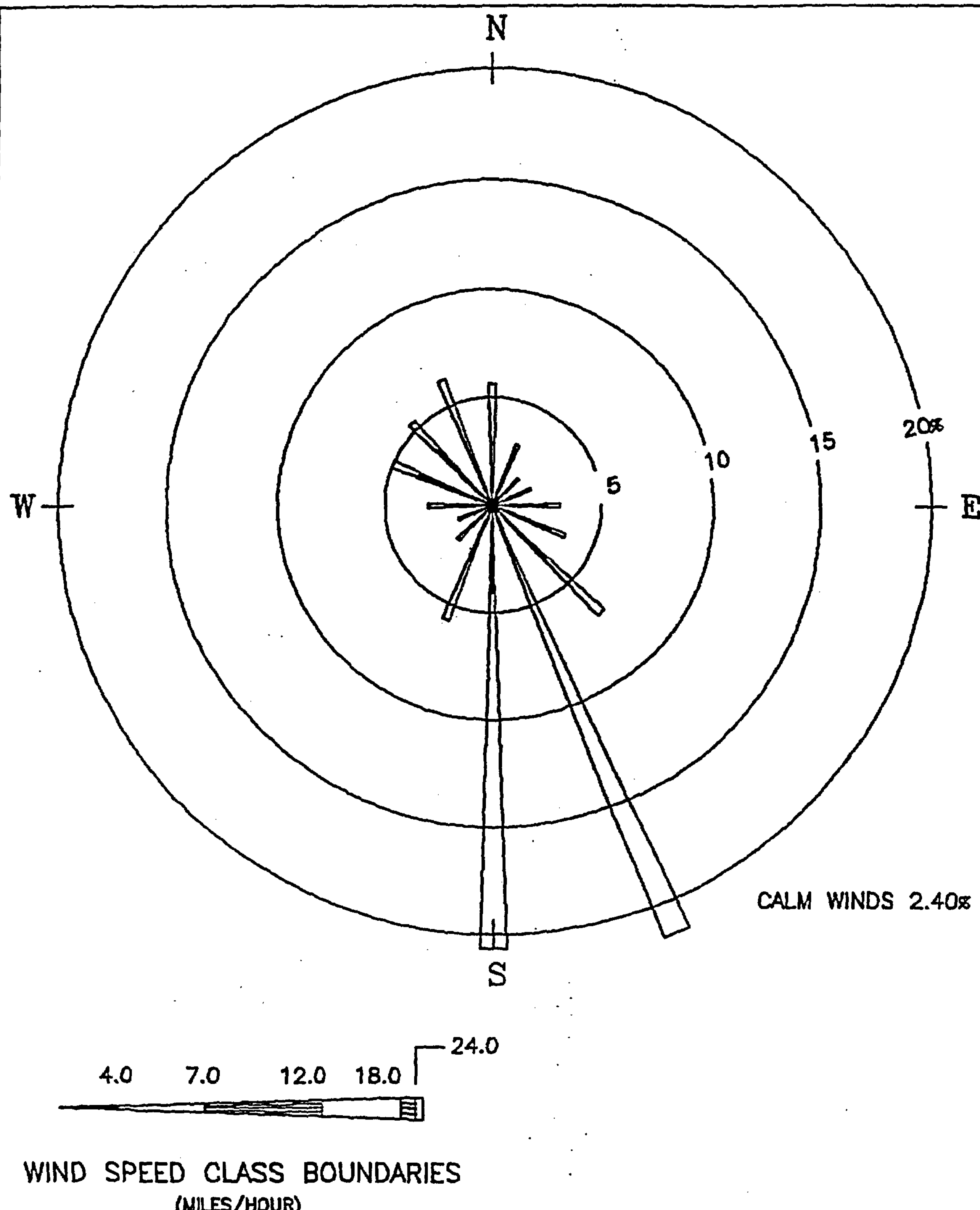


Notes:
Diagram of the frequency of occurrence for each wind direction. Wind direction is the direction from which the wind is blowing. Example - wind is blowing from the north 9.1 percent of the time.

Figure 2.3-2

SALT LAKE CITY WINDROSE: 1988-1992
SPRING

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

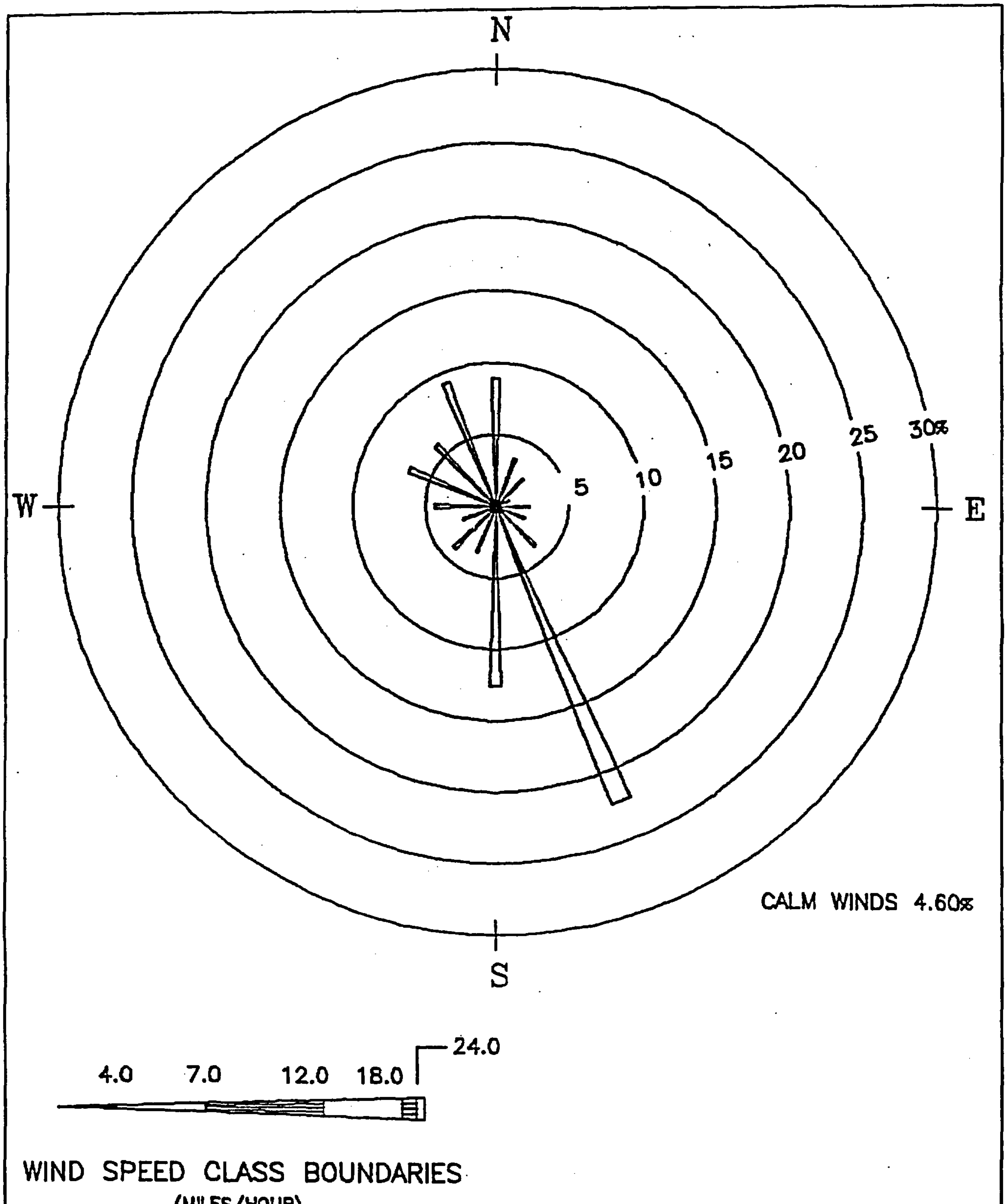


Notes:
Diagram of the frequency of occurrence for each wind direction. Wind direction is the direction from which the wind is blowing. Example - wind is blowing from the north 5.7 percent of the time.

Figure 2.3-3

SALT LAKE CITY WINDROSE: 1988-1992 SUMMER

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

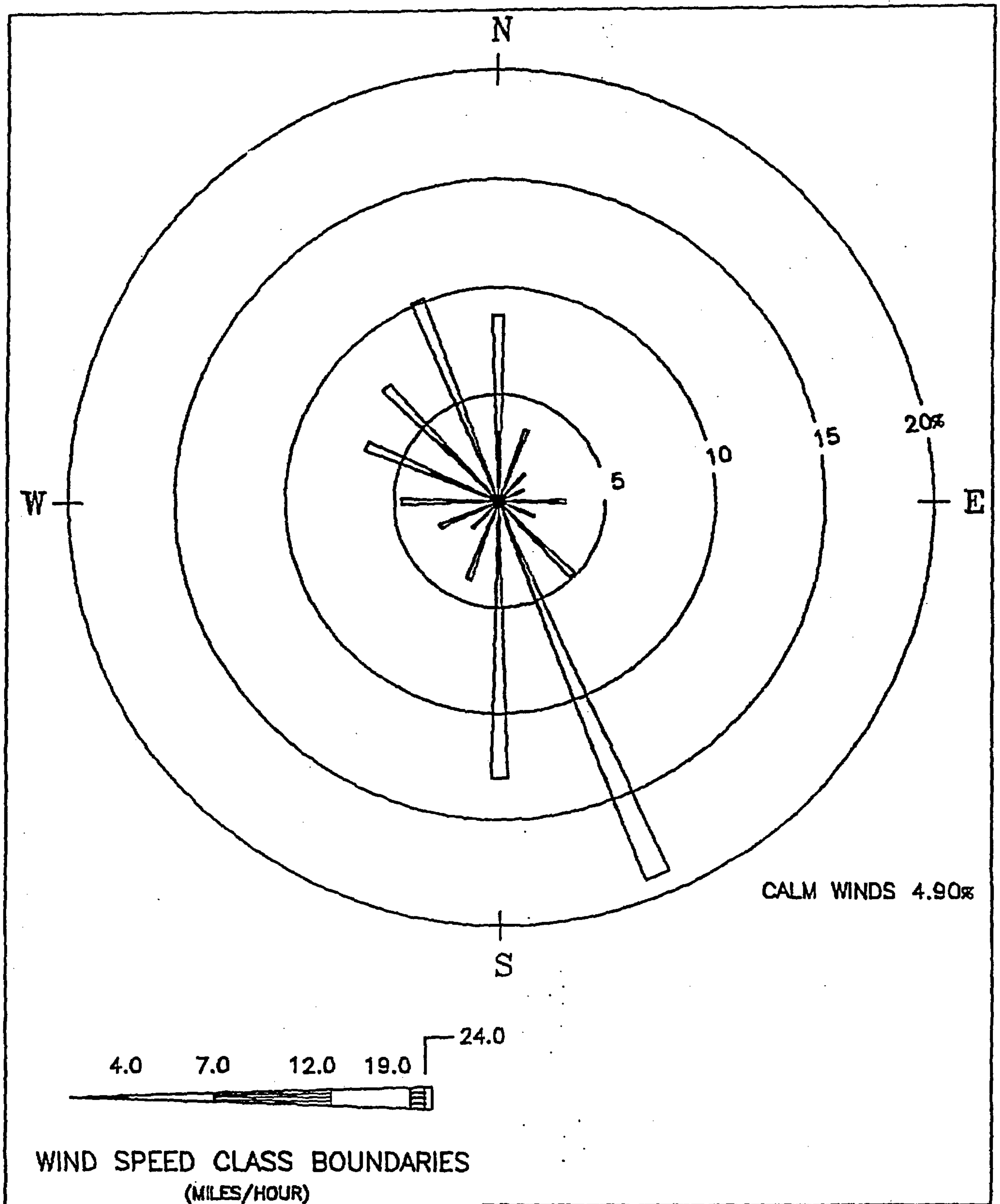


Notes:
Diagram of the frequency of occurrence for each wind direction. Wind direction is the direction from which the wind is blowing. Example - wind is blowing from the north 8.9 percent of the time.

Figure 2.3-4

SALT LAKE CITY WINDROSE: 1988-1992 AUTUMN

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

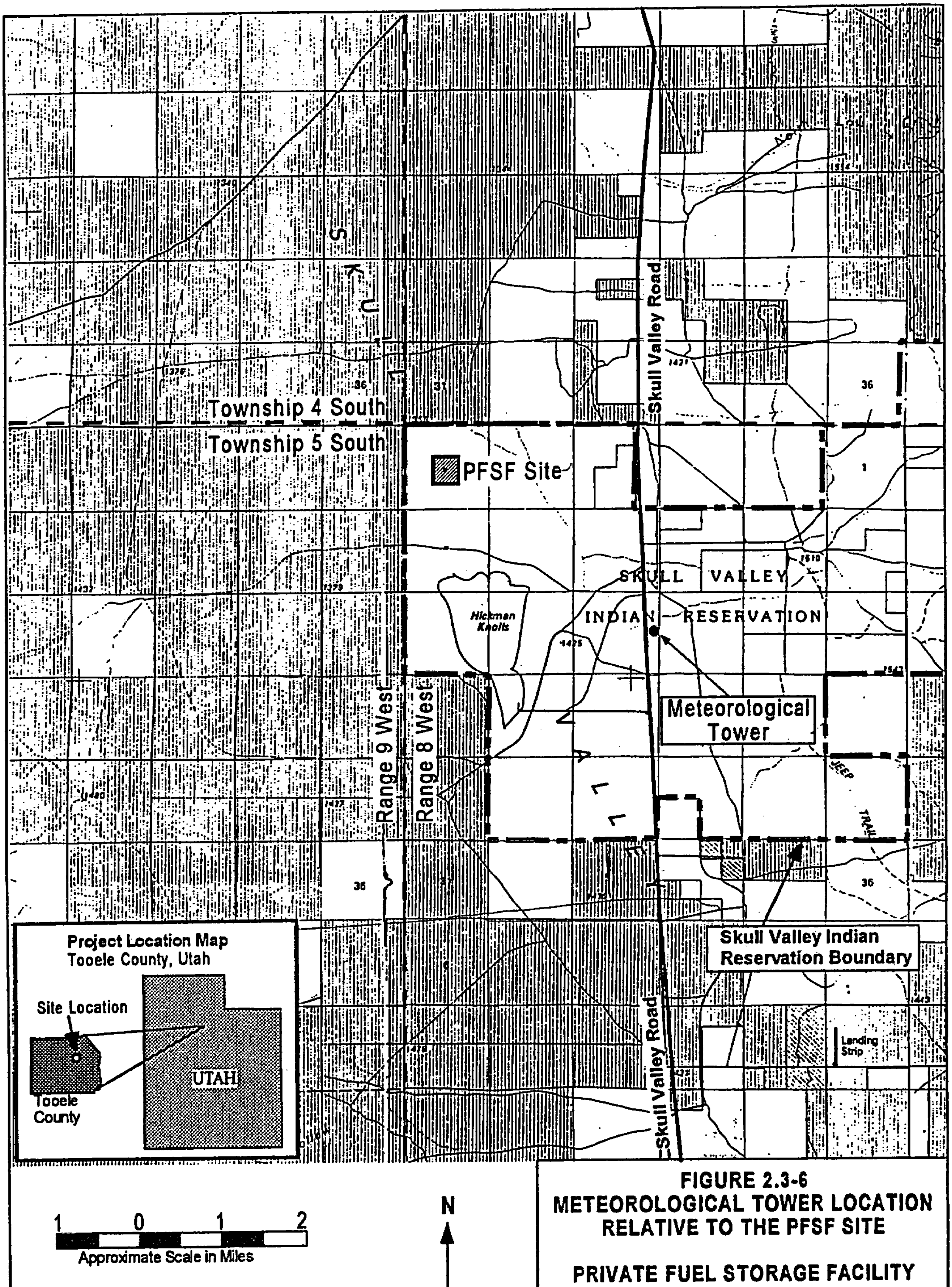


Notes:
Diagram of the frequency of occurrence for each wind direction. Wind direction is the direction from which the wind is blowing. Example - wind is blowing from the north 8.7 percent of the time.

Figure 2.3-5

SALT LAKE CITY WINDROSE: 1988-1992

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



Source: Bureau of Land Management 1:100,000 Scale Topographic Map of Rush Valley, 1979.

FIGURE 2.3-6
METEOROLOGICAL TOWER LOCATION RELATIVE TO THE PFSF SITE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

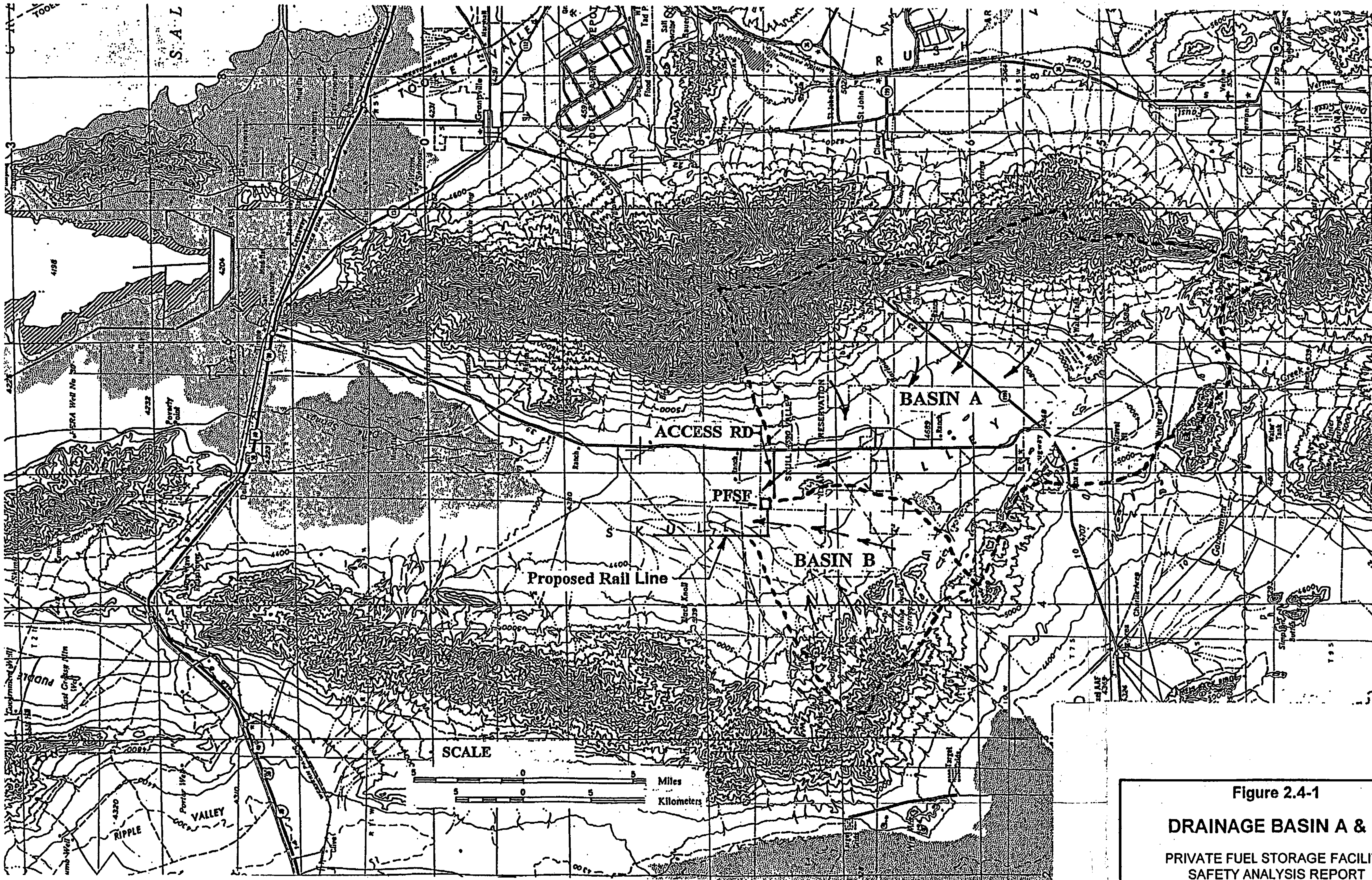
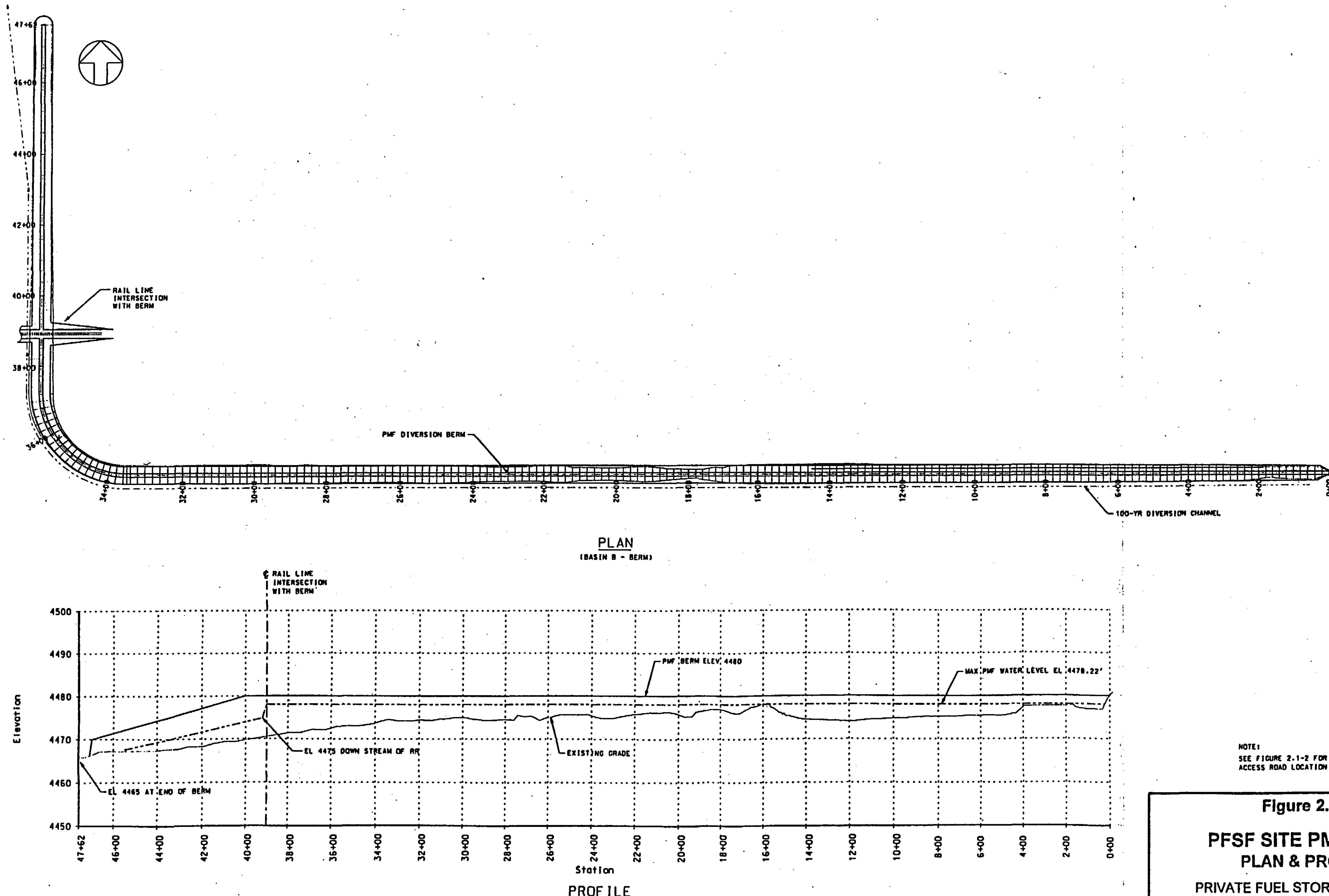


Figure 2.4-1

DRAINAGE BASIN A & B

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

Figure 2.4-2

PFSF SITE PMF BERM
PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

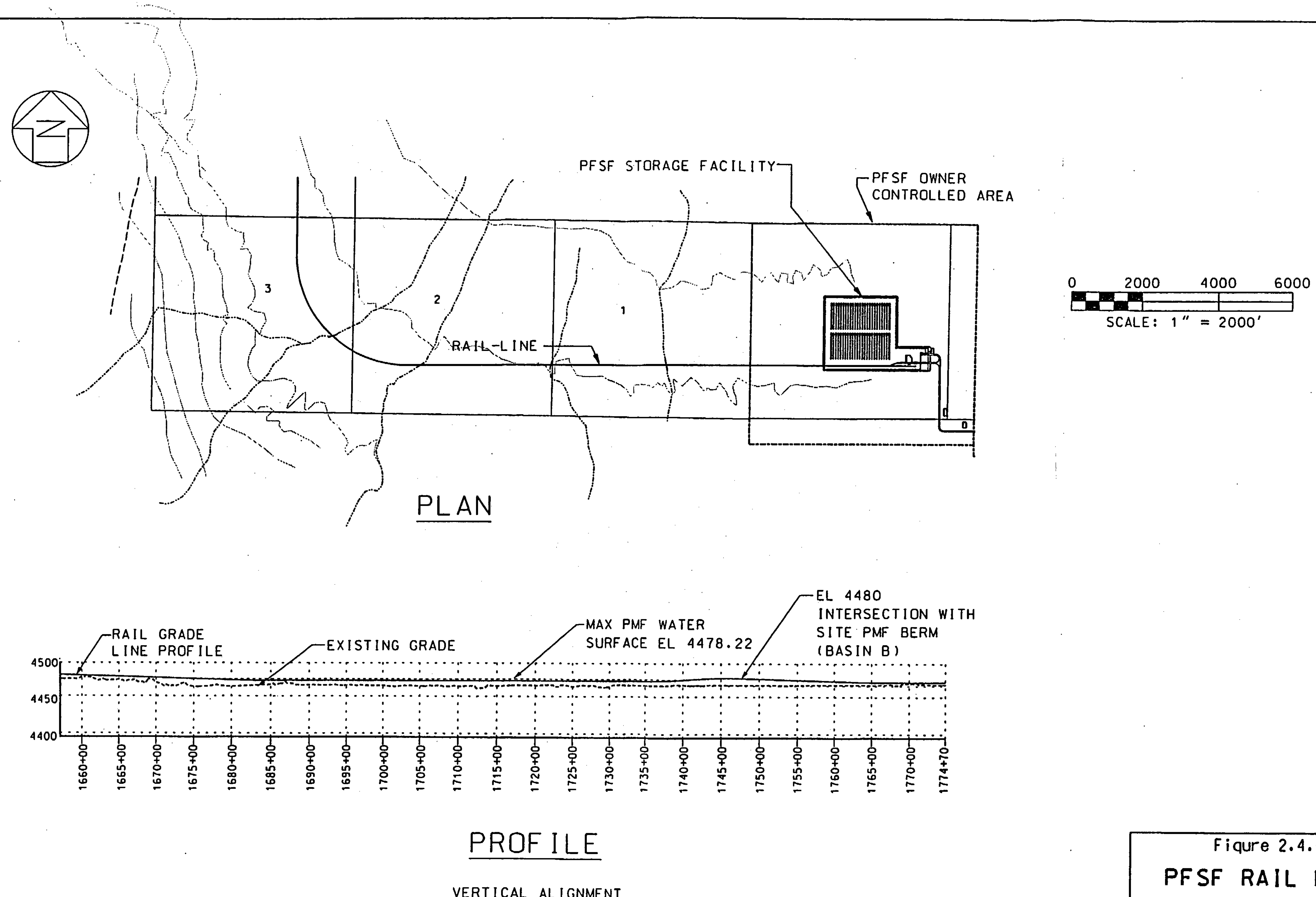
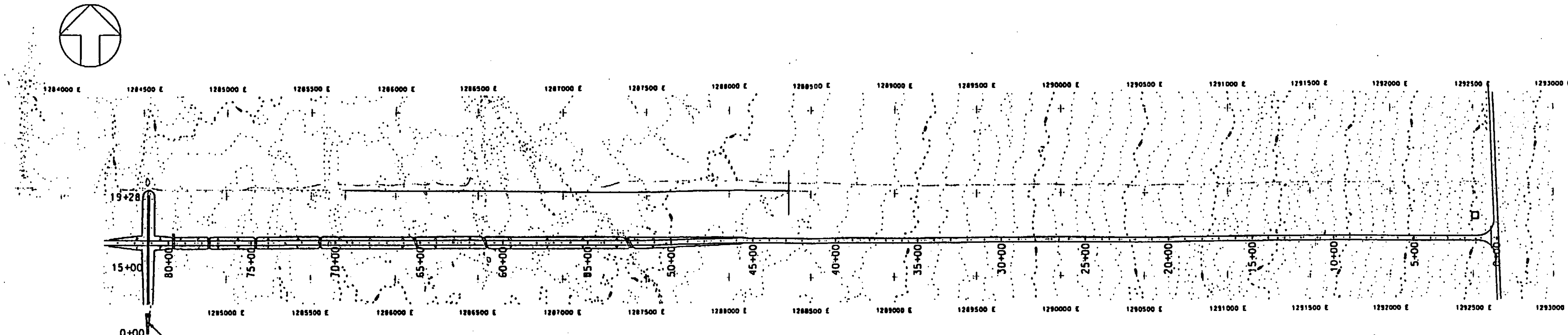


Figure 2.4.-3

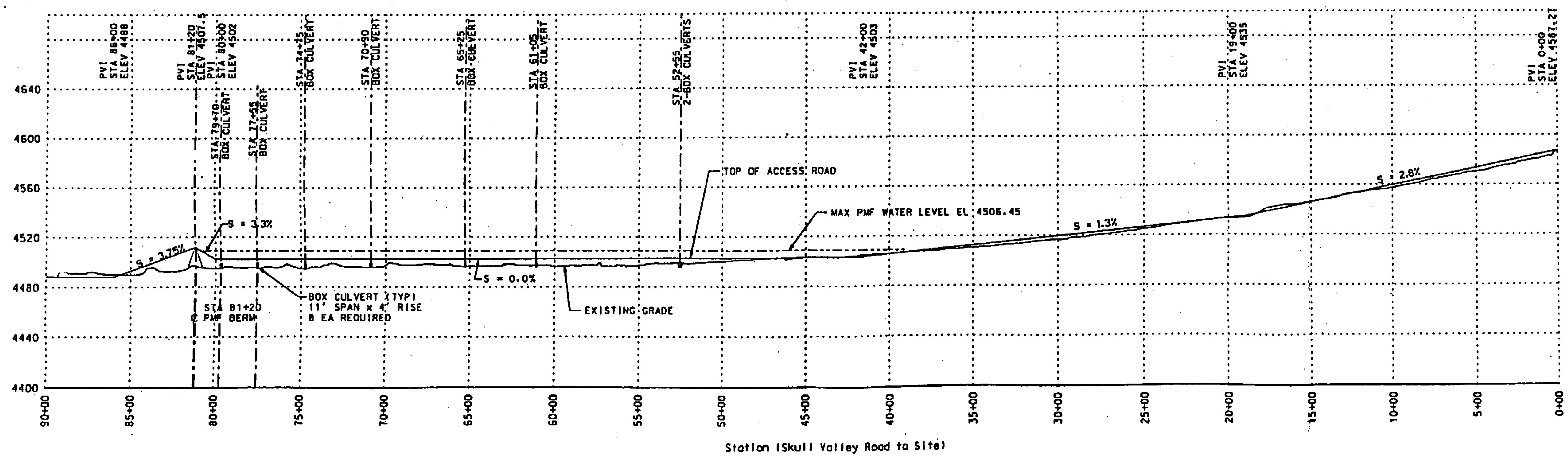
PFSF RAIL LINE PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

Revision 21



ACCESS ROAD PLAN



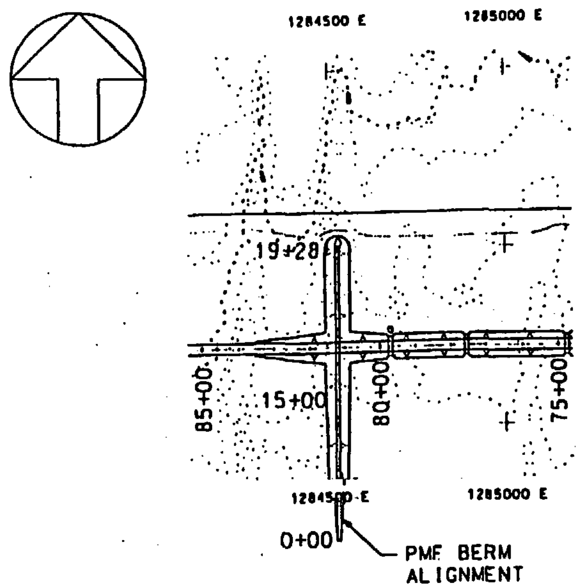
ACCESS ROAD PROFILE

NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

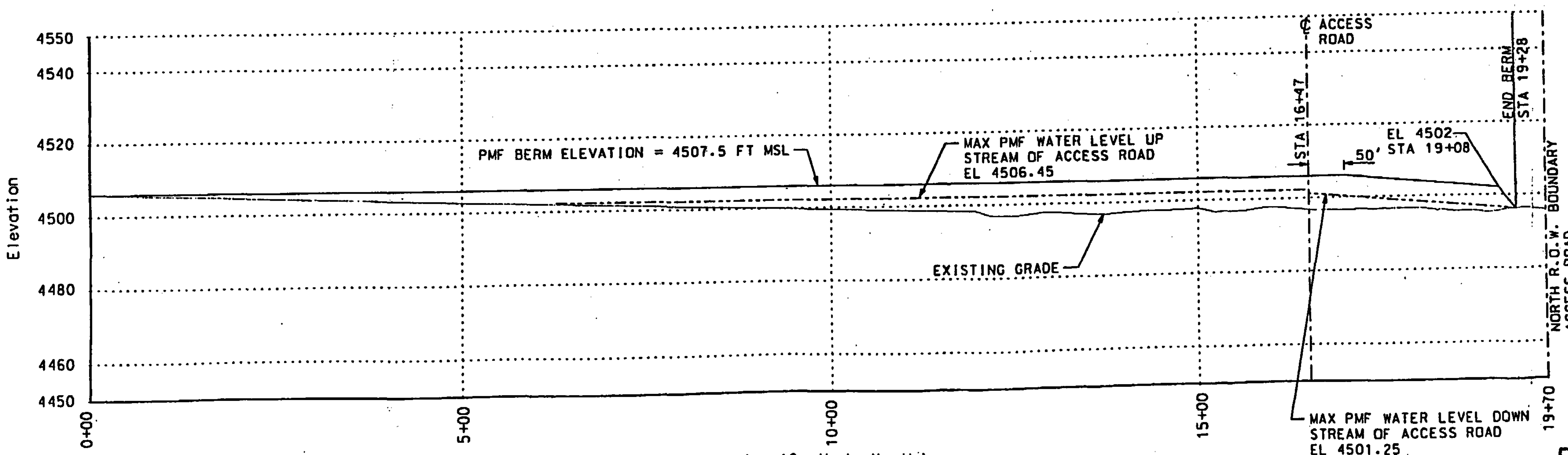
Figure 2.4-4

PFSF ACCESS ROAD
PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



ACCESS ROAD PLAN



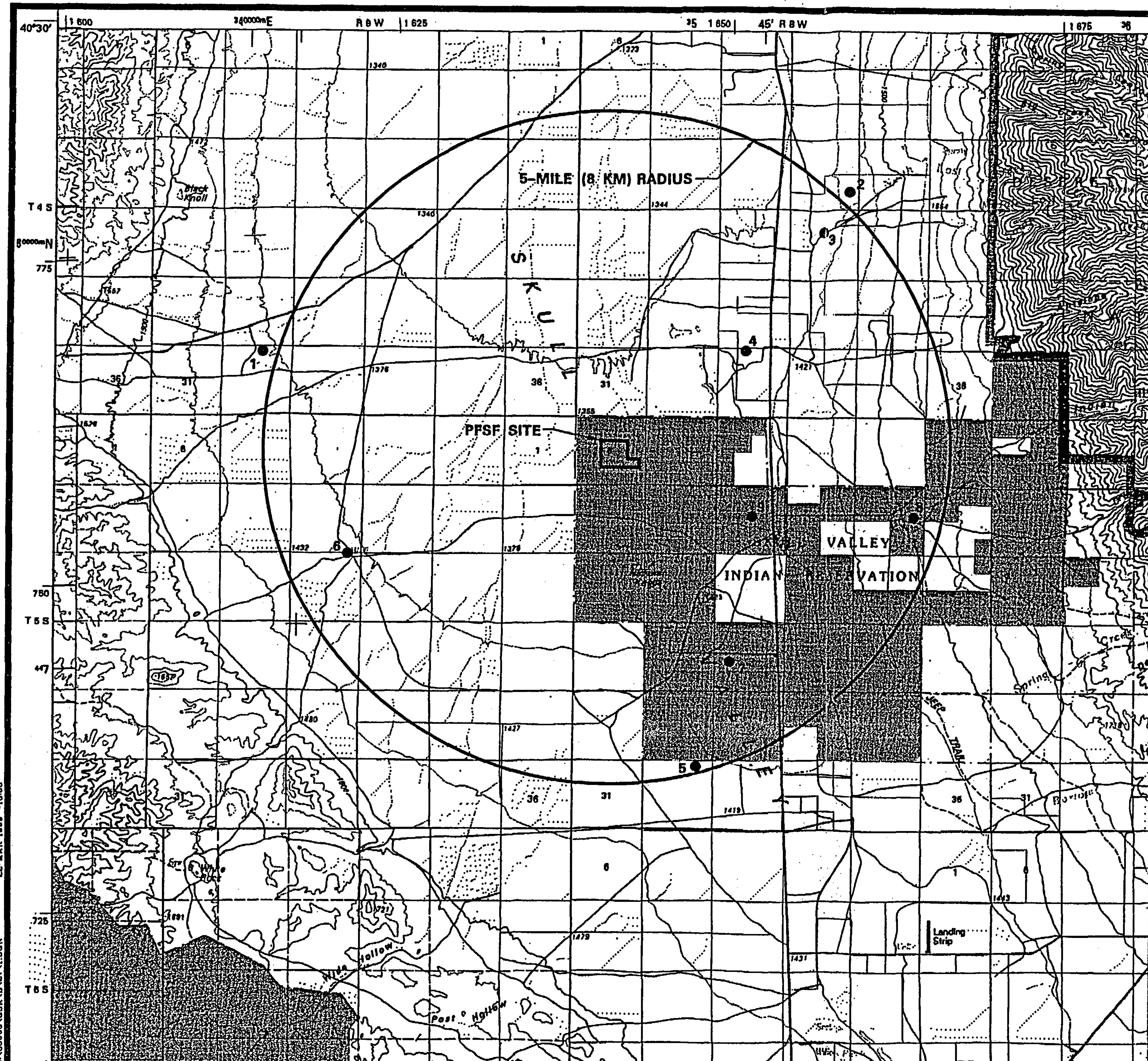
PMF BERM PROFILE
(BETWEEN BASIN A/B)

NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

Figure 2.4-5

PFSF ACCESS ROAD PMF BERM PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



WELL MAP NO.	OWNER	TOTAL DEPTH	DATE DRILLED	USE	DEPTH TO WATER	YIELD	CURRENT WATER RIGHT
1	SKULL VALLEY CO., LTD.	340'	1956	STOCK WATERING	295'	35 gpm	YES (1955) 0.015 cfs
2	ISLAND RANCHING CO., INC.	325'	NO INFO.	IRRIGATION, STOCK	NO DATA	NO DATA	YES (1959) 2.180 cfs
3	ISLAND RANCHING CO., INC.	347'	1954	IRRIGATION, STOCK	90'	NO DATA	YES (1954) 1.226 cfs
4	ANSCHUTZ LAND CO., LTD.	408'	NO INFO.	IRRIGATION, STOCK	NO DATA	NO DATA	YES (1960) 0.7487 cfs
5	ANSCHUTZ LAND CO., LTD.	209'	1948	STOCK WATERING	150'	20 gpm	YES (1948) 0.015 cfs
6	ANSCHUTZ LAND CO., LTD.	292'	1940	STOCK WATERING	280'	12 gpm	YES (1940) 0.015 cfs
7	SKULL VALLEY INDIAN RESERV.	401'	1975	DOMESTIC, INDUSTRIAL	77.5'	15 gpm	NOT REQUIRED
8	SKULL VALLEY INDIAN RESERV.	651'	1976	DOMESTIC	519.5'	60 gpm	NOT REQUIRED
9	SKULL VALLEY INDIAN RESERV.	NO DATA	NO DATA	DOMESTIC	NO DATA	NO DATA	NOT REQUIRED

SOURCES: BASE MAP IS RUSH VALLEY, UT, BUREAU OF LAND MANAGEMENT SPECIAL EDITION, SURFACE MANAGEMENT STATUS, 1:100,000 SCALE METRIC, 1981. WELL DATA ARE FROM STATE OF UTAH, DIVISION OF WATER RIGHTS AND HOOD AND WADDELL (1968).

LEGEND:

● WATER WELL LOCATION

SCALE 1 : 100,000

CONTOUR INTERVAL IS 50 METERS

KILOMETERS 0 1 2 3 4 5 6

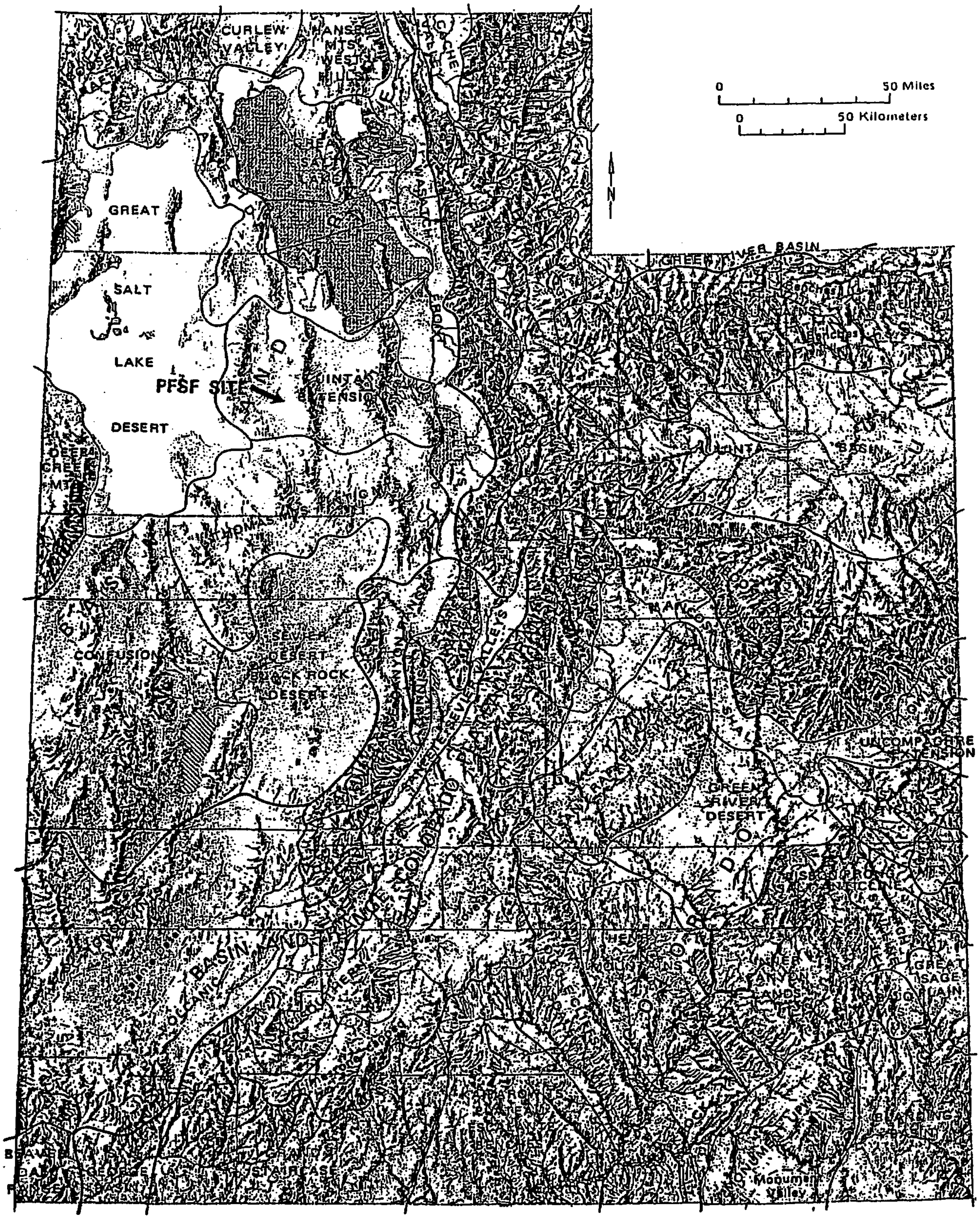
MILES 0 1 2 3 4

Figure 2.5-1

WATER WELLS WITHIN 5 MILES (8 KM) OF PFSF SITE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\05996\GSK\B\27\1.DGN 22-MAR-1999 -16:03



SOURCE: STOKES, 1986

Figure 2.6-1

PHYSIOGRAPHY OF UTAH

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

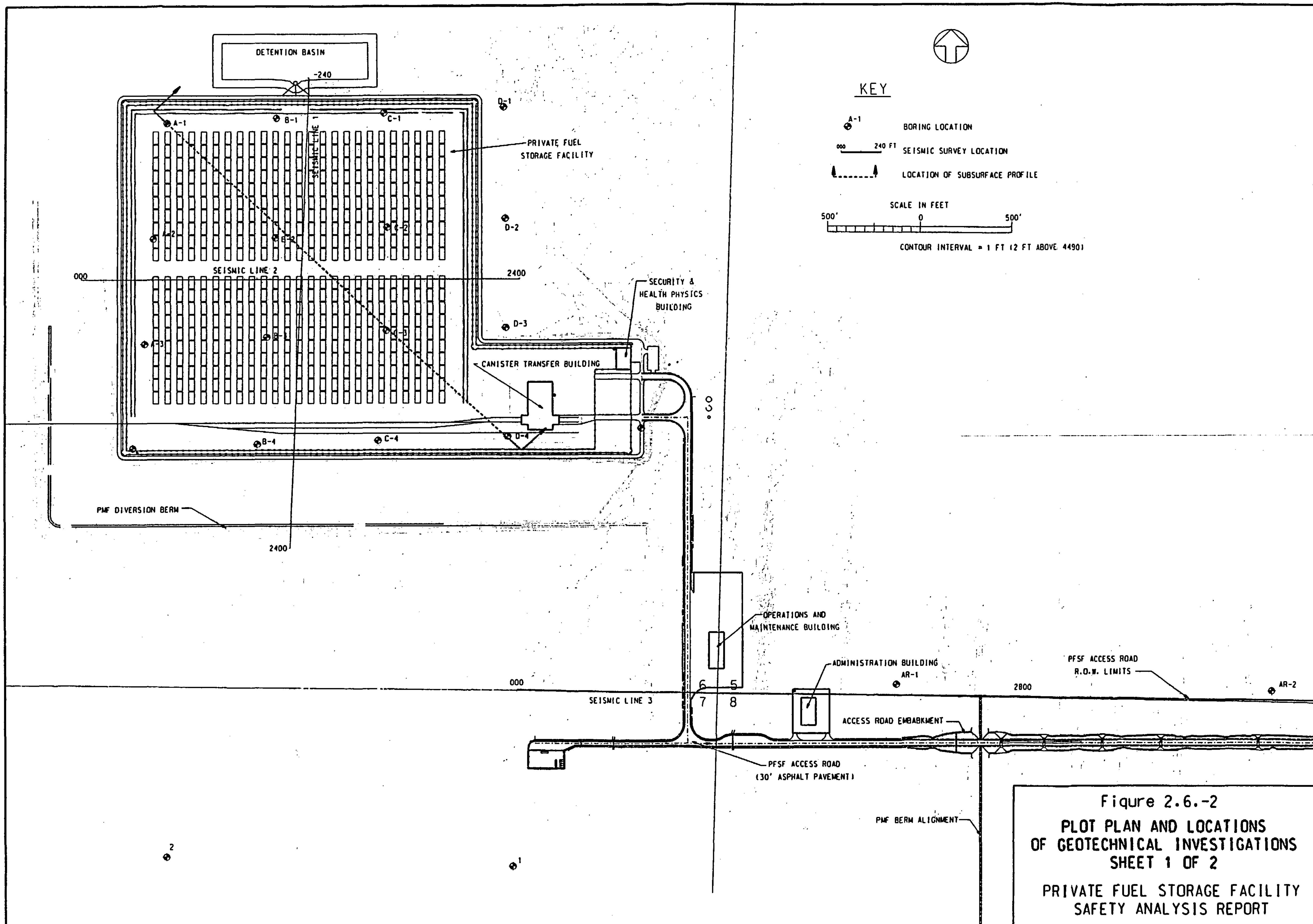


Figure 2.6.-2

PLOT PLAN AND LOCATIONS OF GEOTECHNICAL INVESTIGATIONS SHEET 1 OF 2

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

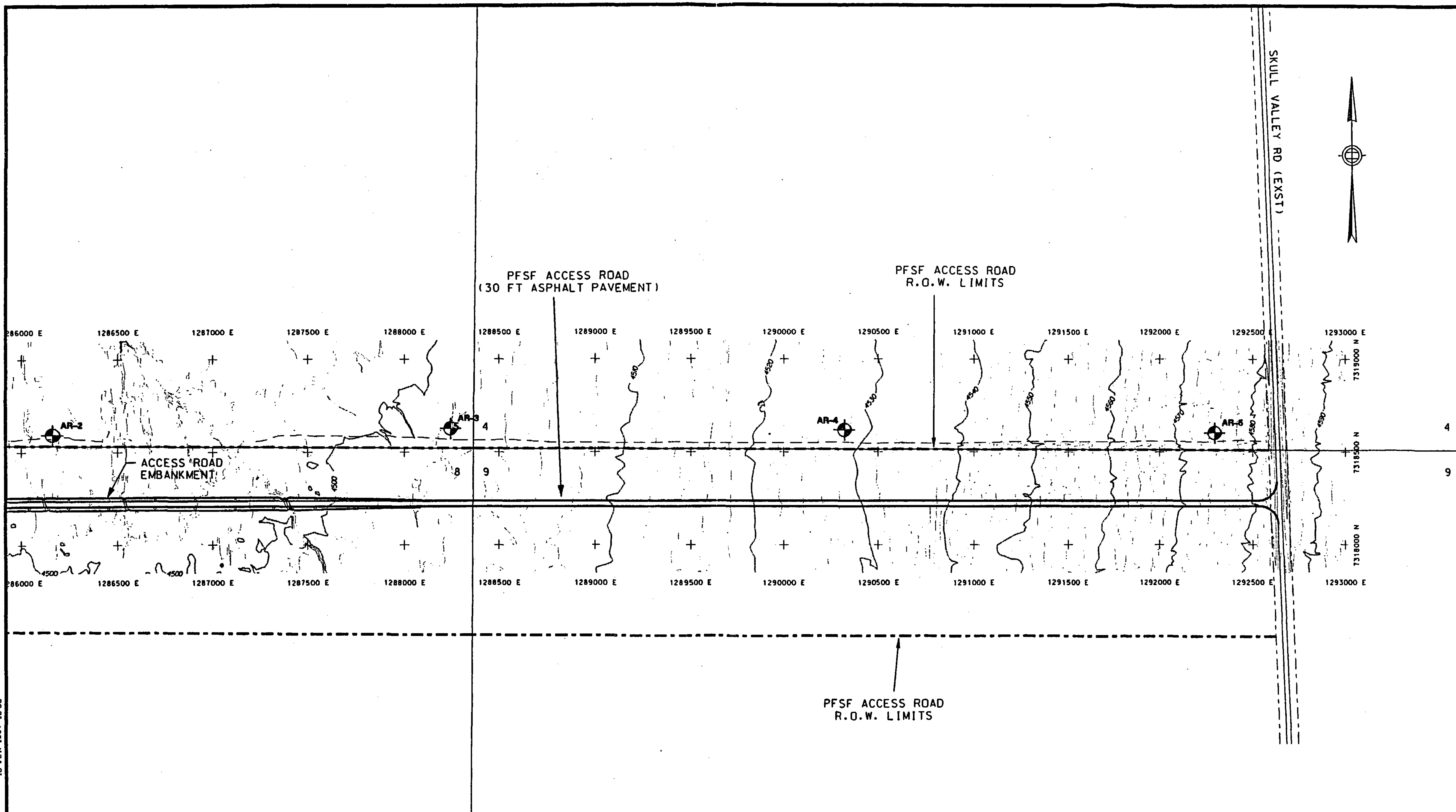
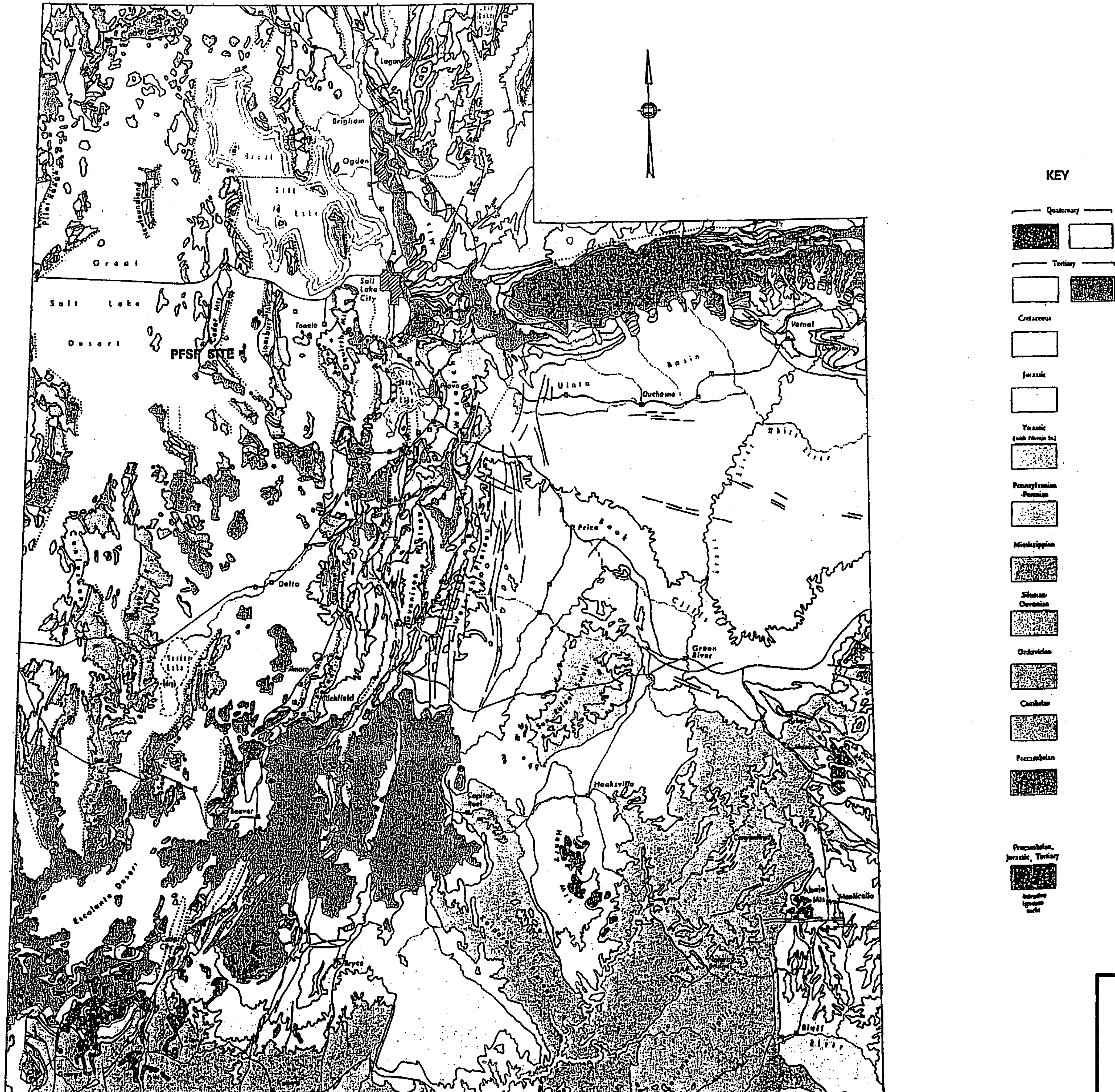


Figure 2.6–2
PLOT PLAN AND LOCATIONS OF GEOTECHNICAL INVESTIGATIONS SHEET 2 OF 2
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\j05998\gsk\b\03\b-0.dgn 13-JUN-1997 13:53



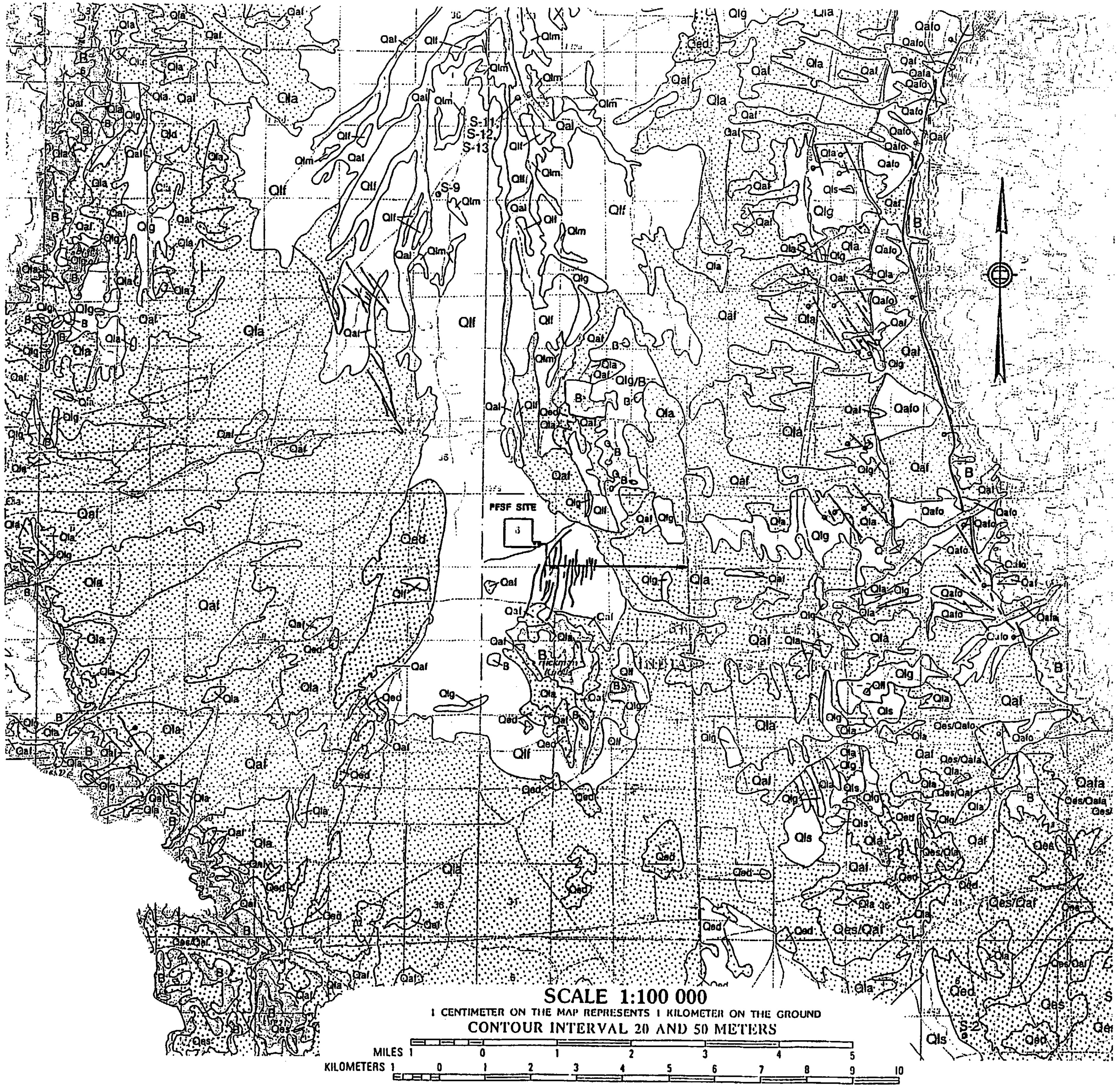
SCALE

SOURCE: HINTZE, in STOKES, 1986

Figure 2.6-3

GEOLOGIC MAP OF UTAH

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



MAP EXPLANATION

Qaf	Active alluvial-fan deposits
Qafa	Abandoned alluvial-fan deposits
Qafo	Inactive alluvial-fan deposits
Qal	Channel alluvium
Qed	Eolian dune deposits
Qes	Nondunal eolian deposits
Qla	Mixed lacustrine and alluvial deposits
Qld	Lacustrine mud
Qlf	Fine-grained lacustrine deposits
Qlg	Lacustrine gravel
Qlm	White marl
Qls	Lacustrine sand
B	Undifferentiated bedrock

Contact

Bonneville shoreline

Provo shoreline

Stansbury shoreline

Gilbert shoreline

Piedmont fault scarps; bar and ball on downthrown side

Faults or fractures having small or undetermined displacement

SOURCE: SACK, 1993

Figure 2.6-4

SURFICIAL GEOLOGY AND PFSF SITE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

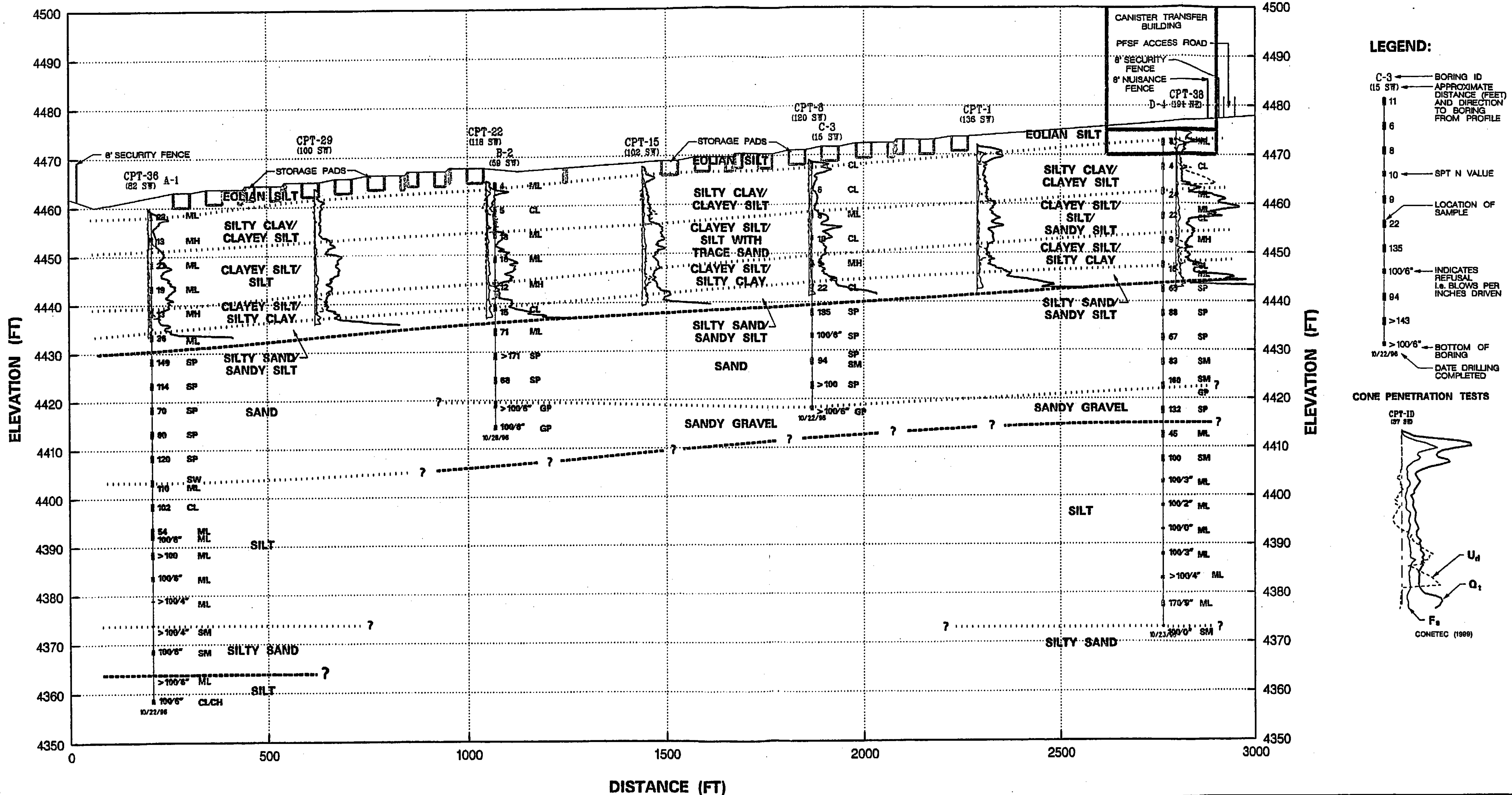
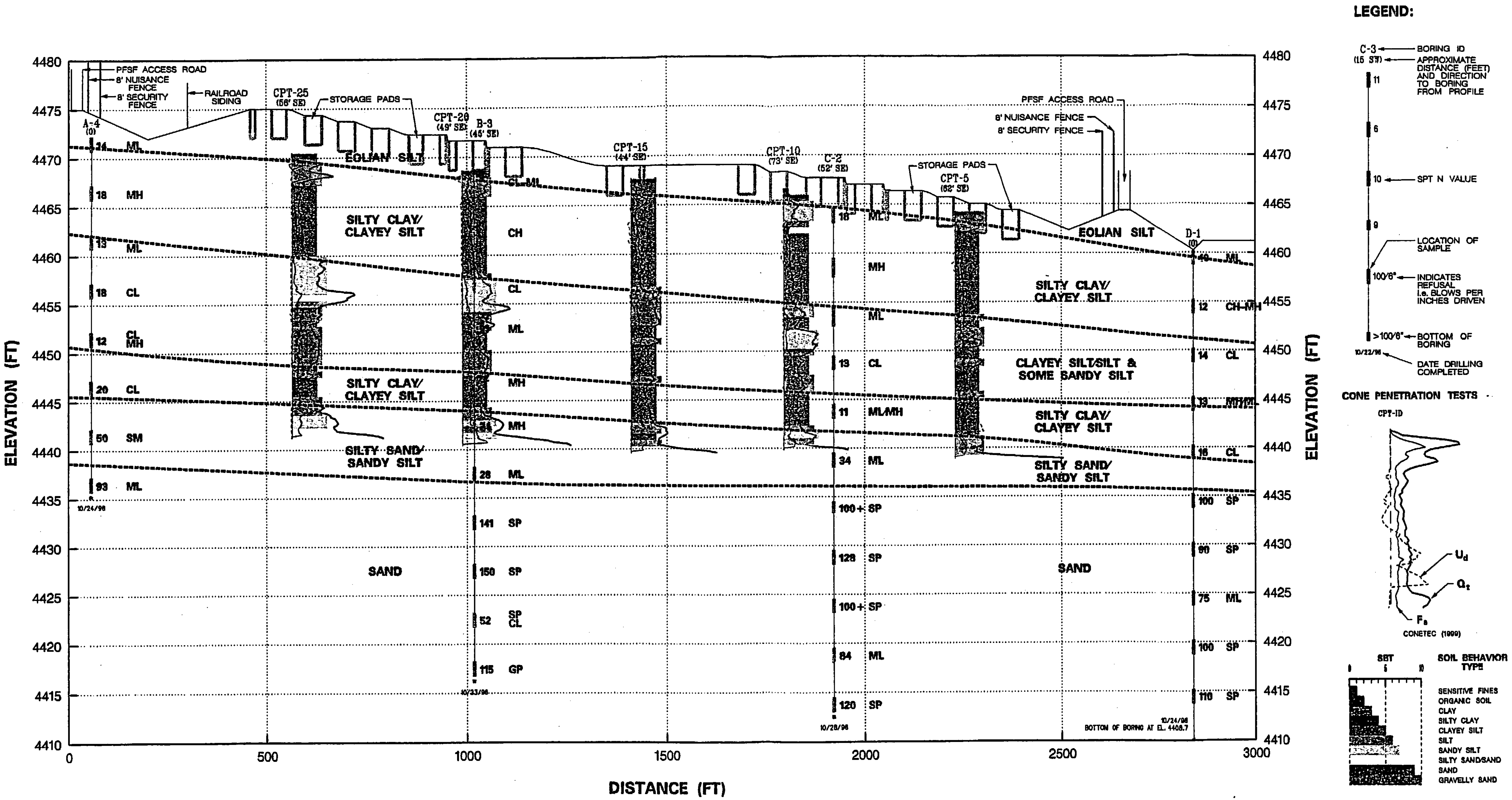


Figure 2.6-5
PAD EMPLACEMENT AREA FOUNDATION PROFILE A-A'
SHEET 1 OF 14
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

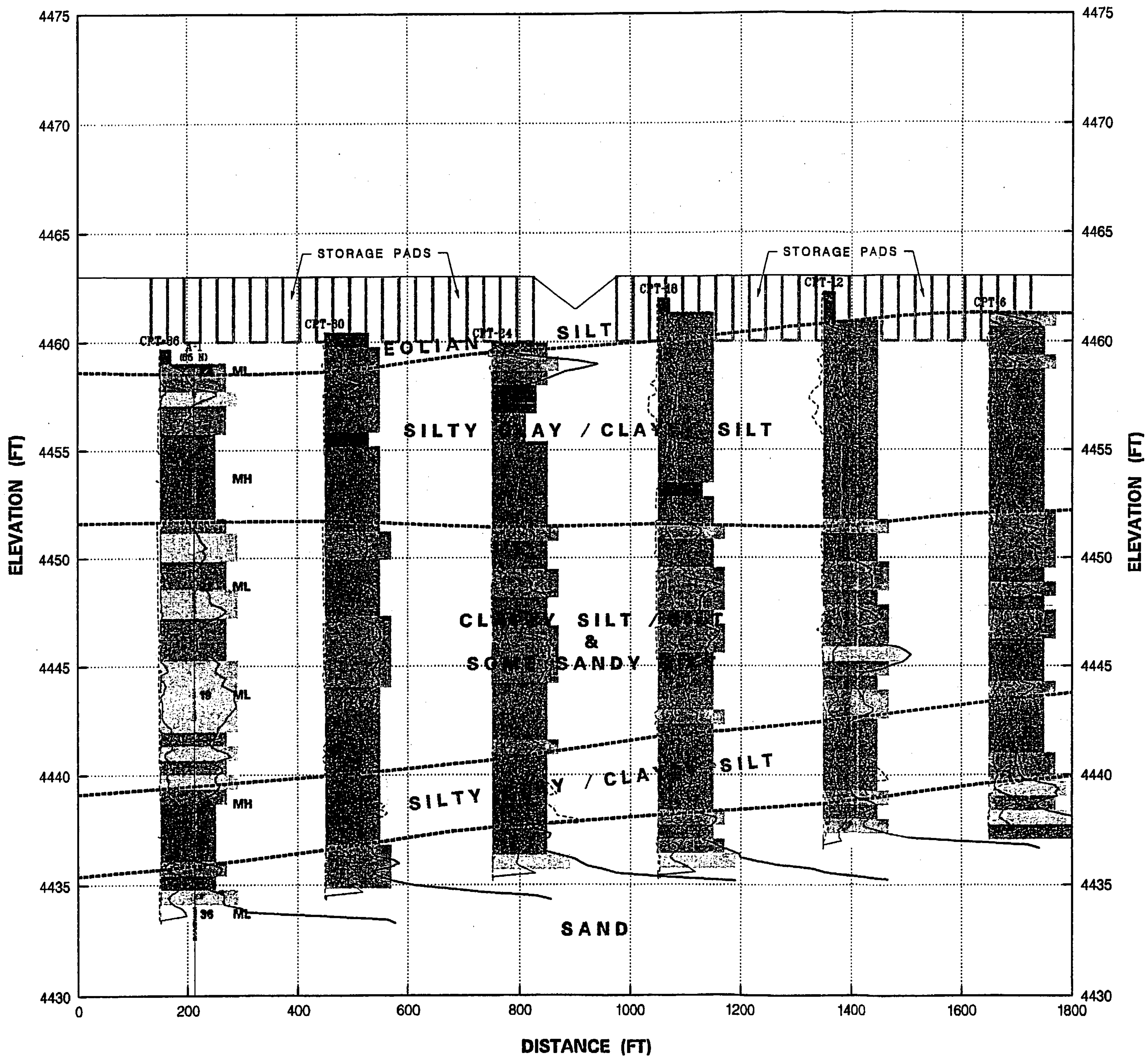


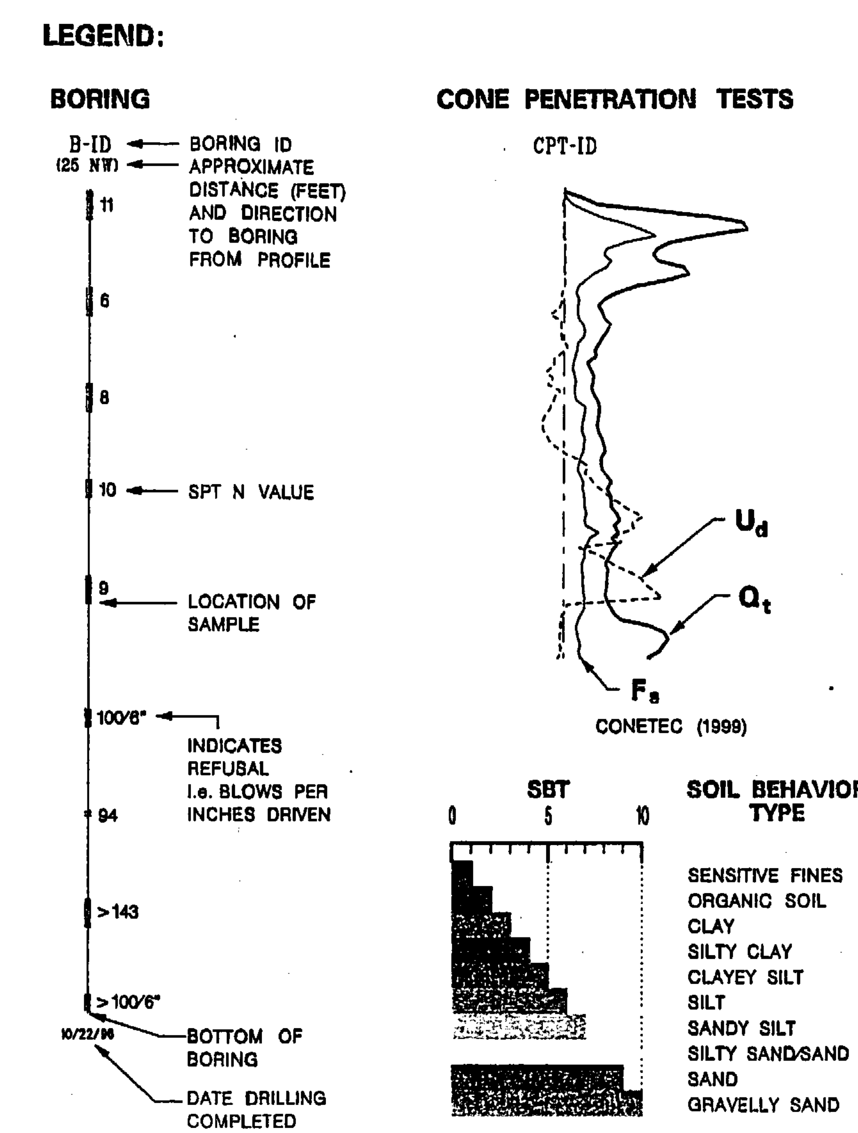
NOTE:
SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

HORIZONTAL 0 250 500 FEET
VERTICAL 0 10 20
SCALE IN FEET

Figure 2.6-5
PAD EMPLACEMENT AREA FOUNDATION PROFILE B-B'
SHEET 2 OF 14
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

22-MAR-2001 16:07





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

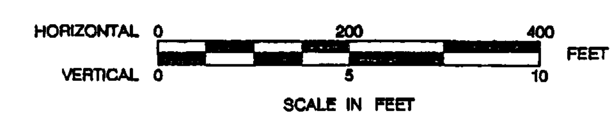
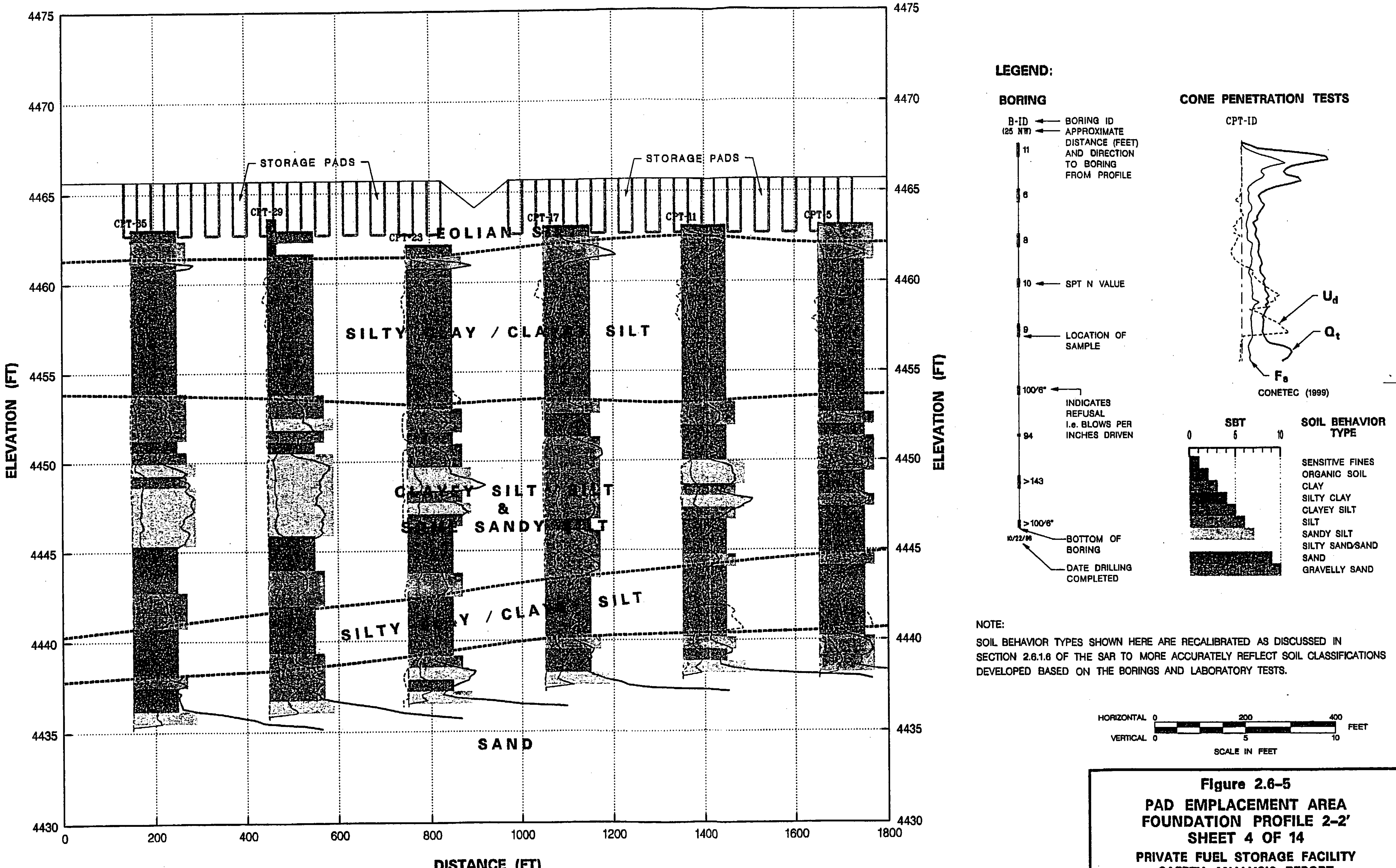


Figure 2.6-5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 1-1'

SHEET 3 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

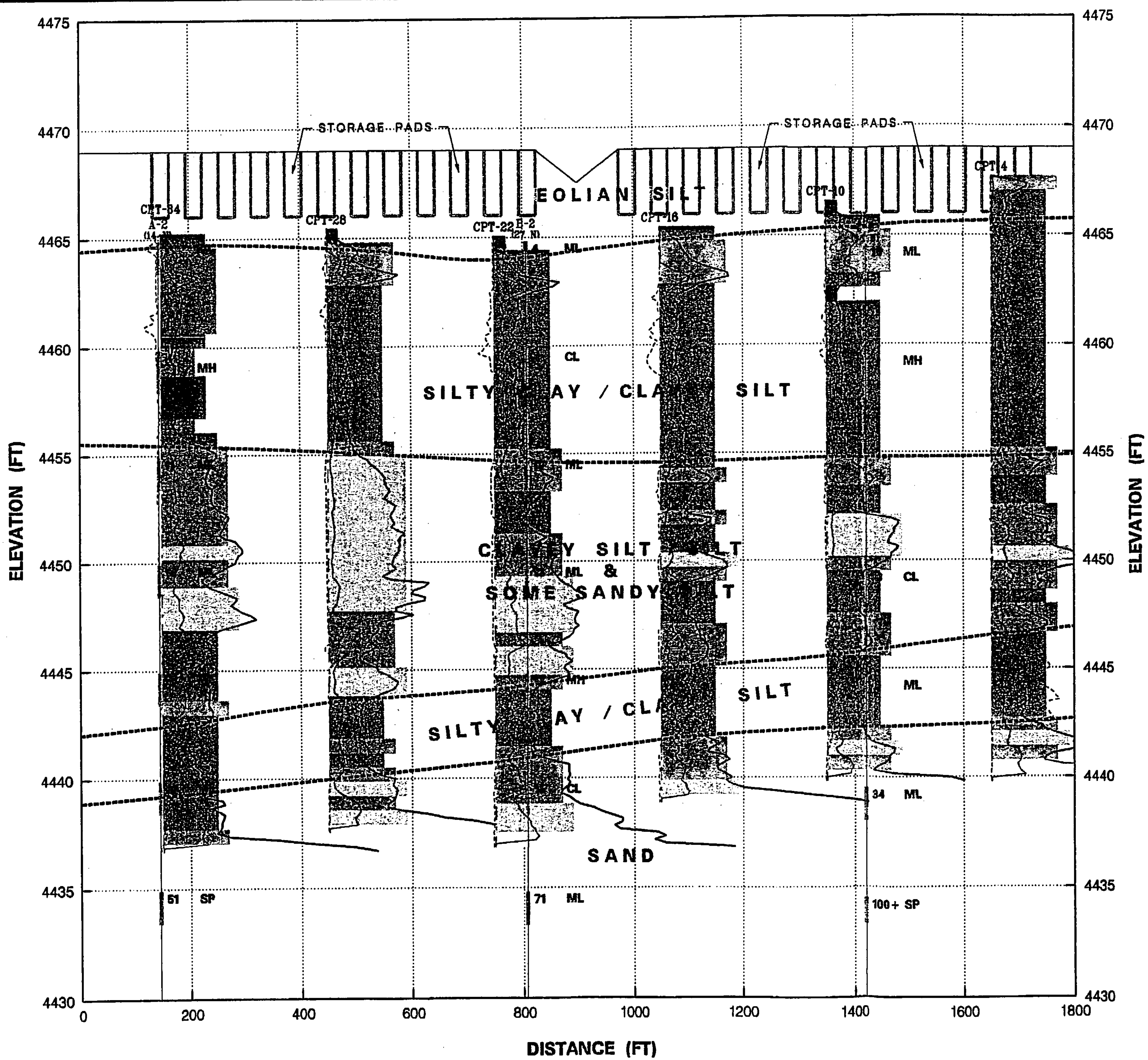


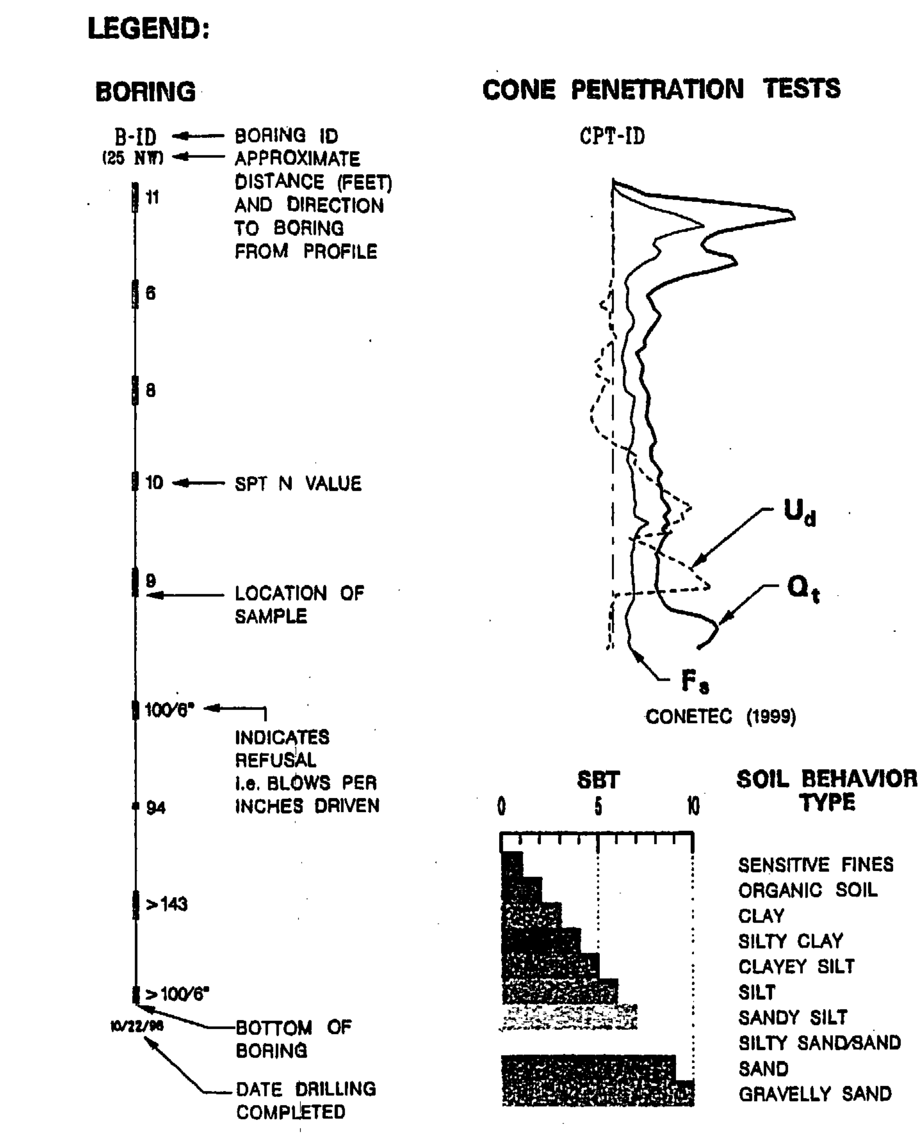
NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

Figure 2.6-5
PAD EMPLACEMENT AREA FOUNDATION PROFILE 2-2' SHEET 4 OF 14
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\b\4.dgn 22-MAR-2001 16:10





NOTE:

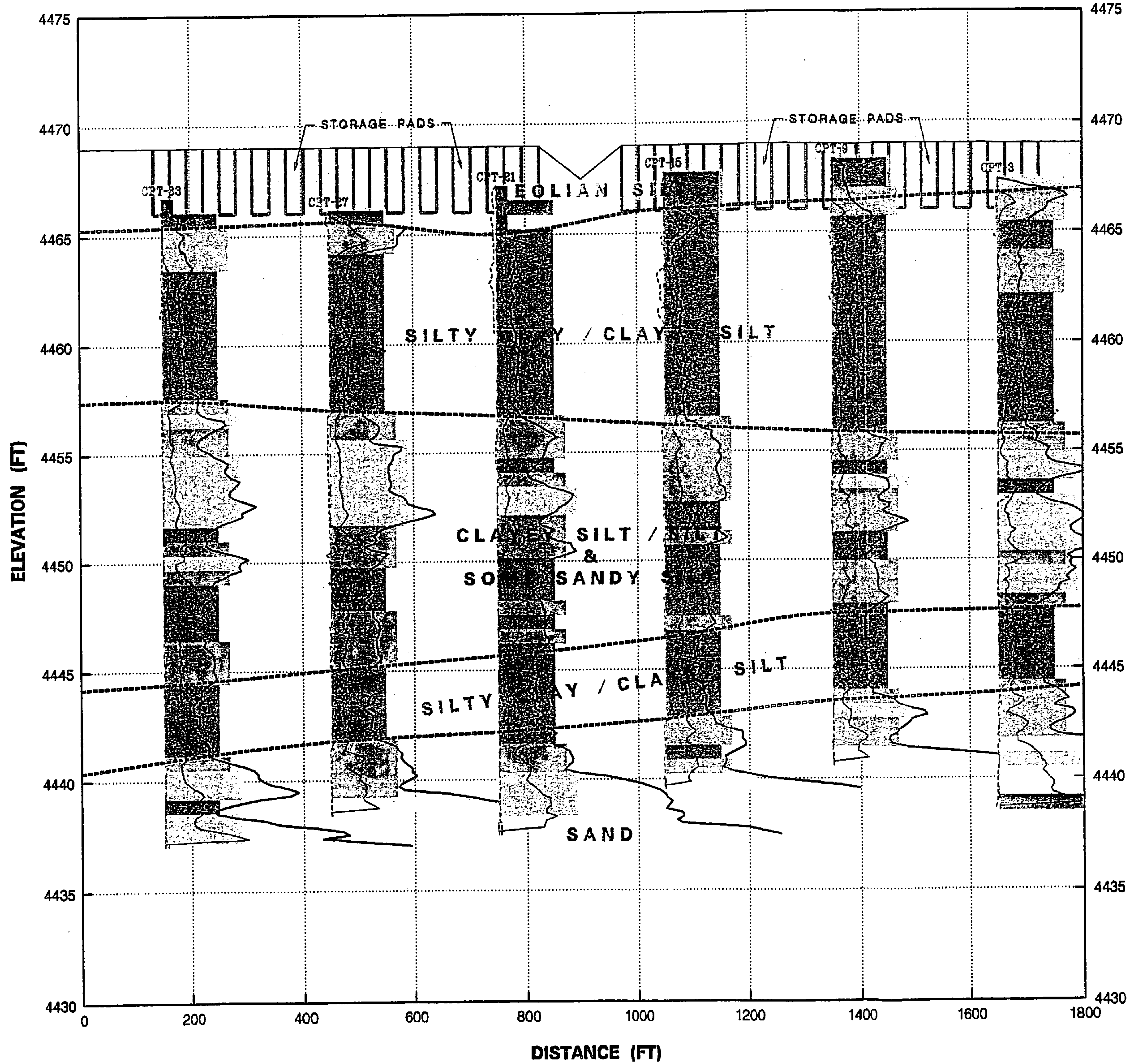
SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

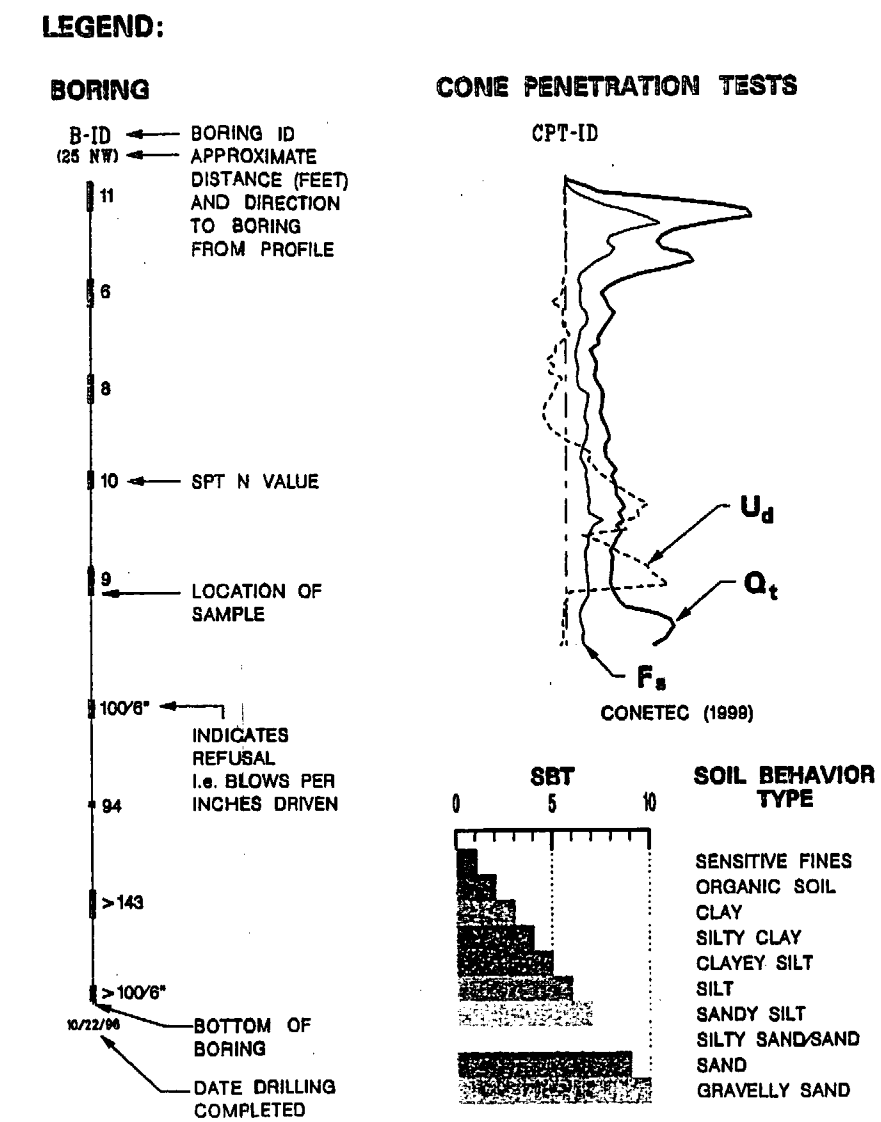
HORIZONTAL 0 200 400 FEET
VERTICAL 0 5 10
SCALE IN FEET

Figure 2.6-5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 3-3'
SHEET 5 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

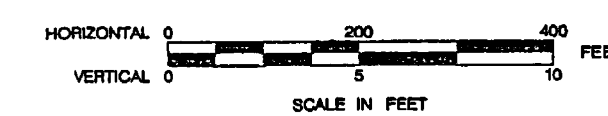
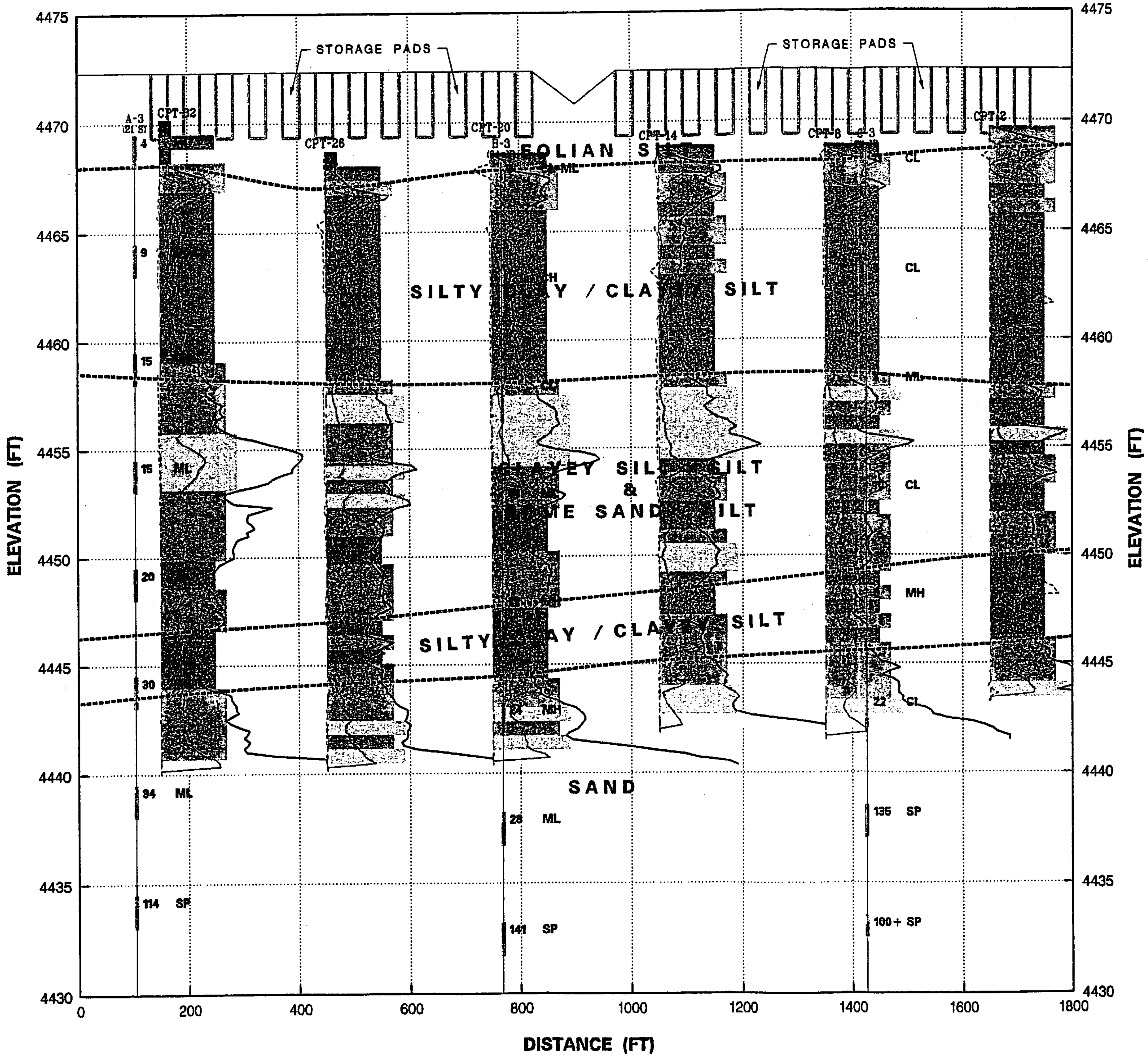


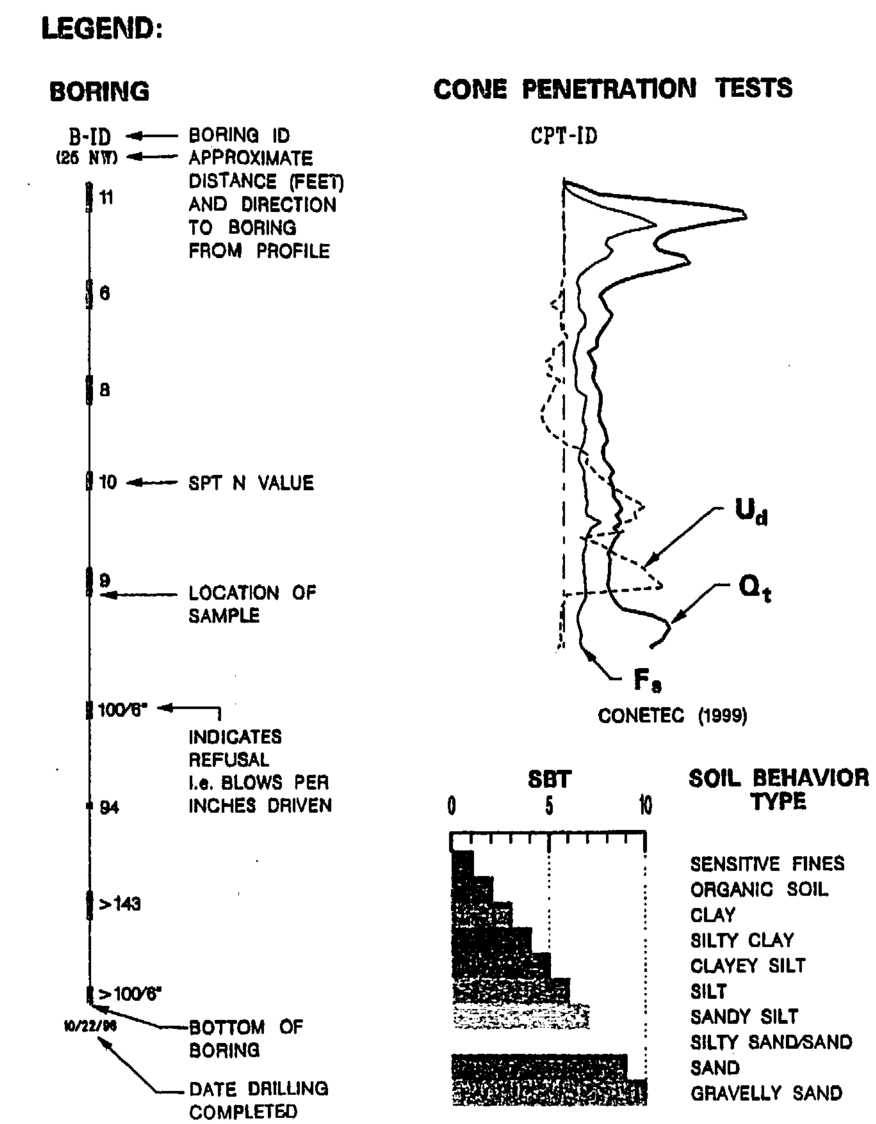
Figure 2.6–5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 4–4'
SHEET 6 OF 14

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

g:\05996\gsk\b\43~.dgn 22-MAR-2001 16:12





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

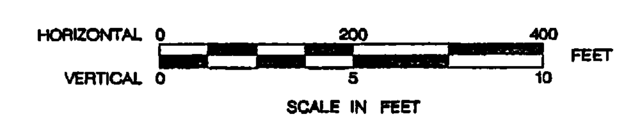
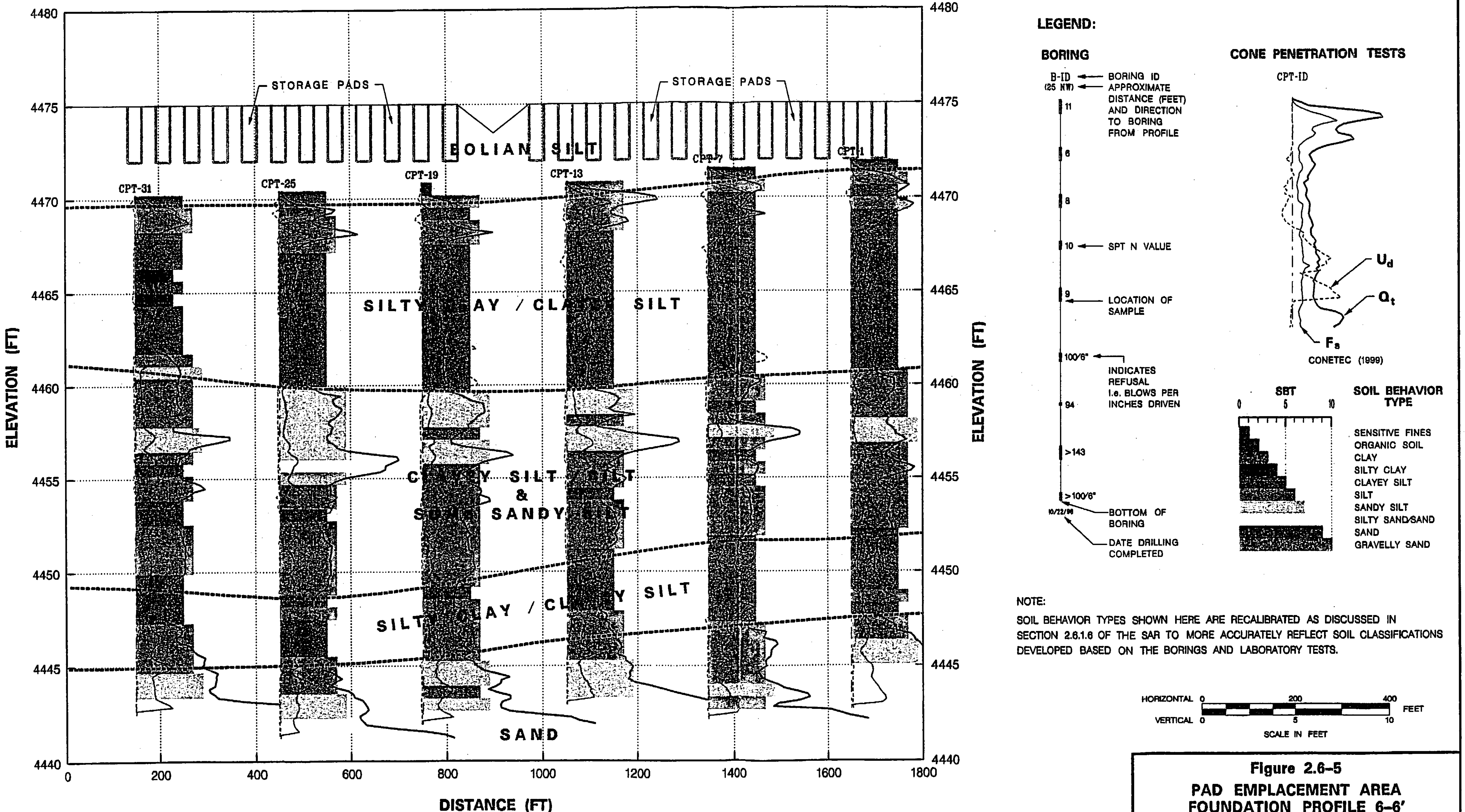


Figure 2.6–5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 5–5'
SHEET 7 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



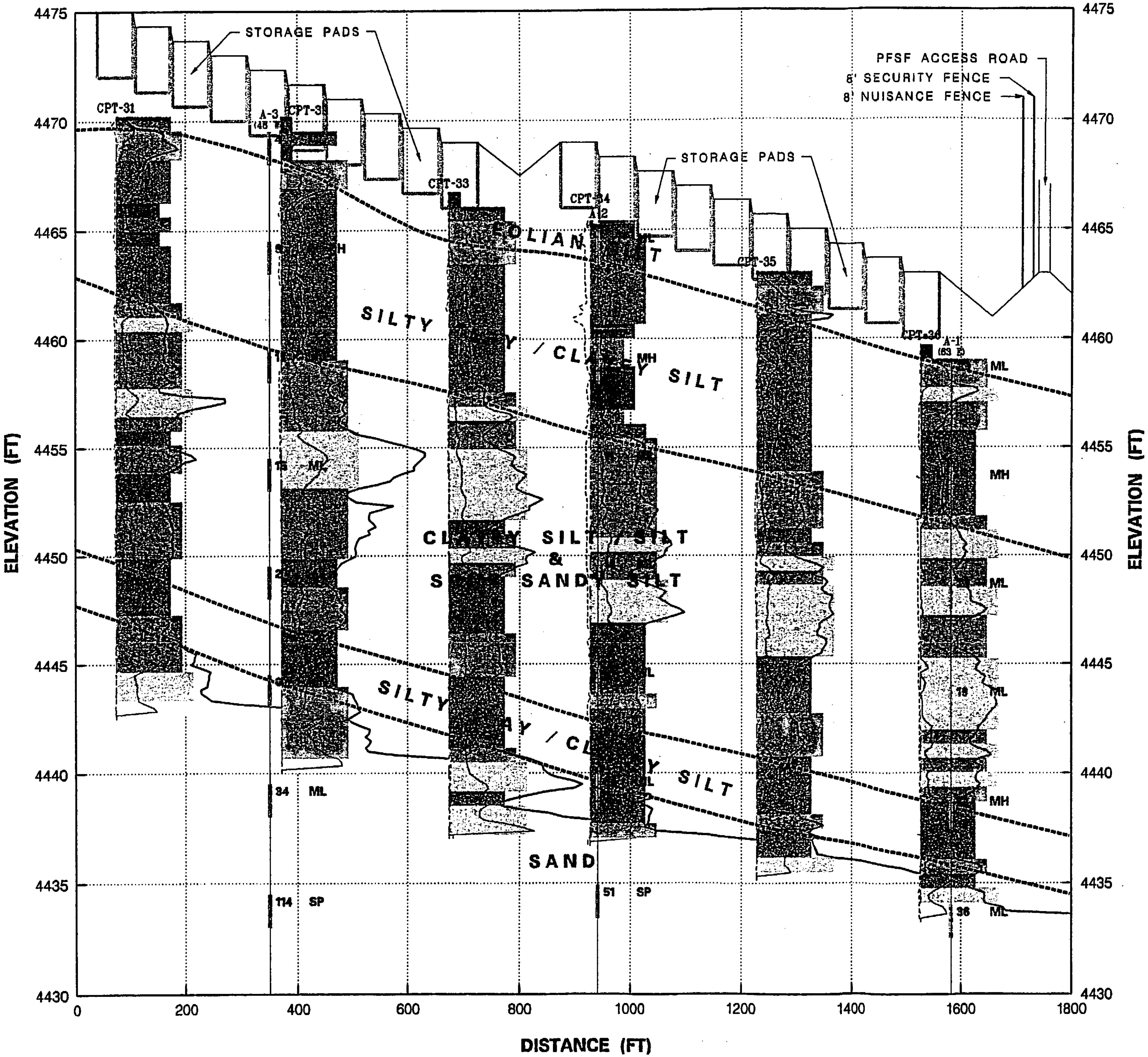
NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

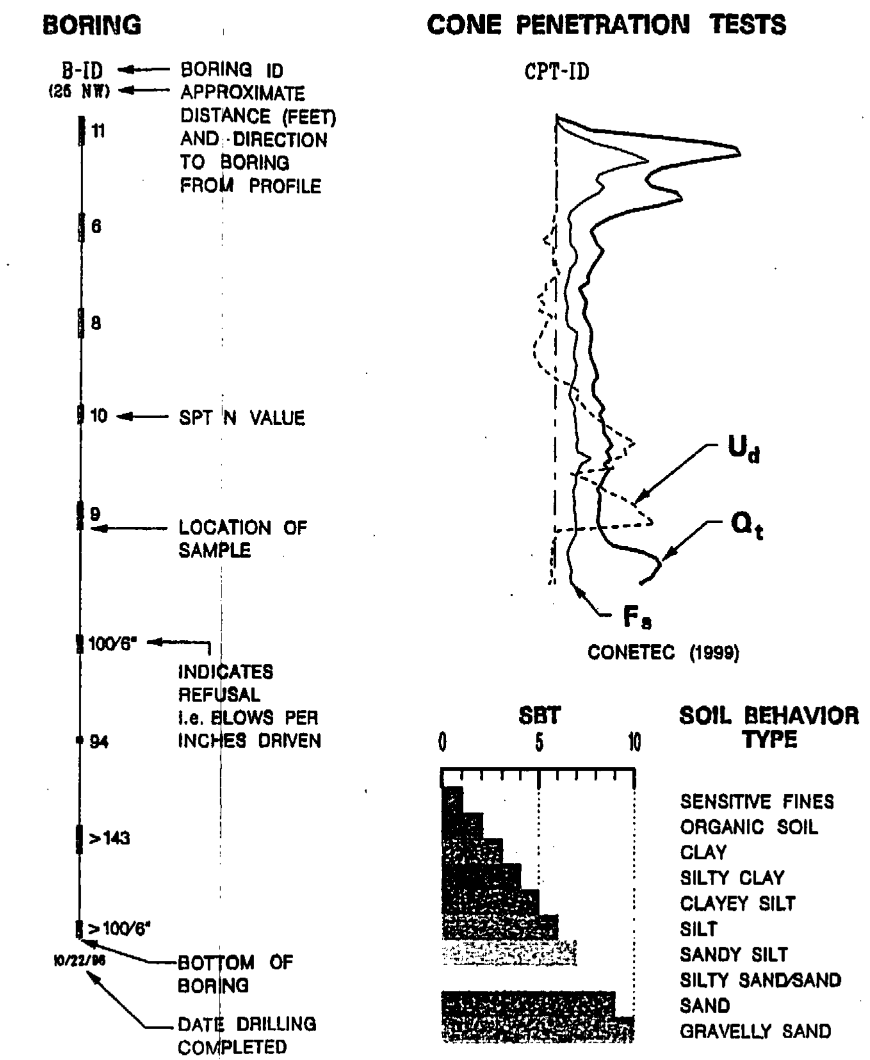
Figure 2.6-5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 6-6'
SHEET 8 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



LEGEND:



NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

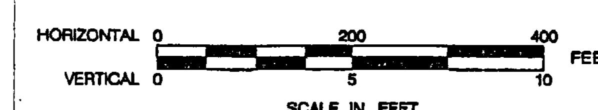


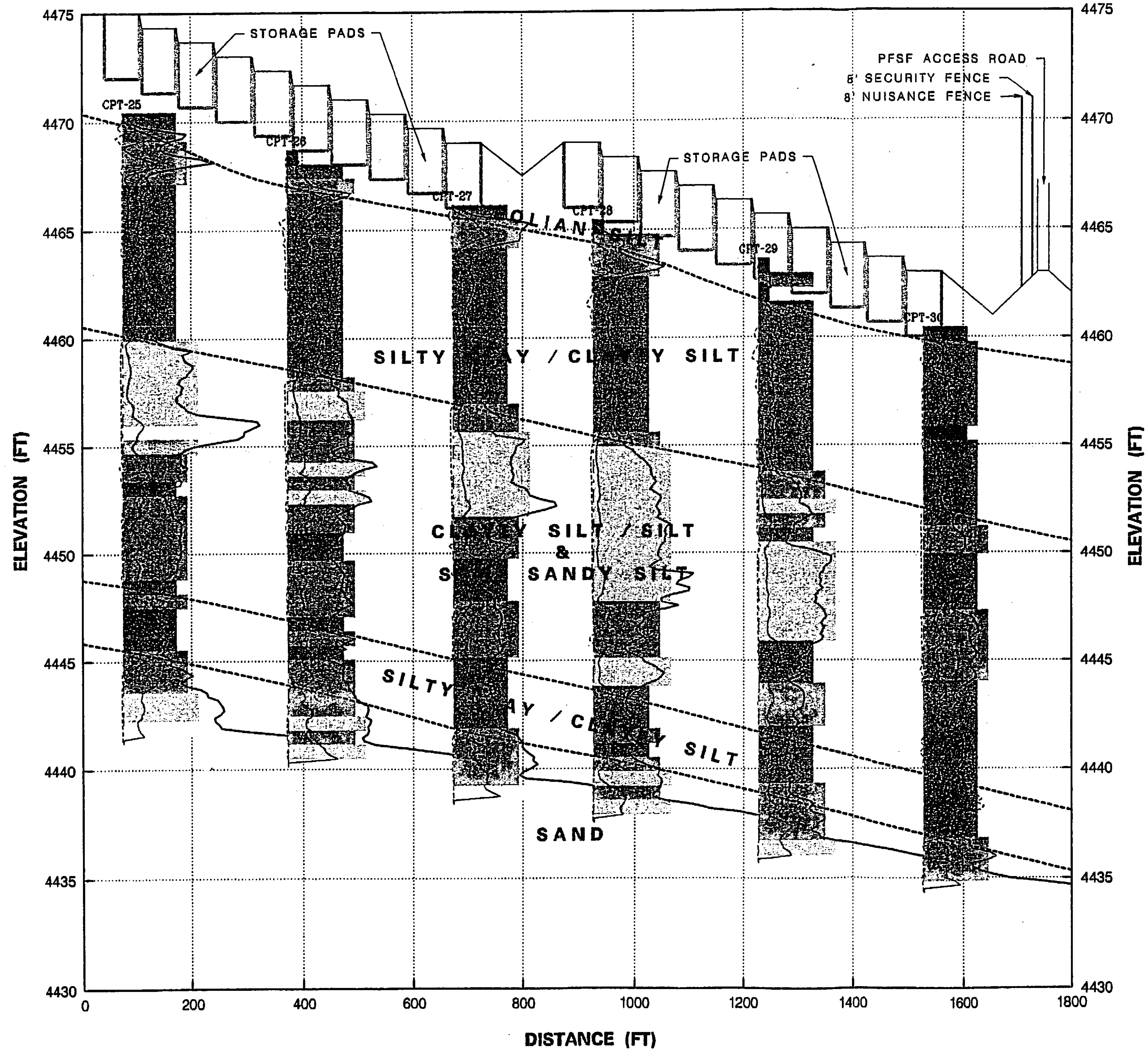
Figure 2.6–5

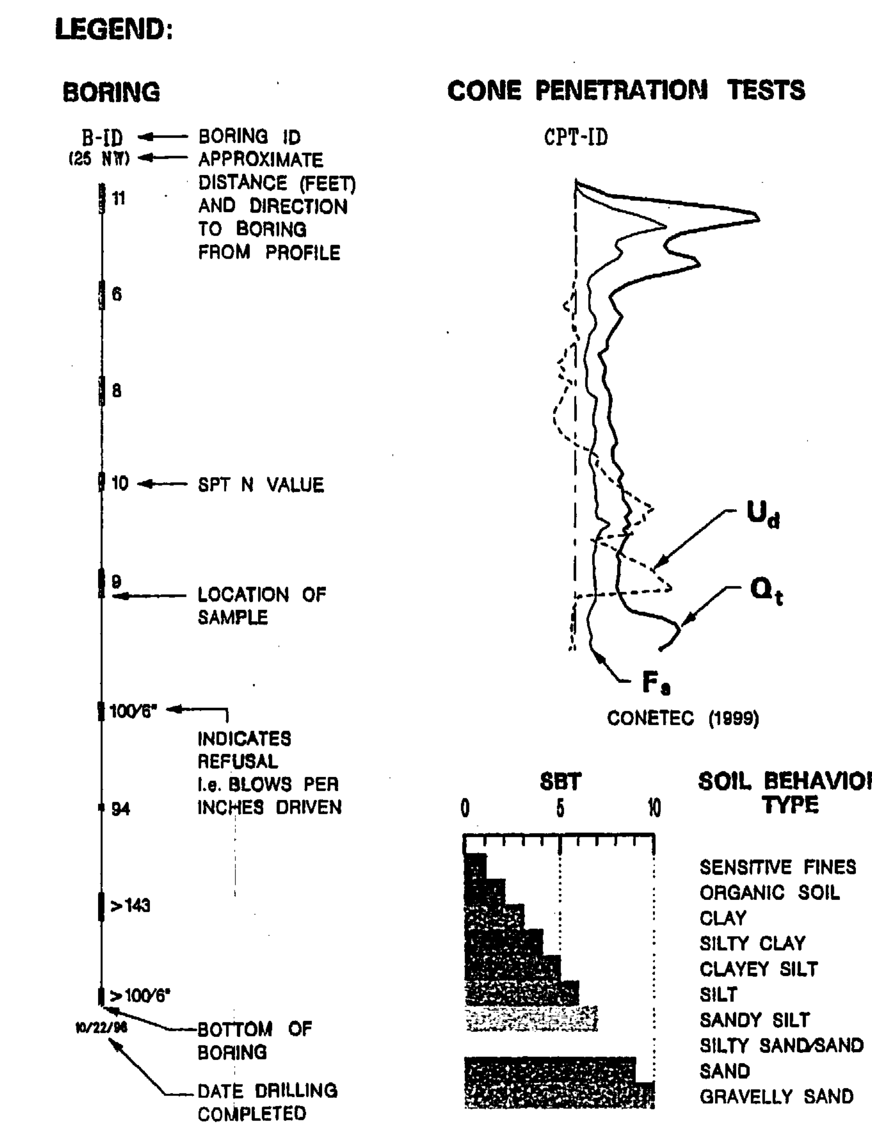
PAD EMPLACEMENT AREA FOUNDATION PROFILE 7–7'

SHEET 9 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\b\4...dgn 22-MAR-2001 16:15





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

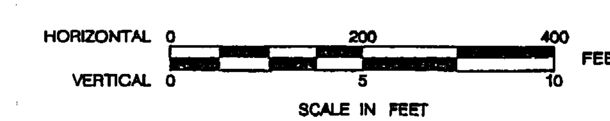
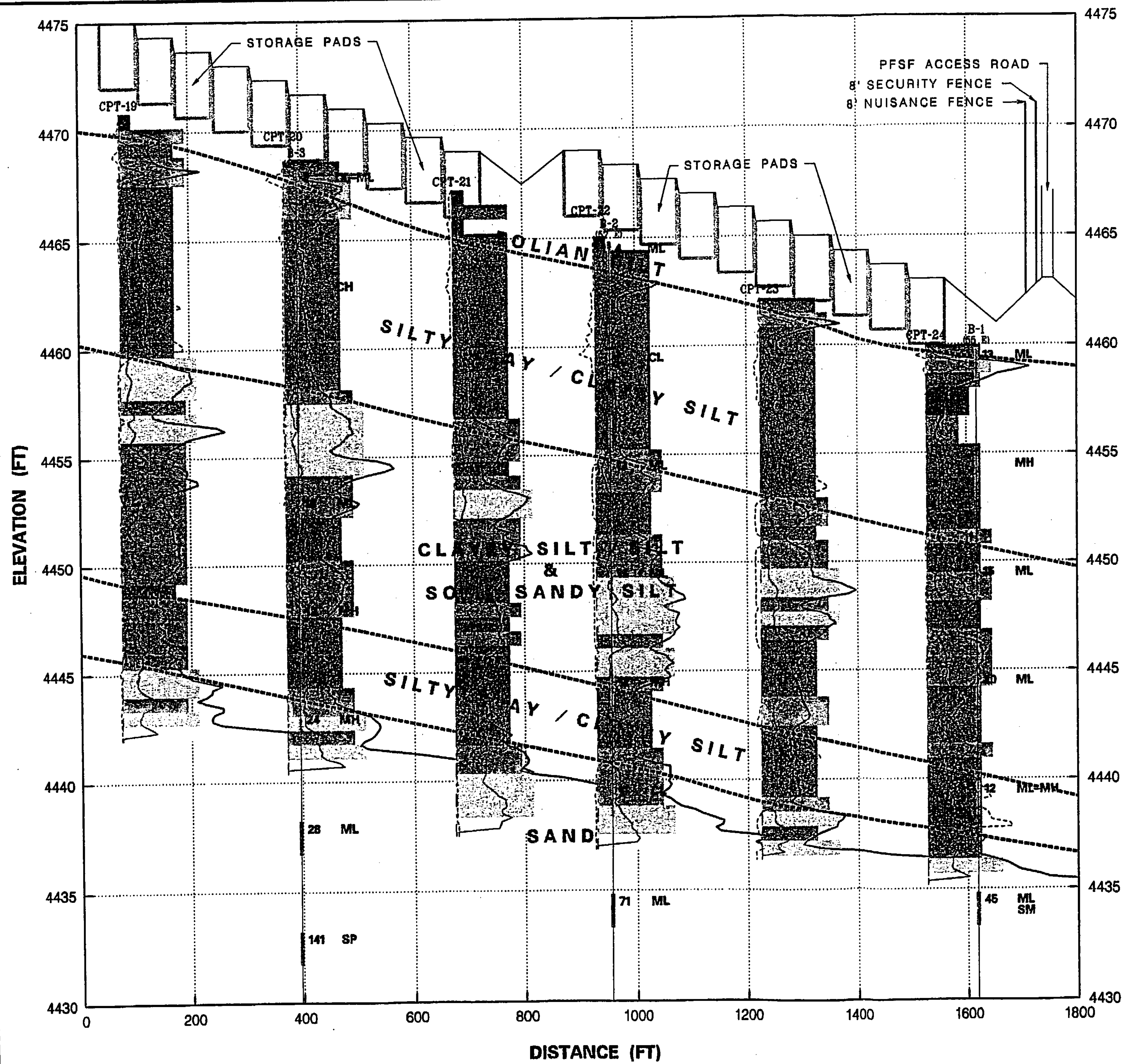


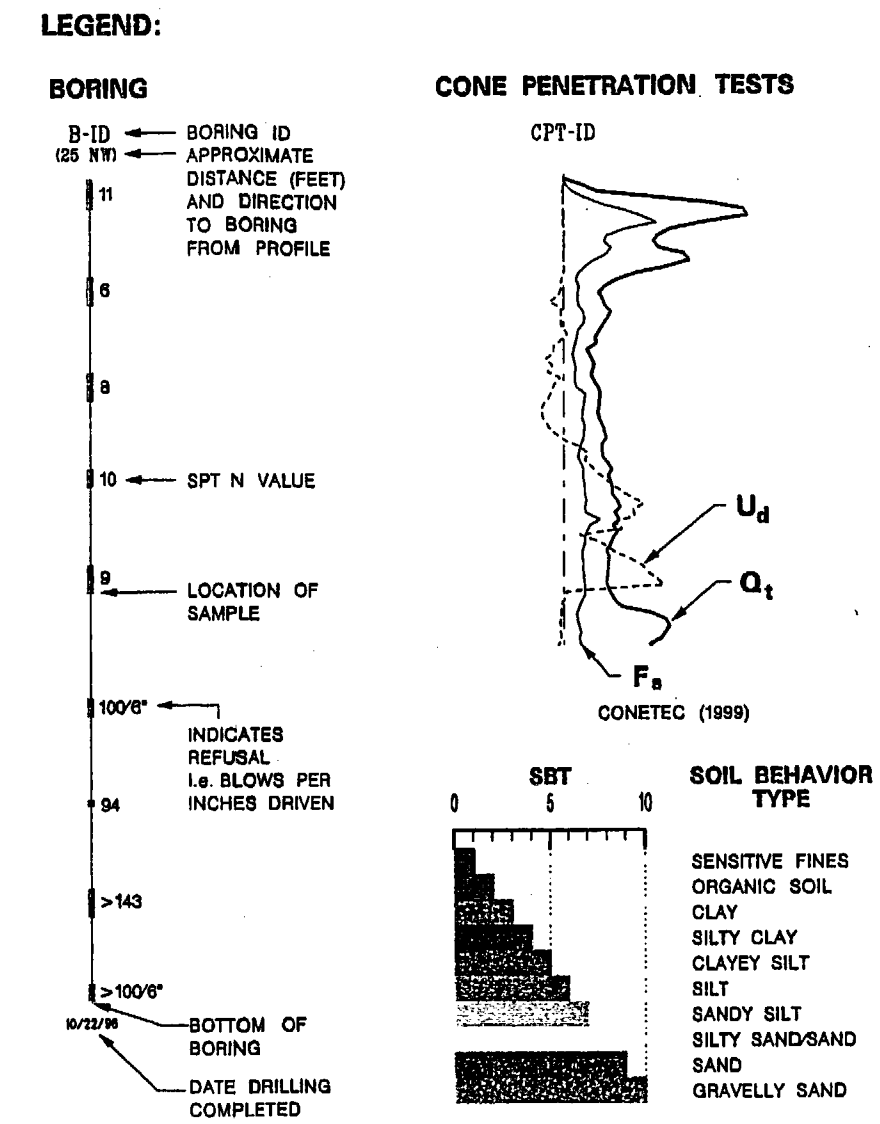
Figure 2.6-5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 8-8' SHEET 10 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\b\46.dgn 22-MAR-2001 16:16





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

HORIZONTAL 0 200 400 FEET
VERTICAL 0 5 10
SCALE IN FEET

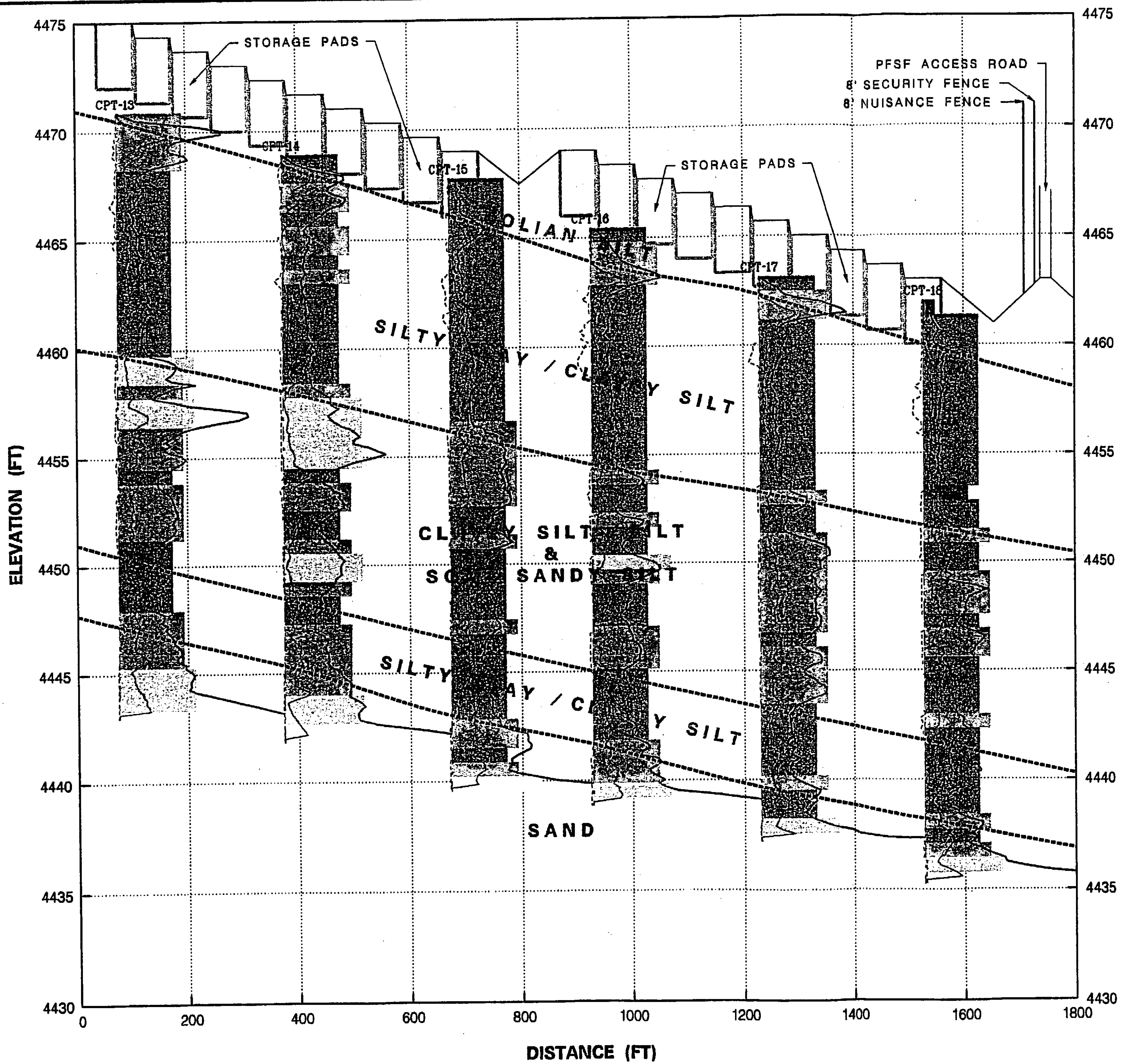
Figure 2.6-5

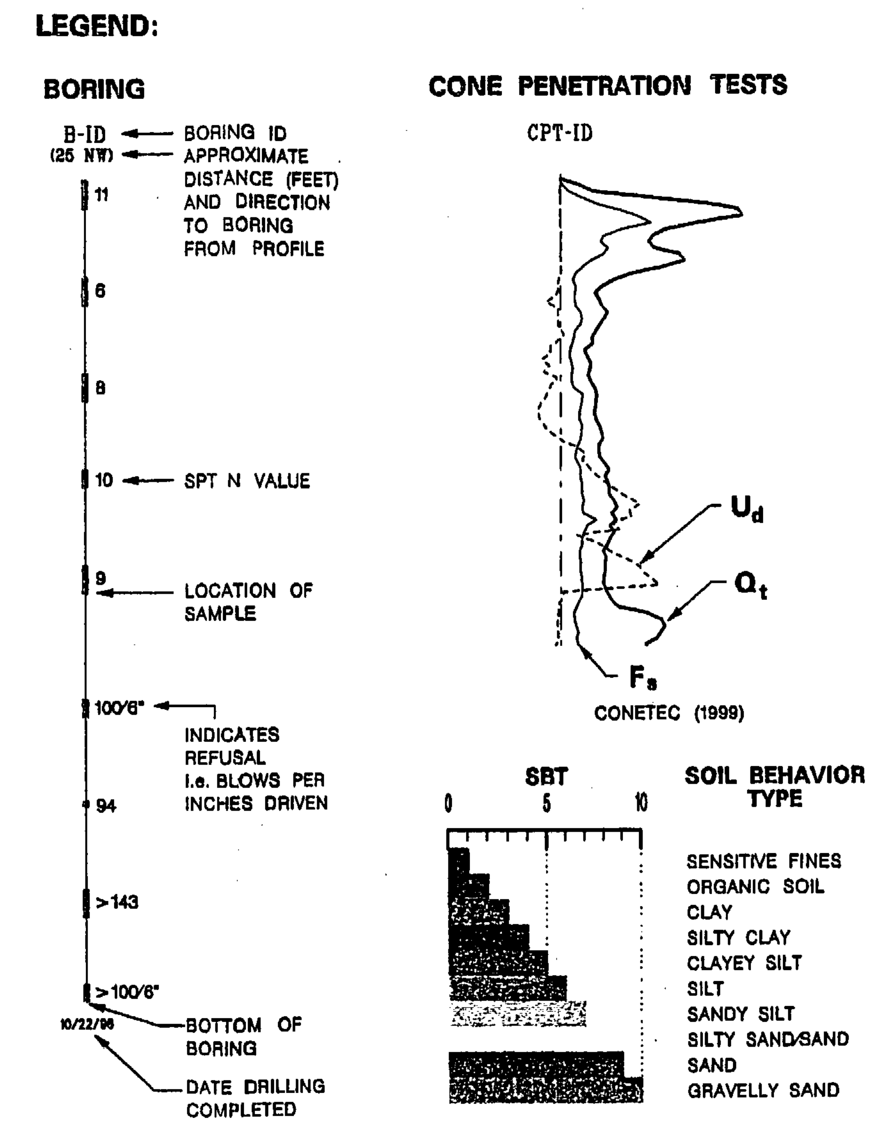
PAD EMPLACEMENT AREA FOUNDATION PROFILE 9-9'

SHEET 11 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\b\4\...dgn 22-MAR-2001 16:18





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

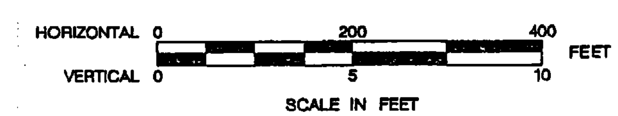


Figure 2.6–5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 10–10'

SHEET 12 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\b\4bve.dgn 22-MAR-2001 16:19

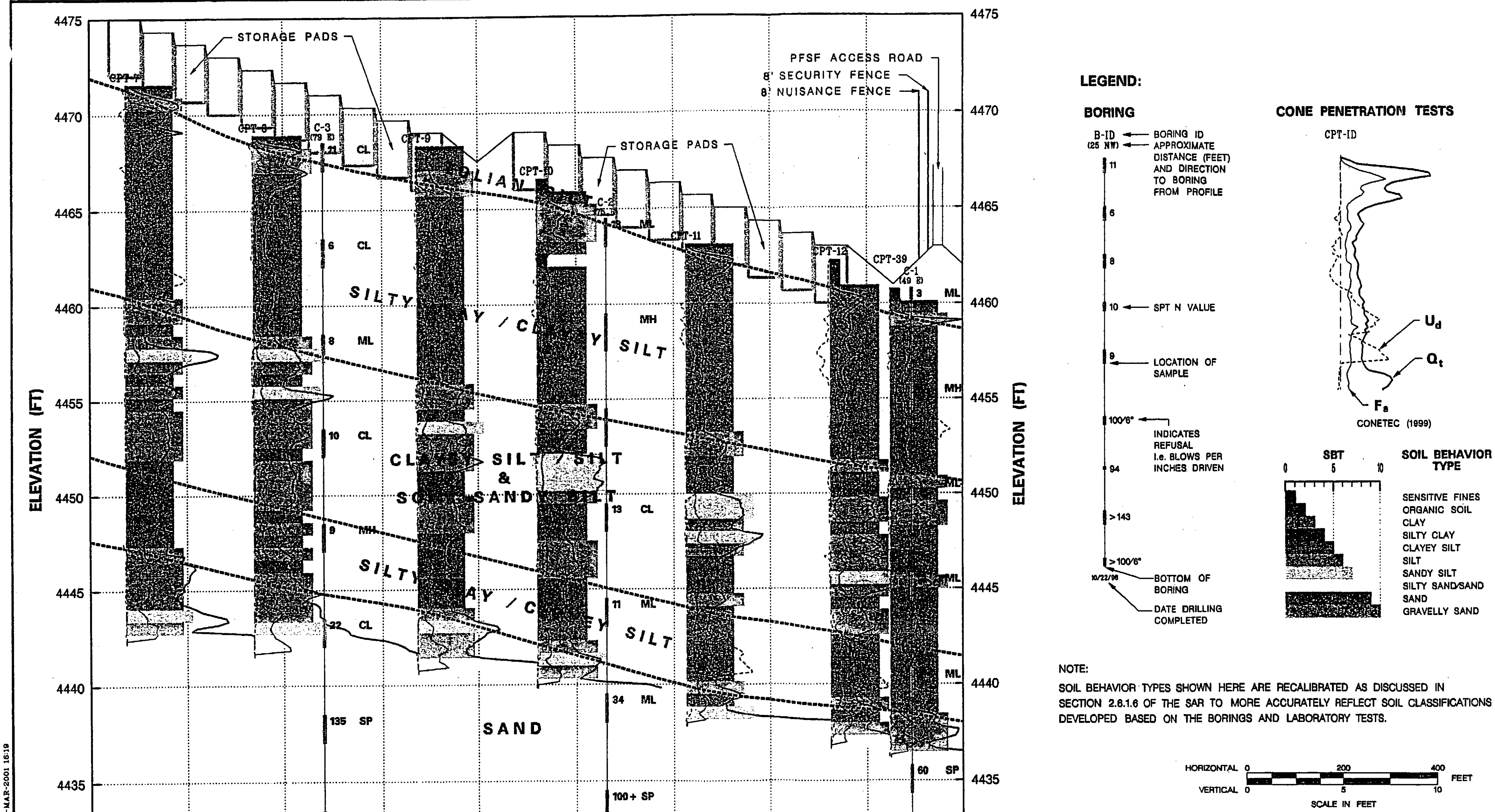
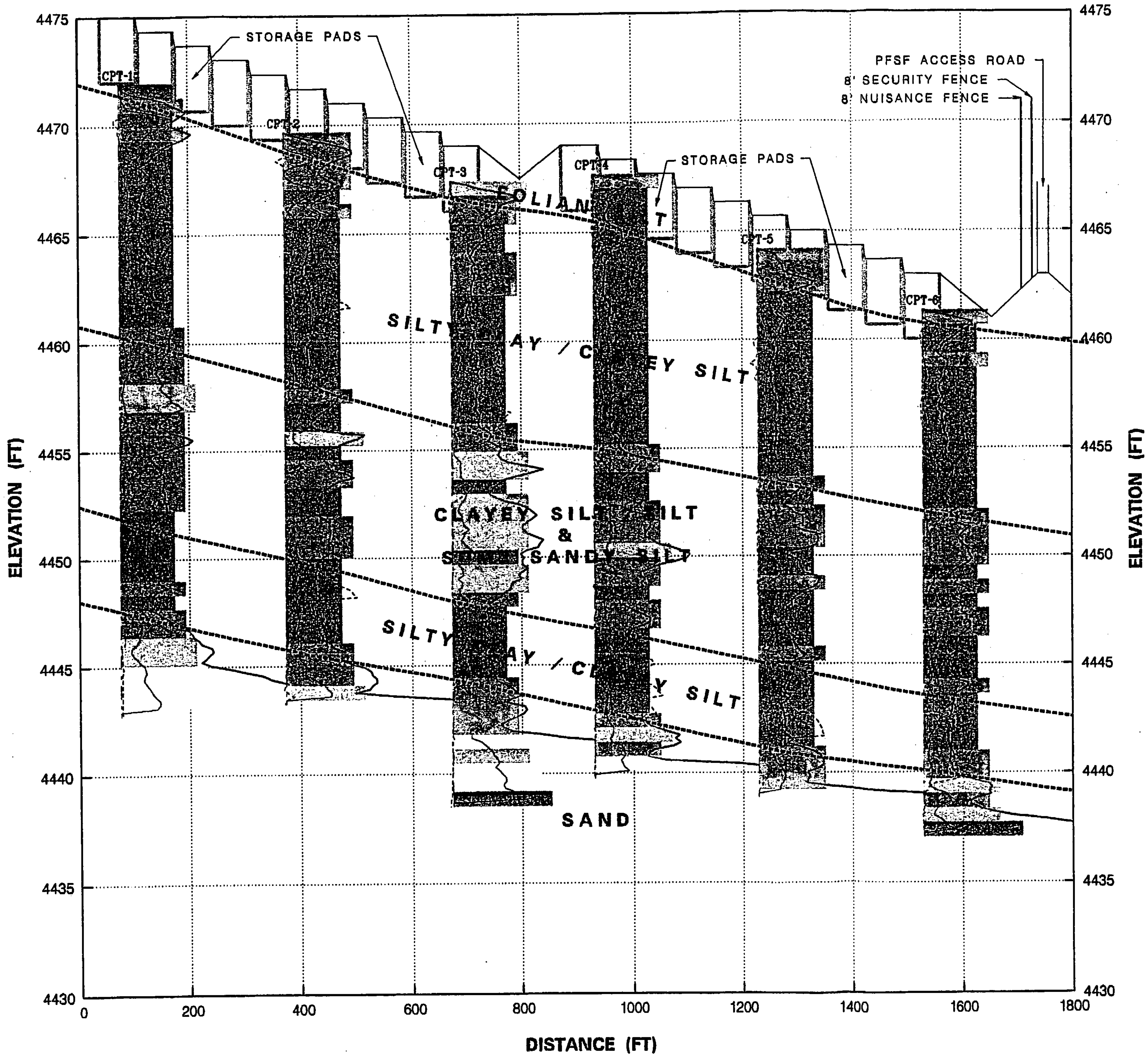
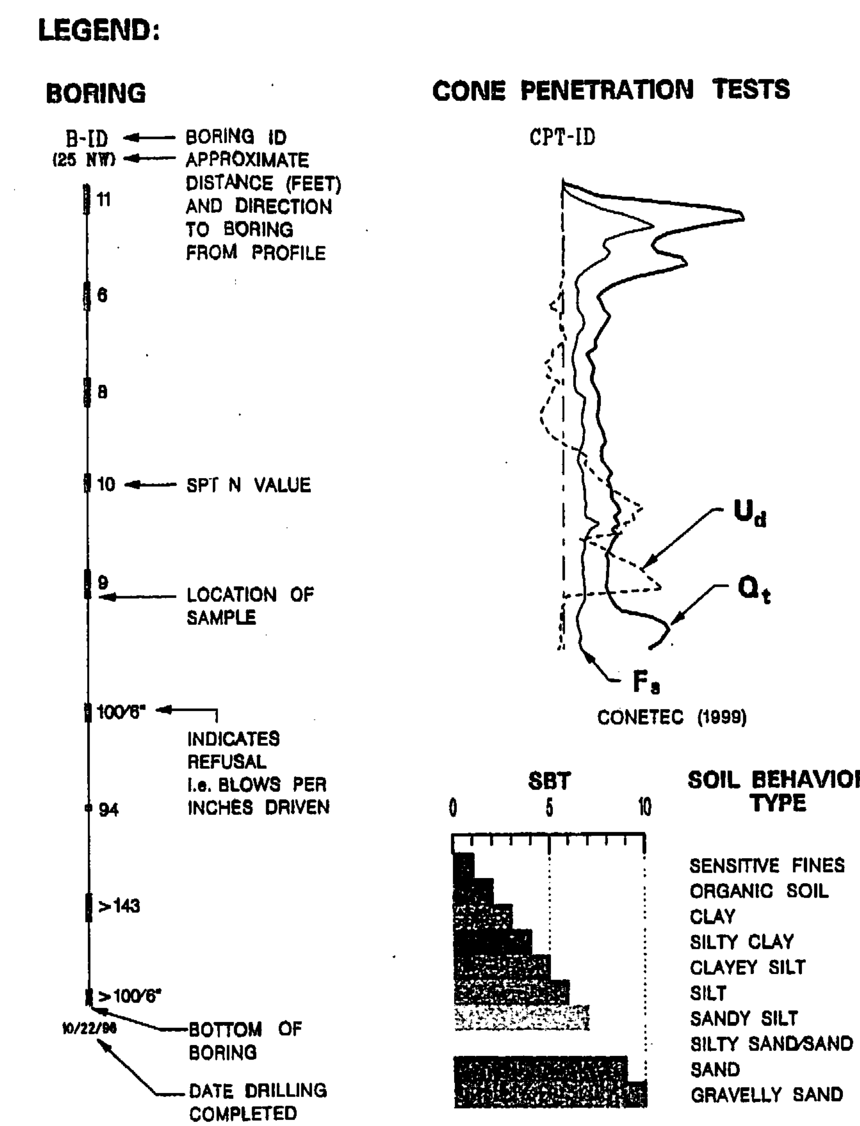


Figure 2.6-5
PAD EMPLACEMENT AREA FOUNDATION PROFILE 11-11'
SHEET 13 OF 14
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT





NOTE:

SOIL BEHAVIOR TYPES SHOWN HERE ARE RECALIBRATED AS DISCUSSED IN SECTION 2.6.1.6 OF THE SAR TO MORE ACCURATELY REFLECT SOIL CLASSIFICATIONS DEVELOPED BASED ON THE BORINGS AND LABORATORY TESTS.

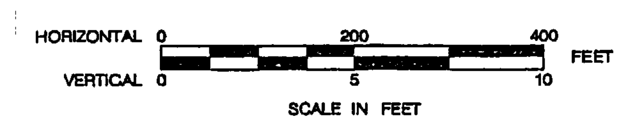


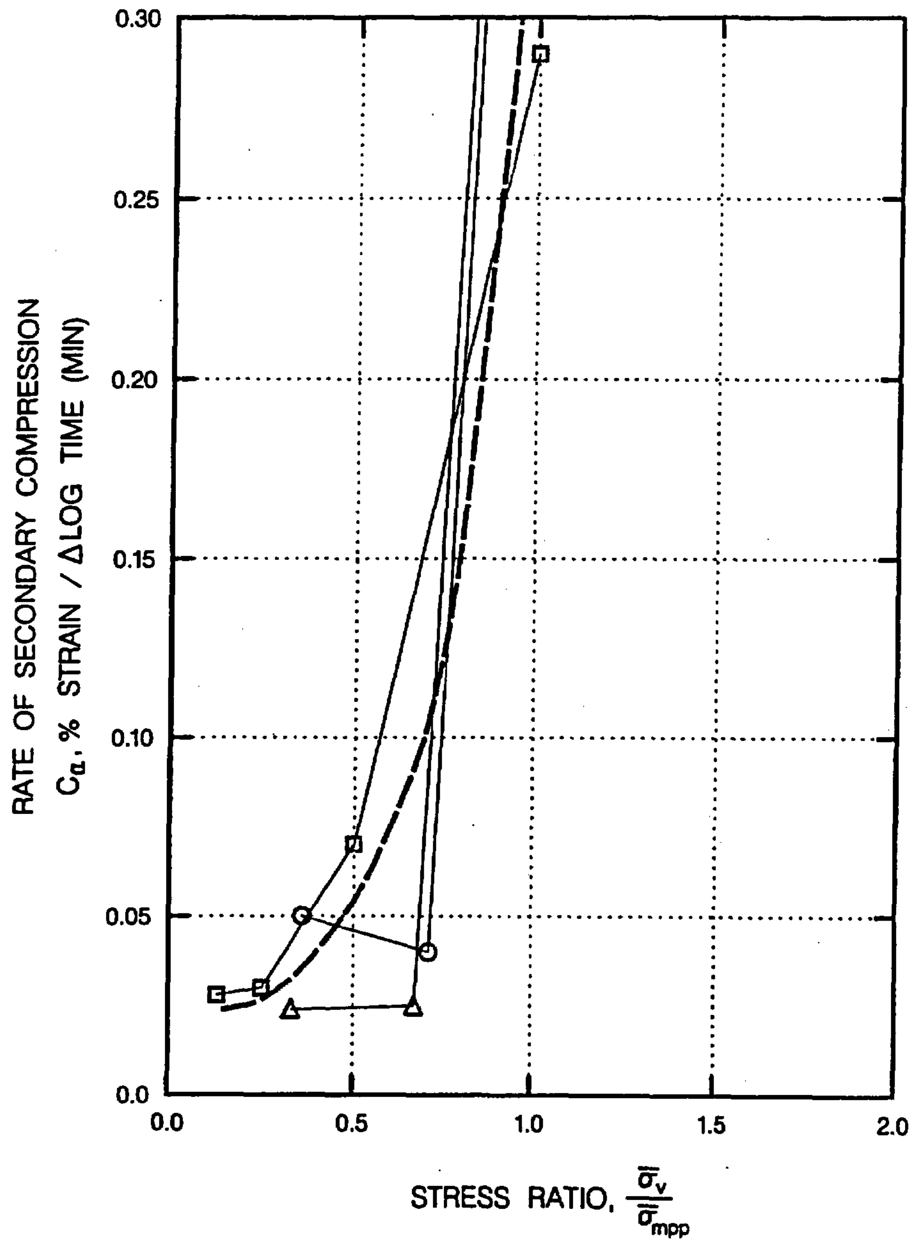
Figure 2.6-5

PAD EMPLACEMENT AREA FOUNDATION PROFILE 12-12'

SHEET 14 OF 14

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

22-MAR-2001 16:00



KEY

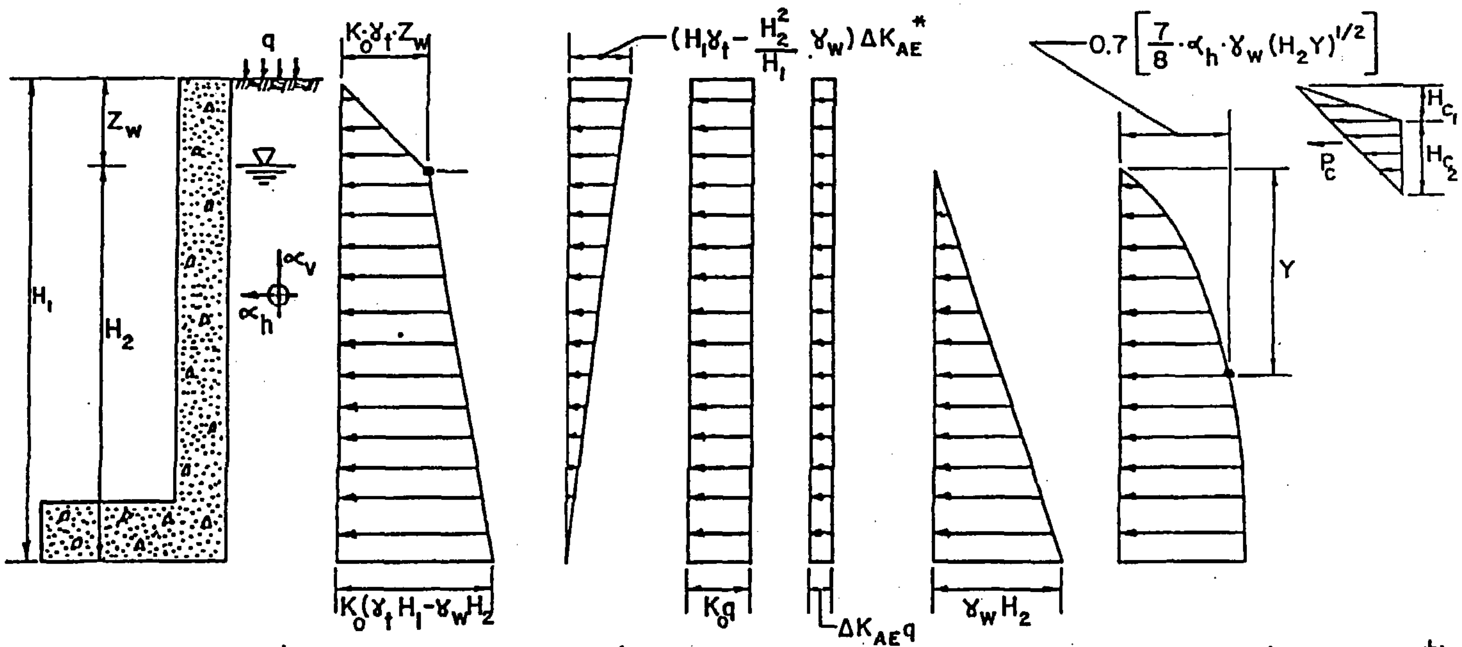
SYMBOL	TEST ID
O	C1-U3C
□	C1-U3D
Δ	C2-U2C

BASED ON RELOADING PORTIONS OF CONSOLIDATION TESTS

Figure 2.6-6

RATE OF SECONDARY COMPRESSION VS STRESS RATIO

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



LOAD/LIN. FT. OF WALL	SOIL	SURCHARGE**	WATER	COMPACTION+
STATIC	$(H_1^2\gamma_t - H_2^2\gamma_w)\frac{K_o}{2}$	$K_o q H_1$	$\frac{1}{2}H_2^2\gamma_w$	$\frac{1}{2}\sigma_c H_{c_2}$
DYNAMIC	$(H_1^2\gamma_t - H_2^2\gamma_w)\frac{1}{2}\Delta K_{AE}$*	$\Delta K_{AE} q H_1$	$0.7\left[\frac{7}{12}\alpha_h\gamma_w H_2^2\right]$	—
COMBINED	$\frac{1}{2}(H_1^2\gamma_t - H_2^2\gamma_w)(K_o+\Delta K_{AE})$	$q(K_o+\Delta K_{AE})H_1$	$\frac{\gamma_w H_2}{2}(1+0.82\alpha_h)$	$\frac{1}{2}\sigma_c H_{c_2}$

*APPLICABLE FOR $H_2 \leq H_1$; IF $H_2 > H_1$, H_2 SHOULD BE TAKEN TO BE EQUAL TO H_1 SINCE STANDING WATER DOES NOT EFFECT THE MAGNITUDE OF EFFECTIVE STRESS.

**FOR UNIFORM SURCHARGE ONLY.

Figure 2.6-7

STATIC AND DYNAMIC LATERAL EARTH PRESSURES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

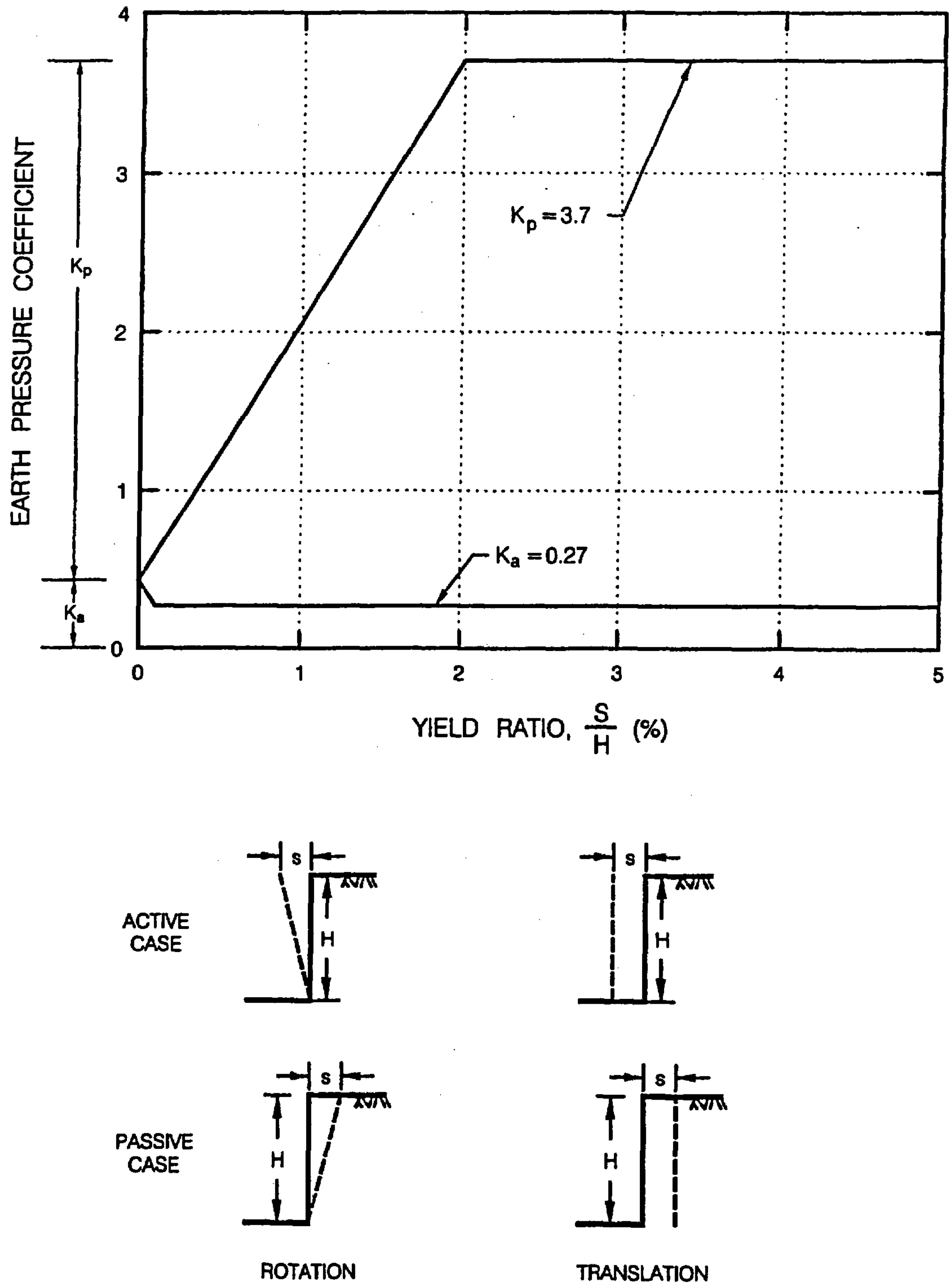


Figure 2.6-8

LATERAL EARTH PRESSURE COEFFICIENTS VS WALL MOVEMENT

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

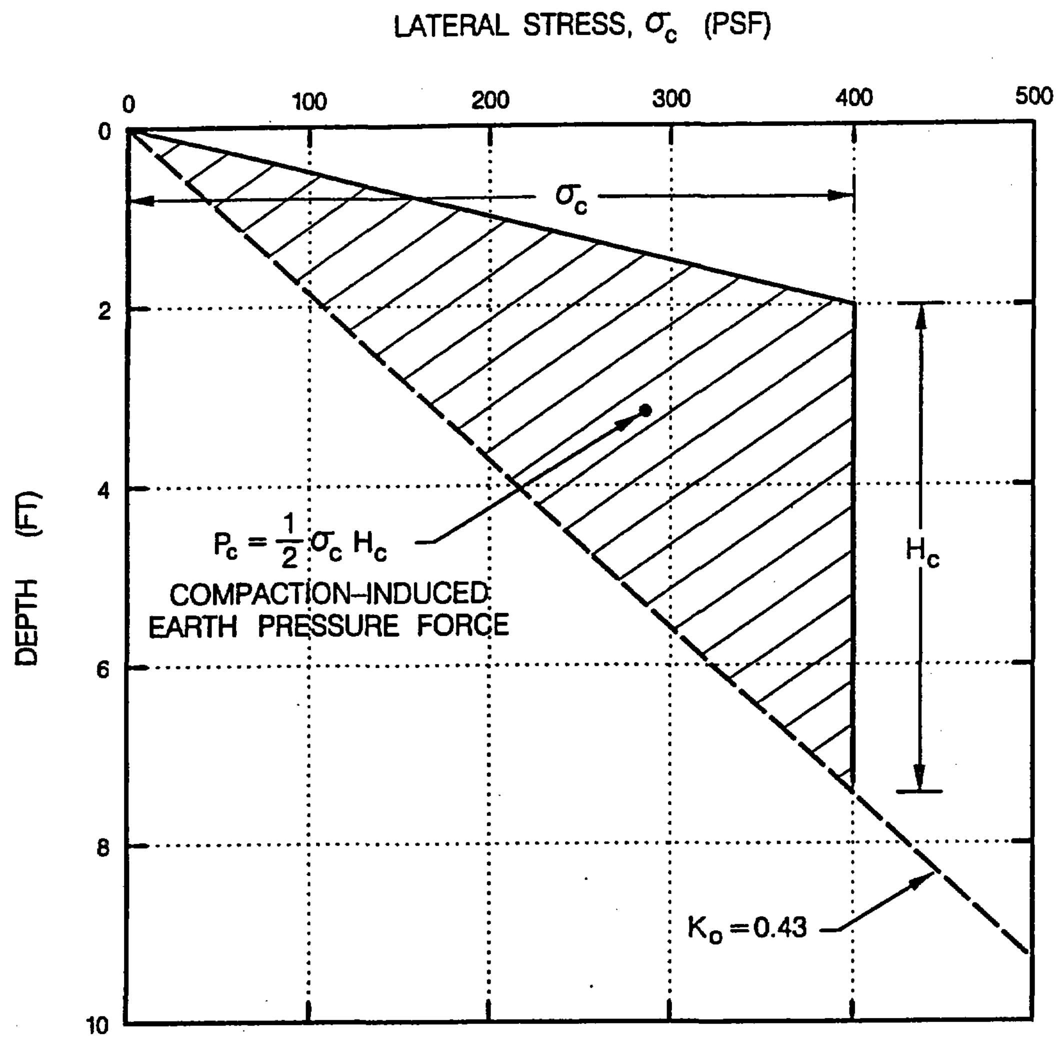
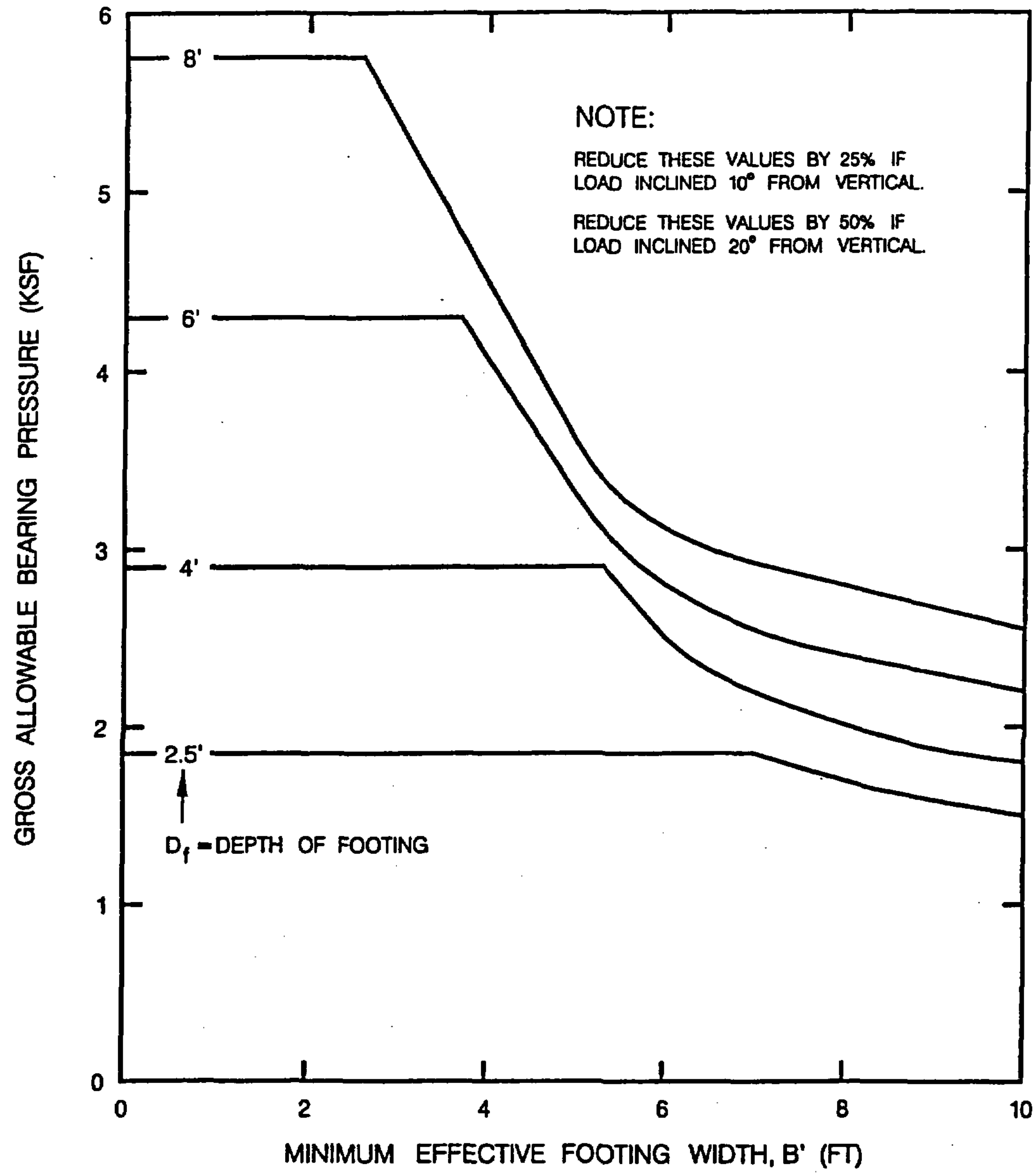
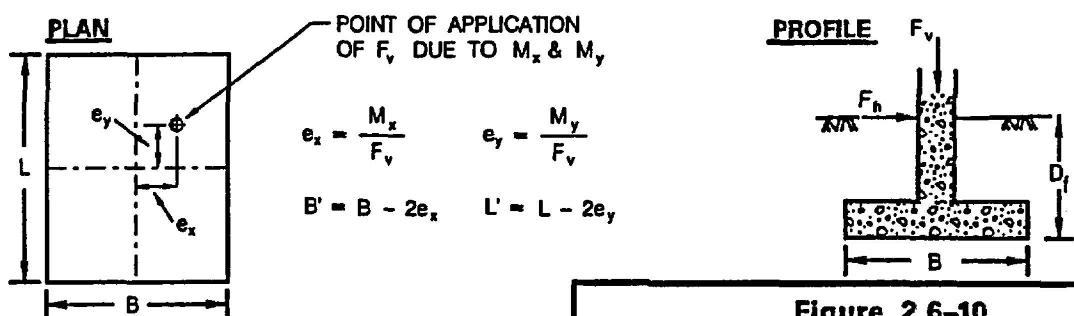


Figure 2.6-9

COMPACTION-INDUCED LATERAL STRESSES

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



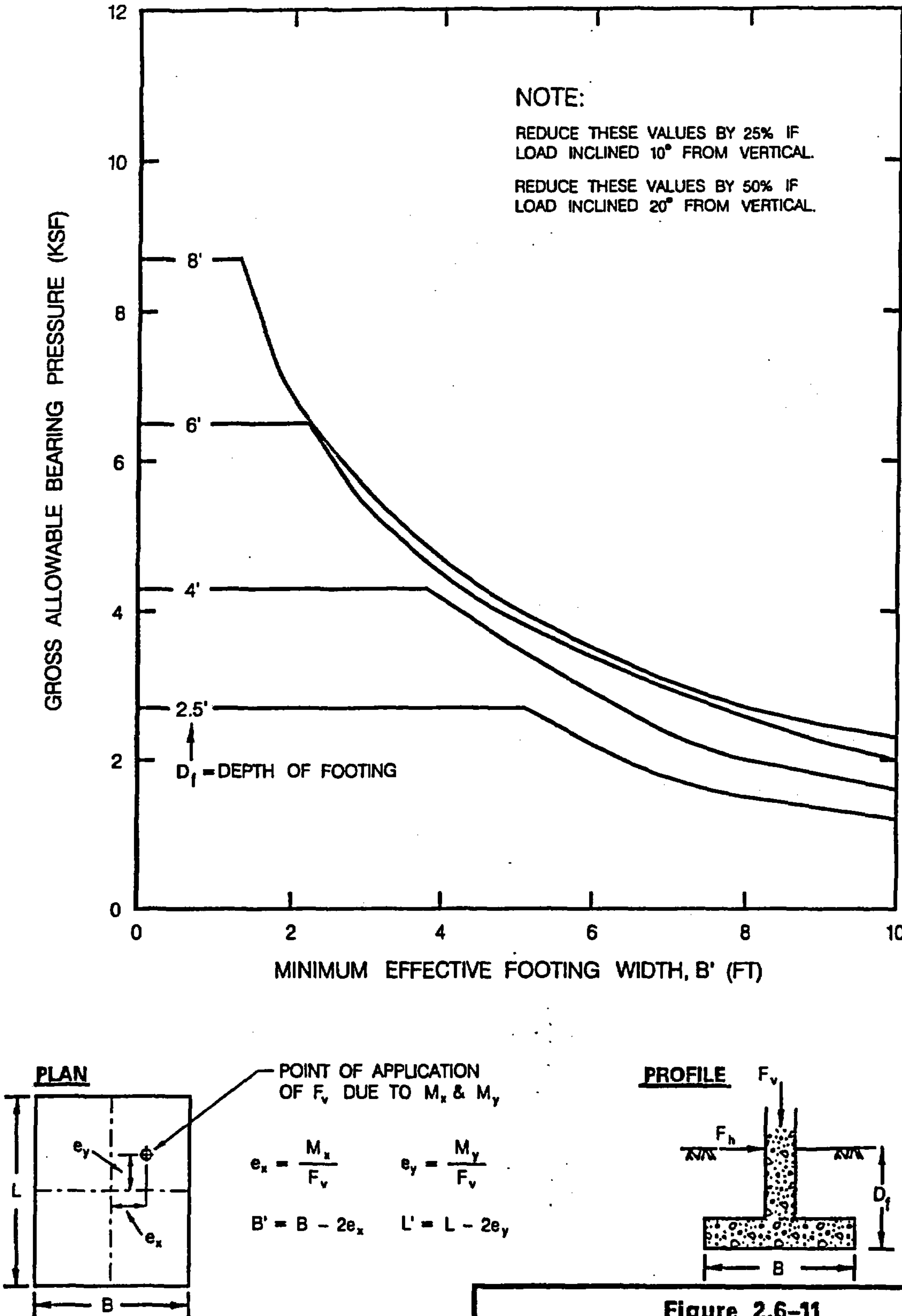


BASED ON MAXIMUM ALLOWABLE SETTLEMENT = 2" @ 40 YRS USING STRAIN-COMPATIBLE MODULI RECOMMENDED BY GEOMATRIX (1997)

Figure 2.6-10

GROSS ALLOWABLE BEARING PRESSURE VS FOOTING WIDTH & DEPTH FOR STRIP FOOTINGS

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



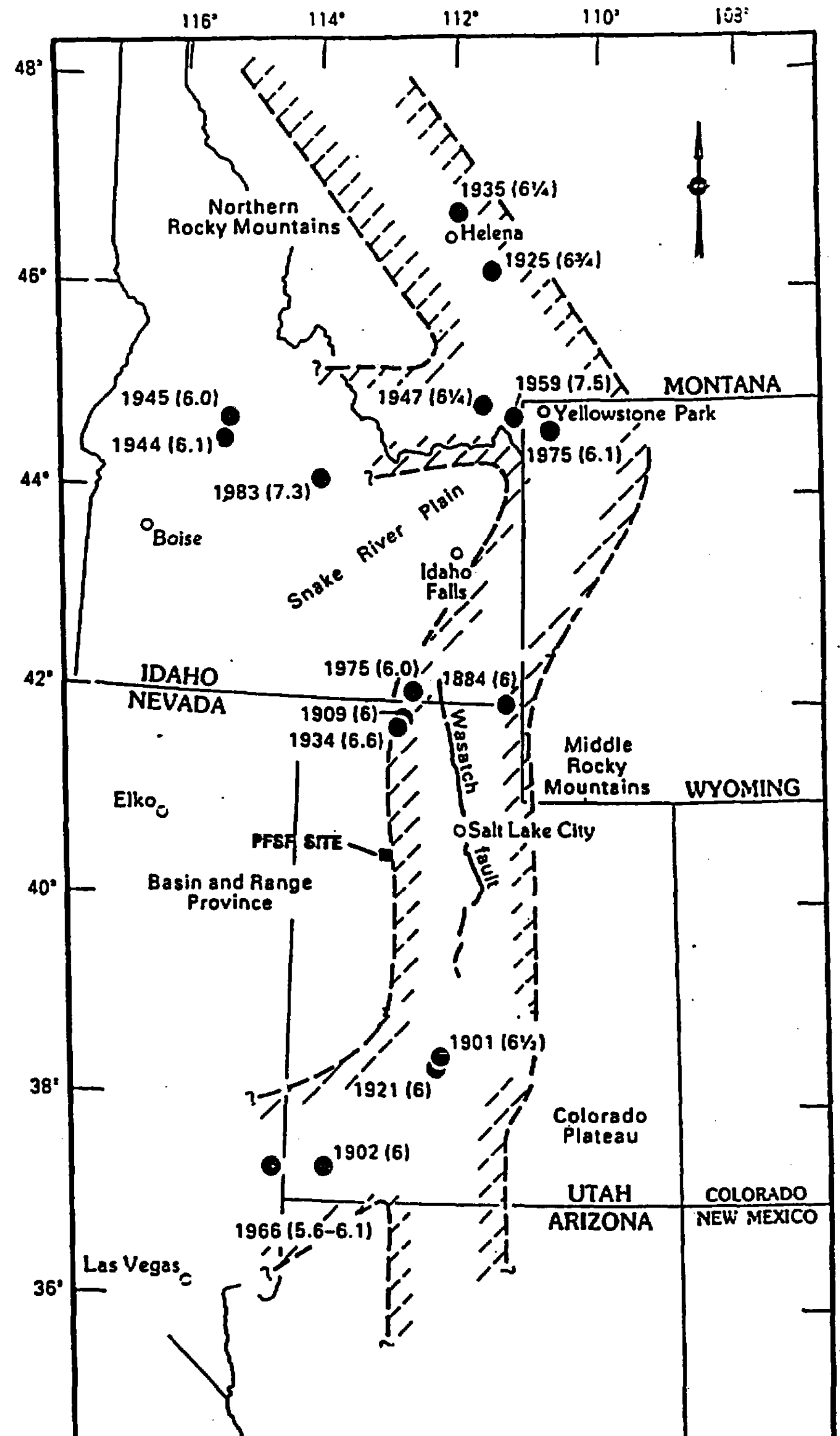
BASED ON MAXIMUM ALLOWABLE SETTLEMENT=1.5" @ 40 YRS USING STRAIN-COMPATIBLE MODULI RECOMMENDED BY GEOMATRIX (1997)

Figure 2.6-11

GROSS ALLOWABLE BEARING PRESSURE VS FOOTING WIDTH & DEPTH FOR SQUARE FOOTINGS

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

Revision 0



SOURCE: ARABASZ ET AL., 1992

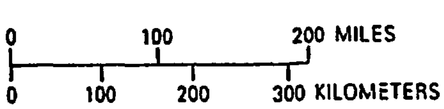
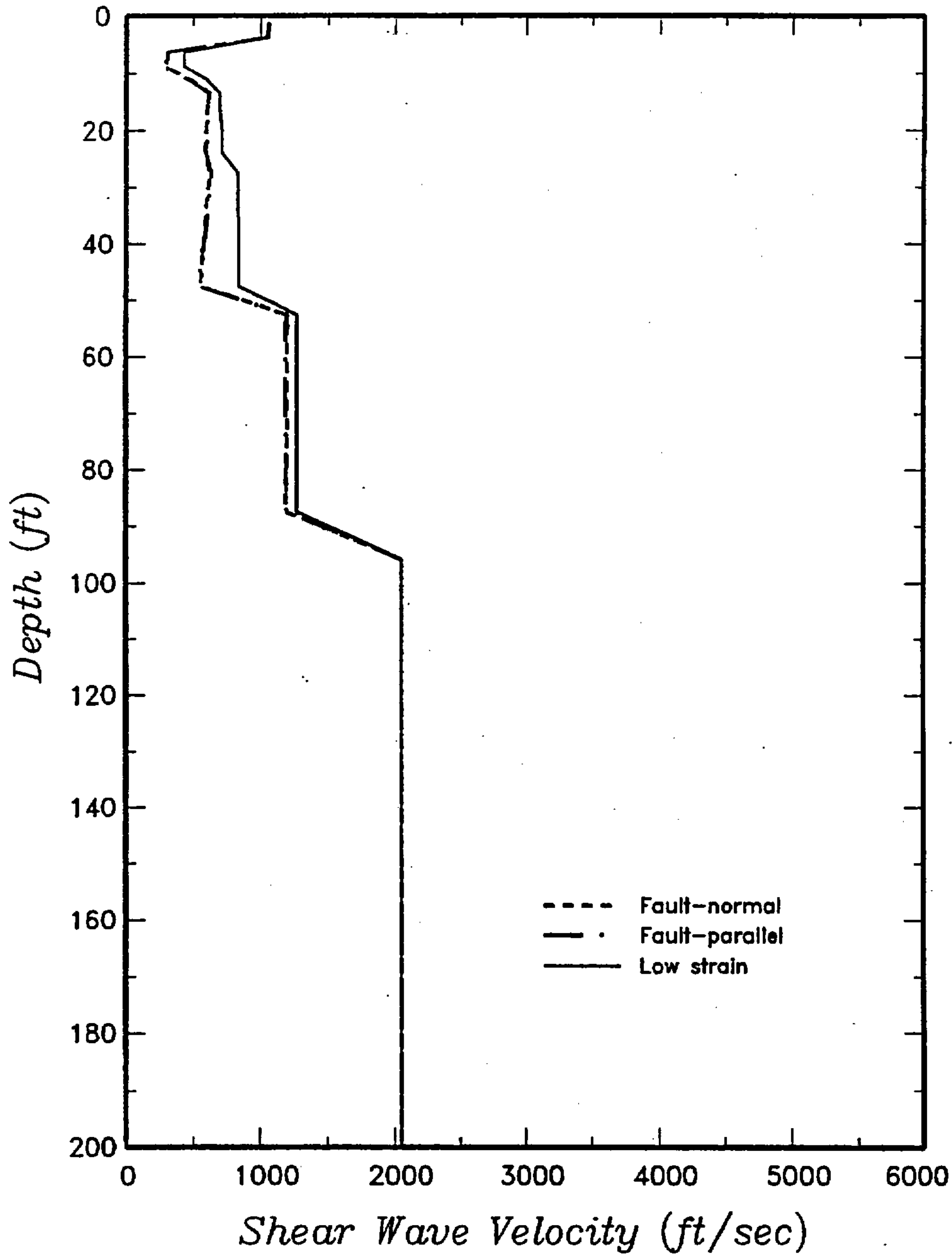


Figure 2.6–12

INTERMOUNTAIN SEISMIC BELT HISTORICAL EARTHQUAKES, MAGNITUDE ≥ 6.0

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

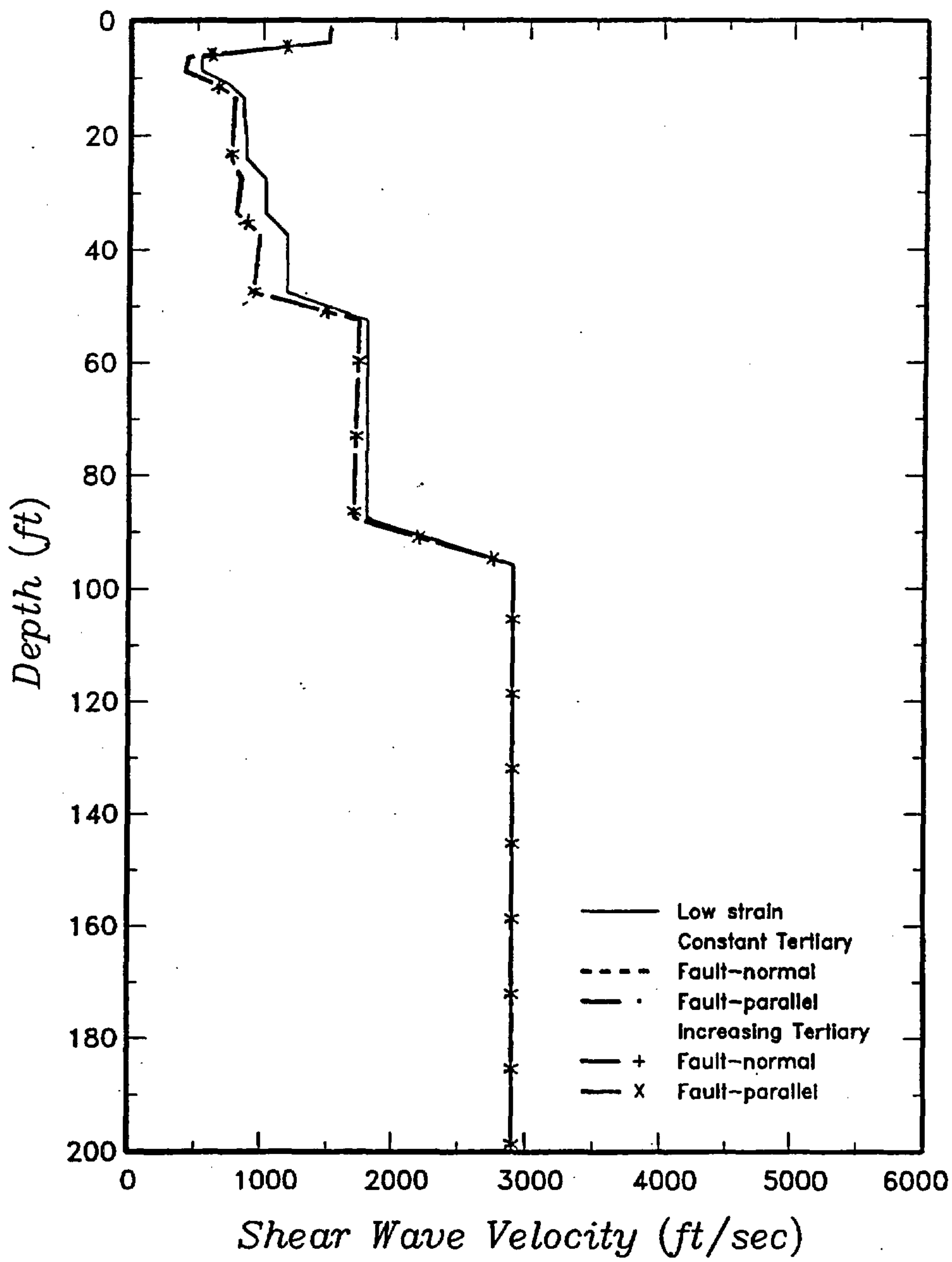


SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)

Figure 2.6-13A

STRAIN-COMPATIBLE SHEAR-WAVE VELOCITY PROFILE LOW RANGE PROPERTIES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)

Figure 2.6-13B

STRAIN-COMPATIBLE SHEAR-WAVE VELOCITY PROFILE BEST-ESTIMATE PROPERTIES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

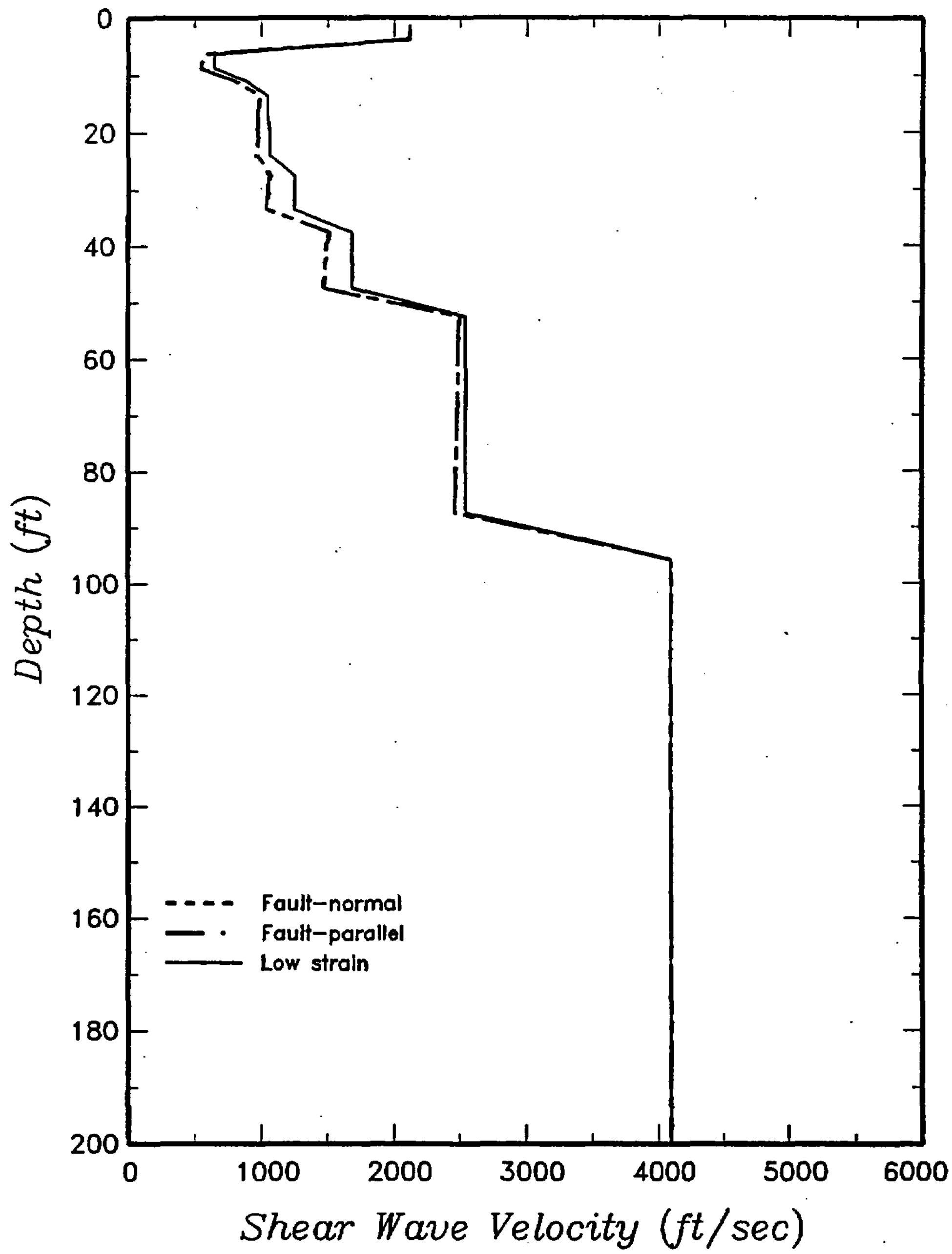


Figure 2.6-13C

STRAIN-COMPATIBLE SHEAR-WAVE VELOCITY PROFILE HIGH RANGE PROPERTIES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)

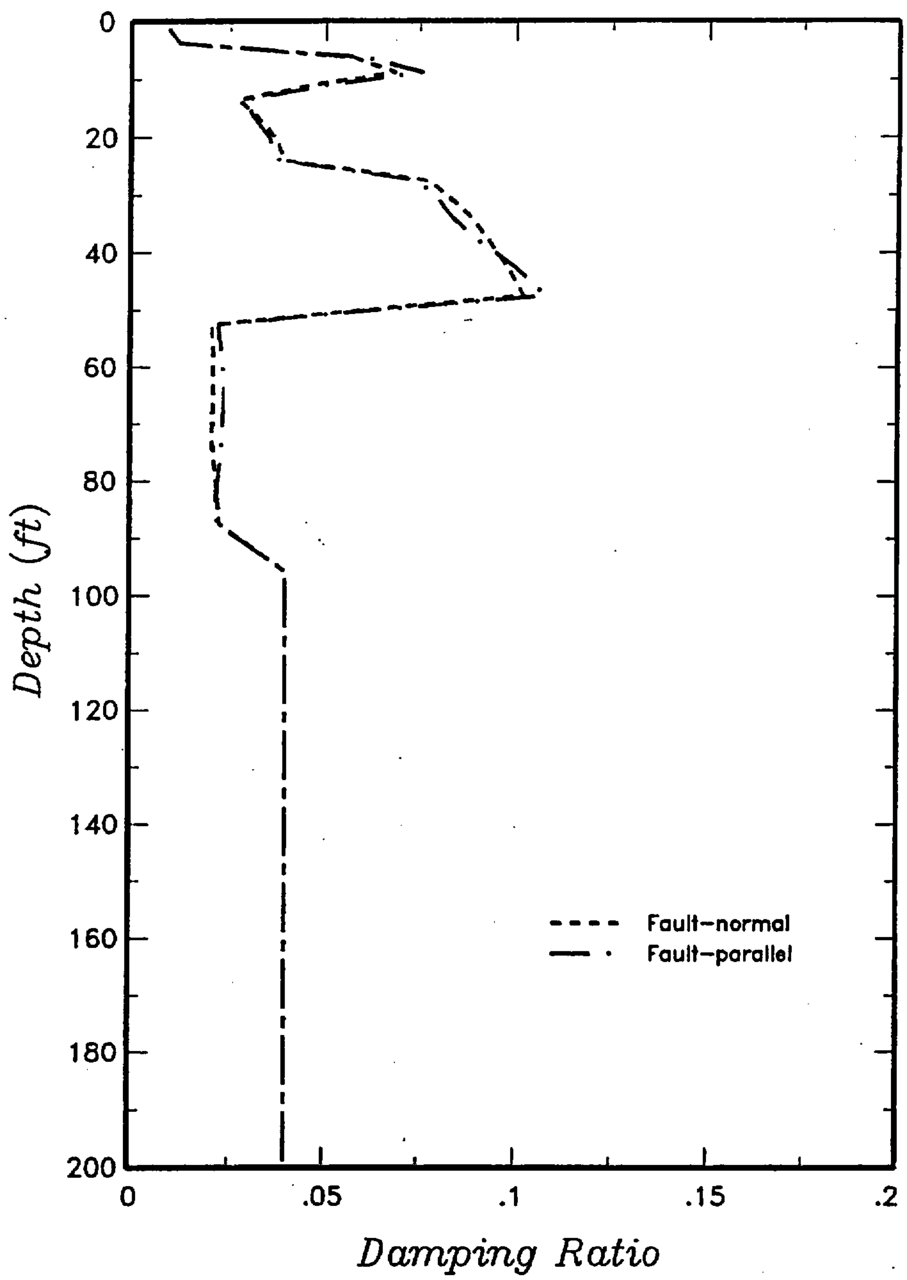
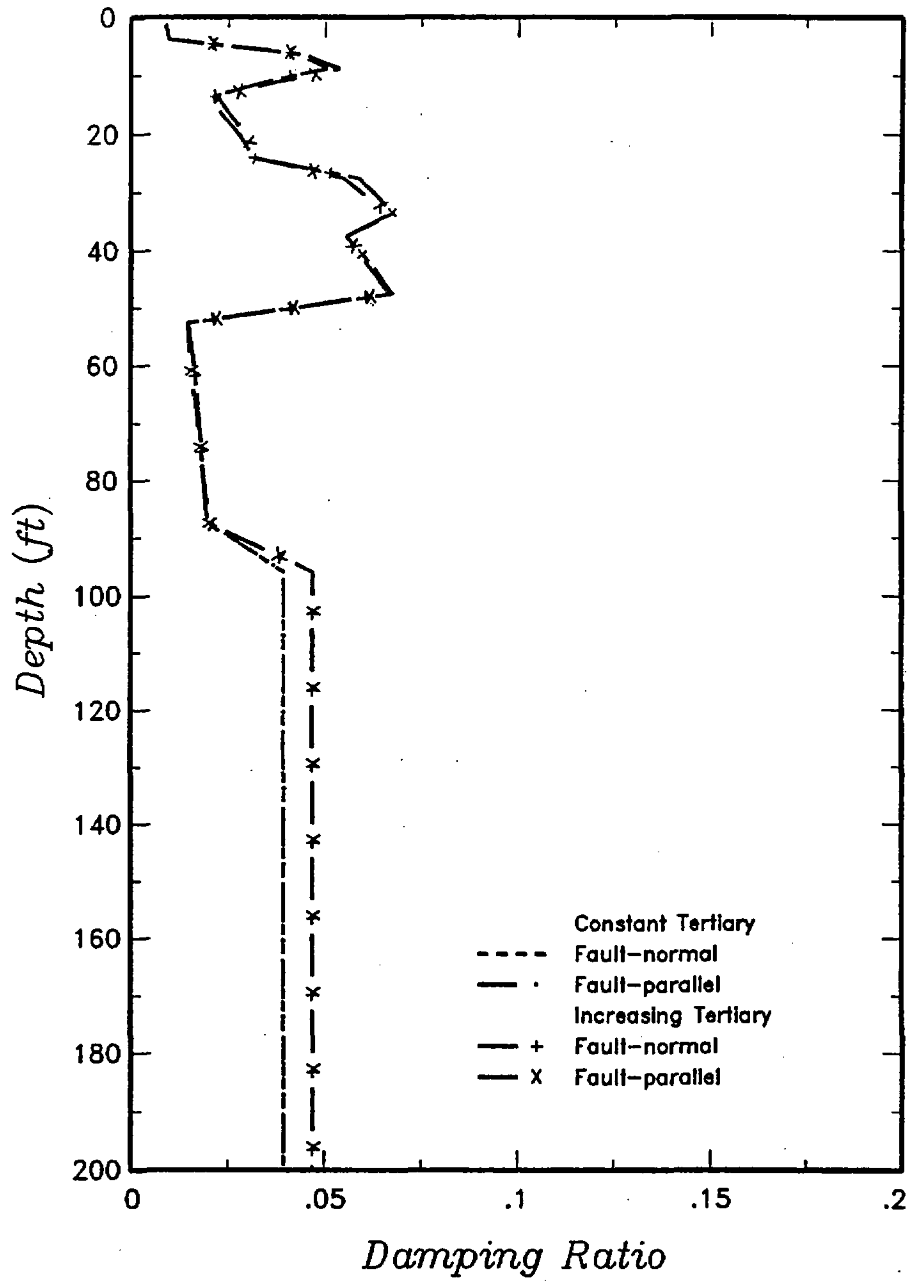


Figure 2.6-14A

STRAIN-COMPATIBLE DAMPING RATIO PROFILE LOW RANGE PROPERTIES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)



SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)

Figure 2.6-14B
STRAIN-COMPATIBLE DAMPING RATIO PROFILE BEST-ESTIMATE PROPERTIES
PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

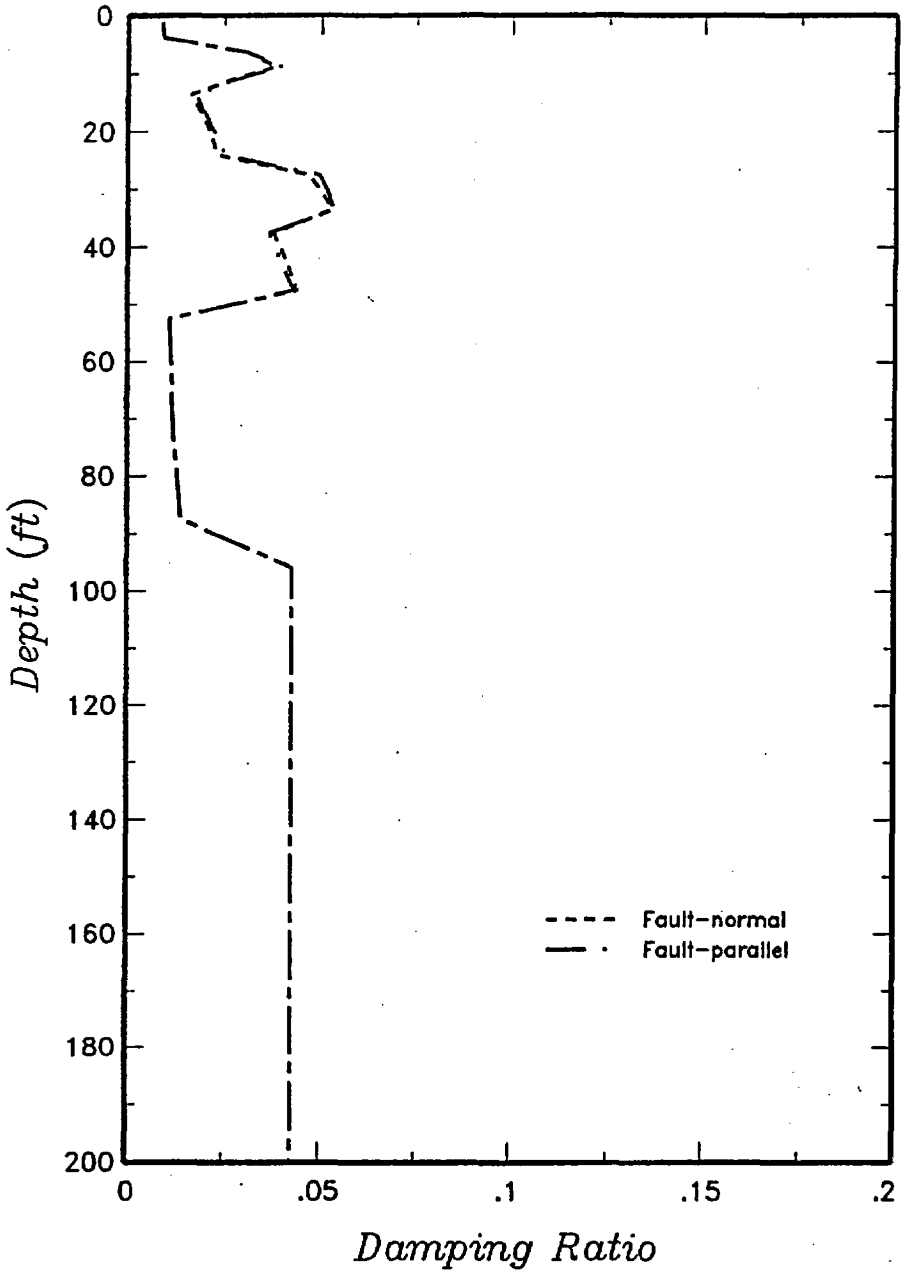


Figure 2.6-14C

STRAIN-COMPATIBLE DAMPING RATIO PROFILE HIGH RANGE PROPERTIES

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

SOURCE: GEOMATRIX CONSULTANTS, INC (2001c)

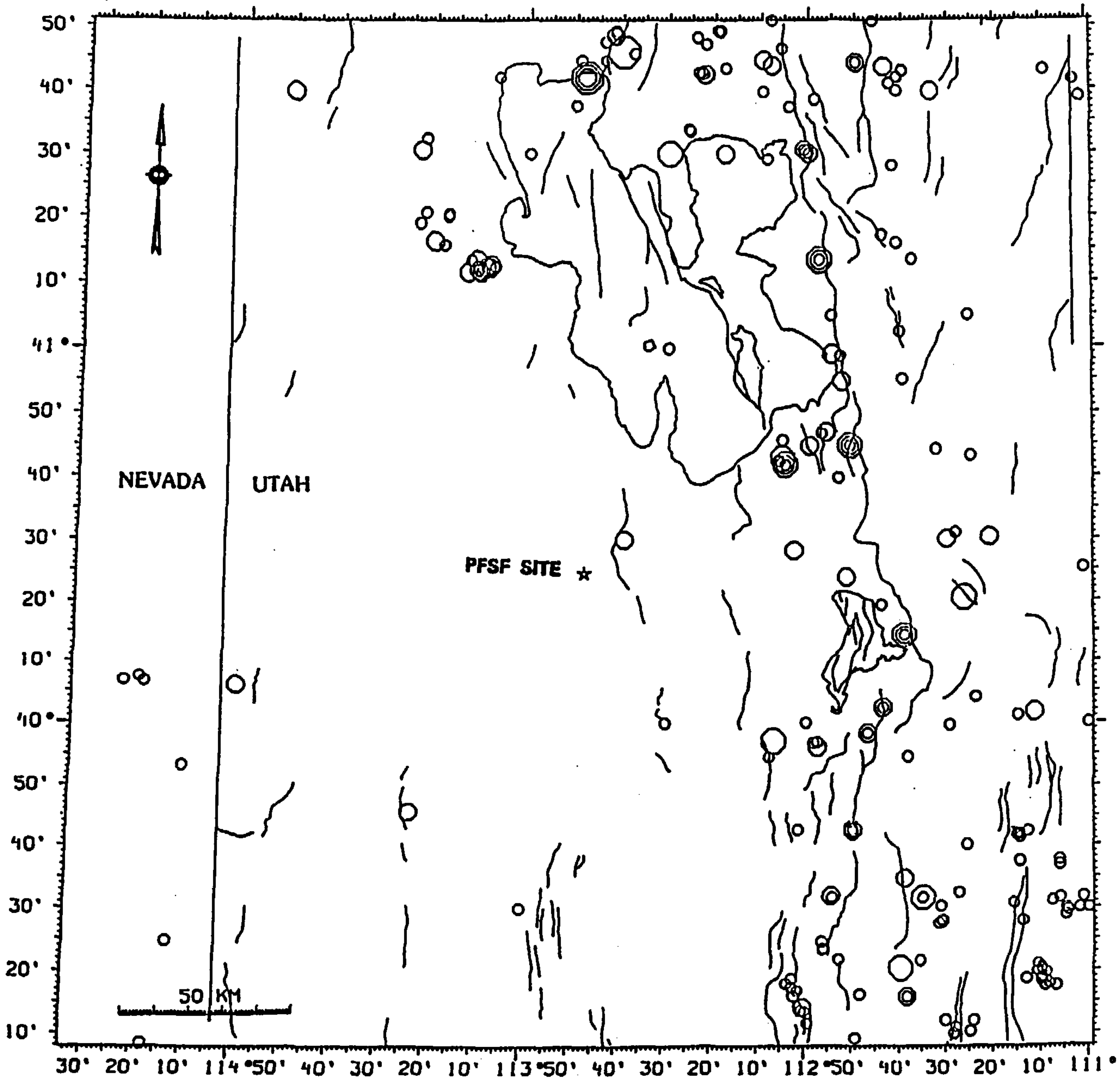
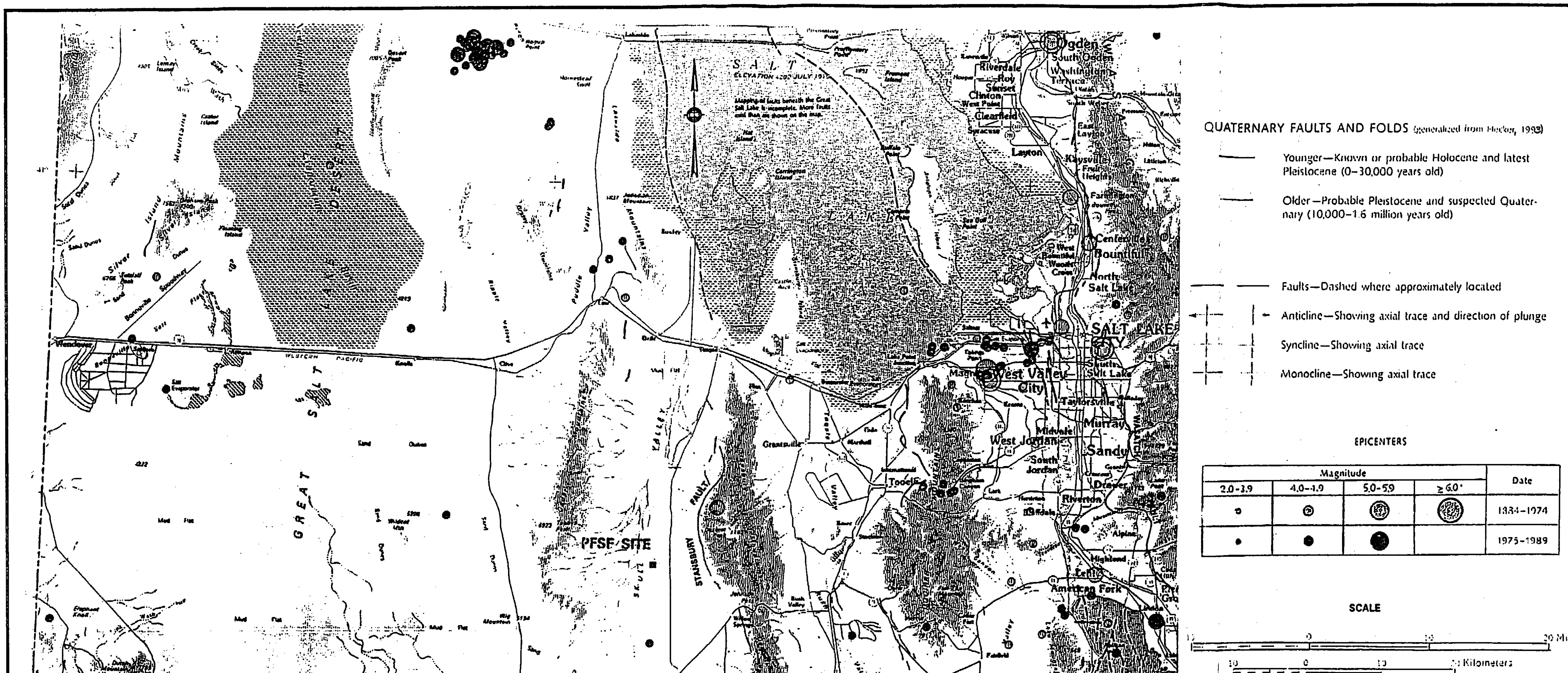


Figure 2.6-15

MAGNITUDE ≥ 3.0 EARTHQUAKES WITHIN 100 MILES OF PFSF 1850 - 1996

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT



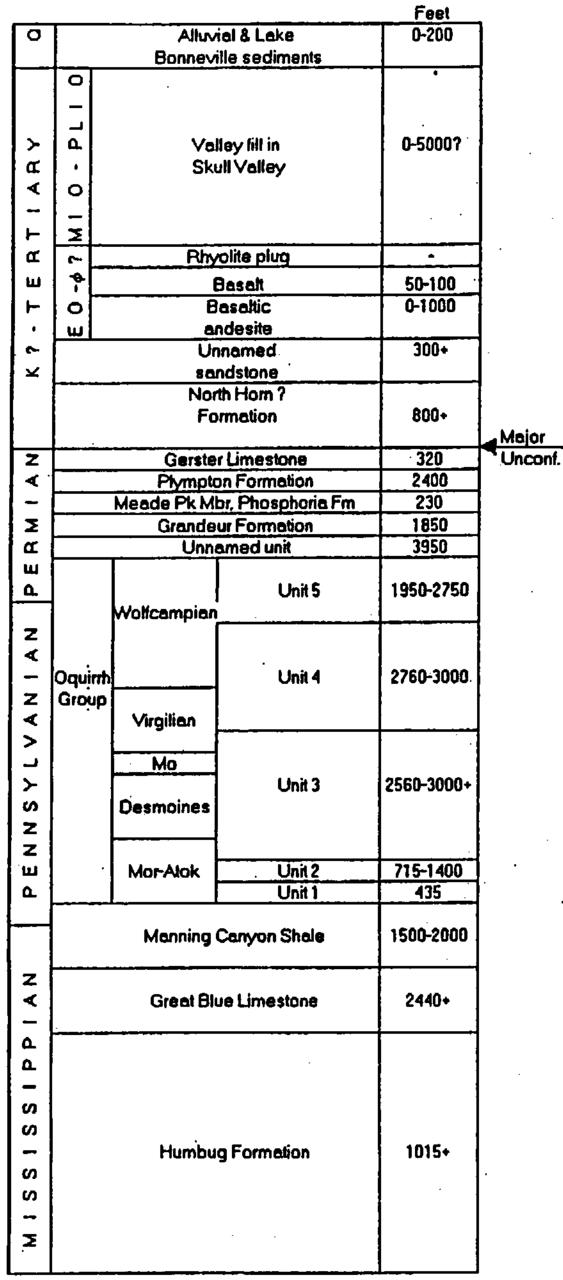
MODIFIED FROM: GOTER, 1990

Figure 2.6–16

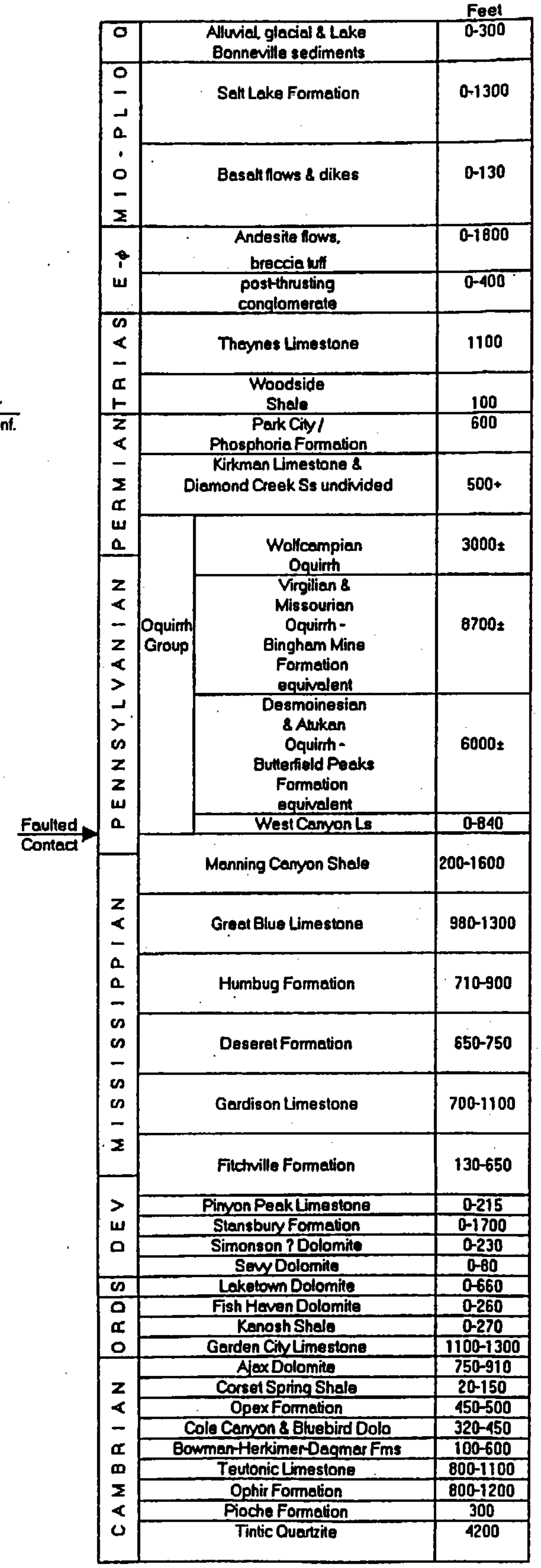
QUATERNARY STRUCTURES AND SEISMICITY, 1884 TO 1989

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

CEDAR MOUNTAINS



STANSBURY MOUNTAINS



CENOZOIC

Quaternary

Alluvial, glacial, and Lake Bonneville sediments – unconsolidated sand, gravel, silt, and clay with some ash beds and marl.

Miocene-Pliocene

Salt Lake Formation (or Group) – valley fill deposits – semi to unconsolidated sand, gravel, silt, clay, tuff and freshwater limestone.

Basalt flows and dikes – olivine basalts believed to mark beginning of Basin and Range rifting.

Eocene-Oligocene

Andesite flows, breccia tuffs – widespread, voluminous rhyolite, dacite, latite, andesite, and welded tuff.

Unnamed post-thrusting conglomerate (= North Horn? Formation?) – reddish pebble conglomerate with argillaceous and calcareous matrix.

MESOZOIC

Triassic

Thaynes Limestone – light gray limestone with red-brown to light gray shaley siltstone and sandstone, minor dolomite.

Woodside Shale – reddish-brown, shaley siltstone and cross-bedded, fine to medium-grained sandstone.

PALEOZOIC

Permian

Park City/Phosphoria Formation – light gray to pink, thin to thick-bedded limestone with brown-black cherty limestone, phosphorite and phosphatic siltstone.

Kirkman Limestone and Diamond Creek Sandstone – Kirkman is light to medium-gray, thin to thick-bedded limestone with chert; Diamond Creek Sandstone is red-brown to light brown, cross-bedded sandstone with some intercalated limestone.

Penn. to Perm.

Oquirrh Group – cyclic alternation of sandy limestone, brown sandstone and minor shale, siltstone, and quartzite; fossiliferous.

Miss. to Penn.

Manning Canyon Shale – (lower) black shale, (middle) dark gray limestone, and (upper) black shale and quartzite, with some pyrite nodules and chert.

Mississippian

Great Blue Limestone – medium to massive bedded, nearly pure, gray to dark gray limestone with some chert; dark green calcareous shale near top.

Humbug Formation – alternating beds of limey sandstone, ortho-quartzite, crinoidal limestone, and sandy limestone; yellow to red-brown and gray alternations.

Deseret Formation – dark gray and blue, somewhat clastic limestone with chert banding and blebs (eyes).

Gardison Limestone – dense, bluish-gray limestone, fossiliferous.

PALEOZOIC (CONT.)

Devonian-Miss.

Fitchville Formation – massive to thin-bedded, light to dark gray dolomite and gray to buff clastic limestone

Devonian

Pinyon Peak Limestone – thin, platy, silty, or argillaceous limestone.

Stansbury Formation – highly variable conglomerate, sandstone, and quartzite with thin beds of gray limestone and dolomite.

Simonson (?) Dolomite – dark gray with minor light gray, medium to coarse crystalline, weakly bedded dolomite.

Sevy Dolomite – very fine crystalline, light gray dolomite with well-defined bedding. Sand layer or dolomitic conglomerate marks the top of the formation.

Silurian

Laketown Dolomite – alternating light to dark gray well-bedded dolomite in lower part and coarse crystalline, massive to obscurely thick-bedded gray dolomite in upper part.

Ordovician

Fish Haven Dolomite – dark gray to black dolomite with some interbeds of light to medium gray dolomite.

Kanosh Shale – green to black, graptolitic shale with interbeds of argillaceous sandstone and limestone or dolomite.

Garden City Limestone – cherty limestone and dolomite; medium gray, argillaceous limestone; interbedded gray argillaceous limestone and green to brown shale or siltstone; sandy limestone with chert and siltstone bands.

Cambrian

Ajax Limestone – thick-bedded, dark gray, ledge-forming dolomite with pisolites, oolites, and chert nodules.

Corset Spring Shale (= Dunderberg Shale?) – thinly bedded, argillaceous limestone and dolomite interbedded with olive to brown-green silty shale.

Opex Formation – gray to black oolitic dolomite, interbedded limestone, dolomite, and shale, light gray to tan dolomite at top.

Cole Canyon and Bluebird Dolomites – thick to massive bedded, dark gray, medium to fine crystalline dolomite (Bluebird); laminated light and dark gray dolomite (Cole Canyon).

Bowman-Herkimer-Dagmar Formations – medium gray, crystalline, laminated dolomite (Dagmar); thin to medium-bedded gray limestone, interbedded light and dark gray dolomite (Herkimer); olive and tan shale with interbedded blue-gray limestone (Bowman).

Teutonic Limestone – blue-gray to dark gray dolomite, thinly interbedded shale and limestone, massive gray dolomite, and argillaceous limestone.

Ophir Formation – calcareous sandstone and sandy limestone, pisolitic limestone, green shale, dark gray limestone.

Pioche Formation – interbedded green phyllitic shale, shale, maroon graywacke and quartzite with prominent cross-bedding.

Tintic Quartzite – light colored (white, light gray, reddish brown), medium grained, medium-bedded quartzite, with a few beds of micaceous shale in the upper part and pebble conglomerate.

SOURCES

Heylmun, E.B., 1965. Reconnaissance of the Tertiary Sedimentary Rocks in Western Utah: Utah Geological and Mineralogical Survey, Bulletin 75, 38 pp.

Hintze, L.F., 1988. Geologic History of Utah, Brigham Young University Geology Studies, Spec. Pub. 7, 203 pp.

Rigby, J.K., 1958. Geology of the Stansbury Mountains, Tooele County, Utah: Utah Geological Society Guidebook 13, 134 pp.

Teichert, J.A., 1959. Geology of the Southern Stansbury Range, Tooele County, Utah: Utah Geological and Mineralogical Survey, Bulletin 65, 75 pp.

Witkind, I.J., 1983. Overthrusts and Salt Diapirs, Central Utah, in D.M. Miller et al., editors, Tectonic and Stratigraphic Studies in the Eastern Great Basin: Geological Society of America, Memoir 157, pp. 45-59.

Figure 2.6-17

STRATIGRAPHIC COLUMN FOR SKULL VALLEY AREA

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

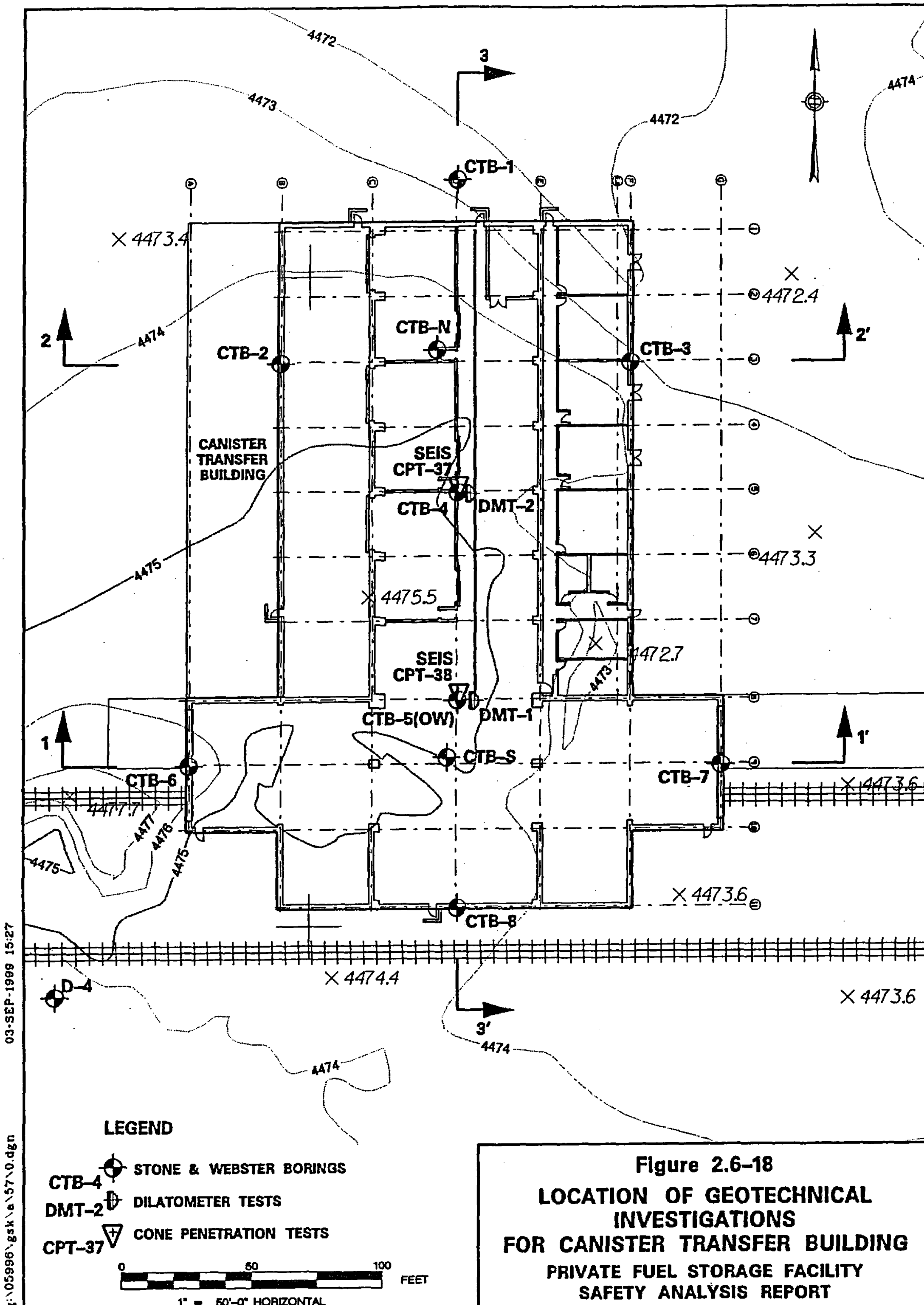


Figure 2.6-18

LOCATION OF GEOTECHNICAL INVESTIGATIONS FOR CANISTER TRANSFER BUILDING

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

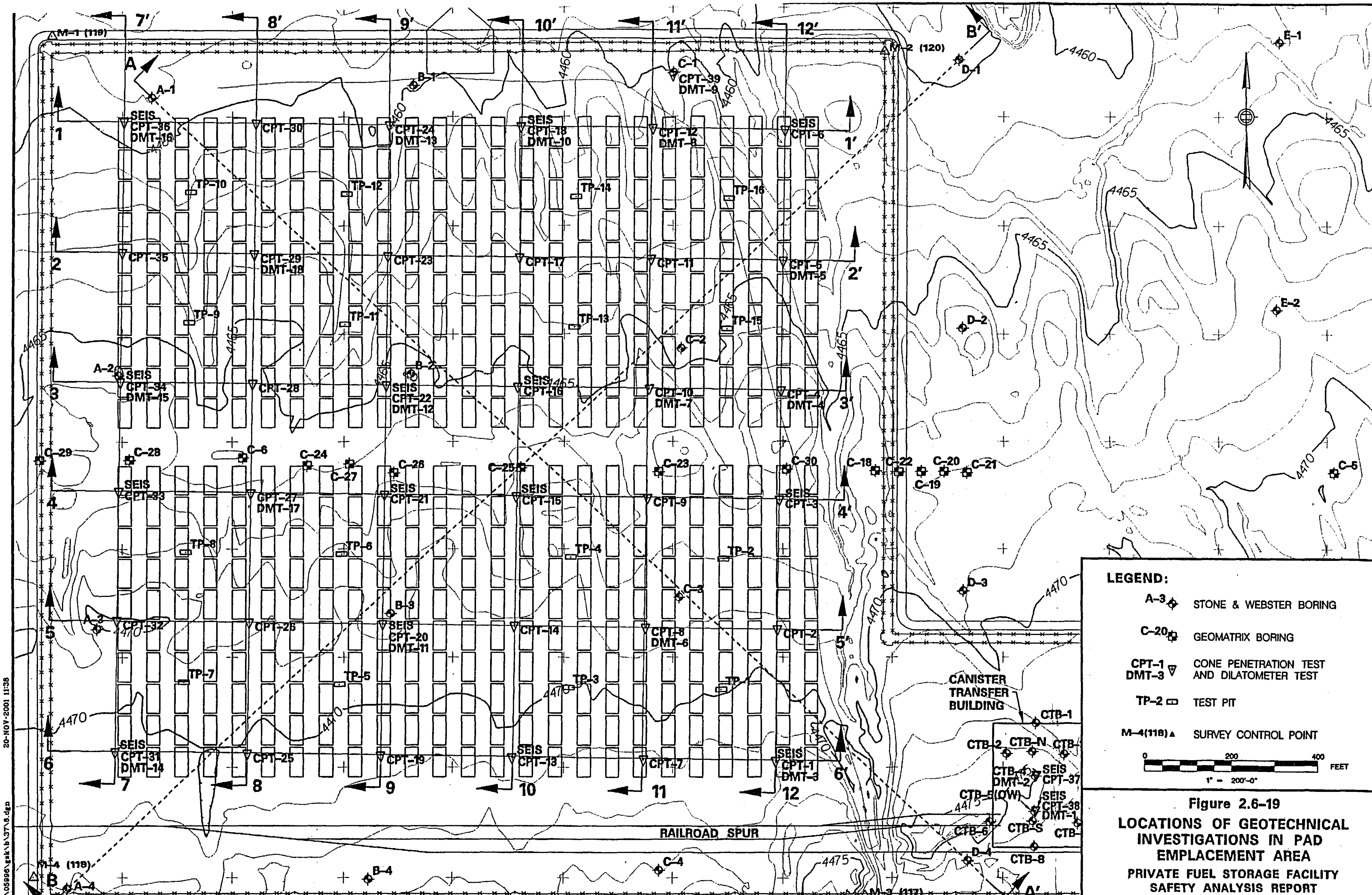


Figure 2.6-19

LOCATIONS OF GEOTECHNICAL INVESTIGATIONS IN PAD EMPLACEMENT AREA

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

20-NOV-2001 11:38

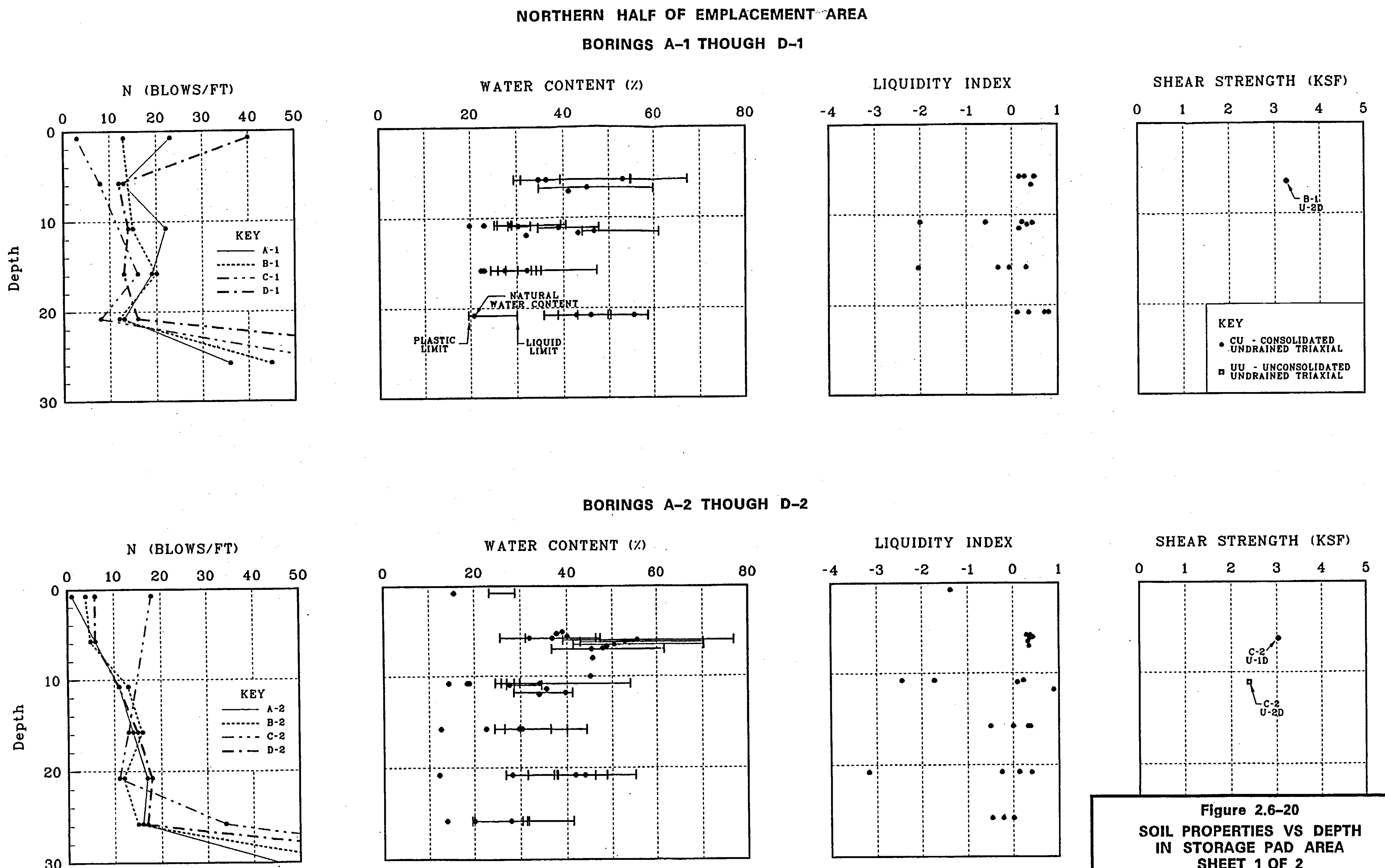
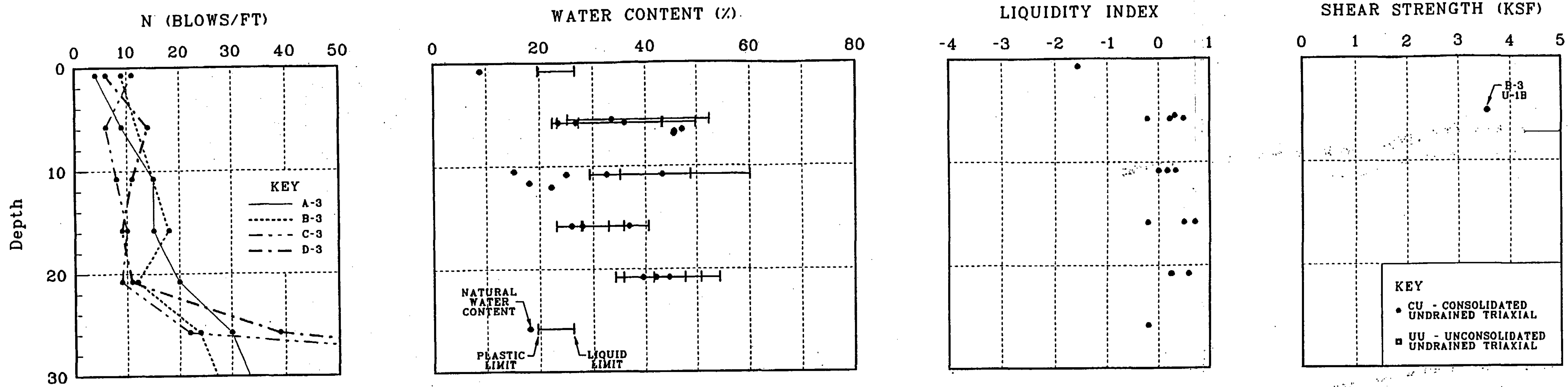


Figure 2.6-20
SOIL PROPERTIES VS DEPTH IN STORAGE PAD AREA SHEET 1 OF 2
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

03-SEP-1999 15:33
g:\05996\gsk\b\35\e-3.dgn

SOUTHERN HALF OF EMPLACEMENT AREA

BORINGS A-3 THOUGH D-3



BORINGS A-4 THOUGH D-4

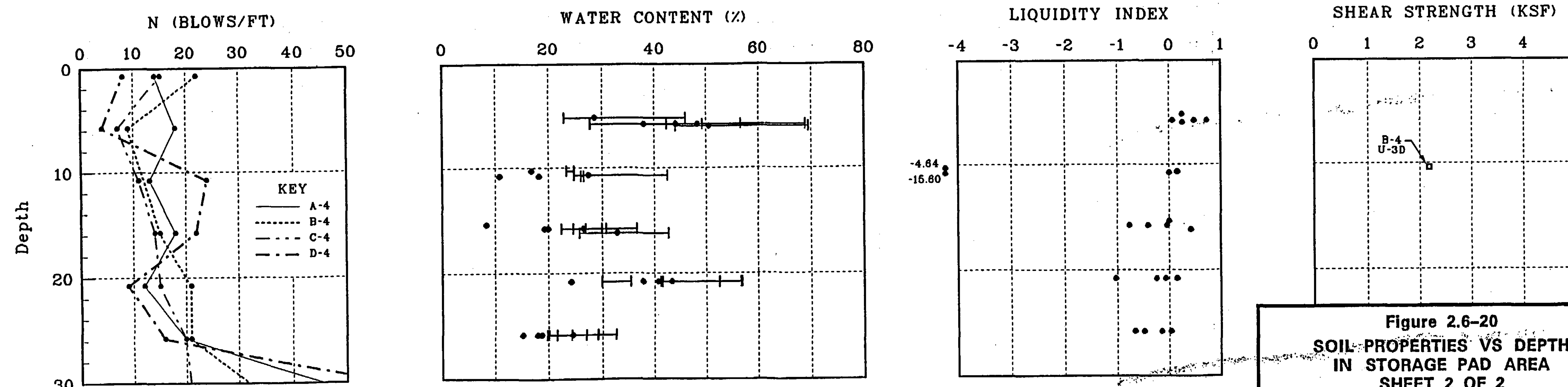


Figure 2.6-20
SOIL PROPERTIES VS DEPTH IN STORAGE PAD AREA SHEET 2 OF 2
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

Revision

03-SEP-1999 16:33
g:\D5996\gzk\b\35\b-3.dgn

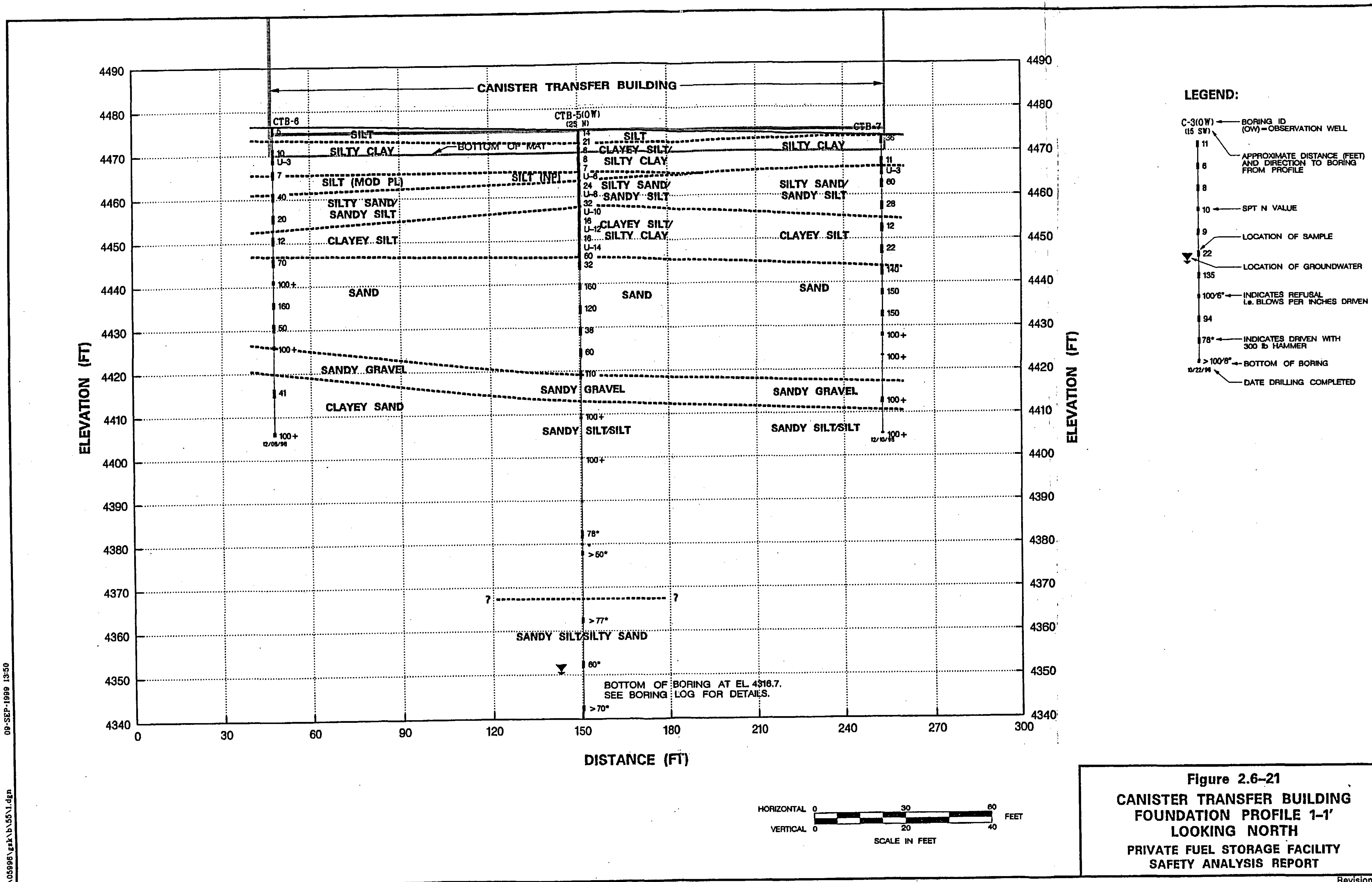
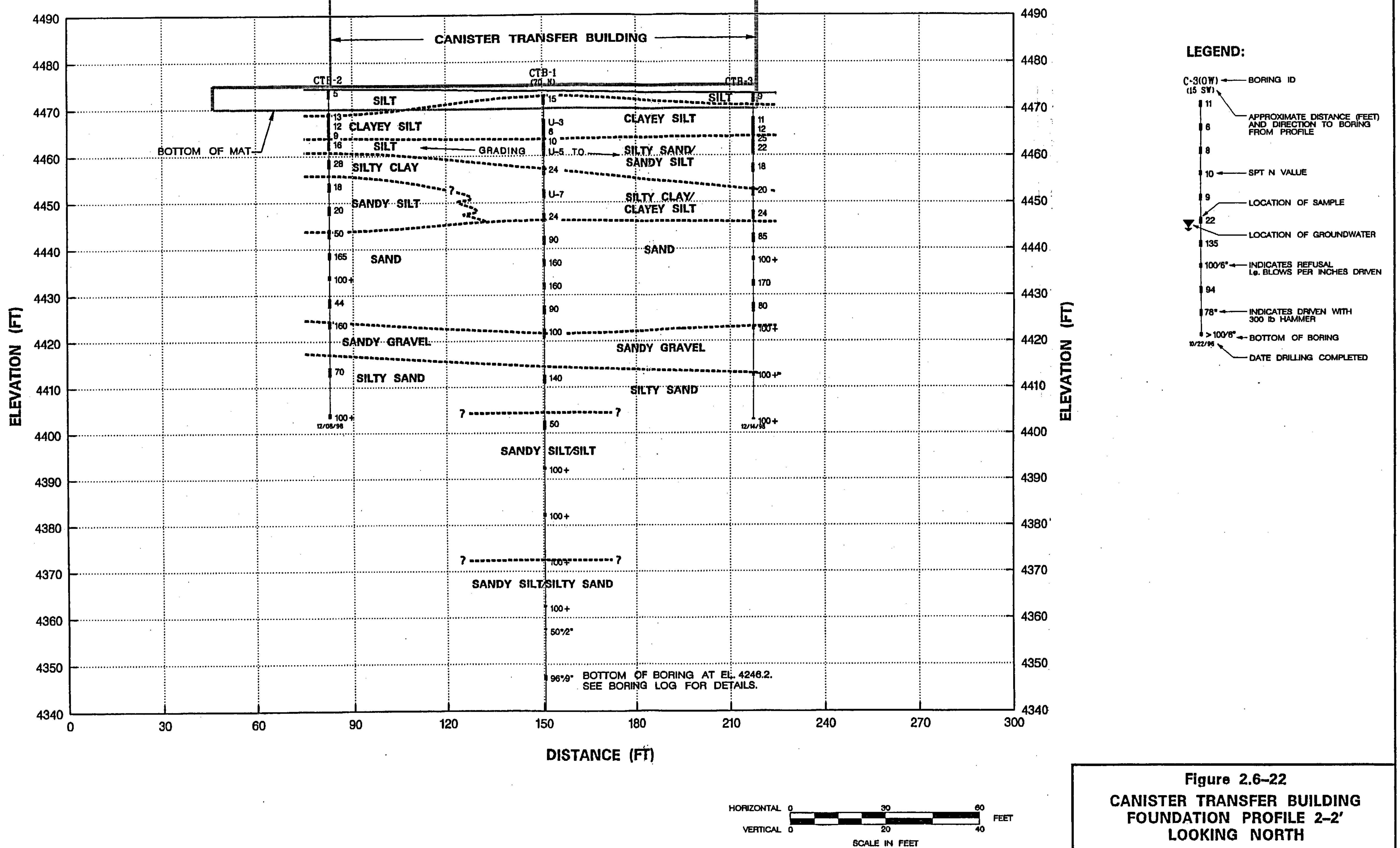


Figure 2.6-21
CANISTER TRANSFER BUILDING FOUNDATION PROFILE 1-1' LOOKING NORTH
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

09-SEP-1999 13:50
g:\05996\gsk\b\55\1.dgn



09-SEP-1999 13:40

g:\05996\gsk\b\58\1.dgn

Figure 2.6-22
CANISTER TRANSFER BUILDING FOUNDATION PROFILE 2-2' LOOKING NORTH
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

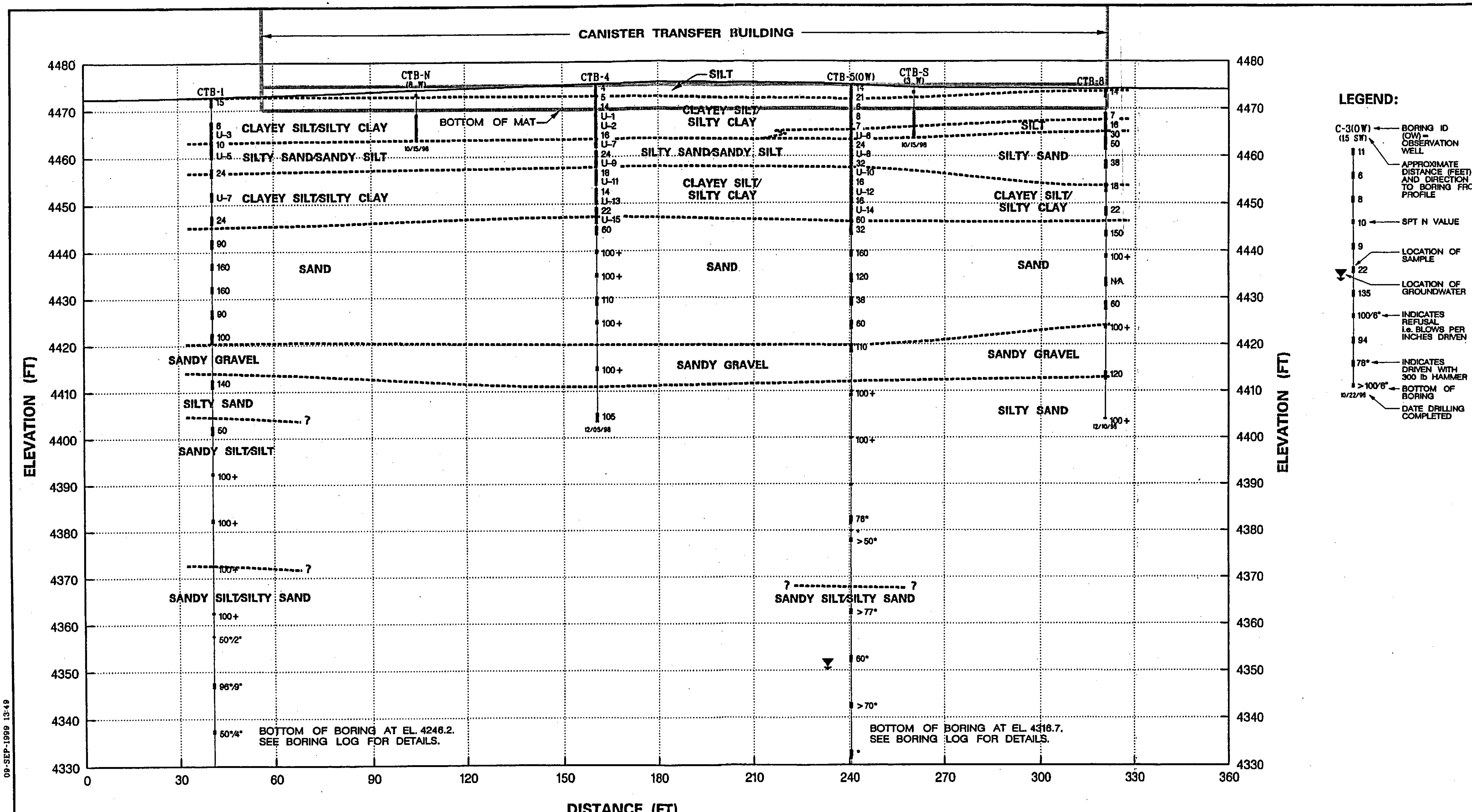
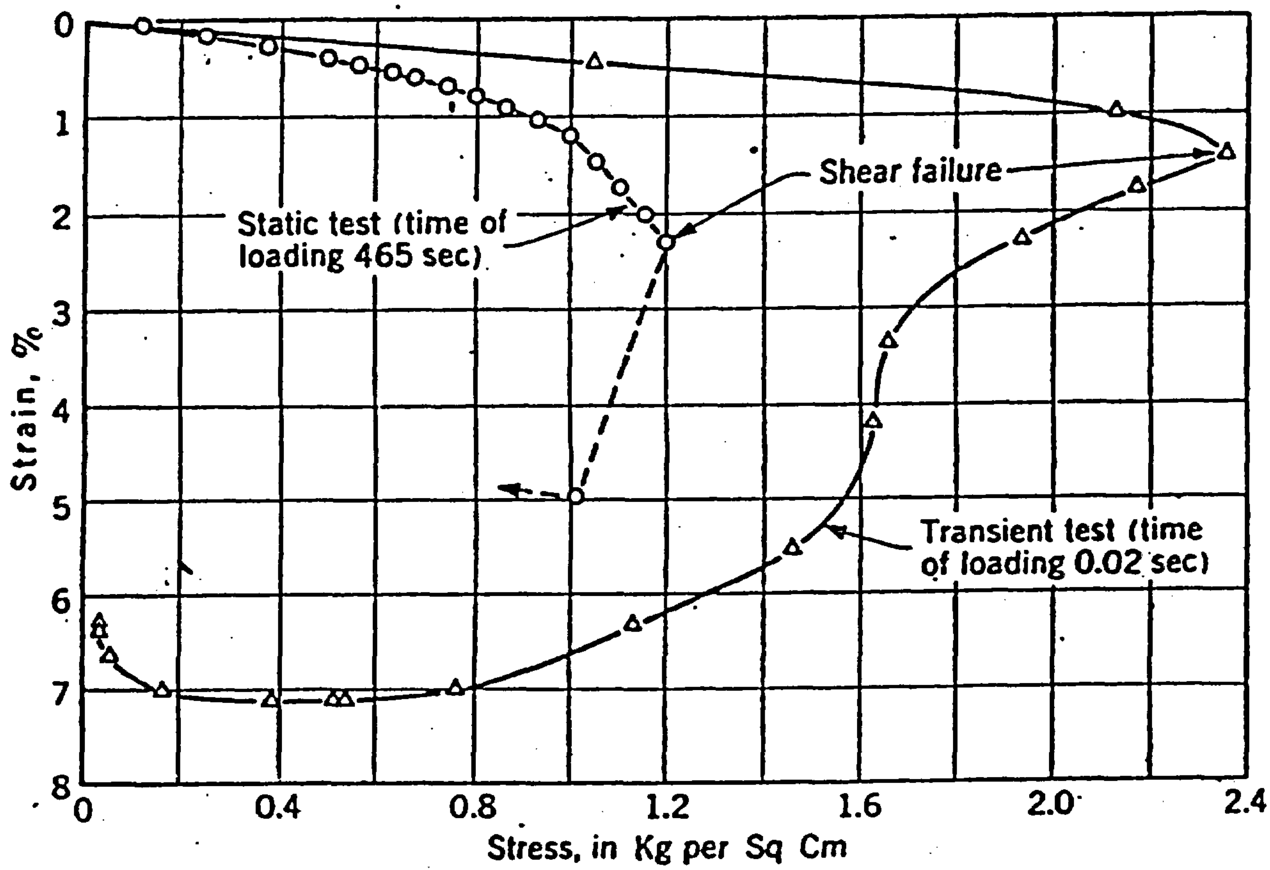


Figure 2.6-23

CANISTER TRANSFER BUILDING FOUNDATION PROFILE 3-3' LOOKING EAST

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

09-SEP-1999 13:49
g:\05996\gsk\b\54\1.dgn



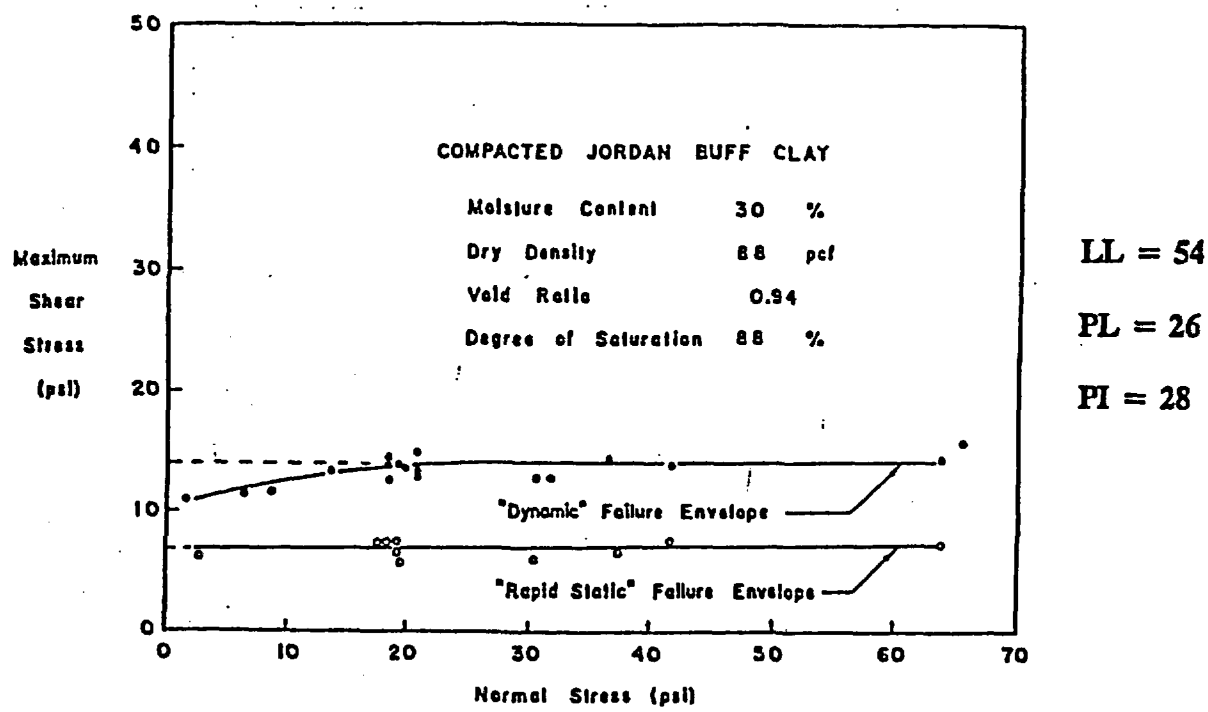
SOURCE: CASAGRANDE AND SHANNON, 1948

Figure 2.6-24

EFFECT OF TIME OF LOADING ON STRESS-STRAIN RELATION FOR CAMBRIDGE CLAY

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\05996\gsk\e\58\0.dgn 07-SEP-1999 16:00



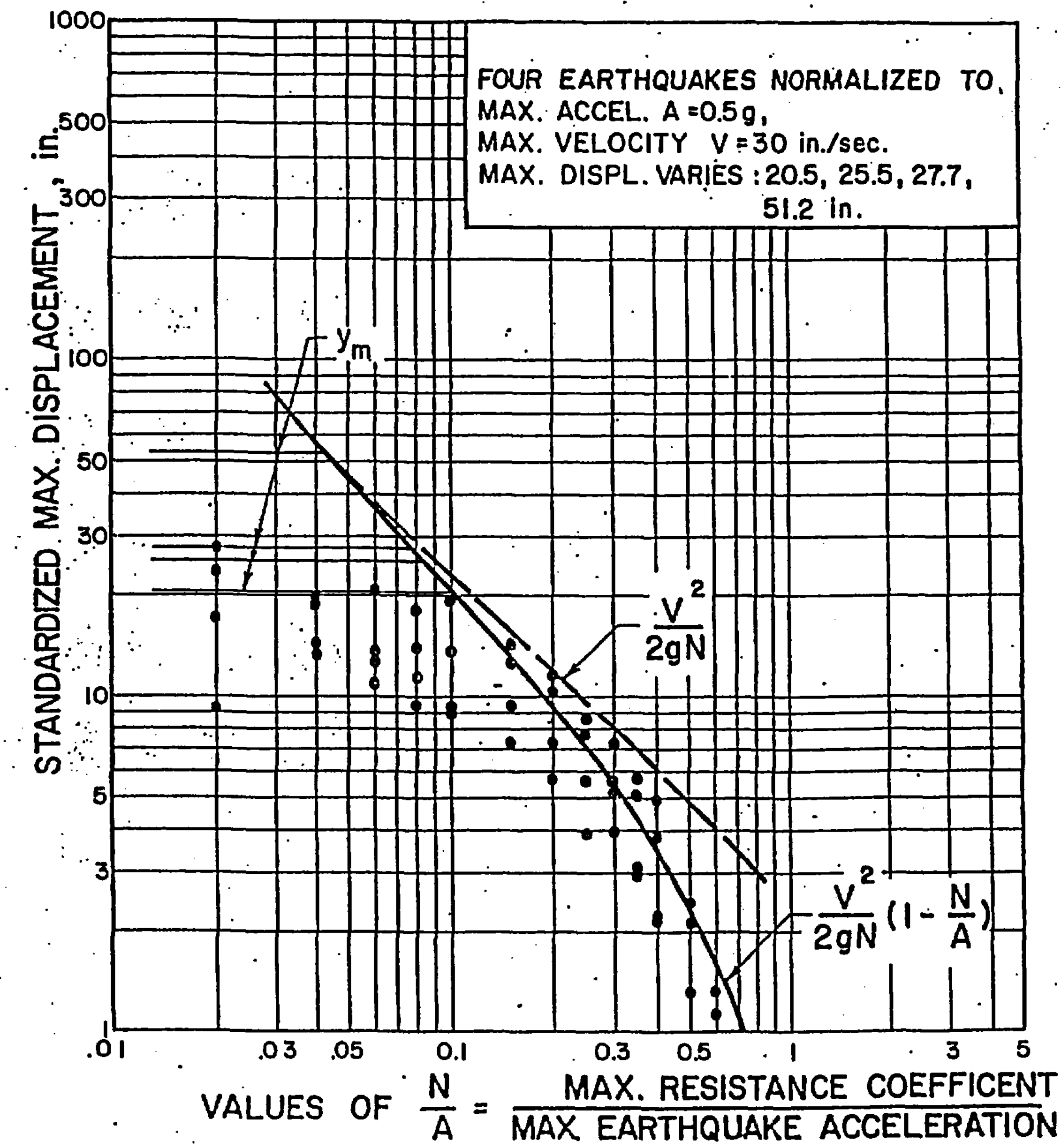
SOURCE: SCHIMMING, HAAS, AND SAXE, 1966

Figure 2.6-25

TYPICAL FAILURE RESPONSE FOR COHESIVE SOIL - "DYNAMIC" VS "RAPID STATIC" TESTING RATES

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\05996\gsk\a\5910.dgn 07-SEP-1999 15:59



SOURCE: NEWMARK, 1965

Figure 2.6–26

STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES SYMMETRICAL RESISTANCE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\05996\gsk\e\60\0.dgn 07-SEP-1999 15:59

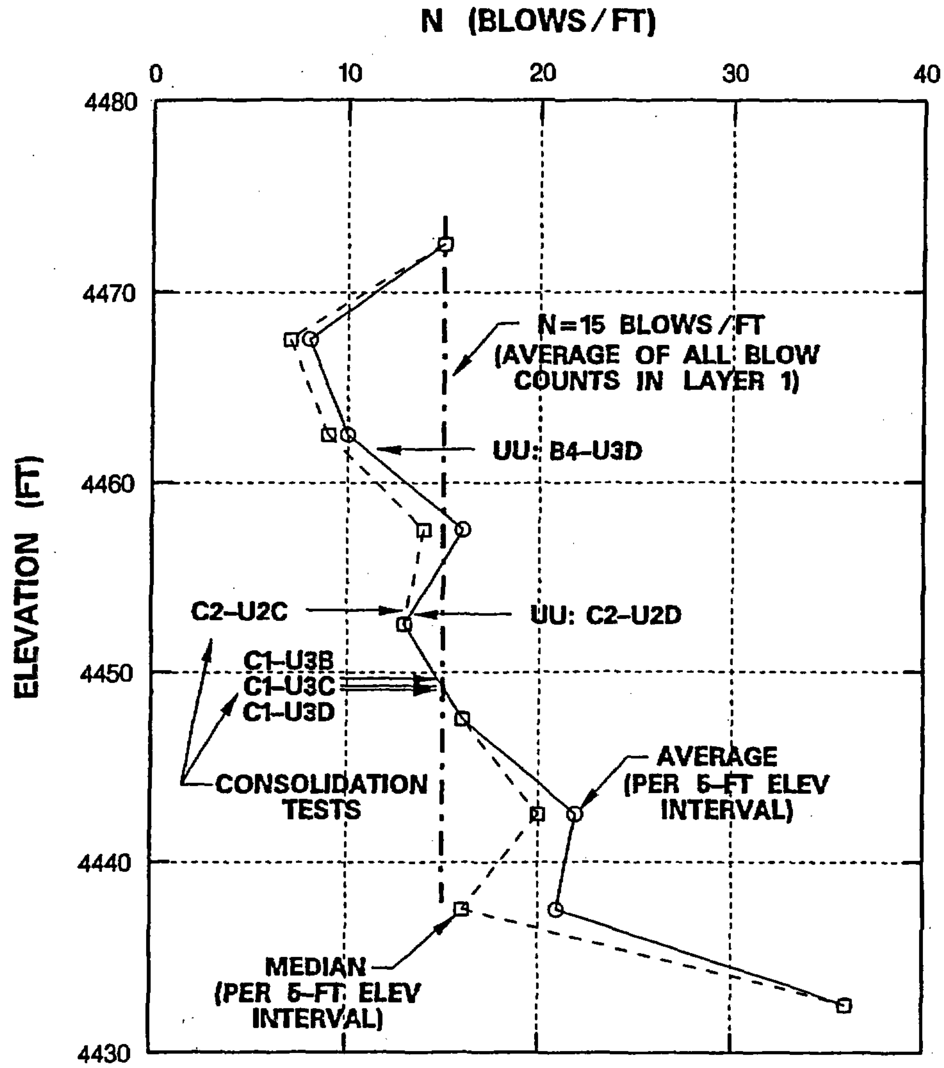
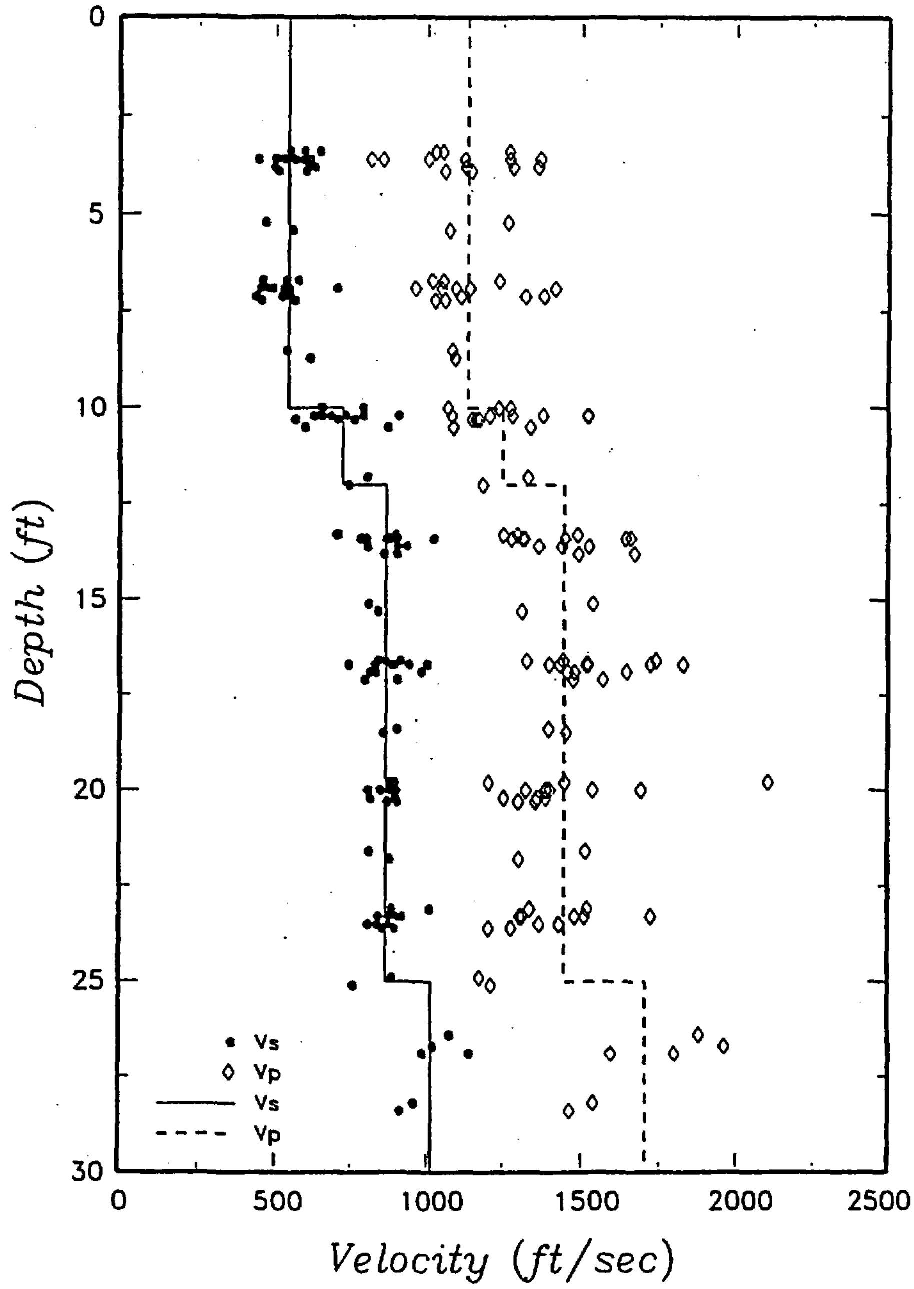


Figure 2.6-27
STANDARD PENETRATION TEST BLOW COUNT DATA FROM LAYER 1 IN STORAGE PAD AREA
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\a\33\1.dgn 07-SEP-1999 16:16



SOURCE: GEOMATRIX CONSULTANTS, INC, 1999d

Figure 2.6–28

SEISMIC CONE PENETRATION TEST DATA AND AVERAGE VELOCITIES

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

G:\05996\gsk\a\61\0.dgn 07-SEP-1999 15:56

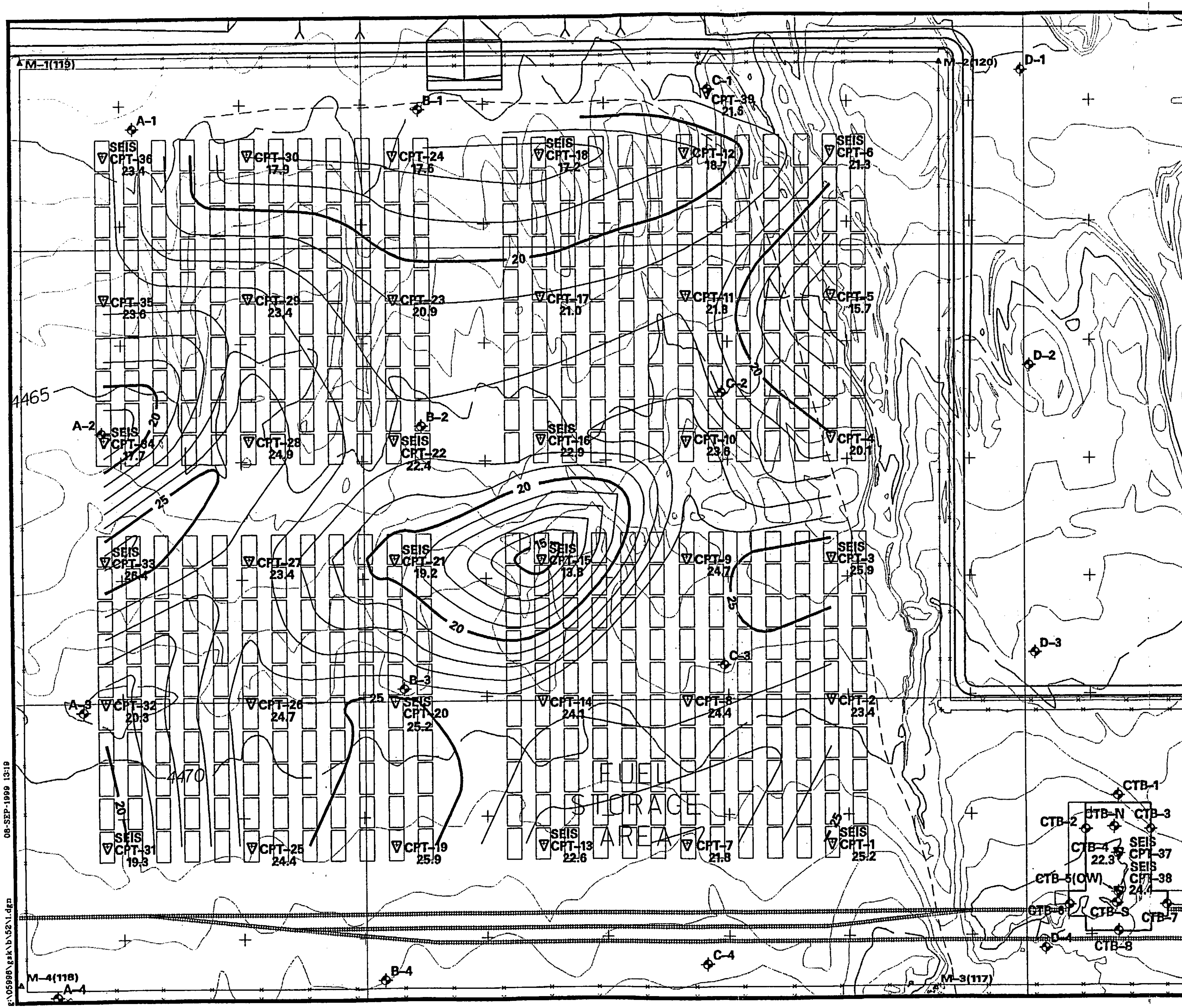


Figure 2.6-29
CONTOUR MAP SHOWING THICKNESS OF SOILS WITH CPT SOIL BEHAVIOR TYPE >5 (SANDY)

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

Revision

08-SEP-1999 13:19

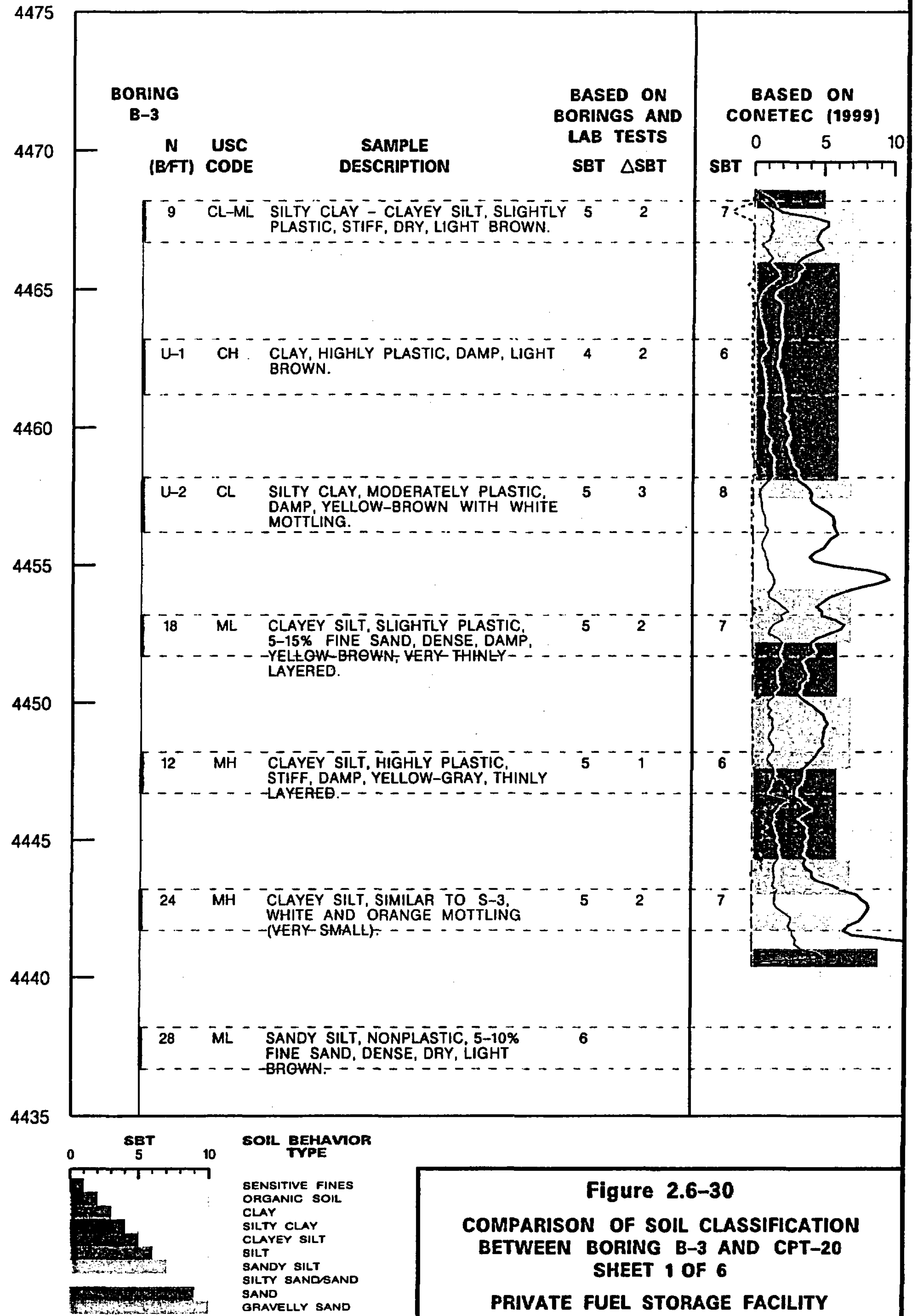


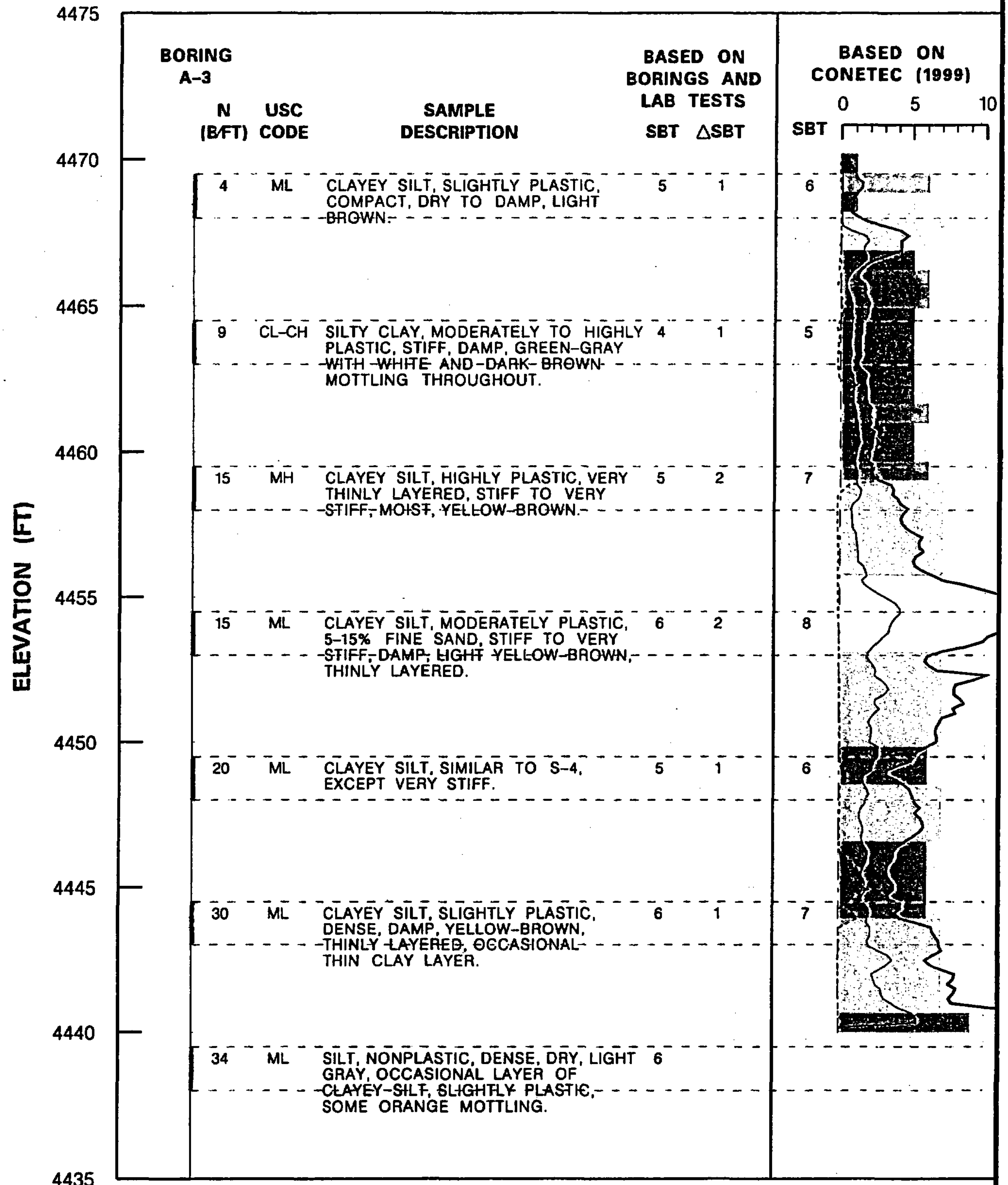
Figure 2.6-30

COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING B-3 AND CPT-20
SHEET 1 OF 6

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

08-DEC-1999 15:45

g:\05996\gsk\a\7510.dgn



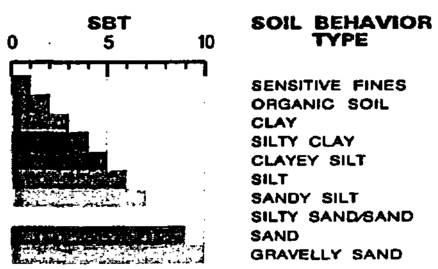
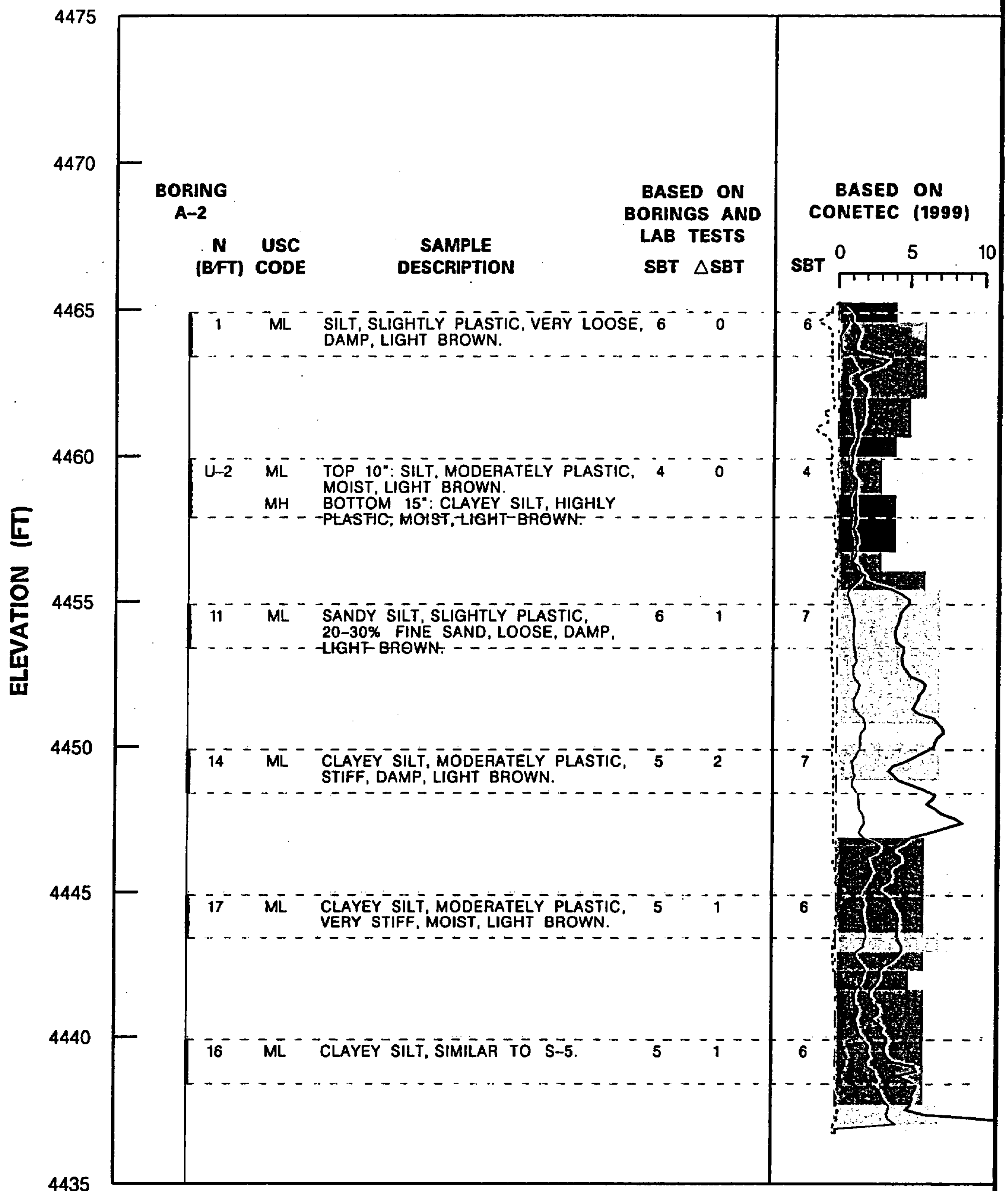


Figure 2.6-30

COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING A-3 AND CPT-32 SHEET 2 OF 6

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

08-DEC-1999 15:45
g:\05996\gsk\a\76\0.dgn



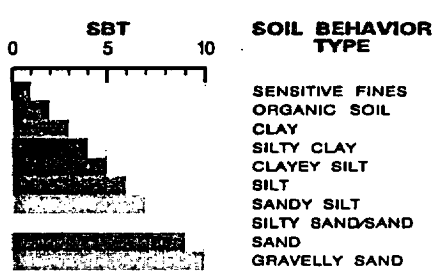


Figure 2.6-30

COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING A-2 AND CPT-34
SHEET 3 OF 6

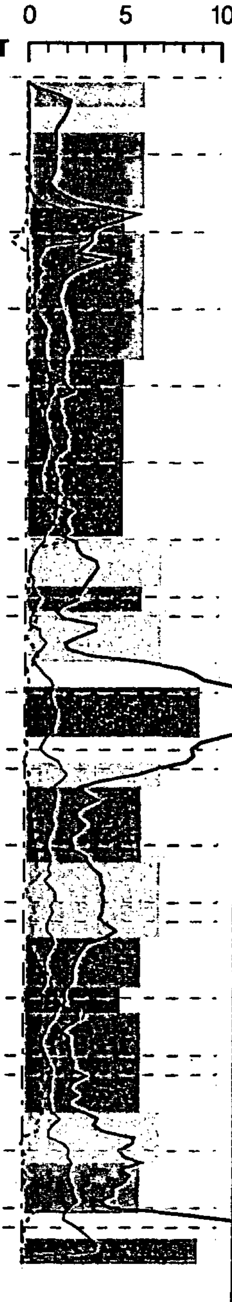
PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

08-DEC-1999 15:45

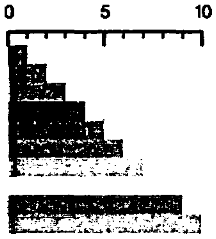
g:\05996\gsk\a\77\0.dgn

ELEVATION (FT): 4480, 4475, 4470, 4465, 4460, 4455, 4450, 4445, 4440

BORING CTB-4 N (B/FT)	USC CODE	SAMPLE DESCRIPTION	BASED ON BORINGS AND LAB TESTS SBT	ΔSBT	BASED ON CONETEC (1999) SBT
4	ML	SILT, MODERATELY PLASTIC, SOFT TO FIRM, DAMP, LIGHT BROWN, EOLIAN.	6	0	6
5	ML	TOP 6": SILT, SIMILAR TO ABOVE. BOTTOM 7": CLAYEY SILT, MODERATELY PLASTIC, FIRM, DAMP, BROWN WITH WHITE MOTTLING.	5	1	6
14	CL	SILTY CLAY, MODERATELY PLASTIC, STIFF, DAMP, GREEN-GRAY WITH WHITE MOTTLING THROUGHOUT.	5	1	6
U-1	CL MH	TOP 14": SILTY CLAY, MODERATELY PLASTIC, GRAY. BOTTOM 10": CLAYEY SILT, HIGHLY PLASTIC, DAMP, YELLOW-BROWN.	5	1	6
U-2	CH	SILTY CLAY, HIGHLY PLASTIC, YELLOW-BROWN.	4	1	5
16	ML	CLAYEY SILT, MODERATELY PLASTIC, VERY STIFF, DAMP, YELLOW-BROWN, SOME PARTINGS OF FINE SAND.	5	0	5
U-7	SP	TOP: SANDY SILT, SLIGHTLY PLASTIC, 25-35% FINE SAND, DAMP, YELLOW-BROWN. BOTTOM: SAND, UNIFORM, FINE, <10% NONPLASTIC FINES, DAMP, LIGHT YELLOW-BROWN.	6	1	7
24	ML SM	TOP 8": SANDY SILT, SLIGHTLY PLASTIC, 30-40% FINE SAND, DAMP, YELLOW-BROWN. BOTTOM 7": SILTY SAND, UNIFORM, FINE 35-40% NONPLASTIC FINES, DENSE, DRY, GRAY-BROWN...	6	1	7
U-9	ML SM	TOP 1": SANDY SILT, NONPLASTIC, 15-25% FINE SAND, DAMP, LIGHT RED-BROWN. BOTTOM 18": SILTY SAND, FINE, 10-20% NP FINES, DAMP, BROWN-GRAY, OCCASIONAL...	7	2	9
18	CL	SILTY CLAY, MODERATELY PLASTIC, <10% FINE SAND, VERY STIFF, DAMP, LIGHT YELLOW-GRAY, NUMEROUS THIN LAYERS OF SILT.	5	1	6
U-11	ML	SANDY SILT, MODERATELY PLASTIC, 10-20% FINE SAND, DRY, LIGHT YELLOW-BROWN.	7	0	7
14	MH	CLAYEY SILT, HIGHLY PLASTIC, STIFF, DRY, YELLOW-BROWN, THINLY BEDDED, WITH OSTRACODES.	5	1	6
U-13	ML	CLAYEY SILT, MODERATELY PLASTIC, DAMP, YELLOW-BROWN.	5	1	6
22	CL	SILTY CLAY, SLIGHTLY PLASTIC, VERY STIFF, DAMP, YELLOW-BROWN.	5	1	6
U-15	ML SM	TOP 15": SANDY SILT, NONPLASTIC, 20-30% FINE SAND, YELLOW-BROWN. BOTTOM 2": SAND, UNIFORM, FINE, 5-15% NONPLASTIC FINES, DRY, LIGHT BROWN.	6	0	6
60	SM	SAND, UNIFORM, FINE, 30-35% NONPLASTIC FINES, VERY DENSE, DRY, LIGHT BROWN, LAYERED, TRACE OF CLAY AT BOTTOM.	8	1	9



SBT 0 5 10



SOIL BEHAVIOR TYPE

SENSITIVE FINES
ORGANIC SOIL
CLAY
SILTY CLAY
CLAYEY SILT
SILT
SANDY SILT
SILTY SAND/SAND
SAND
GRAVELLY SAND

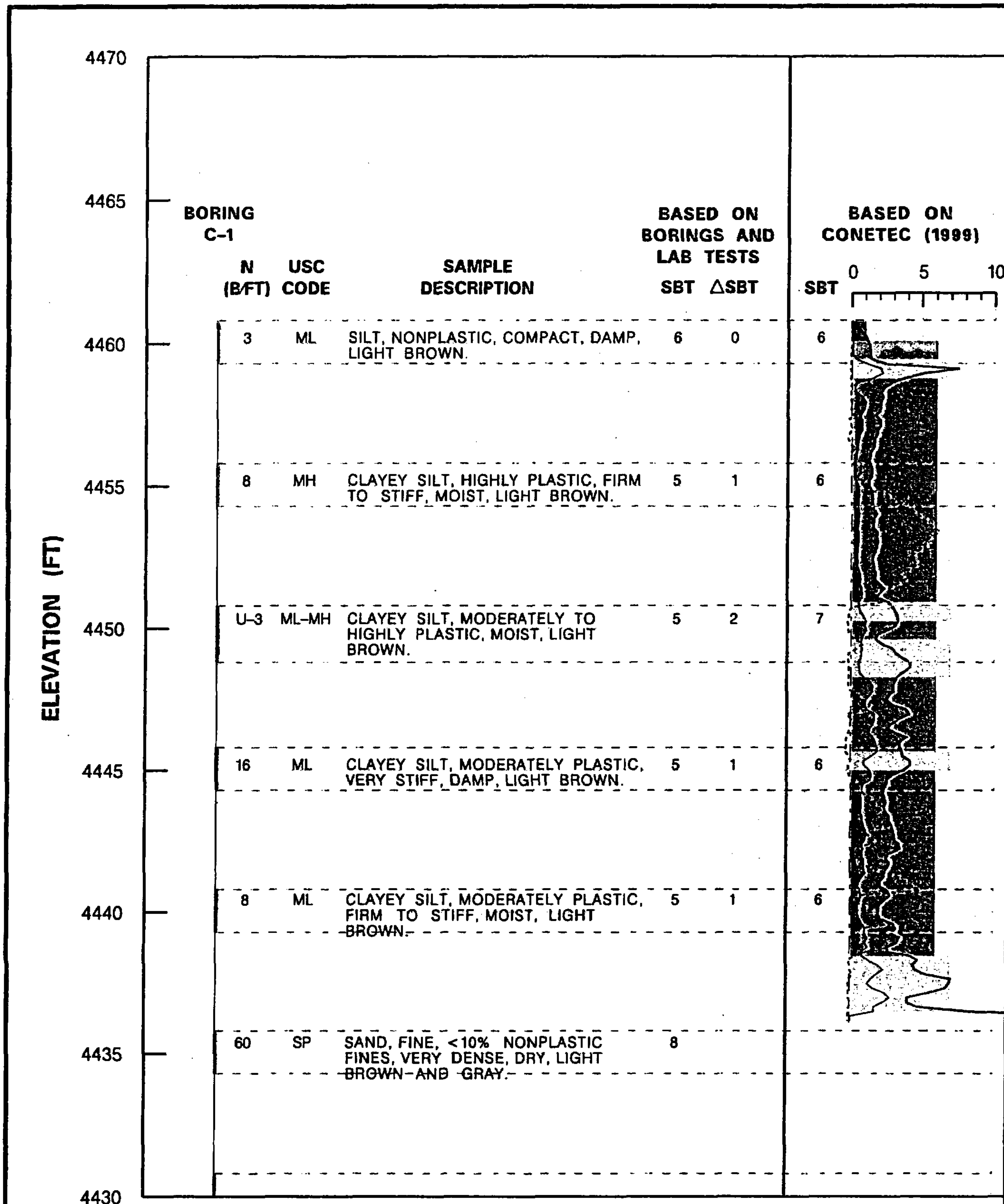
Figure 2.6-30

COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING CTB-4 AND CPT-37 SHEET 4 OF 6

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

08-DEC-1999 15:45

g:\05996\gsk\a\78\0.dgn



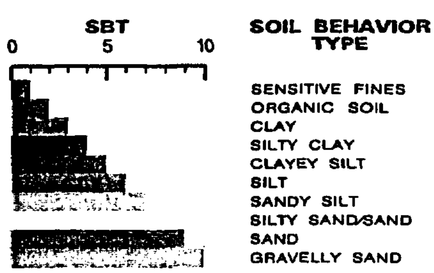


Figure 2.6-30

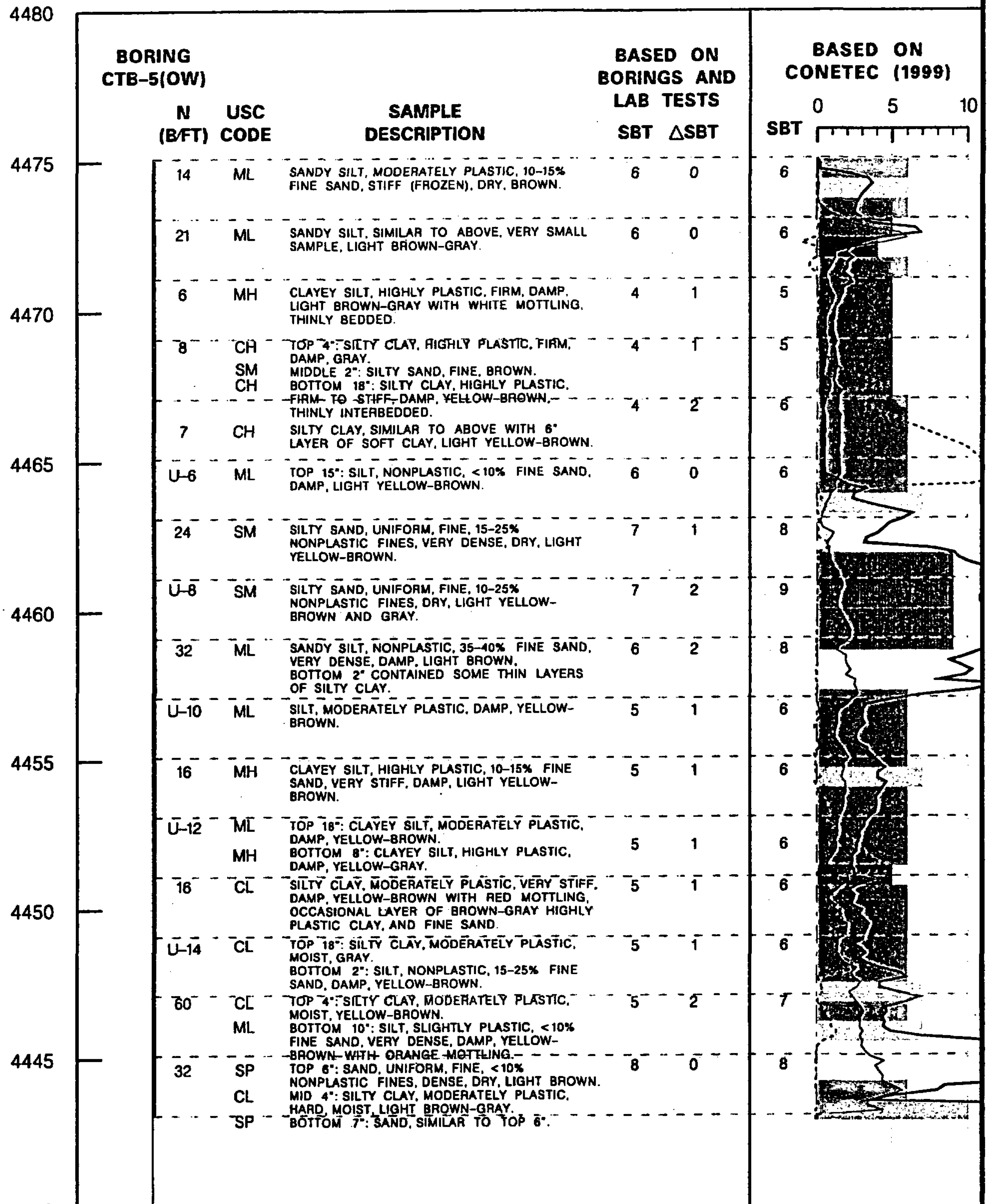
COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING C-1 AND CPT-39 SHEET 5 OF 6

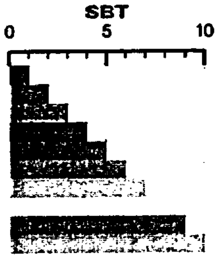
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\a\790.dgn 08-DEC-1999 15:45

ELEVATION (FT)

BORING CTB-5(OW) N (B/FT)	USC CODE	SAMPLE DESCRIPTION	BASED ON BORINGS AND LAB TESTS SBT	ΔSBT	BASED ON CONETEC (1999) SBT
14	ML	SANDY SILT, MODERATELY PLASTIC, 10-15% FINE SAND, STIFF (FROZEN), DRY, BROWN.	6	0	6
21	ML	SANDY SILT, SIMILAR TO ABOVE, VERY SMALL SAMPLE, LIGHT BROWN-GRAY.	6	0	6
6	MH	CLAYEY SILT, HIGHLY PLASTIC, FIRM, DAMP, LIGHT BROWN-GRAY WITH WHITE MOTTLING, THINLY BEDDED.	4	1	5
8	CH SM CH	TOP 4": SILTY CLAY, HIGHLY PLASTIC, FIRM, DAMP, GRAY. MIDDLE 2": SILTY SAND, FINE, BROWN. BOTTOM 18": SILTY CLAY, HIGHLY PLASTIC, FIRM TO STIFF, DAMP, YELLOW-BROWN, THINLY INTERBEDDED.	4	1	5
7	CH	SILTY CLAY, SIMILAR TO ABOVE WITH 6" LAYER OF SOFT CLAY, LIGHT YELLOW-BROWN.	4	2	6
U-6	ML	TOP 15": SILT, NONPLASTIC, <10% FINE SAND, DAMP, LIGHT YELLOW-BROWN.	6	0	6
24	SM	SILTY SAND, UNIFORM, FINE, 15-25% NONPLASTIC FINES, VERY DENSE, DRY, LIGHT YELLOW-BROWN.	7	1	8
U-8	SM	SILTY SAND, UNIFORM, FINE, 10-25% NONPLASTIC FINES, DRY, LIGHT YELLOW-BROWN AND GRAY.	7	2	9
32	ML	SANDY SILT, NONPLASTIC, 35-40% FINE SAND, VERY DENSE, DAMP, LIGHT BROWN. BOTTOM 2" CONTAINED SOME THIN LAYERS OF SILTY CLAY.	6	2	8
U-10	ML	SILT, MODERATELY PLASTIC, DAMP, YELLOW-BROWN.	5	1	6
16	MH	CLAYEY SILT, HIGHLY PLASTIC, 10-15% FINE SAND, VERY STIFF, DAMP, LIGHT YELLOW-BROWN.	5	1	6
U-12	ML MH	TOP 18": CLAYEY SILT, MODERATELY PLASTIC, DAMP, YELLOW-BROWN. BOTTOM 8": CLAYEY SILT, HIGHLY PLASTIC, DAMP, YELLOW-GRAY.	5	1	6
16	CL	SILTY CLAY, MODERATELY PLASTIC, VERY STIFF, DAMP, YELLOW-BROWN WITH RED MOTTLING, OCCASIONAL LAYER OF BROWN-GRAY HIGHLY PLASTIC CLAY, AND FINE SAND.	5	1	6
U-14	CL	TOP 18": SILTY CLAY, MODERATELY PLASTIC, MOIST, GRAY. BOTTOM 2": SILT, NONPLASTIC, 15-25% FINE SAND, DAMP, YELLOW-BROWN.	5	1	6
60	CL ML	TOP 4": SILTY CLAY, MODERATELY PLASTIC, MOIST, YELLOW-BROWN. BOTTOM 10": SILT, SLIGHTLY PLASTIC, <10% FINE SAND, VERY DENSE, DAMP, YELLOW-BROWN WITH ORANGE MOTTLING.	5	2	7
32	SP CL SP	TOP 6": SAND, UNIFORM, FINE, <10% NONPLASTIC FINES, DENSE, DRY, LIGHT BROWN. MID 4": SILTY CLAY, MODERATELY PLASTIC, HARD, MOIST, LIGHT BROWN-GRAY. BOTTOM 7": SAND, SIMILAR TO TOP 6".	8	0	8





SOIL BEHAVIOR TYPE

SENSITIVE FINES
ORGANIC SOIL
CLAY
SILTY CLAY
CLAYEY SILT
SILT
SANDY SILT
SILTY SAND/SAND
SAND
GRAVELLY SAND

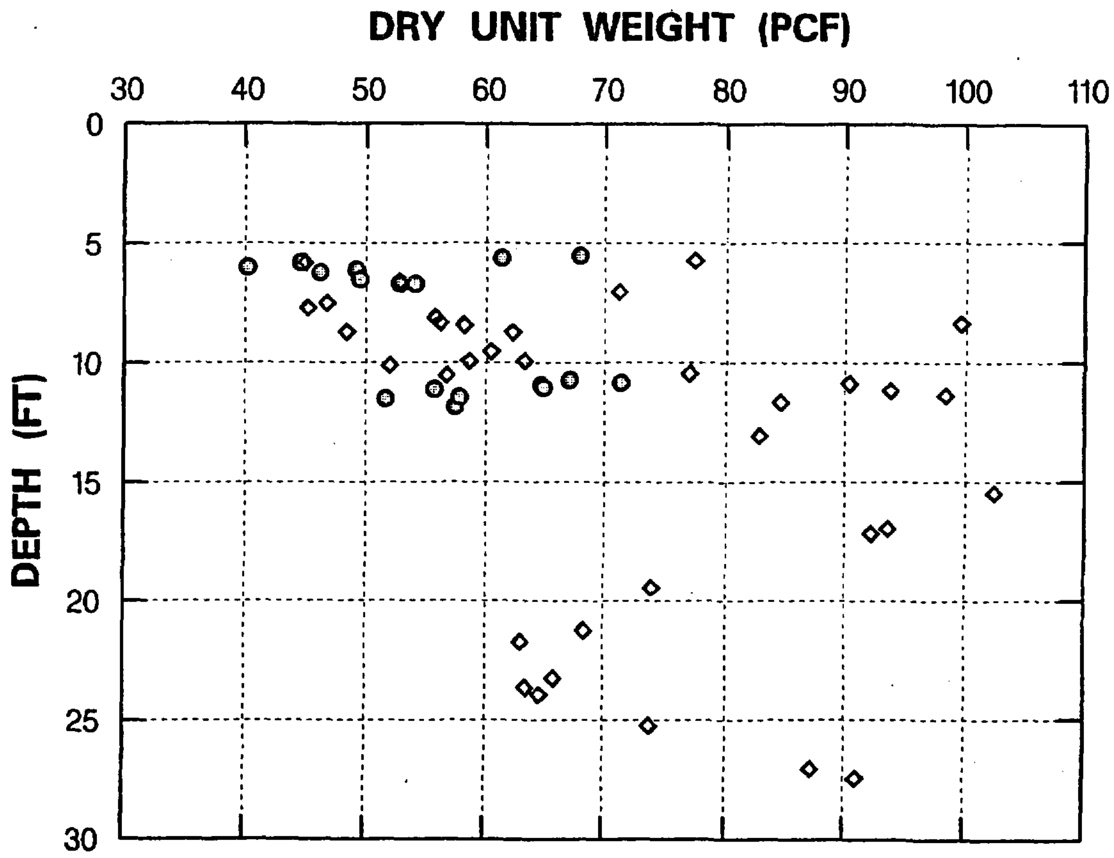
Figure 2.6-30

COMPARISON OF SOIL CLASSIFICATION BETWEEN BORING CTB-5(OW) AND CPT-38 SHEET 6 OF 6

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

08-DEC-1999 15:45

g:\05996\gsk\a\80\0.dgn



LEGEND:

- ⊚ PAD EMPLACEMENT AREA
- ◇ CANISTER TRANSFER BUILDING AREA

Figure 2.6-31

DRY DENSITIES OF SUBSURFACE SOILS AT THE SITE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

g:\05996\gsk\a\81\0.dgn 04-APR-2000 16:52

APPENDIX 2A

GEOTECHNICAL DATA

Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

Prepared for:

Private Fuel Storage Facility
Private Fuel Storage, LLC

Prepared by: RP Gillespie by NTG / Alan C Smith 6/11/97
Date

Reviewed by: Paul J. Trudeau 6/11/97
Date

Independent Review by: Paul J. Trudeau 6/11/97
Date

Approved by: Mani V. George 6/11/97
Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
Copyright 1997

CHAPTER 2

APPENDIX 2A

GEOTECHNICAL DATA

TABLE OF CONTENTS

Generalized Subsurface Profile

The subsurface profile at the site was investigated with a series of exploratory borings up to 100 ft deep, as well as seismic refraction (S and P-wave) and reflection surveys. These borings were drilled in accordance with the requirements of SWEC, 1996A & B, and Standard Penetration Test (SPT) samples were obtained at 5 ft intervals. The locations of these borings and the geophysical survey lines are shown in Figure 1, and logs of these borings are presented in Attachment 1.

Based on these borings, the generalized subsurface profile, which is shown in Figure 2, consists of three layers. The uppermost layer extends to a depth of between 25 and 35 ft and is mainly interlayered silt, silty clay, and clayey silt. The clayey silts and silty clays are commonly slightly to moderately plastic, with some being highly plastic. SPT N-values are mostly between 8 and 20 blows per ft, indicating "stiff" or "medium dense" materials. Most samples were dry or damp.

A distinct change in material occurs at about 25 to 35 ft, depending upon location at the site. SPT N-values commonly exceed 100 blows per ft, and refusal (>100 blows per 6 in.) conditions are often encountered. The upper 25 to 30 ft consists of very dense, dry, fine sand. This layer is underlain by very dense silt. Thin layers of fine gravel and coarse sand also are evident, indicating a near-shore environment of deposition. A few clayey zones were encountered, but they had no apparent effect on the blow counts.

The groundwater table was not encountered in the borings, which were completed to 100 ft depth; therefore, the groundwater table is greater than 100 ft below existing grade.

These borings did not encounter bedrock; however, interpretation of the seismic reflection survey data indicates that the depth to bedrock is between 520 ft and 880 ft below the surface at the site (Geosphere, 1997).

Geotechnical laboratory tests, performed on samples obtained in these borings, are described in detail in Attachment 2. These tests included determination of water content, Atterberg Limits, percent fines, and specific gravity. Unconsolidated-undrained triaxial compression tests and consolidation tests also were performed. See Attachment 2 for additional information regarding these laboratory tests.

Hickman Knolls

Recent personal communication with Dr. James Baer at Brigham Young University introduced the possibility that Hickman Knolls may be a detached slide block, which is "floating" in the valley-fill sediments. A gravity survey performed by him at the north end of Skull Valley revealed that several rock knobs are apparently detached from the bedrock surface and appear to be surrounded on all sides by unconsolidated sediments. He speculated that Hickman Knolls could be a similar structure.

Summary

Results of the test boring program, the geotechnical laboratory testing program, and the geophysical survey program formed the bases for the geotechnical design criteria and the earthquake determination analysis. For results of these analyses and foundation recommendations, see Section 4, "Geotechnical Design Criteria," of the PFSF Project Design Criteria Manual (SWEC, 1997), and Geomatrix and Lettis (1997).

References

Geomatrix and Lettis, 1997, "Deterministic Earthquake Ground Motions Analysis, Private Fuel Storage Facility, Skull Valley, Utah," PFSF Project Report No. 0599601-G(PO5)-1, Revision 0, prepared by Geomatrix Consultants, Inc and William Lettis & Associates, Inc, San Francisco, California, March 1997.

Geosphere, 1997, "Seismic Survey of the Private Fuel Storage Facility, Skull Valley, Utah," PFSF Project Report No. 0599601-G(PO9)-1, Revision 0, prepared by Geosphere Midwest, Inc, Midland, Michigan, February 1997.

SWEC, 1996A, "Geotechnical Requirements Document," PFSF Project Report No. 05996.01-G(B)-01, Revision 0, SWEC Project No. 05996.01, prepared by Stone & Webster Engineering Corporation, Boston, Massachusetts, 1996.

SWEC, 1996B, "ESSOW for Test Borings and Laboratory Testing," PFSF Project ESSOW No. 05996.01-G001, prepared by Stone & Webster Engineering Corporation, Boston, Massachusetts, September 24, 1996.

SWEC, 1997, "Design Criteria Manual, Private Fuel Storage Facility," prepared by Stone & Webster Engineering Corporation, Denver, Colorado, June 1997.

Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

ATTACHMENT 1

BORING LOGS

Private Fuel Storage Facility
Skull Valley
Private Fuel Storage, LLC

Responsible Engineer	RP Gillespie by NTG	6/11/97 Date
Reviewer	Paul J. Trudeau (I)	6/11/97 Date
Approved	Nani V. Sanger	6/11/97 Date

(I) = Independent Review

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
Copyright 1997

APPENDIX 2A

ATTACHMENT 1

BORING LOGS

APPENDIX 2A

ATTACHMENT 1

BORING LOGS

TABLE OF CONTENTS

Stone & Webster Engineering Corporation	BORING LOG	Boring 1 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C Smith
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/17/96 - 10/17/96
Coordinates: N 7317572.25 E 1281965.96 — Ground Elevation: 4489.9 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc. — Driller: W. Westbrook — Rig Type: Mobile B-80

Methods: — Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2" O.D. Split Spoon, SPT, 18" long
Drilling Rock:

Comments: No bedrock or groundwater encountered. Backfilled with soil to surface, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-4-6 (4.0")	10	ML	SILT, nonplastic, dense, damp, light brown.
4485	5	S	2	5-13-36 (18.0")	49	ML SP	Top 12": SILT, nonplastic, very dense, moist, brown, occasional 1/4" layer of silty clay, moderately plastic. Bottom 6": SAND, fine, <10% nonplastic fines, very dense, dry, light brown.
4480	10	S	3	13-30-35 (18.0")	65	ML	SILT, nonplastic, 5-15% fine sand, very dense, dry, light brown.
4475	15	S	4	8-9-14 (15.0")	23	ML	Sandy SILT, nonplastic, 10-20% fine sand, dense, moist, brown, 4" section in middle, slightly plastic.
4470	20	S	5	7-14-13 (15.0")	27	ML	Top 11": SILT, stratified, nonplastic, <10% fine sand, dense, moist, mottled light gray and orange brown. Bottom 4": SILT, slightly plastic, dense, moist, light gray and orange brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Dudson	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring 1 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4465	25	S	6	15-28-50 (14.0")	78	SM	SAND, uniform, fine, 10-20% nonplastic fines, very dense, damp, light brown.
4460	30	S	7	9-9-13 (14.0")	22	ML	SILT, nonplastic, compact, dry, light brown.
4455	35	S	8	38-50-50/4" (5.0")	100+	ML	SILT, nonplastic, <10% fine sand, very dense, dry, light brown, trace of coarse sand.
4450	40	S	9	35-55 (10.0")	100+	GM	GRAVEL, fine, up to 1/2", 20-30% sand, 10-20% nonplastic fines, very dense, dry, light brown.
4445	45	S	10	50/5" (5.0")	100+	GM	Silty GRAVEL, up to 1 1/2", 10-20% sand, 30-40% nonplastic fines, very dense, dry, light gray, (gravel mostly 2 pieces of 1 1/2" diameter.)
4440	50	S	11	60-65-62 (10.0")	127	GM	silty GRAVEL, up to 1 1/2", subangular to subrounded, 20-30% sand, 20-30% nonplastic fines, very dense, dry, light brown.
							BOTTOM OF BORING AT 51.5 FEET
4435	55						
4430	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: [signature] Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring 2 J.O. 05996.01 Sheet 1 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** — Logged by: **R. Gillespie**
Client: **Private Fuel Storage, L.L.C.** — Date Start - Finish: **10/12/96 - 10/12/96**
Coordinates: **N 7317598.43 E 1280074.22** — Ground Elevation: **4488.4 ft**
Groundwater Depth: **N/A** ft — Depth to Bedrock: **N/A** ft — Total Depth Drilled: **49.3 ft**
Contractor: **Earthcore, Inc.** — Driller: **Strickland** — Rig Type: **Acker Soil Sentry**

Methods: — Casing Used:
Drilling Soil: **3 1/4" Hollow Stem Augers 6 1/4" O.D**
Sampling Soil: **2.0" O.D. Split Spoon, 140 lb hammer, 30" fall.**
Drilling Rock:

Comments: **No groundwater or bedrock encountered.**

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-2-2 (14.0")	4	CL	Silty CLAY, slightly plastic, soft to firm, damp, light yellow-brown, very thinly layered.
4485							
	5	S	2	2-2-2 (19.0")	4	CL	Silty CLAY, similar to above, mottles of white calcareous material (?)
4480							
	10	S	3	5-9-12 (18.0")	21	SP	Silty SAND, uniform, fine, 5-8% nonplastic fines, dense, dry, light red-brown at top to light brown at bottom. Calcareous (?)
4475							
	15	S	4	5-6-8 (16.0")	14	ML	SILT, nonplastic, compact, dry, light yellow-brown, trace fine sand, very thin layers.
4470							
	20	S	5	13-15-23 (16.0")	38	ML	SILT, nonplastic, very dense, dry, light brown-gray, trace clay in very thin layers, minor red-brown mottling.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Dudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring 2 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4465							
	25	S	6	12-17-19 (18.0")	36	ML SP	Top 14": SILT, nonplastic, very dense, dry, very light brown to gray-white. Bottom 4": Silty SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, light brown.
4460							
	30	S	7	15-34-61 (15.0")	95	ML	Top 11": SILT, nonplastic, very dense, dry, light brown, trace subrounded medium gravel. Bottom 3": SILT, slightly plastic, hard, dry, light gray-white, trace medium to coarse gravel.
4455							
	35	S	8	45-40-30 (12.0")	70	GC	Clayey GRAVEL, poorly graded, coarse to fine, 15-25% slightly plastic fines, very dense, dry, whitish-gray, most gravel with caliche coating.
4450							
	40	S	9	100/4" (5.0")	100+	ML	Gravelly SILT, slightly plastic, 10-20% coarse sand to fine gravel, very dense, dry, very light gray, caliche coating.
4445							
	45	S	10	15-32-55 (15.0")	87	CL	Silty CLAY, slightly plastic, trace fine sand, hard, dry, olive-gray, desication cracks.
4440		S	11	70-89-100/4" (18.0")	100+	ML	Clayey SILT, slightly plastic, very dense, dry, light olive-gray. Interlayers of clay, moderately plastic.
	50						BOTTOM OF BORING AT 49.3 FEET
4435							
	55						
4430							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: R.J. Hudson (signature) — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-1 J.O. 05996.01 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/17/96 - 10/22/96

Coordinates: N 7321702.84 E 1280027.61 — Ground Elevation: 4459.0 ft

Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 101 ft

Contractor: Earthcore, Inc. — Driller: W. Westbrook — Rig Type: Mobile B-80

Methods: — Casing Used:

Drilling Soil: 3 1/4" I.D. Hollow Stem Augers to 65'. 65-100' roller cone bit with compressed air.

Sampling Soil: 2.0" O.D. Split-barrel Spoon, 18" long.

Drilling Rock:

Comments: No rock or groundwater encountered. Backfilled with soil to surface, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	6-13-10 (4.0")	23	ML	SILT, nonplastic, very stiff, dry, light brown, bottom 1" clayey.
4455	5	S	2	4-5-8 (12.0")	13	MH	Clayey SILT, highly plastic, numerous silt partings, stiff, damp, light brown and gray.
4450	10	S	3	6-9-13 (13.0")	22	ML	SILT, slightly plastic, very stiff, damp, brown, stratified.
4445	15	S	4	7-9-10 (15.0")	19	ML	SILT, moderately plastic, very stiff, damp, brown, stratified.
4440	20	S	5	5-6-7 (18.0")	13	MH	Clayey SILT, highly plastic, stiff, moist, light gray.

Legend/Notes

- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-1 J.O. 05996.01 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4435	25	S	6	7-15-21 (12.0")	36	ML	SILT, nonplastic, very dense, dry, light gray, occasional 2-4 mm thick layer of clayey silt; bottom 4" sandy silt, 10-20% fine sand.
4430	30	S	7	20-55-94 (10.0")	149	SP	SAND, fine, <5% nonplastic fines, very dense, dry, some stratification.
4425	35	S	8	20-54-60 (11.0")	114	SP	SAND, similar to S-7, except bottom 3" contained few coarse sand pieces.
4420	40	S	9	20-35-35 (12.0")	70	SP	SAND, similar to S-7, except bottom 2" sandy silt, nonplastic, 10-20% fine sand.
4415	45	S	10	20-35-45 (13.0")	80	SP	SAND, similar to S-7, except two 1/2" thick layers of silty clay.
4410	50	S	11	20-50-70 (12.0")	120	SP	SAND, similar to S-7, except occasional piece of fine gravel and 1" layer with trace coarse sand.
4405	55	S	12	45-60-50 (12.0")	110	SW ML	Top 6": Gravelly SAND, coarse to fine, 10-20% fine gravel, <5% nonplastic fines, very dense, dry, light brown. Bottom 6": SILT, nonplastic, trace coarse sand, 20-30% fine sand, very dense, dry, light brown.
4400	60	S	13	27-57-45 (14.0")	102	CL	CLAY, moderately plastic, few pieces of fine gravel, trace sand, hard, damp, light brown.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature]   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-1 J.O. 05996.01 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4395							
	65	S	14	10-20-34 (14.0")	54	ML	Clayey SILT, slightly plastic, 10-20% mostly fine sand, 5-10% fine gravel, dense, damp, brown. (At 66', augers advancing very slow.)
		S	15	42-100 (9.0")	100+	ML	Clayey SILT, similar to S-14, except very dense and 15-25% gravel up to 1".
4390							
	70	S	16	100+ (10.0")	100+	ML	SILT, nonplastic, <10% fine sand, very dense, dry, light brown.
							Driller took out 65' of 3" auger, replaced with 4 1/4" casing, advanced boring using compressed air and rollerbit.
4385							
	75	S	17	45-100 (10.0")	100+	ML	Sandy SILT, nonplastic, 10-20% fine sand, very dense, dry, light brown, contained two 1/2" layers of clayey silt, slightly plastic.
4380							
	80	S	18	100/4"	100+	ML	No recovery, second attempt recovered 2". Sandy SILT, similar to S-17.
4375							
	85	S	19	100/4" (4.0")	100+	SM	Silty SAND, fine, uniform, 20-30% nonplastic fines, very dense, dry, light gray.
4370							
	90	S	20	50-100 (10.0")	100+	SM	Silty SAND, similar to S-19.
4365							
	95	S	21	100/6" (6.0")	100+	ML	SILT, nonplastic, <10% fine sand, very dense, dry, light brown.
4360							
	100	S	22	50-100 (12.0")	100+	CL-CH	Silty CLAY, moderately to highly plastic, hard, slightly damp, brown.
							BOTTOM OF BORING AT 101 FEET

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-2 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/28/96 - 10/28/96

Coordinates: N 7321062.28 E 1279960.29 — Ground Elevation: 4464.9 ft

Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 51.0 ft

Contractor: Earthcore, Inc. — Driller: W. Westbrook — Rig Type: Mobile B-80

Methods: — Casing Used:

Drilling Soil: 3 1/4" I.D. Hollow Stem Augers

Sampling Soil: 2.0" O.D. Split Spoon, 24" long. 3" Shelby Sampler, 30" long.

Drilling Rock:

Comments: No bedrock or groundwater encountered, backfilled with soil to ground surface, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1/12"-1 (3.0")	1	ML	SILT, slightly plastic, very loose, damp, light brown.
4460	5	U	2	PUSH (25.0")		ML MH	Top 10": SILT, moderately plastic, moist, light brown. Bottom 15": Clayey SILT, highly plastic, moist, light brown.
4455	10	S	3	5-5-6 (14.0")	11	ML	Sandy SILT, slightly plastic, 20-30% fine sand, loose, damp, light brown.
4450	15	S	4	7-7-7 (13.0")	14	ML	Clayey SILT, moderately plastic, stiff, damp, light brown.
4445	20	S	5	5-6-11 (14.0")	17	ML	Clayey SILT, moderately plastic, very stiff, moist, light brown.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Andrean	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-2 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4440	25	S	6	5-6-10 (15.0")	16	ML	Clayey SILT, similar to S-5.
4435	30	S	7	13-25-26 (13.0")	51	SP	SAND, fine, <10% nonplastic fines, very dense, dry, light brown, contained 1 1/2" layer of clayey silt, slightly plastic.
4430	35	S	8	25-75-100/5" (12.0")	100+	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown, trace coarse sand.
4425	40	S	9	35-100/4" (8.0")	100+	SP	SAND, similar to S-8.
4420	45	S	10	30-50-55 (12.0")	105	SW	Gravelly SAND, coarse to fine, 20-30% fine gravel, <5% nonplastic fines, very dense, dry, light brown.
4415	50	S	11	35-100 (9.0")	100+	SP	Top 5": SAND, fine, <5% nonplastic fines, very dense, dry, light brown, trace coarse sand. Bottom 4": Gravelly SAND, coarse to fine, 30-40% fine gravel, <5% nonplastic fines, very dense, dry, light brown.
							BOTTOM OF BORING AT 51.0 FEET
4410	55						
4405	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: [signature] — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 10/24/96 - 10/24/96
Coordinates: N 7320471.57 E 1279918.87 | Ground Elevation: 4469.5 ft
Groundwater Depth: N/A ft | Depth to Bedrock: N/A ft | Total Depth Drilled: 50.9 ft
Contractor: Earthcore, Inc. | Driller: Strickland | Rig Type: Acker Soil Sentry

Methods: | Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-2-2 (11.0")	4	ML	Clayey SILT, slightly plastic, compact, dry to damp, light brown.
4465	5	S	2	4-4-5 (15.0")	9	CL-CH	Silty CLAY, moderately to highly plastic, stiff, damp, green-gray with white and dark brown mottling throughout.
4460	10	S	3	4-5-10 (14.0")	15	MH	Clayey SILT, highly plastic, very thinly layered, stiff to very stiff, moist, yellow-brown.
4455	15	S	4	6-7-8 (13.0")	15	ML	Clayey SILT, moderately plastic, 5-15% fine sand, stiff to very stiff, damp, light yellow-brown, thinly layered.
4450	20	S	5	6-8-12 (13.0")	20	ML	Clayey SILT, similar to S-4, except very stiff.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

Sample Type:
S = 2" OD Split Spoon
U = 3" OD Thin-Walled Tube

Approved	Date
(signature)	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-3 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445	25	S	6	8-12-18 (16.0")	30	ML	Clayey SILT, slightly plastic, dense, damp, yellow-brown, thinly layered, occasional thin clay layer.
4440	30	S	7	12-14-20 (14.0")	34	ML	SILT, nonplastic, dense, dry, light gray, occasional layer of clayey silt, slightly plastic, some orange mottling.
4435	35	S	8	28-50-64 (18.0")	114	SP	SAND, uniform, fine, trace coarse sand and fine gravel, very dense, dry, light gray, 3-5% nonplastic fines.
4430	40	S	9	24-75-100 (16.0")	100+	SP	SAND, mostly uniform, fine, 3-5% nonplastic fines, few layers of medium to coarse sand, very dense, dry, light brown.
4425	45	S	10	17-30-52 (16.0")	82	CL ML	Top 4": Silty CLAY, slightly plastic, very dense, damp, brown. Bottom 12": Sandy SILT, nonplastic, 15-20% fine sand, very dense, damp, light brown, slightly cemented.
4420	50	S	11	38-100/5" (12.0")	100+	ML	Clayey SILT, slightly plastic, very dense, damp, slightly cemented, very light brown. BOTTOM OF BORING AT 50.9 FEET
4415	55						
4410	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P.J. Trudeau Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-4 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/24/96 - 10/24/96
Coordinates: N 7319880.79 E 1279861.82 — Ground Elevation: 4472.2 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 36.5 ft
Contractor: Earthcore, Inc. — Driller: Strickland — Rig Type: Acker Soil Sentry

Methods: — Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-6-8 (6.0")	14	ML	Clayey SILT, slightly plastic, very dense, dry, light brown.
4470							
	5	S	2	4-8-10 (14.0")	18	MH	Clayey SILT, highly plastic, very stiff, damp, light green-gray, very thinly layered, white and orange mottling.
4465							
	10	S	3	6-6-7 (12.0")	13	ML	Top 6": SILT, nonplastic, compact, damp, light brown. Bottom 6": SILT, nonplastic, compact, dry, light gray.
4460							
	15	S	4	6-8-10 (14.0")	18	CL	Silty CLAY, slightly to moderately plastic, very stiff, damp, light yellow-brown, thinly layered, occasional clay layer.
4455							
	20	S	5	6-6-6 (18.0")	12	CL MH	Top 10": Silty CLAY, similar to S-4. Bottom 8": Clayey SILT, highly plastic, stiff, damp, light yellow-gray with white mottling, thinly layered.

Legend/Notes

- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P. J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring A-4 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	6	4-8-12 (12.0")	20	CL	Silty CLAY, slightly plastic, very stiff, damp, brown-gray with orange mottling.
4445							
	30	S	7	15-20-30 (16.0")	50	SM	Silty SAND, fine, 10-20% nonplastic fines, very dense, dry, light gray.
4440							
	35	S	8	17-42-51 (18.0")	93	ML	Sandy SILT, nonplastic, 30-40% fine sand, very dense, dry, light brown.
4435							BOTTOM OF BORING AT 36.5 FEET
	40						
4430							
	45						
4425							
	50						
4420							
	55						
4415							
	60						
4410							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-1 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/18/96 - 10/18/96
Coordinates: N 7318603.98 E 1284062.70
Ground Elevation: 4487.5 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 31.5 ft
Contractor: Earthcore, Inc.
Driller: Strickland
Rig Type: Acker Soil Sentry

Methods:
Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2" O.D. Split Spoon, 140# hammer.
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-4-5 (9.0")	9	ML	Clayey SILT, slightly plastic, compact, dry, light brown with white mottling and roots.
4485							
	5	S	2	8-11-12 (12.0")	23	SP	SAND, uniform, fine, <3% nonplastic fines, very dense, dry, light brown.
4480							
	10	S	3	7-8-12 (11.0")	20	SP	SAND, uniform, fine, 3-5% nonplastic fines, dense, dry, light brown.
4475							
	15	S	4	7-7-10 (15.0")	17	ML,CL	Interlayered SILT and CLAY, slightly plastic, trace fine sand, very stiff, damp, yellow-gray and yellow-brown.
4470							
	20	S	5	6-9-10 (18.0")	19	CL	Silty CLAY, slightly plastic, very stiff, damp, yellow-gray, very thinly layered, trace white mottling.

Legend/Notes

- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-1 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4465							
	25	S	6	10-15-14 (16.0")	29	CL ML	Top 12": Silty CLAY, moderately to slightly plastic, very stiff, damp, yellow-gray and yellow-brown. Bottom 4": SILT, nonplastic, trace fine sand, dense, slightly damp, light gray.
4460							
	30	S	7	17-27-24 (16.0")	51	ML	SILT and Clayey SILT, interlayered, non to slightly plastic, very dense, slightly damp, some fine sand, light yellow-gray. BOTTOM OF BORING AT 31.5 FEET
4455							
	35						
4450							
	40						
4445							
	45						
4440							
	50						
4435							
	55						
4430							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-2 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/18/96 - 10/18/96

Coordinates: N 7318592.99 E 1286161.25 — Ground Elevation: 4494.9 ft

Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 31.5 ft

Contractor: Earthcore, Inc. — Driller: Strickland — Rig Type: Acker Soil Sentry

Methods:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Casing Used:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-2-4 (8.0")	6	ML	SILT, non to slightly plastic, compact, damp, light brown, trace roots.
4490	5	S	2	4-10-12 (14.0")	22	SP	SAND, uniform, fine, trace silt, very dense, slightly damp, light brown with orange streaks.
4485	10	S	3	7-8-10 (15.0")	18	SP	SAND, similar to S-2.
4480	15	S	4	9-9-14 (14.0")	23	ML	Sandy SILT, nonplastic, fine sand, dense, dry, red-brown, trace of clay in pockets and seams, moderately plastic, gray.
4475	20	S	5	17-20-30 (13.0")	50	SP	SAND, uniform, fine, 5-8% nonplastic silt, very dense, dry, light brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
P. J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-2 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4470	25	S	6	11-14-11 (16.0")	25	ML	SILT, nonplastic, dense, light gray with orange mottling. Occasional thin layer of silty clay, slightly plastic, light gray, dry to slightly damp.
4465	30	S	7	2-10-12 (18.0")	22	CL	Silty CLAY, slightly to moderately plastic, very stiff, damp, light gray, very thinly layered, some orange-brown stained layers, trace of perched water. BOTTOM OF BORING AT 31.5 FEET
4460	35						
4455	40						
4450	45						
4445	50						
4440	55						
4435	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/21/96 - 10/21/96
Coordinates: N 7318625.48 E 1288244.35 — Ground Elevation: 4501.4 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 31.5 ft
Contractor: Earthcore, Inc. — Driller: Strickland — Rig Type: Acker Soil Sentry

Methods: — Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-3-4 (8.0")	7	ML	SILT, slightly plastic, compact, dry, light brown.
4500							
4495	5	S	2	4-4-7 (9.0")	11	CL	Silty CLAY, slightly plastic, stiff, dry, light brown, some roots.
4490	10	S	3	4-6-14 (10.0")	20	ML	SILT, slightly plastic, dense, dry, thinly layered, very light gray and light brown. (calichified?)
4485	15	S	4	7-12-13 (8.0")	25	ML	Clayey SILT, slightly plastic, dense, dry, very thinly layered, very light gray, (calichified?)
4480	20	S	5	20-36-57 (16.0")	93	SP-SM	Silty SAND, uniform, fine, 8-12% nonplastic fines, very dense, dry, light brown. Occasional layer of gravel and pebbles intermixed with sand. Gravel to 3/4" max., subangular to subrounded. (Fractured by spoon)

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
R.F. Tindeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-3 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
	25	S	6	100/5" (8.0")	100+	GM	Silty GRAVEL, 20-30% nonplastic fines, coarse to fine gravel, subangular to subrounded gravel to 1.5" max., very dense, dry, light brown.
4475							
	30	S	7	7-10-14 (14.0")	24	GM CL	Top 4": Silty GRAVEL, similar to S-6. Bot. 10": Silty CLAY, slightly to moderately plastic, very stiff, dry, light gray.
4470							BOTTOM OF BORING AT 31.5 FEET
	35						
4465							
	40						
4460							
	45						
4455							
	50						
4450							
	55						
4445							
	60						
4440							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-4 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7318616.84 E 1290320.39
Groundwater Depth: N/A ft
Contractor: Earthcore, Inc.

Depth to Bedrock: N/A ft
Driller: Strickland

Logged by: R. Gillespie
Date Start - Finish: 10/21/96 - 10/21/96
Ground Elevation: 4527.3 ft
Total Depth Drilled: 25.0 ft
Rig Type: Acker Soil Sentry

Methods:
Drilling Soil: 3 1/4" I.D Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Casing Used:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-2-3 (6.0")	5	CL	Silty CLAY, slightly plastic, firm, dry, light brown.
4525							
	5	S	2	2-2-3 (5.0")	5	ML	Clayey SILT, moderately plastic, firm, dry, light gray-yellow.
4520							
	10	S	3	7-8-11 (9.0")	19	ML	Sandy SILT, nonplastic, 10-15% fine sand, dense, dry, brown.
4515							
	15	S	4	8-12-17 (10.0")	29	ML	Clayey SILT, slightly plastic, very dense, dry, brown, trace fine sand.
4510							
	20	S	5	100 (7.0")	100+	GP	Sandy GRAVEL, poorly graded, 10-15% slightly plastic fines, coarse to fine sand, subrounded to subangular gravel to 1.5", very dense, dry, light brown. Gravel to 3-4" upon augers.

Legend/Notes
- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
P. J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-4 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4505							
	25	S	6	50/0"	100+		No recovery. Very hard augering through gravel. BOTTOM OF BORING AT 25 FEET
4500							
	30						
4495							
	35						
4490							
	40						
4485							
	45						
4480							
	50						
4475							
	55						
4470							
	60						
4465							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: R.J. Trudeau   Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring AR-5 J.O. 05996.01 Sheet 1 of 1

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/21/96 - 10/22/96
Coordinates: N 7318601.97 E 1292299.14 — Ground Elevation: 4575.3 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 14 ft
Contractor: Earthcore, Inc. — Driller: Strickland — Rig Type: Acker Soil Sentry

Methods: Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-2-4 (8.0")	6	ML	SILT, nonplastic, compact, dry, brown, trace fine sand and gravel.
4570	5	S	2	7-10-47 (16.0")	57	ML GM	Top 8": Sandy SILT, slightly plastic, 10-15% fine sand, very dense, dry, very light tan, trace gravel. Bottom 8": Silty GRAVEL, 10-20% nonplastic fines, coarse to fine sand, angular to subrounded gravel to 2" max., very dense, dry, brown.
4565	10	S	3	24-34-63 (6.0")	97	GM	Silty GRAVEL, similar to S-2 bot. 8". (3.0" spoon used)
		S	4	100/1"	100+		No recovery BOTTOM OF BORING AT 14 FEET
4560	15						
4555	20						

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
R.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-1 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: A.C. Smith
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/28/96 - 10/28/96
Coordinates: N 7321739.31 E 1280619.03
Ground Elevation: 4459.8 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc.
Driller: W. Westbrook
Rig Type: Mobile B-80

Methods:
Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, 24" long. 3" O.D. Shelby Sampler, 30" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled to ground surface with soil, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-4-9 (8.0")	13	ML	SILT, nonplastic, very dense, damp, light brown, bottom 1" slightly plastic.
4455	5	U	2	PUSH (26.5")		MH	Clayey SILT, highly plastic, moist, light gray and brown.
4450	10	S	3	7-7-8 (13.0")	15	ML	Clayey SILT, moderately plastic, stiff to very stiff, damp, light brown.
4445	15	S	4	6-10-10 (16.0")	20	ML	Clayey SILT, moderately plastic, very stiff, moist, light brown.
4440	20	S	5	4-5-7 (16.0")	12	ML-MH	Clayey SILT, moderately to highly plastic, stiff, moist, light brown (pale green?)

Legend/Notes
- Datum is MSL - NGVD29.
- ⊽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-1 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4435	25	S	6	8-15-30 (12.0")	45	ML SM	Top 6": SILT, nonplastic, very dense, damp, light brown. Bottom 6": Silty SAND, fine, 20-30% nonplastic fines, very dense, nearly dry, light brown.
4430	30	S	7	12-35-35 (14.0")	70	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown.
4425	35	S	8	20-35-55 (13.0")	90	SP	SAND, similar to S-7, 1" layer with some coarse sand.
4420	40	S	9	10-14-19 (11.0")	33	CL	Silty CLAY, moderately plastic, hard, damp, light gray.
4415	45	S	10	25-33-30 (15.0")	63	SW ML	Top 6": Gravelly SAND, coarse to fine, 20-30% mostly fine gravel, up to 1", <5% nonplastic fines, very dense, dry, light brown. Bottom 9": Sandy SILT, nonplastic, 20-30% fine sand, very dense, dry, yellow-brown.
4410	50	S	11	25-75-50 (12.0")	125	SP SM	Top 8": SAND, <5% nonplastic fines, very dense, dry, light brown. Bottom 4": Silty SAND, fine, 10-20% nonplastic fines, very dense, dry, yellow brown. BOTTOM OF BORING AT 51.5 FEET
4405	55						
4400	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-2 J.O. 05996.01 Sheet 1 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** — Logged by: **R. Gillespie**

Client: **Private Fuel Storage, L.L.C.** — Date Start - Finish: **10/28/96 - 10/28/96**

Coordinates: **N 7321074.91 E 1280621.39** — Ground Elevation: **4464.8 ft**

Groundwater Depth: **N/A** ft — Depth to Bedrock: **N/A** ft — Total Depth Drilled: **51.0** ft

Contractor: **Earthcore, Inc.** — Driller: **Strickland** — Rig Type: **Acker Soil Sentry**

Methods: — Casing Used:

Drilling Soil: **3 1/4" I.D. Hollow Stem Augers**

Sampling Soil: **2.0" O.D. Split Spoon, SPT. 3" Shelby Tube, 30" long.**

Drilling Rock:

Comments: **No groundwater or bedrock encountered**

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-2-2 (3.0")	4	ML	Clayey SILT, slightly plastic, loose, dry, light brown.
4460	5	S	2	2-2-3 (8.0")	5	CL	Silty CLAY, moderately plastic, firm, damp, light gray with white and orange-brown mottling.
		U	1	PUSH (25.0")		ML	Top: SILT, slightly plastic, damp, yellow-brown. Bottom: Clayey SILT, moderately plastic, damp, brown.
4455	10	S	3	4-6-7 (12.0")	13	ML	Clayey SILT, slightly to moderately plastic, 5-10% fine sand, stiff, damp, light brown with white mottling.
4450	15	S	4	3-7-9 (14.0")	16	ML	SILT, nonplastic, 5-10% fine sand, very stiff, damp, light yellow-brown and light gray.
4445	20	S	5	5-6-6 (18.0")	12	MH	Clayey SILT, highly plastic, stiff, damp, light yellow-brown and light gray.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
R.J. Hudson	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-2 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4440	25	S	6	6-7-8 (13.0")	15	CL	Silty CLAY, moderately plastic, stiff to very stiff, damp, light yellow and light gray.
4435	30	S	7	21-34-37 (18.0")	71	ML	Sandy SILT, nonplastic, 10-15% fine sand, very dense, dry, light gray.
4430	35	S	8	46-71-100/5" (18.0")	100+	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, light brown.
4425	40	S	9	16-32-36 (16.0")	68	SP	SAND, uniform, fine, 3-8% nonplastic fines, very dense, dry, light brown, trace fine gravel. Bot. 2" becomes gravelly silt and silty sand, slightly cemented.
4420	45	S	10	65-100/5" (7.0")	100+	GP	Sandy GRAVEL, poorly graded, 30-40% sand, mostly fine, 3-8% nonplastic fines, gravel to 1.0", very dense, dry, light brown.
4415	50	S	11	40-100 (14.0")	100+	GP	Sandy GRAVEL, similar to S-10, 20-30% sand, 8-12% nonplastic fines, slightly cemented.
							BOTTOM OF BORING AT 51.0 FEET
4410	55						
4405	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: R.J. Vrudoan (signature) — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/23/96 - 10/23/96
Coordinates: N 7320517.02 E 1280582.67 — Ground Elevation: 4468.2 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc. — Driller: Strickland — Rig Type: Acker Soil Sentry

Methods: — Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT. 3" Shelby Tube, 30" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-4-5 (4.0")	9	CL-ML	Silty CLAY - Clayey SILT, slightly plastic, stiff, dry, light brown.
4465							
	5	U	1	PUSH (24.0")		CH	CLAY, highly plastic, damp, light brown.
4460							
	10	U	2	PUSH (25.0")		CL	Silty CLAY, moderately plastic, damp, yellow-brown with white mottling.
4455							
	15	S	2	5-8-10 (14.0")	18	ML	Clayey SILT, slightly plastic, 5-15% fine sand, dense, damp, yellow-brown, very thinly layered.
4450							
	20	S	3	4-6-6 (18.0")	12	MH	Clayey SILT, highly plastic, stiff, damp, yellow-gray, thinly layered.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-3 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445							
	25	S	4	8-10-14 (16.0")	24	MH	Clayey SILT, similar to S-3, white and orange mottling (very small.)
4440							
	30	S	5	8-12-16 (16.0")	28	ML	Sandy SILT, nonplastic, 5-10% fine sand, dense, dry, light brown.
4435							
	35	S	6	28-45-96 (18.0")	141	SP	SAND, uniform, fine, <3% nonplastic fines, very dense, dry, trace of coarse sand, light brown.
4430							
	40	S	7	26-50-100 (18.0")	150	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, light brown, trace fine gravel.
4425							
	45	S	8	18-18-34 (15.0")	52	SP CL	Top 6": SAND, similar to S-7. Bottom 9": Silty CLAY, slightly plastic, trace fine sand, hard, damp, light brown-gray.
4420							
	50	S	9	46-50-65 (18.0")	115	GP	Sandy GRAVEL, poorly graded, 5-8% nonplastic fines, 20-30% coarse to fine sand, subrounded to subangular gravel to 3/4", very dense, dry, light brown.
							BOTTOM OF BORING AT 51.5 FEET
4415							
	55						
4410							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Dudean — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-4 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: A.C. Smith
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/28/96 - 10/28/96
Coordinates: N 7319912.39 E 1280540.31
Ground Elevation: 4472.5 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc.
Driller: W. Westbrook
Rig Type: Mobile B-80

Methods:
Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT, 24" long. 3" O.D. Shelby Tube Sampler, 30" long
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-9-13 (6.0")	22	ML	SILT, nonplastic, very dense, damp, light brown.
4470							
	5	S	2	3-4-5 (13.0")	9	CH	Silty CLAY, highly plastic, stiff, damp, light brown.
4465							
	10	U	3	PUSH (27.0")		CL ML	Top 12": Silty CLAY, moderately plastic, moist, light brown. Bottom 15": SILT, nonplastic, 5-15% fine sand, dry, light brown, a few thin layers of clay.
4460							
	15	S	4	8-7-8 (12.0")	15	ML	SILT, moderately plastic, <10% fine sand, stiff to very stiff, damp, light brown.
4455							
	20	S	5	5-10-11 (13.0")	21	ML	SILT, moderately plastic, very stiff, damp, with 1" layers of slightly plastic silt.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
R.Y. Dudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring B-4 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	6	6-10-11 (14.0")	21	ML	SILT, similar to S-4.
4445							
	30	S	7	9-14-20 (14.0")	34	ML	Sandy SILT, nonplastic, 30-40% fine sand, dense, dry, light brown.
4440							
	35	S	8	5-11-30 (13.0")	41	SP-ML SP	Top 7": Stratified, 1/4" to 1/2" thick layers of fine SAND, <10% nonplastic fines, 1/4" to 1/2" thick layers of clayey SILT, slightly to moderately plastic, damp, light gray. Bottom 6": SAND, fine, <5% nonplastic fines, dense, dry, light brown.
4435							
	40	S	9	20-45-50 (13.0")	95	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown.
4430							
	45	S	10	14-45-55 (14.0")	100	SP	SAND, similar to S-9, except trace coarse sand and fine gravel (top 2" moderately plastic clay), dry, light brown.
4425							
	50	S	11	25-45-90 (14.0")	135	SP	SAND, similar to S-9, except trace coarse sand and fine gravel, 1/2" layer of nonplastic silt.
							BOTTOM OF BORING AT 51.5 FEET
4420							
	55						
4415							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-1 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: A.C. Smith
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 10/28/96 - 10/28/96
Coordinates: N 7321775.38 E 1281211.00 | Ground Elevation: 4460.8 ft
Groundwater Depth: N/A ft | Depth to Bedrock: N/A ft | Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc. | Driller: W. Westbrook | Rig Type: Mobile B-80

Methods: | Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, 24" long. 3" O.D. Shelby Sampler, 30" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil to ground surface, marked with a stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
***** 4460	0	S	1	2-1-2 (4.0")	3	ML	SILT, nonplastic, compact, damp, light brown.
4455	5	S	2	3-4-4 (16.0")	8	MH	Clayey SILT, highly plastic, firm to stiff, moist, light brown.
4450	10	U	3	PUSH (24.0")		ML-MH	Clayey SILT, moderately to highly plastic, moist, light brown.
4445	15	S	4	8-8-8 (15.0")	16	ML	Clayey SILT, moderately plastic, very stiff, damp, light brown.
4440	20	S	5	4-4-4 (18.0")	8	ML	Clayey SILT, moderately plastic, firm to stiff, moist, light brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
R.J. Vudoan	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-1 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4435	25	S	6	13-25-35 (13.0")	60	SP	SAND, fine, <10% nonplastic fines, very dense, dry, light brown and gray.
4430	30	S	7	17-36-55 (13.0")	91	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown.
4425	35	S	8	20-20-25 (15.0")	45	SM	Top 5": SAND, similar to S-7. Bottom 10": Silty SAND, 10-20% nonplastic fines, dense, dry, light brown.
4420	40	S	9	20-45-70 (14.0")	115	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown, gravelly layers every 2 to 3", 1" thick, 20-30% fine gravel and coarse sand.
4415	45	S	10	20-45-40 (10.0")	85	ML	SILT, nonplastic, very dense, dry, light brown, a few 1/2" layers of fine sand, top and bottom, 1" of clayey silt and silty clay.
4410	50	S	11	25-50-60 (13.0")	110	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown, 2" layer of 5-15% nonplastic fines. BOTTOM OF BORING AT 51.5 FEET
4405	55						
4400	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-2 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/28/96 - 10/28/96
Coordinates: N 7321142.12 E 1281237.18
Ground Elevation: 4464.5 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc.
Driller: Strickland
Rig Type: Acker Soil Sentry

Methods:
Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT. 3.0" O.D. Shelby Tube, 30" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	3-8-10 (4.0")	18	ML	Clayey SILT, slightly plastic, very dense, dry, light brown, trace fine sand, roots.
4460	5	U	1	PUSH (24.5")		MH	Clayey SILT, highly plastic, damp, yellow-brown.
4455	10	U	2	PUSH (25.0")		ML	Clayey SILT, moderately plastic, damp, yellow-brown.
4450	15	S	2	5-6-7 (13.0")	13	CL	Silty CLAY, moderately plastic, stiff, damp, yellow-brown with orange mottling, very thinly layered with clayey silt.
4445	20	S	3	4-5-6 (18.0")	11	ML	Clayey SILT, moderately plastic, stiff, damp, yellow-gray and yellow-brown, very thinly bedded.

Legend/Notes
- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
[signature]	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-2 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4440	25	S	4	7-14-20 (14.0")	34	ML ML	Top 6": Clayey SILT, similar to S-3. Bottom 8": Sandy SILT, nonplastic, 10-15% fine sand, very dense, damp, light brown-gray.
4435	30	S	5	30-87-100/2" (14.0")	100+	SP	SAND, uniform, fine, 3-5% nonplastic fines, trace medium to coarse sand and fine gravel, very dense, dry, light brown.
4430	35	S	6	21-51-77 (18.0")	128	SP	SAND, uniform, fine, 8-12% nonplastic fines, very dense, damp, light brown.
4425	40	S	7	30-81-100/5" (18.0")	100+	SP	Top 10": SAND, similar to S-6, except slightly cemented. Bot. 8": Gravelly SAND, poorly graded, 30-40% coarse to fine gravel, subangular to subrounded, mostly fine sand, 8-12% nonplastic fines, very dense, dry, brown.
4420	45	S	8	41-40-44 (16.0")	84	ML	Sandy SILT, nonplastic, 25-35% fine sand, very dense, dry, light brown, slightly cemented.
4415	50	S	9	31-40-80 (12.0")	120	SP	SAND, uniform, fine, 3-8% nonplastic fines, very dense, dry, light brown, few very thin black mineral layers. Bot. 2" Sandy gravel. BOTTOM OF BORING AT 51.5 FEET
4410	55						
4405	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved	Date
R.E. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7320563.12 E 1281241.31
Groundwater Depth: N/A ft
Contractor: Earthcore, Inc.

Depth to Bedrock: N/A ft
Driller: Strickland

Logged by: R. Gillespie
Date Start - Finish: 10/22/96 - 10/22/96
Ground Elevation: 4468.5 ft
Total Depth Drilled: 51.0 ft
Rig Type: Acker Soil Sentry

Methods:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Casing Used:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-4-7 (6.0")	11	CL	Silty CLAY, moderately plastic, stiff, dry, brown.
4465							
	5	S	2	3-3-3 (6.0")	6	CL	Silty CLAY, moderately plastic, firm, damp, green-gray with white and oragne mottling.
4460							
	10	S	3	3-3-5 (14.0")	8	ML	Clayey SILT, moderately plastic, firm to stiff, damp, yellow-green with minor orange and white mottling, trace sand and piece of gravel. Thinly layered.
4455							
	15	S	4	4-5-5 (12.0")	10	CL	Silty CLAY, moderately plastic, stiff, damp, yellow-brown, trace sand, little mottling.
4450							
	20	S	5	3-3-6 (18.0")	9	MH	Clayey SILT, highly plastic, stiff, damp, yellow-brown, thinly layered, some white mottling.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-3 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445							
	25	S	6	6-8-14 (15.0")	22	CL	Silty CLAY, slightly plastic, 5-15% fine sand, very stiff, damp, few layers of silt, thinly laminated, some with white and orange mottling.
4440							
	30	S	7	16-56-79 (17.0")	135	SP	SAND, uniform, fine, very dense, dry, light brown, trace medium sand, 3-5% nonplastic fines.
4435							
	35	S	8	33-100 (12.0")	100+	SP	SAND, uniform, fine, <3% nonplastic fines, trace medium to coarse sand, very dense, dry, light brown.
4430							
	40	S	9	27-34-60 (14.0")	94	SP SM	Top 7": SAND, similar to S-8. Bottom 7": Silty SAND, 15-20% nonplastic fines, very dense, dry, light green-brown, slightly cemented.
4425							
	45	S	10	27-43-100/5" (15.0")	100+	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, light brown.
4420							
	50	S	11	95-100/5" (10.0")	100+	GP	Sandy GRAVEL, poorly graded, 3-5% nonplastic fines, 15-25% coarse to fine sand, subangular to subrounded gravel to maximum 1.0", very dense, dry, brown. BOTTOM OF BORING AT 51.0 FEET
4415							
	55						
4410							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-4 J.O. 05996.01 Sheet 1 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT**
Client: **Private Fuel Storage, L.L.C.**
Coordinates: **N 7319942.04 E 1281203.84**
Groundwater Depth: **N/A** ft
Contractor: **Earthcore, Inc.**

Logged by: **A.C. Smith**
Date Start - Finish: **10/24/96 - 10/24/96**
Ground Elevation: **4473.2 ft**
Depth to Bedrock: **N/A** ft
Total Depth Drilled: **51.5** ft
Driller: **W. Westbrook**
Rig Type: **Mobile B-80**

Methods:
Drilling Soil: **3 1/4" I.D Hollow Stem Augers**
Sampling Soil: **2.0" O.D. Split Spoon, SPT, 24" long.**
Drilling Rock:

Casing Used:

Comments: **No bedrock or groundwater encountered. Backfilled with soil, marked with a stick.**

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-6-9 (7.0")	15	ML	SILT, slightly plastic, very dense, dry, light brown.
4470							
	5	S	2	4-3-4 (12.0")	7	CL MH	Top 4": CLAY, moderately plastic, firm, moist, gray. Bottom 8": Clayey SILT, highly plastic, firm, damp, light gray.
4465							
	10	S	3	3-4-7 (15.0")	11	ML	SILT, slightly plastic, <5% fine sand, stiff, damp, light brown.
4460							
	15	S	4	6-6-8 (12.0")	14	ML	SILT, similar to S-3, except moderately plastic.
4455							
	20	S	5	5-7-8 (13.0")	15	MH	Clayey SILT, highly plastic, stiff to very stiff, damp, thinly laminated, light brown and gray.

Legend/Notes
- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring C-4 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	6	10-10-10 (14.0")	20	CL	Silty CLAY, slightly plastic, <10% fine sand, very stiff, damp, light brown.
4445							
	30	S	7	8-10-11 (13.0")	21	SM ML	Top 3": Silty SAND, fine, 30-40% nonplastic fines, compact, dry, light brown. Bottom 10": SILT, nonplastic, compact, dry, light brown.
4440							
	35	S	8	16-30-50 (13.0")	80	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown.
4435							
	40	S	9	30-50-100 (13.0")	150	SP	SAND, similar to S-8, trace coarse sand.
4430							
	45	S	10	25-30-40 (14.0")	70	SP SM	Top 5": Gravelly SAND, coarse to fine, mostly fine, 30-40% rounded fine gravel, <5% nonplastic fines, very dense, dry, light brown. Bottom 9": Silty SAND, fine, 35-50% nonplastic fines, very dense, dry, light brown.
4425							
	50	S	11	15-35-100/5" (13.0")	100+	SP	SAND, fine, trace fine gravel, <5% nonplastic fines, dry, light brown. Bottom 5", 30-40% fine gravel and little coarse sand. BOTTOM OF BORING AT 51.5 FEET
4420							
	55						
4415							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved	Date
P. J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-1 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7321814.43 E 1281856.37
Groundwater Depth: N/A ft
Contractor: Earthcore, Inc.

Depth to Bedrock: N/A ft
Driller: W. Westbrook

Logged by: A.C. Smith
Date Start - Finish: 10/24/96 - 10/24/96
Ground Elevation: 4460.2 ft
Total Depth Drilled: 51.5 ft
Rig Type: Mobile B-80

Methods:
Drilling Soil: 3.0" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT, 24" long.
Drilling Rock:

Casing Used:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	6-20-20 (6.0")	40	ML	SILT, nonplastic, very dense, dry, light brown, damp at tip.
4455	5	S	2	3-5-7 (14.0")	12	CH-MH	Silty CLAY - Clayey SILT, highly plastic, stiff, damp, light brown, two 1" layers of slightly plastic clayey silt.
4450	10	S	3	5-7-7 (14.0")	14	CL	Silty CLAY, moderately plastic, <10% fine sand, stiff, damp, light brown.
4445	15	S	4	4-6-7 (16.0")	13	ML	Clayey SILT, moderately plastic, stiff, damp to moist, thinly laminated, light brown and light gray.
4440	20	S	5	6-7-9 (15.0")	16	CL	Silty CLAY, slightly to moderately plastic, <10% fine sand, very stiff, damp, laminated, light gray and yellow brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-1 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4435	25	S	6	20-40-60 (6.0")	100	SP	SAND, fine, <7% nonplastic fines, very dense, dry, light brown.
4430	30	S	7	15-40-50 (12.0")	90	SP	SAND, similar to S-6.
4425	35	S	8	15-37-38 (15.0")	75	ML	SILT, nonplastic, 10-20% fine sand, very dense, dry, light gray, trace of clay, 3" layer of silty fine sand, 30-40% nonplastic fines.
4420	40	S	9	20-40-60 (11.0")	100	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown. Two 2" layers with little medium and coarse sand, 20-30% fine gravel up to 1".
4415	45	S	10	20-60-50 (13.0")	110	SP	SAND, fine, <10% fines, very dense, dry, light brown.
4410	50	S	11	25-55-83 (13.0")	138	SP	SAND, similar to S-10.
							BOTTOM OF BORING AT 51.5 FEET
4405	55						
4400	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-2 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7321198.17 E 1281873.25
Groundwater Depth: N/A ft
Contractor: Earthcore, Inc.

Depth to Bedrock: N/A ft
Driller: W. Westbrook

Logged by: A.C. Smith
Date Start - Finish: 10/24/96 - 10/24/96
Ground Elevation: 4467.7 ft
Total Depth Drilled: 51.5 ft
Rig Type: Mobile B-80

Methods:
Casing Used:
Drilling Soil: 4 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT, 24" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil, marked with a stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-3-3 (6.0")	6	ML	SILT, nonplastic, compact, dry, light brown.
4465							
	5	S	2	3-3-3 (15.0")	6	ML	Clayey SILT, moderately plastic, firm, damp, light brown, middle 3" nonplastic.
4460							
	10	S	3	3-4-7 (16.0")	11	CH	Silty CLAY, highly plastic, stiff, damp, light brown.
4455							
	15	S	4	5-7-8 (16.0")	15	ML	Clayey SILT, moderately plastic, stiff to very stiff, damp, light brown, a few 1/4" layers of silty fine sand.
4450							
	20	S	5	6-9-9 (15.0")	18	ML	SILT, moderately plastic, very stiff, damp, thinly laminated, light brown and light gray.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Dudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-2 J.O. 05996.01 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **A.C. Smith**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445							
	25	S	6	6-8-9 (15.0")	17	CL	Silty CLAY, moderately plastic, very stiff, damp, thinly laminated, light brown and light gray.
4440							
	30	S	7	21-47-60 (13.0")	107	SP	SAND, fine, <10% nonplastic fines, very dense, dry, light brown and gray.
4435							
	35	S	8	15-50-71 (12.0")	121	SP	SAND, similar to S-7, trace coarse sand.
4430							
	40	S	9	10-50-55 (12.0")	105	SP	SAND, similar to S-7, trace coarse sand and fine gravel.
4425							
	45	S	10	20-40-40 (12.0")	80	SP CL	Top 7": Gravelly SAND, coarse to fine, mostly fine, 20-30% fine gravel, <5% nonplastic fines, very dense, dry, light brown. Bottom 5": CLAY, highly plastic, hard, dry, gravelly at top of layer, contained 1" layer of silty clay.
4420							
	50	S	11	20-60-90 (11.0")	150	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown, 2" thick layer of coarse to fine sand with 10-20% fine gravel. BOTTOM OF BORING AT 51.5 FEET
4415							
	55						
4410							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/22/96 - 10/22/96
Coordinates: N 7320587.09 E 1281884.33
Ground Elevation: 4469.2 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 51.5 ft
Contractor: Earthcore, Inc.
Driller: Strickland
Rig Type: Acker Soil Sentry

Methods:
Casing Used:
Drilling Soil: 3 1/4" I.D. Hollow Stem Augers
Sampling Soil: 2.0" O.D Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-3-3 (7.0")	6	CL	Silty CLAY, moderately plastic, firm, slightly damp, light green with white and orange-brown mottling.
4465	5	S	2	1-5-9 (11.0")	14	ML	Clayey SILT, moderately plastic, stiff, dry, light brown, trace roots.
4460	10	S	3	3-4-7 (15.0")	11	ML	Top 10": Clayey SILT, similar to S-2. Bottom 5" : SILT, nonplastic, compact, slightly damp, light brown with white mottling.
4455	15	S	4	4-4-5 (18.0")	9	ML	Clayey SILT, moderately plastic, stiff, damp, yellow-brown, thinly layered, minor orange mottling.
4450	20	S	5	4-5-6 (18.0")	11	ML	Clayey SILT, similar to above, few thin layers of silt, some white mottling.

Legend/Notes
- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
[signature]	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-3 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
4445	25	S	6	8-14-25 (16.0")	39	ML	SILT, nonplastic, very dense, slightly damp, light gray, minor thin layers of silty clay, some orange mottling.
4440	30	S	7	37-100/4" (12.0")	100+	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, light gray-brown.
4435	35	S	8	40-100/5" (12.0")	100+	SP	SAND, similar to S-7.
4430	40	S	9	38-100/5" (16.0")	100+	SP	SAND, similar to S-7, few thin layers of silt.
4425	45	S	10	78-100/4" (18.0")	100+	SP SW	Top 8": SAND, similar to S-7, few thin layers of silt, some medium to coarse sand. Bottom 10": Gravelly SAND, well graded, 3-5% nonplastic fines, coarse to fine gravel, coarse to fine sand, mostly fine, very dense, dry, light brown.
4420	50	S	11	70-75 (14.0")	100+	CL-CH	Silty CLAY, moderately to highly plastic, hard, slightly damp, trace sand and gravel, occasional silt layer, green-brown.
							BOTTOM OF BORING AT 51.5 FEET
4415	55						
4410	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-4 J.O. 05996.01 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: A.C. Smith
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 10/22/96 - 10/23/96
Coordinates: N 7319973.03 E 1281903.10
Ground Elevation: 4473.2 ft
Groundwater Depth: N/A ft
Depth to Bedrock: N/A ft
Total Depth Drilled: 100.5 ft
Contractor: Earthcore, Inc.
Driller: W. Westbrook
Rig Type: Mobile B-80

Methods:
Casing Used:
Drilling Soil: 4 1/4" I.D. Hollow Stem Augers to 65", below 65' roller cone bit with compressed air, open hole.
Sampling Soil: 2.0" O.D. Split Spoon, SPT, 18" and 24" long.
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	4-4-4 (3.0")	8	ML	SILT, nonplastic, <10% fine sand, compact, dry, light brown.
4470							
	5	S	2	3-2-2 (3.0")	4	CL	Silty CLAY, moderately plastic, soft to firm, damp, light brown.
4465							
	10	S	3	5-10-14 (8.0")	24	ML SM	Top 7": SILT, slightly plastic, very stiff, damp, bottom 2" contained 20-30% fine sand. Bottom 1": Silty SAND, fine, 20-30% nonplastic fines, very dense, dry, light brown.
4460							
	15	S	4	10-12-10 (12.0")	22	ML CL	Top 7": SILT, nonplastic, <10% fine sand, dense, slightly damp. Bottom 5": Silty CLAY, moderately plastic, very stiff, moist, brown.
4455							
	20	S	5	4-4-5 (14.0")	9	MH	Clayey SILT, highly plastic, stiff, moist, stratified with light brown and light gray bands 1/8-1/2" thick.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Vudean	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-4 J.O. 05996.01 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	6	6-7-9 (14.0")	16	CL-ML	Silty CLAY - Clayey SILT, slightly plastic, 10-20% fine sand, very stiff, damp, stratified light brown and light gray.
4445							
	30	S	7	8-31-34 (10.0")	65	SP	SAND, uniform, fine, <5% nonplastic fines, very dense, dry, light brown-gray, stratified with thin black layers at top 1/8-1/4".
4440							
	35	S	8	15-38-50 (13.0")	88	SP	SAND, uniform, fine, <10% nonplastic fines, very dense, dry, light brown and gray, two silty layers near bottom 1/4" thick.
4435							
	40	S	9	14-27-40 (12.0")	67	SP	SAND, similar to S-8, contained 1" layer with 20-30% nonplastic fines.
4430							
	45	S	10	18-33-50 (12.0")	83	SM	Silty SAND, fine, 10-20% nonplastic fines, very dense, dry, light brown, occasional 1/8-1/4" layer of silt.
4425							
	50	S	11	20-70-90 (13.0")	160	SM GP	Top 6": Silty SAND, fine 30-40% nonplastic fines, very dense, dry, light brown. Bottom 7": Sandy GRAVEL, mostly fine, up to 1 1/2", subrounded, 30-40% coarse to fine sand, <10% nonplastic fines, very dense, dry, light gray.
4420							
	55	S	12	28-57-75 (12.0")	132	SP	SAND, fine, 10-20% fine gravel, <5% nonplastic fines, very dense, dry, light brown.
4415							
	60	S	13	14-22-23 (11.0")	45	ML	SILT, nonplastic, 10-20% fine sand, dense, dry, light brown.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: R.J. Dudean(?) — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring D-4 J.O. 05996.01 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410							
	65	S	14	30-50-30 (13.0")	100	SM	Silty SAND, fine, 30-40% nonplastic fines, very dense, dry, light brown.
4405							
	70	S	15	60-100/3" (9.0")	100+	ML	SILT, nonplastic, <10% fine sand, very dense, dry, light brown.
4400							
	75	S	16	65-100/2" (8.0")	100+	ML	SILT, similar to S-15, slightly damp.
4395							
	80	S	17	50-50/0" (6.0")	100+	ML	SILT, similar to S-15, slightly damp.
4390							
	85	S	18	35-100/3" (7.0")	100+	ML	SILT, similar to S-15, slightly damp, piece of gravel in tip.
4385							
	90	S	19	100/4" (4.0")	100+	ML	SILT, similar to S-15, except damp.
4380							
	95	S	20	50-70-100/3" (14.0")	100+	ML	Clayey SILT, slightly plastic, very dense, slightly damp, light brown, some areas of silty clay, some nonplastic silt up to 2" thick.
4375							
	100	S	21	50-50/0" (6.0")	100+	SM	Silty SAND, fine, 20-30% nonplastic fines, very dense, dry, light brown, top 2" coarse to fine sand. BOTTOM OF BORING AT 100.5 FEET

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-1 J.O. 05996.02 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 12/03/98 - 12/03/98
Coordinates: N7,321,865 E1,282,600
Ground Elevation: 4462.6 ft
Groundwater Depth:
Depth to Bedrock:
Total Depth Drilled: 51.5 ft
Contractor: Earthcore Drilling
Driller: T. Kern
Rig Type: Mobile B-80

Methods:
Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem auger
Sampling Soil: 2.0" OD split-spoon, SPT, 18" drive; 3.0" Shelby tube, 30"
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-7-14 (5.0")	21	ML	Sandy SILT, moderately plastic, very stiff, dry, lt. brown.
4460							
	5	S	2	4-5-8 (12.0")	13	MH	SILT, highly plastic, stiff, damp, lt. gray mottled with white, thinly layered.
4455							
	10	S	3	4-5-7 (18.0")	12	CL	Silty CLAY, moderately plastic, stiff, damp, lt. yellow-brown, layered.
4450							
	15	S	4	6-7-8 (18.0")	15	ML	Clayey SILT, moderately plastic, stiff to very stiff, damp, yellow-brown, layered.
4445							
	20	S	5	4-4-7 (18.0")	11	ML-MH	Clayey SILT, moderately to highly plastic, stiff, damp, yellow-brown.

Legend/Notes

- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon

Approved	Date
R.J. Hudean	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-1 J.O. 05996.02 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4440							
	25	S	6	20-28-50 (14.0")	78	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, lt. gray-brown, trace of medium to coarse sand.
4435							
	30	S	7	20-50-60 (18.0")	110	SP	SAND, uniform, fine, very dense, dry, lt. brown.
4430							
	35	S	8	20-18-18 (12.0")	36	SP CL	Top 9": SAND, similar to above. Bottom 3": Silty CLAY, slightly to moderately plastic, hard, damp, green-gray.
4425							
	40	S	9	35-35-80 (15.0")	115	SP	SAND, uniform, fine, 5-8% nonplastic fines, very dense, dry, lt. gray.
4420							
	45	S	10	30-70-100 (13.0")	170	SW	Gravelly SAND, well graded, sub-angular to sub-round gravel to 1.0", <3% non plastic fines, very dense, dry, lt. brown.
4415							
	50	S	11	30-50-80 (9.0")	130	SP	SAND, uniform, fine to medium, mostly fine, trace of coarse, very dense, dry, v. lt. brown. End of Boring at 51.5 ft.
4410							
	55						
4405							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-2 J.O. 05996.02 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 12/04/98 - 12/04/98
Coordinates: N7,321,250 E1,282,600 | Ground Elevation: 4468.0 ft
Groundwater Depth: | Depth to Bedrock: | Total Depth Drilled: 51.5 ft
Contractor: Earthcore Drilling | Driller: T. Kern | Rig Type: Mobile B-80

Methods: | Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem auger
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive; 3.0" Shelby tube
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-4-4-5 (10.0")	8	ML	SILT, slightly to moderately plastic, firm to stiff, dry brown, eolian.
4465							
	5	U	1	PUSH (26.0")		ML MH	Top: SILT, slightly to moderately plastic, dry, brown. Bottom: SILT, highly plastic, damp, green-gray.
4460							
	10	S	2	4-5-5-7 (23.0")	10	MH	Clayey SILT, highly plastic, stiff, damp, lt. yellow-brown with white mottling, thinly layered.
4455							
	15	S	3	9-7-9-12 (2.0")	16	ML	Sandy SILT, slightly to moderately plastic, 10-20% fine sand, very stiff, damp, yellow-brown.
4450							
	20	S	4	6-8-8-12 (22.0")	16	MH	SILT, highly plastic, very stiff, damp, lt. yellow-brown, thinly layered.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
R.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-2 J.O. 05996.02 Sheet 2 of 2

Site: **Private Fuel Storage Facility, Skull Valley, UT** Logged by: **R. Gillespie**

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445							
	25	S	5	8-12-18-35 (20.0")	30	ML-MH SP	Top 8": Clayey SILT, moderately to highly plastic, very stiff to hard, damp, yellow-brown, thinly layered. Bottom 12": SAND, uniform, fine, dense, damp, lt. gray. Thin layer of gray clay at contact.
4440							
	30	S	6	40-50-70-100 (13.0")	120	SP	SAND, uniform, fine, 5-8% fines, very dense, dry, gray, some thin layering.
4435							
	35	S	7	40-100 (11.0")	100+	SP	SAND, uniform, fine, 3-5% fines, very dense, dry, v. lt. brown.
4430							
	40	S	8	30-40-40-90 (13.0")	80	SP	SAND, similar to above, becomes slightly cemented toward bottom, trace of medium gravel.
4425							
	45	S	9	18-25-50-80 (16.0")	75	SM SW	Top 10": Silty SAND, mostly fine, 15-20% nonplastic fines, very dense, dry, gray, slightly cemented. Bottom 6": Gravelly SAND, well graded, 20-30% sub-round gravel to 1/2", 5-8% nonplastic fines, very dense, dry, v. lt. brown.
4420							
	50	S	10	45-60-100 (13.0")	160	SP	SAND, uniform, fine, 3-5% nonplastic fines, very dense, dry, lt. brown. End of Boring at 51.5 ft.
4415							
	55						
4410							
	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved E.J. Lindeau Date 08/31/99

Stone & Webster Engineering Corporation	**BORING LOG**	Boring E-3 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/23/96 - 10/23/96
Coordinates: N 7320635 E 1282600 — Ground Elevation: 4471.3 ft
Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 51.1 ft
Contractor: Earthcore, Inc. — Driller: W. Westbrook — Rig Type: Mobile B-80

Methods: — Casing Used:
Drilling Soil: 4 1/4" I.D Hollow Stem Augers
Sampling Soil: 2.0" O.D. Split Spoon, SPT
Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-4-5 (7.0")	9	ML	SILT, nonplastic, dense, dry, light brown.
4470							
4465	5	S	2	2-4-7 (14.0")	11	CL	CLAY, moderately plastic, stiff, moist, light gray, numerous silt partings.
4460	10	S	3	3-5-7 (16.0")	12	CL-CH	Silty CLAY, moderately to highly plastic, stiff, moist, light brown, bottom 4" nonplastic.
4455	15	S	4	7-10-12 (16.0")	22	ML	SILT, nonplastic, <10% fine sand, dense, damp, light brown, 1/4" layer of gray clay every 3-4".
4450	20	S	5	4-7-8 (16.0")	15	ML-CL	Clayey SILT - Silty CLAY, slightly plastic, stiff to very stiff, moist, thinly laminated, light brown and gray.

Legend/Notes

- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-3 J.O. 05996.01 Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4445	25	S	6	5-6-12 (15.0")	18	ML	SILT, nonplastic, compact, damp, light brown and gray, a few zones of clayey silt. Bottom 4" contained 10-20% fine sand.
4440	30	S	7	15-50-75 (14.0")	125	SP	SAND, fine, <5% nonplastic fines, very dense, dry, light brown.
4435	35	S	8	20-50-80 (12.0")	130	SP	SAND, similar to S-7, trace coarse sand.
4430	40	S	9	15-50-60 (13.0")	110	SP	SAND, similar to S-7, mottled with yellow brown.
4425	45	S	10	15-55-50 (12.0")	105	SP	SAND, similar to S-7, contained two layers 1-2" thick of gravelly sand, coarse to fine, mostly fine, 20-30% fine gravel, subrounded, <5% nonplastic fines.
4420	50	S	11	32-55-95 (13.0")	150	SP	SAND, similar to S-7, contained 2" thick layer of gravelly sand similar to S-10.
							BOTTOM OF BORING AT 51.5 FEET
4415	55						
4410	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P. J. Trudeau (signature) Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring E-4 J.O. 05996.01 Sheet 1 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: A.C. Smith

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/23/96 - 10/23/96

Coordinates: N 7319988 E 1282615 — Ground Elevation: 4474.6 ft

Groundwater Depth: N/A ft — Depth to Bedrock: N/A ft — Total Depth Drilled: 51.5 ft

Contractor: Earthcore, Inc. — Driller: W. Westbrook — Rig Type: Mobile B-80

Methods: — Casing Used:

Drilling Soil: 4 1/4" Hollow Stem Augers

Sampling Soil: 2.0" O.D. Split Spoon, SPT, 24" long.

Drilling Rock:

Comments: No groundwater or bedrock encountered. Backfilled with soil, marked with stake.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	3-7-10 (7.0")	17	ML	SILT, nonplastic, very dense, dry, light brown, a few roots.
4470	5	S	2	9-10-16 (13.0")	26	SM	Silty SAND, fine, 20-30% nonplastic fines, very dense, dry, light brown. Top 1" clayey silt.
4465	10	S	3	13-25-27 (12.0")	52	SM	Silty SAND, fine, 10-20% nonplastic fines, very dense, dry, light brown.
4460	15	S	4	8-20-20 (12.0")	40	ML	SILT, nonplastic, < 10% fine sand, very dense, dry, light brown. Bottom 2" stratified with 1/8" layers of clay.
4455	20	S	5	6-6-10 (17.0")	16	ML	Clayey SILT, slightly plastic, thinly laminated, very stiff, damp, light brown and light gray.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
B.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation

BORING LOG

Boring E-4
J.O. 05996.01
Sheet 2 of 2

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: A.C. Smith

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450	25	S	6	7-10-14 (16.0")	24	ML	SILT, nonplastic, <10% fine sand, dense, damp, light brown, occasional thin layer of clay.
4445	30	S	7	30-50-65 (12.0")	115	SP	SAND, fine, <10% nonplastic fines, very dense, dry, light brown, top 1" damp silt, nonplastic.
4440	35	S	8	17-55-75 (13.0")	130	SP	SAND, similar to S-7, slightly damp.
4435	40	S	9	12-25-50 (13.0")	75	SP	SAND, similar to S-7, slightly damp, contained 2" layer of sandy silt.
4430	45	S	10	12-55-75	130	SP	SAND, similar to S-7, slightly damp.
4425	50	S	11	20-80-50 (12.0")	130	SP	Gravelly SAND, coarse to fine, mostly fine, 10-20% mostly fine gravel, up to 1 1/2", rounded, <5% nonplastic fines, very dense, dry, light brown.
							BOTTOM OF BORING AT 51.5 FEET
4420	55						
4415	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved RJ Trudeau   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 1 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 12/14/98 - 01/02/99
Coordinates: N7,320,287 E1,282,057 | Ground Elevation: 4472.7 ft
Groundwater Depth: | Depth to Bedrock: | Total Depth Drilled: 226.5 ft
Contractor: Earthcore and Layne | Driller: Kern and Mott | Rig Type: See below

Methods: Casing Used: 6 5/8" from 110' to 226.5'
Drilling Soil: 3 1/4" ID hollow stem auger to 70'; air rotary with 3 1/8" open hole to 110'; mud rotary from 110'.
Sampling Soil: 2.0" OD split-spoon w/SPT to 110'; 2.5" spoon w/300# hammer, 30" drop from 115'; 3.0" Shelby
Drilling Rock:

Comments: Mobile B-80 used to 110 ft; Failing-Speedstar 30K from 110 to 226.5 ft.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	3-6-9-9 (7.0")	15	ML	Clayey SILT, moderately plastic, stiff to very stiff, damp, brown, trace fine sand.
4470							
	5	S	2	4-3-3-3 (12.0")	6	CL MH	Top 2": Silty CLAY, moderately plastic, firm, damp, light gray. Bottom 10": Clayey SILT, highly plastic, firm, damp, gray with white mottling.
4465		U	3	PUSH (26.0")		CH	Silty CLAY, highly plastic, damp, gray-brown with white mottling.
	10	S	4	5-5-5-7 (24.0")	10	CL SM	Top 13": Silty CLAY, moderately plastic, stiff, damp, light brown-gray. Bottom 11": Silty SAND, uniform, fine, 20-30% slightly plastic fines, compact, damp, light brown-gray.
		U	5	PUSH (11.0")		SC SM	Top 10": Clayey SAND, fine, 20-30% moderately plastic fines, damp, yellow-brown. Bottom 1": Silty SAND, uniform, fine, damp, 10-20% nonplastic fines, yellow-brown.
4460							
	15	S	6	10-14-10-14 (12.0")	24	ML	Sandy SILT, slightly plastic, 40-50% fine sand, very stiff, damp, lt. yellow-brown, occasional layer of moderately to highly plastic clay, gray.
4455							
	20	U	7	PUSH (25.5")		MH	Clayey SILT, highly plastic, damp, lt. yellow-brown.

Legend/Notes

- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  S = 2" or 2.5" OD Split Spoon
  U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 2 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	8	10-10-14-16 (15.0")	24	ML	Sandy SILT, slightly plastic, 10-20% fine sand, dense, damp, yellow-brown with orange mottling, thinly bedded, occasional thin layer of clay.
4445							
	30	S	9	16-40-50-80 (15.0")	90	SM	Silty SAND, uniform, fine, 10-15% nonplastic fines, very dense, dry, lt. brown. Two 2" layers of clay, moderately to highly plastic, green-brown.
4440							
	35	S	10	30-60-100 (12.0")	160	SM	Silty SAND, similar to above, except no clay and 1 piece of gravel.
4435							
	40	S	11	20-60-100 (14.0")	160	SP	SAND, uniform, fine, very dense, dry, lt. brown.
4430							
	45	S	12	30-40-50-100 (4.0")	90	ML-MH	Clayey SILT, moderately to highly plastic, 10-20% fine sand, hard, dry, lt. brown.
4425							
	50	S	13	40-60-40-60 (12.0")	100	ML-MH SP GP	Top 1": Clayey SILT, similar to above. Middle 2": SAND, uniform, fine, lt. brown. Bottom 9": Sandy GRAVEL, poorly graded, coarse to fine gravel to 1.5", mostly fine sand, 3-5% slightly plastic fines, very dense, dry, lt. brown. Silty Clay in tip of spoon, highly plastic, green-brown.
4420							
	55						
4415							
	60	S	14	30-70-70-100 (11.0")	140	SM	Silty SAND, uniform, fine, 10-20% non to slightly plastic fines, very dense, dry (water added), light brown.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P.J. Trudeau (signature) Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 3 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410							
	65						
4405							
	70	S	15	12-20-30-50 (18.0")	50	CL&ML	Thinly interbedded Silty CLAY and Clayey SILT, moderately plastic, dry, hard, brown, with thin layers of fine sand. Switched to air-rotary at 70 ft. Cuttings mainly fine sand, silt, and fine gravel between 70 to 77 ft. White volcanic ash between 77 and 80 ft.
4400							
	75						
4395							
	80	S	16	40-100/3" (8.0")	100+	ML	Top 2": Sandy SILT, nonplastic, fine sand, v. dense, dry, lt. gray, volcanic ash. Bottom 6": Clayey SILT, slightly plastic, very dense, dry, brown.
4390							
	85						
4385							
	90	S	17	60-100/5" (8.0")	100+	ML	SILT, very slightly plastic, 5-10% fine sand, very dense, dry, lt. brown.
4380							
	95						
4375							
	100	S	18	100/5" (1.0")	100+		Mix of Sandy SILT (volcanic ash) and Silty SAND, very small sample, not meaningful.
4370							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 4 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
	105						
4365							
	110	S	19	100 (5.0")	100+	SM	Silty SAND, uniform, medium to fine, mostly fine, 10-20% sl. plastic fines, very dense, damp, brown.
4360							Switched to mud-rotary, 300 lb hammer, and larger dia. spoon as noted.
	115	S	20	50/3"-50*/2" (5.0")	*	ML	Sandy SILT, slightly to moderately plastic, 10-20% fine sand, very dense, dry, lt. brown.
4355							
	120						
4350							
	125	S	21	25*-46*-50*/3" (18.0")	*	SM	Silty SAND, uniform, fine, 20-30% slightly plastic fines, damp(?), brown.
4345							
	130						
4340							
	135	S	22	30*-50*/4 (14.0")	*	ML	Sandy SILT, slightly to moderately plastic, 10-15% fine sand, very dense, damp(?), lt. brown. (2.25" OD spoon)
4335							
	140						
4330							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 5 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
	145	S	23	31*-50*/2 (10.0")	*	ML	Sandy SILT, similar to above. (2.5" OD spoon used to end of boring)
4325							
	150						
4320							
	155	S	24	50*/5" (2.5")	*	ML	Sandy SILT, nonplastic, 10-15% fine sand, very dense, saturated, lt. gray, volcanic ash.
4315							
	160						
4310							
	165						
4305							
	170	S	25	47*-50*/3.5' (10.0")	*	MH	Clayey SILT, moderately to highly plastic, 8-12% fine sand, very dense, damp, yellow-brown and brown, thinly bedded.
4300							
	175						
4295							
	180						
4290							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: [signature] Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 6 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
	185	S	26	16*-27*-50*/4" (16.0")	*	MH SM	Top 6": Clayey SILT, similar to above, few cemented pieces. Bottom 10": Becomes Silty SAND, uniform, fine to medium, 15-20% slightly plastic fines, very dense, wet, brown, thinly bedded. (Separate jars)
4285							
	190						
4280							
	195	S	27	50*/3" (3.0")	*	SP	SAND, uniform, fine, 5-10% nonplastic fines, very dense, wet, lt. brown.
4275							
	200						
4270							
	205						
4265							
	210	S	28	42*-50*/2 (11.0")	*	SM	Silty SAND, uniform, fine, 10-20% slightly plastic fines, very dense, damp, lt. brown, few thin layers of clay.
4260							
	215						
4255							
	220						
4250							

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-1 J.O. 05996.02 Sheet 7 of 7

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
	225	S	29	15*-16*-30* (14.0")	*	CL	Silty CLAY, slightly plastic, 10-15% fine sand, hard, damp, brown. Clay appears desiccated (flaky). End of Boring at 226.5 ft.
4245							
	230						
4240							
	235						
4235							
	240						
4230							
	245						
4225							
	250						
4220							
	255						
4215							
	260						
4210							
	265						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] — Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-2 J.O. 05996.02 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 12/07/98 - 12/08/98
Coordinates: N7,320,217 E1,281,989 | Ground Elevation: 4474.3 ft
Groundwater Depth: | Depth to Bedrock: | Total Depth Drilled: 71 ft
Contractor: Earthcore Drilling | Driller: T. Kern | Rig Type: Mobile B-80

Methods: | Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem augers
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	2-2-3-3 (22.0")	5	ML	SILT, moderately plastic, 5-10% fine sand, firm, damp, brown.
4470	5	S	2	6-6-7-9 (16.0")	13	ML CL	Top 6": SILT, same as above. Bottom 10": Silty CLAY, moderately plastic, stiff, damp, green-gray with white mottling.
		S	3	3-5-7-7 (10.0")	12	MH	Clayey SILT, highly plastic, stiff, damp, green-gray with white mottling, thinly bedded.
4465	10	S	4	3-4-5-6 (20.0")	9	MH	Clayey SILT, similar to above.
		S	5	5-7-9-11 (3.0")	16	ML	SILT, slightly plastic, dense, damp, lt. brown-gray, trace of fine sand.
4460	15	S	6	7-12-16-18 (16.0")	28	CL	Silty CLAY, moderately plastic, 10-15% fine sand, very stiff, damp, lt. brown.
4455	20	S	7	6-8-10-10 (18.0")	18	ML	Sandy SILT, moderately plastic, 10-20% fine sand in thin layers, very stiff, damp, lt. brown with some white on some bedding surfaces.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon

Approved	Date
R.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-2 J.O. 05996.02 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450	25	S	8	6-8-12-12 (17.0")	20	ML	Sandy SILT, similar to above, occasional thin layer of clay.
4445	30	S	9	10-10-40-50 (18.0")	50	ML SP	Top 3": Sandy SILT, similar to above. Bottom 15": SAND, uniform, fine, <10% nonplastic fines, very dense, dry, lt. brown. Three inch layer of clay near top of sand, highly plastic, damp, green-gray. Some iron oxide mottling.
4440	35	S	10	35-65-100 (12.0")	165	SP	SAND, uniform, fine, <8% nonplastic fines, very dense, dry, lt. brown, trace of medium sand.
4435	40	S	11	30-100 (10.0")	100+	SP	SAND, similar to above, trace of medium gravel.
4430	45	S	12	12-14-30-28 (15.0")	44	CL	Silty CLAY, moderately plastic, hard, damp, gray-green,with thin layers of silt and fine sand. Becomes sandy silt toward bottom with few fossil fragments.
4425	50	S	13	40-60-100 (13.0")	160	GW	Sandy GRAVEL, coarse to fine, sub-round to angular, coarse to fine sand, mostly fine, 3-5% nonplastic fines, very dense, dry, lt. brown. Three inch layer of fine sand near top, clean, lt. brown.
4420	55						
4415	60	S	14	35-35-35-100 (17.0")	70	SM	Silty SAND, uniform, fine, 15-20% nonplastic fines, very dense, dry, lt. brown. Several pieces of gravel at top, very thinly layered.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P.J. Trudeau — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-2 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410	65						
4405	70	S	15	50-100 (7.0")	100+	SM	Silty SAND, widely graded, coarse to fine sand, mostly fine, 10-15% slightly plastic fines, 10-15% gravel, very dense, lt. gray-brown, thinly layered. Water added to lift cuttings.
							End of Boring at 71 ft.
4400	75						
4395	80						
4390	85						
4385	90						
4380	95						
4375	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-3 J.O. 05996.02 Sheet 1 of 3

Site: **Private Fuel Storage Facility, Skull Valley, UT**
Client: **Private Fuel Storage, L.L.C.**
Coordinates: **N7,320,217 E1,282,124**
Groundwater Depth:
Contractor: **Earthcore Drilling**

Depth to Bedrock:
Driller: **T. Kern**

Logged by: **R. Gillespie**
Date Start - Finish: **12/14/98 - 12/14/98**
Ground Elevation: **4473.2 ft**
Total Depth Drilled: **70.45 ft**
Rig Type: **Mobile B-80**

Methods:
Casing Used: **None**

Drilling Soil: **3 1/4" ID hollow-stem augers**
Sampling Soil: **2.0" OD split-spoon, SPT, 24-inch drive**
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	6-5-4-5 (20.0")	9	ML	SILT, moderately plastic, 5-15% fine sand, stiff (frozen), damp, lt. brown, eolian, becomes green-gray with white in tip.
4470							
	5	S	2	4-5-6-9 (23.0")	11	MH	Clayey SILT, highly plastic, stiff, damp, green-gray to yellow-brown with white mottling, thinly layered.
		S	3	6-5-7-9 (22.0")	12	MH	Clayey SILT, similar to above, yellow-brown.
4465							
	10	S	4	8-12-13-13	25		No Recovery
		S	5	6-10-12-12 (19.0")	22	ML	Sandy SILT, moderately plastic, 10-20% fine sand, damp, very stiff, lt. brown.
4460							
	15	S	6	8-9-9-10 (17.0")	18	SM ML	Top 10": Silty SAND, uniform, fine, 15-25% slightly plastic fines, dense, damp, lt. brown-gray, few layers of highly plastic Clay. Bottom 7": Sandy SILT, moderately plastic, very stiff, damp, yellow-brown.
4455							
	20	S	7	7-10-10-12 (19.0")	20	SM MH	Top 4": Silty SAND, uniform, fine, 15-20% slightly plastic fines, dense, damp, yellow-brown. Bottom 15": Clayey SILT, highly plastic, very stiff, damp, lt. yellow-brown with white mottling on bedding planes, thinly layered, with ostracodes.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-3 J.O. 05996.02 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	8	5-9-15-15 (16.0")	24	CL	Silty CLAY, moderately plastic, very stiff, damp, yellow-brown, interlayered with Sandy Silt, moderately plastic, 15-20% fine sand.
4445							
	30	S	9	25-35-50-100 (15.0")	85	SP	SAND, uniform, fine, <10% nonplastic fines, very dense, dry, lt. brown.
4440							
	35	S	10	30-100/5" (9.0")	100+	SP	SAND, similar to above.
4435							
	40	S	11	35-70-100 (12.0")	170	SP	SAND, similar to above.
4430							
	45	S	12	20-30-50-50 (12.0")	80	ML SM	Top 7": Clayey SILT, moderately plastic, hard, dry, gray-brown. Bottom 5": Silty SAND, uniform, fine, 10-20% slightly plastic fines, very dense, dry, lt. brown, slightly cemented.
4425							
	50	S	13	60-100/5" (10.0")	100+	GW	Sandy GRAVEL, well graded, sub-round to sub-angular gravel to 1.5", coarse to fine sand, mostly fine, 3-5% nonplastic fines, very dense, dry, lt. brown, trace fossils (gastropods), very thin caliche rinds on some gravel.
4420							
	55						
4415							
	60	S	14	60-100 (10.0")	100+	SP	Gravelly SAND, poorly graded, coarse to fine sand, mostly fine, 25-35% gravel to 1.0", angular to sub-round, 5-8% slightly plastic fines, very dense, dry (water added), lt. brown-gray, till-like.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P. J. Trudeau — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-3 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410							
	65						
4405							
	70	S	15	100/5" (6.0")	100+	SP	SAND, poorly graded, coarse to fine, mostly fine, 10-15% gravel, very dense, dry, lt. brown, water added. End of Boring at 70.45 ft.
4400							
	75						
4395							
	80						
4390							
	85						
4385							
	90						
4380							
	95						
4375							
	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature]   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-4 J.O. 05996.02 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 12/04/98 - 12/05/98
Coordinates: N7,320,167 E1,282,057 — Ground Elevation: 4475.3 ft
Groundwater Depth: — Depth to Bedrock: — Total Depth Drilled: 72 ft
Contractor: Earthcore Drilling — Driller: T. Kern — Rig Type: Mobile B-80

Methods: — Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem augers
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive; 3.0" OD Shelby tube, 30" and 24"
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-1-3-3 (5.0")	4	ML	SILT, moderately plastic, soft to firm, damp, lt. brown, eolian.
		S	2	3-1-4-5 (13.0")	5	ML	Top 6": SILT, similar to above. Bottom 7": Clayey SILT, moderately plastic, firm, damp, brown with white mottling.
4470	5	S	3	5-7-7-8 (11.0")	14	CL	Silty CLAY, moderately plastic, stiff, damp, green-gray with white mottling throughout.
		U	1	PUSH (24.0")		CL MH	Top 14": Silty CLAY, moderately plastic, gray. Bottom 10": Clayey SILT, highly plastic, damp, yellow-brown.
		U	2	PUSH (26.0")		CH	Silty CLAY, highly plastic, yellow-brown.
4465	10	S	6	5-8-8-8 (13.0")	16	ML	Clayey SILT, moderately plastic, very stiff, damp, yellow-brown, some partings of fine sand.
		U	7	PUSH (16.0")		ML SP	Top: Sandy SILT, slightly plastic, 25-35% fine sand, damp, yellow-brown. Bottom: SAND, uniform, fine, <10% nonplastic fines, damp, lt. yellow-brown.
4460	15	S	8	8-12-12-12 (15.0")	24	ML SM	Top 8": Sandy SILT, slightly plastic, 30-40% fine sand, compact, damp, lt. brown. Bottom 7": Silty SAND, uniform, fine, 35-40% nonplastic fines, dense, dry, gray-brown, a few thin layers of clay.
		U	9	PUSH (19.0")		ML SM	Top 1": Sandy SILT, nonplastic, 15-25% fine sand, damp, lt. red-brown. Bottom 18": Silty SAND, fine, 10-20% nonplastic fines, damp, brown-gray, occasional thin layer of silty clay.
		S	10	8-8-10-8 (18.0")	18	CL	Silty CLAY, moderately plastic, <10% fine sand, very stiff, damp, lt. yellow-gray, numerous thin layers of silt.
4455	20	U	11	PUSH (20.0")		ML	Sandy SILT, moderately plastic, 10-20% fine sand, dry, lt. yellow-brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
R.J. Hudson	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-4 J.O. 05996.02 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
		S	12	6-6-8-14 (18.0")	14	MH	Clayey SILT, highly plastic, stiff, dry, yellow-brown, thinly bedded, with ostracodes.
4450	25	U	13	PUSH (19.5")		ML	Clayey SILT, moderately plastic, damp, yellow-brown.
		S	14	7-10-12-12 (20.0")	22	CL	Silty CLAY, slightly plastic, very stiff, damp, yellow-brown.
		U	15	PUSH (16.0")		ML SM	Top 15": Sandy SILT, nonplastic, 20-30% fine sand, yellow-brown. Bottom 2": SAND, uniform, fine, 5-15% nonplastic fines, dry, lt. brown.
4445	30	S	16	20-30-30-50 (19.0")	60	SM	SAND, uniform, fine, 30-35% nonplastic fines, very dense, dry, lt. brown, layered, trace of clay at bottom.
4440	35	S	17	40-100 (8.0")	100+	SP	SAND, uniform, fine, <10% nonplastic fines, very dense, dry, lt. brown.
4435	40	S	18	35-100 (10.0")	100+	SP	SAND, similar to above.
4430	45	S	19	12-40-70-30 (18.0")	110	SP	SAND, similar to above.
4425	50	S	20	20-100 (11.0")	100+	SP	SAND, similar to above.
4420	55						
4415	60	S	21	70-100 (10.0")	100+	SW	Sandy GRAVEL, well graded, sub-round to angular gravel to 3/4", coarse to fine sand, 10-15% nonplastic fines, very dense, dry, v. lt. brown.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: RJ Trudeau — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-4 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410	65						
4405	70	S	22	30-35-70-100 (21.0")	105	SM	Top 11": Silty SAND, 15-20% gravel to 0.5", coarse to fine sand, 10-20% slightly plastic fines, very dense, dry (water added), brown. Bottom 10": Silty SAND, uniform, fine, 15-20% slightly plastic fines, very dense, dry, lt. brown, layered. End of Boring at 72 ft.
4400	75						
4395	80						
4390	85						
4385	90						
4380	95						
4375	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P. Trudeau (signature) — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-5(OW) J.O. 05996.02 Sheet 1 of 5

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 12/16/98 - 01/08/99
Coordinates: N7,320,087 E1,282,057 — Ground Elevation: 4475.1 ft
Groundwater Depth: 124.5 ft on 01/08/99 — Depth to Bedrock: — Total Depth Drilled: 158.4 ft
Contractor: Earthcore/Layne — Driller: Kern/Franklin — Rig Type: See below

Methods: — Casing Used: 6 5/8"
Drilling Soil: 3 1/4" ID HSA to 95'; ODEX system (7 1/2" bit) from 92' to 158.4'.
Sampling Soil: 2.0" OD split-spoon w/SPT to 95'; 2.5" OD spoon w/300# hammer from 92' to 158.4', 30" drop.
Drilling Rock: None

Comments: Mobile B-80 used to 95 ft, Failing-Speedstar 30K from 92' to 158.4'. Monitoring well installed.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	4-7-7-10 (4.0")	14	ML	Sandy SILT, moderately plastic, 10-15% fine sand, stiff(frozen), dry, brown.
		S	2	10-10-11-10 (2.0")	21	ML	Sandy SILT, similar to above, very small sample, lt. brown-gray.
		S	3	2-2-4-4 (22.0")	6	MH	Clayey SILT, highly plastic, firm, damp, lt. brown-gray with white mottling, thinly bedded.
4470	5						
		S	4	2-4-4-6 (24.0")	8	CH SM CH	Top 4": Silty CLAY, highly plastic, firm, damp, gray Middle 2": Silty SAND, fine, brown. Bottom 18": Silty CLAY, highly plastic, firm to stiff, damp, yellow-brown, thinly interbedded.
		S	5	2-3-4-6 (21.0")	7	CH	Silty CLAY, similar to above with 6" layer of soft clay, lt. yellow-brown.
4465	10	U	6	PUSH (19.0")		ML	Top 15": SILT, nonplastic, <10% fine sand, damp, lt. yellow-brown. Bottom 4": Sandy SILT, nonplastic, 15-25% fine sand, damp, yellow-brown.
		S	7	9-12-12-20 (17.0")	24	SM	Silty SAND, uniform, fine, 15-25% nonplastic fines, very dense, dry, lt. yellow-brown.
		U	8	PUSH (20.0")		SM	Silty SAND, uniform, fine, 10-25% nonplastic fines, dry, lt. yellow-brown and gray.
4460	15						
		S	9	12-14-18-14 (8.0")	32	ML	Sandy SILT, nonplastic, 35-40% fine sand, very dense, damp, lt. brown, bottom 2" contained some thin layers of silty clay.
		U	10	PUSH (22.0")		ML	SILT, moderately plastic, damp, yellow-brown.
4455	20	S	11	6-6-10-10 (18.0")	16	MH	Clayey SILT, highly plastic, 10-15% fine sand, very stiff, damp, lt. yellow-brown.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" or 2.5" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
R.F. Vandeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-5(OW) J.O. 05996.02 Sheet 2 of 5

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
		U	12	PUSH (26.0")		ML MH	Top 18": Clayey SILT, moderately plastic, damp, yellow-brown. Bottom 8": Clayey SILT, highly plastic, damp, yellow-gray.
4450	25	S	13	4-8-8-10 (22.0")	16	CL	Silty CLAY, moderately plastic, very stiff, damp, yellow-brown with red mottling, occasional layer of brown-gray highly plastic clay, and fine sand.
		U	14	PUSH (20.0")		CL	Top 18": Silty CLAY, moderately plastic, moist, gray. Bottom 2": SILT, nonplastic, 15-25% fine sand, damp, yellow-brown.
		S	15	12-25-35-30 (14.0")	60	CL ML	Top 4": Silty CLAY, moderately plastic, moist, yellow-brown. Bottom 10": SILT, slightly plastic, <10% fine sand, very dense, damp, yellow-brown with orange mottling.
4445	30	S	16	6-12-20-40 (17.0")	32	SP CL SP	Top 6": SAND, uniform, fine, <10% nonplastic fines, dense, dry, lt. brown. Mid 4": Silty CLAY, moderately plastic, hard, moist, lt. brown-gray. Bottom 7": SAND, similar to top 6".
4440	35	S	17	35-60-100 (14.0")	160	SP	SAND, uniform, fine, <10% nonplastic fines, very dense, dry, lt. brown.
4435	40	S	18	25-50-70 70 (14.0")	120	SP	SAND, uniform, fine, 3-8% nonplastic fines, very dense, dry, lt. brown.
4430	45	S	19	10-16-22 26 (14.0")	38	CL-CH	Silty CLAY, moderately to highly plastic, hard, dry, lt. brown-gray, few thin layers of fine sand, slightly fossiliferous (molluscs).
4425	50	S	20	25-20-40-40 (14.0")	60	SM	Silty SAND, uniform, fine, 10-15% slightly plastic fines, very dense, dry, lt. brown, trace of gravel to 3/4", slightly cemented. Clay in tip, moderately plastic, hard, dry, green-gray. (Water being added to help lift cuttings)
4420	55	S	21	35-40-70-100 (20.0")	110	SP GP	Top 14": SAND, uniform, fine to medium, mostly fine, 5-8% nonplastic fines, very dense, dry, lt. brown, trace of clay in thin layers. Bottom 6": Sandy GRAVEL, poorly graded, subangular to sub-round gravel to 2.0", coarse to fine sand, mostly fine, very dense, dry, brown.
4415	60						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-5(OW) J.O. 05996.02 Sheet 3 of 5

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410	65	S	22	100/5" (8.0")	100+	CL ML	Top 5": Sandy CLAY, moderately plastic, 15-20% medium to fine sand, 5-10% gravel, hard, dry, lt. gray-brown. Bot 3": SILT, slightly plastic, very dense, dry, lt. gray, volcanic ash. Switch to air rotary at 70' (open hole). Lt. brown and gray dust between 70 and 78 ft. (ash?)
4405	70						
4400	75	S	23	100/5" (12.0")	100+	ML	Sandy SILT, nonplastic, 15-25% fine sand, very dense, dry, lt. gray, volcanic ash.
4395	80						
4390	85	S	24	Not valid (4.0")		SP	Silty SAND, uniform, fine, very dense, 5-8% nonplastic fines, dry, brown. (Mechanical problems with hammer system, no valid blows)
4385	90						
		S	26	12*-28*-50*/5" (22.0")	*	ML	SILT, slightly to moderately plastic, very dense, dry, lt. brown, re-worked ash in tip. (Note: original auger hole completed to 95 ft. (S-25). Boring relocated 3.5 ft. to east and re-drilled and sampled with ODEX (air) from 92 ft (S-26).)
4380	95	S	25	Not valid (3.0")	*	ML	SILT, nonplastic, very dense, dry, lt. brown, volcanic ash (possibly re-worked).
		S	27	30*-50*/5 (12.0")	*	ML	SILT, nonplastic, very dense, damp, lt. brown to gray, volcanic ash.
4375	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-5(OW) J.O. 05996.02 Sheet 4 of 5

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4370	105						
4365	110						
		S	28	34*-27*-50*/3" (18.0")	*	ML	SILT, slightly to moderately plastic, 10-15% fine sand, very dense, damp, lt. brown, few pieces of cemented siltstone, thinly bedded.
4360	115						
4355	120						Ash in cuttings at 119 ft.
		S	29	20*-25*-35* (18.0")	*	SM MH	Top 10": Silty SAND, uniform, fine to medium, 10-20% nonplastic fines, very dense, damp, brown. Bottom 8": Clayey SILT, highly plastic, hard, sl. damp, lt. brown, few pieces of cemented siltstone.
4350	125						
4345	130						
		S	30	15*-20*-50*/4" (18.0")	*	SM ML	Top 6": Silty SAND, uniform, medium to fine, very dense, wet, brown. Bottom 12": Clayey SILT, very slightly plastic, very dense, wet, lt. gray-brown.
4340	135						
4335	140						
		S	31	Rods fell (22.0")	*	ML	Sandy SILT, slightly to moderately plastic, 15-20% fine sand, very dense, wet, layered, lt. brown at top to lt. gray at bottom, reworked ash (?).

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-5(OW) J.O. 05996.02 Sheet 5 of 5

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4330	145						
		S	32	15*-23*-25* (16.0")	*	SM	Silty SAND, uniform, fine, 10-20% slightly plastic fines, very dense, saturated, brown, trace of ash.
4325	150						
4320	155						
		S	33	15*-22*-50*/5" (22.0")	*	SM	Silty SAND at top, uniform, medium to fine, very dense, saturated, lt. brown-gray, reworked ash.
						ML	Sandy SILT, nonplastic, 15-20% fine sand, very dense, saturated, lt. gray, volcanic ash, thinly layered.
4315	160						End of Boring at 158.45 ft.
4310	165						
4305	170						
4300	175						
4295	180						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-6 J.O. 05996.02 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT
Client: Private Fuel Storage, L.L.C.
Coordinates: N7,320,062 E1,281,954
Groundwater Depth:
Contractor: Earthcore Drilling

Depth to Bedrock:
Driller: T. Kern

Logged by: R. Gillespie
Date Start - Finish: 12/08/98 - 12/08/98
Ground Elevation: 4476.5 ft
Total Depth Drilled: 71.0 ft
Rig Type: Mobile B-80

Methods:
Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem augers
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	1-1-4-3 (14.0")	5	ML	SILT, moderately plastic, < 10% fine sand, firm, damp, lt. brown, eolian.
4475							
4470	5	S	2	3-5-5-7 (12.0")	10	CL	Silty CLAY, highly plastic, stiff, damp, lt. green-gray with white mottling, thinly layered.
		U	3	PUSH (21.0")		CH	Silty CLAY, highly plastic, damp, brown-gray.
4465	10	S	4	3-3-4-5 (20.0")	7	CH ML	Top 12": Silty CLAY, similar to above. Bottom 8": SILT, moderately plastic, firm, damp, lt. yellow-brown.
4460	15	S	5	10-20-20-20 (14.0")	40	ML SM	Top 5": SILT, similar to above. Bottom 9": Silty SAND, uniform, fine, 35-45% nonplastic fines, very dense, dry, lt. brown.
4455	20	S	6	7-10-10-12 (19.0")	20	ML	Sandy SILT, moderately plastic, 10-20% fine sand, very stiff, damp, lt. yellow-brown, thinly bedded.

Legend/Notes
- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
[signature]	08/31/99

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450	25	S	7	3-5-7-8 (24.0")	12	ML	Clayey SILT, moderately plastic, stiff, damp, lt. yellow-brown.
4445	30	S	8	16-30-40-40 (19.0")	70	ML SP	Top 6": Sandy SILT, nonplastic, 30-40% fine sand, very dense, dry, lt. brown. Bottom 13": SAND, uniform, fine, <10% nonplastic fines, very dense, dry, lt. brown.
4440	35	S	9	40-100 (8.0")	100+	SP	SAND, similar to above, thinly bedded.
4435	40	S	10	35-60-100 (13.0")	160	SP	SAND, similar to above.
4430	45	S	11	25-25-25-15 (15.0")	50	SP CH	Top 7": SAND, similar to above. Bottom 8": becomes interbedded CLAY and Sandy CLAY, highly plastic, fine sand, hard, dry, lt. gray.
4425	50	S	12	60-100 (10.0")	100+	SP	Gravelly SAND, poorly graded, fine sand, 15-25% sub-angular to sub-round gravel to 1.0", very dense, dry, lt. brown.
4420	55						
4415	60	S	13	20-25-16-25 (17.0")	41	SC&CL	Interbedded Clayey SAND and CLAY, moderately plastic, fine sand, hard, dry (water added), lt. yellow-gray.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved P.J. Trudeau   Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-6 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT   Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410	65						
4405	70	S	14	40-100 (7.0")	100+	SC	Clayey SAND, uniform, fine, slightly plastic, very dense, dry (water added), lt. gray-brown.
							End of Boring at 71 ft.
4400	75						
4395	80						
4390	85						
4385	90						
4380	95						
4375	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: P. Trudeau   Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-7 J.O. 05996.02 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT
Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 12/10/98 - 12/10/98
Coordinates: N7,320,062 E1,282,159
Ground Elevation: 4473.4 ft
Groundwater Depth:
Depth to Bedrock:
Total Depth Drilled: 68.45 ft
Contractor: Earthcore Drilling
Driller: T. Kern
Rig Type: Mobile B-80

Methods:
Casing Used: None
Drilling Soil: 3 1/4" ID hollow-stem augers
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	13-15-21-25 (4.0")	36	CL	Silty CLAY, moderately plastic, hard(frozen), damp, gray-brown.
4470							
	5	S	2	4-5-6-22 (19.0")	11	CH	Silty CLAY, highly plastic, stiff, damp, yellow-brown with white on bedding planes.
		U	3	PUSH (19.0")		SP	SAND, uniform, fine, < 10% nonplastic fines, yellow-brown.
4465							
	10	S	4	16-25-35-40 (12.0")	60	SM	Silty SAND, uniform, fine, 25-35% nonplastic fines, very dense, dry, lt. brown.
4460							
	15	S	5	18-18-10-12 (12.0")	28	ML	Top 5": Sandy SILT, nonplastic, 15-25% fine sand, very dense, dry, yellow-brown. Bottom 7": SILT, moderately plastic, very stiff, damp, lt. yellow-brown.
4455							
	20	S	6	6-6-6-9 (21.0")	12	MH	Clayey SILT, highly plastic, stiff, damp, lt. yellow-brown, with ostracodes.

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
B.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-7 J.O. 05996.02 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450							
	25	S	7	10-10-12-18 (18.0")	22	ML	SILT, slightly plastic, compact, damp, yellow-gray with orange mottling, thinly bedded, trace clay in thin layers.
4445							
	30	S	8	35-70-70-100/5" (14.0")	140	SP	SAND, uniform, fine, < 10% nonplastic fines, very dense, dry, lt. brown.
4440							
	35	S	9	35-50-100 (14.0")	150	SP	SAND, similar to above.
4435							
	40	S	10	25-50-100 (12.0")	150	SP	SAND, similar to above.
4430							
	45	S	11	60-100/5" (5.0")	100+	SP	Top 1": SAND, similar to above. Bottom 4": Gravelly SAND, coarse to fine, mostly fine, sub-round to sub-angular gravel to 3/4", 3-5% nonplastic fines, very dense, dry. lt. browh.
4425							
	50	S	12	100/5" (5.0")	100+	ML	Sandy SILT, slightly to moderately plastic, 20-30% fine sand, very dense, dry, lt. brown, layer of clay at top.
4420							
	55						
4415							
	60	S	13	50-100/5" (8.0")	100+	GM SM	Top 4": Silty GRAVEL, poorly graded, sub-round to sub-angular to 1", coarse to fine sand, mostly fine, 15-25% slightly plastic fines, very dense, dry (water added), lt. brown-gray. Bottom 4": Silty SAND, uniform, fine to medium, 10-20% slightly plastic fines, very dense, lt. gray-brown.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved: RJ Duncan — Date: 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-7 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
1410							Very slow augering below 60 ft.
	65						
1405		S	14	100/5" (5.0")	100+	ML	Sandy SILT, non to slightly plastic, 30-40% fine sand, trace of coarse sand-fine gravel, very dense, dry, layered yellow-brown and gray-brown.
	70						End of Boring at 68.45 ft.
1400							
	75						
1395							
	80						
1390							
	85						
1385							
	90						
1380							
	95						
1375							
	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-8 J.O. 05996.02 Sheet 1 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 12/09/98 - 12/10/98
Coordinates: N7,320,007 E1,282,057 — Ground Elevation: 4474.2 ft
Groundwater Depth: — Depth to Bedrock: — Total Depth Drilled: 70.5 ft
Contractor: Earthcore Drilling — Driller: T. Kern — Rig Type: Mobile B-80

Methods: — Casing Used: None
Drilling Soil: 3 1/4" ID hollow stem augers
Sampling Soil: 2.0" OD split-spoon, SPT, 24" drive
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	S	1	4-7-7-7 (14.0")	14	ML CL	Top 2": Sandy SILT, moderately plastic, fine sand, firm, damp, brown, eolian. Bottom 12": Silty CLAY, moderately plastic, stiff, damp, green-gray.
4470							
	5	S	2	2-3-4-6 (20.0")	7	CH	Silty CLAY, highly plastic, firm, damp, lt. yellow-brown mottled with white and red-brown.
		S	3	5-9-7-8 (19.0")	16	ML	SILT, slightly plastic, 5-15% fine sand, dense, damp, lt. yellow-gray.
4465	10	S	4	18-12-18-20 (13.0")	30	SM	Silty SAND, uniform, fine, 10-15% nonplastic fines, very dense, dry, lt. brown.
		S	5	20-20-30-35 (12.0")	50	SM	Silty SAND, similar to above, except 15-25% nonplastic fines.
4460	15	S	6	18-20-18-16 (9.0")	38	SM	Silty SAND, uniform, fine, 30-35% nonplastic fines, very dense, dry, lt. brown.
4455	20	S	7	6-8-10-12 (20.0")	18	SM MH	Top 3": Silty SAND, similar to above. Bottom 17": Clayey SILT, highly plastic, very stiff, damp, lt. yellow-brown, thinly bedded, white on bedding surfaces, ostracodes.

Legend/Notes
- Datum is MSL - NGVD29.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  S = 2" OD Split Spoon

Approved	Date
P.J. [signature]	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-8 J.O. 05996.02 Sheet 2 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4450	25	S	8	8-8-14-14 (20.0")	22	CL	Silty CLAY, moderately plastic, very stiff, damp, lt. yellow-gray, interbeds of silt and clay.
4445	30	S	9	35-50-100 (11.0")	150	SM	Silty SAND, uniform, fine, 10-15% nonplastic fines, very dense, dry, lt. brown, thinly layered, trace of clay.
4440	35	S	10	35-100 (10.0")	100+	SP	SAND, uniform, fine, very dense, dry, lt. brown, water added.
4435	40	S	11	Not valid (15.0")		SP	SAND, uniform, fine, <3% nonplastic fines, very dense, dry, lt. brown. (Mechanical problem with hammer, water added to clear cuttings)
4430	45	S	12	35-40-20-30 (13.0")	60	SP ML	Top 8": SAND, similar to above. Bottom 5": Clayey SILT, slightly to moderately plastic, hard, dry, lt. gray-brown with orange mottling.
4425	50	S	13	50-100 (9.0")	100+	GW	Sandy GRAVEL, well graded, coarse to fine, sub-round to sub-angular gravel to 3/4", <5% nonplastic fines, very dense, dry, lt. brown.
4420	55						
4415	60	S	14	40-60-60-60 (13.0")	120	SM	Silty SAND, uniform, fine, 10-15% slightly plastic fines, very dense, dry, lt. brown, thinly bedded.

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-8 J.O. 05996.02 Sheet 3 of 3

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Elev (ft)	depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
4410	65						
4405	70	S	15	100/5" (5.0")	100+	SM	Silty SAND, uniform, fine, nonplastic fines, very dense, dry, lt. gray, volcanic ash.
							End of Boring at 70.45 ft.
4400	75						
4395	80						
4390	85						
4385	90						
4380	95						
4375	100						

Note: See Sheet 1 for Boring Summary and Legend Information

Approved [signature] Date 08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-N J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility, Skull Valley, UT — Logged by: R. Gillespie

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 10/15/98 - 10/15/98

Coordinates: N7,320,222 E1,282,049 — Ground Elevation: 4474.1 ft

Groundwater Depth: — Depth to Bedrock: — Total Depth Drilled: 11.0 ft

Contractor: Layne-Christensen — Driller: Crowley — Rig Type: Mobile

Methods: — Casing Used:

Drilling Soil: 4.25 in. ID hollow-stem auger

Sampling Soil: 3.0 in. OD Shelby tube, galvanized

Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0						NO SAMPLES TAKEN 0 TO 5 FT.
4470	5	U	1	PUSH (24.0")		CL MH	Top 21.5": Silty CLAY, moderately plastic, gray. Bottom 2.5": Clayey SILT, highly plastic, light brown, very thinly laminated.
		U	2	PUSH (23.0")		MH	Clayey SILT, similar to above.
4465	10	U	3	PUSH (23.0")		CH	Silty CLAY, highly plastic, light yellow-brown.
							End of Boring at 11.0 ft.
4460	15						
4455	20						

Legend/Notes

- Datum is MSL - NGVD29.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
P.J. Trudeau	08/31/99

Stone & Webster Engineering Corporation	BORING LOG	Boring CTB-S J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility, Skull Valley, UT | Logged by: R. Gillespie
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 10/15/98 - 10/15/98
Coordinates: N7,320,065 E1,282,053 | Ground Elevation: 4474.5 ft
Groundwater Depth: | Depth to Bedrock: | Total Depth Drilled: 11.0 ft
Contractor: Layne-Christensen | Driller: Crowley | Rig Type: Mobile

Methods: | Casing Used:
Drilling Soil: 4.25 in. ID hollow-stem auger
Sampling Soil: 3.0 in. OD Shelby tube, galvanized
Drilling Rock:

Comments:

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0						NO SAMPLES TAKEN 0 TO 5 FT.
4470	5	U	1	PUSH (24.0")		MH	Clayey SILT, highly plastic, light brown-gray, moist, thinly laminated.
		U	2	PUSH (23.0")		CH	Silty CLAY, highly plastic, damp, light brown-gray.
4465	10	U	3	PUSH (24.0")		MH SM&ML ML	Top 14": Clayey SILT, highly plastic, damp, light brown. Middle 7": Silty SAND, fine, 10-20% nonplastic fines, and SILT, nonplastic, light yellow-brown. Bottom 3": SILT, moderately plastic, yellow-brown.
							End of Boring at 11.0 ft.
4460	15						
4455	20						

Legend/Notes
- Datum is MSL - NGVD29.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  - S = 2" OD Split Spoon
  - U = 3" OD Thin-Walled Tube

Approved	Date
[signature]	08/31/99

Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

ATTACHMENT 2

GEOTECHNICAL LABORATORY TESTING
January 1997

Private Fuel Storage Facility
Skull Valley
Private Fuel Storage, LLC

Responsible Engineer	Alan C Smith	6/11/97 Date
Reviewer	Paul J. Dudeau (I)	6/11/97 Date
Approved	[illegible]	6/11/97 Date

(I) = Independent Review

QUALITY ASSURANCE CATEGORY I AND III
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
Copyright 1997

APPENDIX 2A

ATTACHMENT 2

GEOTECHNICAL LABORATORY TESTING
JANUARY 1997

Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

ATTACHMENT 2

TABLE OF CONTENTS

INTRODUCTION

The Geotechnical Laboratory received 20 boxes of split spoon jar samples and 9 undisturbed tube samples from the Skull Valley site on November 13, 1996. A testing program was developed identifying types of tests to be performed and which samples to test. Testing began on November 15, 1996 and ended on January 10, 1997.

The tests performed were water content, Atterberg limits, percent fines, specific gravity, consolidation, and unconsolidated - undrained triaxial compression. They were conducted in accordance with the following American Society for Testing and Materials standards.

C-136	1996	Test Method for Sieve Analysis of Fine and Coarse Aggregates
D-854	1992	Test Method for Specific Gravity of Soils
D-1140	1992	Test Method for Amount of Material in Soils Finer Than the No. 200 Sieve
D-2216	1992	Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D-2435	1990	Test Method for One-Dimensional Consolidation Properties of Soils
D-2850	1995	Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression
D-4318	1995	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

TEST RESULTS

The results of testing for percent passing the no. 200 sieve are presented in Table 1. The Atterberg Limit test results are shown in Table 2.

Two undisturbed tube samples were tested for unconsolidated-undrained compressive strength. The plots and data are shown in Appendix A. We were unable to conduct the triaxial test in accordance with ASTM D-2166, Unconfined Compressive Strength of Cohesive Soils, due to the weak soil structure. The samples would have collapsed while unconfined before the test began. We used ASTM D-2850 because we could extrude the sample directly into a membrane preventing a collapse of the sample. The compressive strength measured using this procedure should be the same obtained if D-2166 had been used. An attempt was made to perform a third test but could not find a suitable sample in the tubes taken between 8 and 12 ft depth.

A total of 5 consolidation tests were conducted on undisturbed tube samples from borings C-1 and C-2. The stress vs. strain plots, strain vs. time plots, and data are presented in Appendix B. Initially two tests were started (C-1/U-3D and C-2/U2E). When the applied stress reached 0.5 tsf they were inundated with distilled water. The incremental loads were added every 30 to 60 minutes until the applied stress was 2 tsf. At that load the strain vs. log of time plot appeared to indicate primary consolidation was not completed. The 2 tsf applied stress was left on the samples for more than 10,000 minutes. Another sample was trimmed (C-2/U-2C) and started, when we assumed we had loaded the first two tests too rapidly. But, when we examined the strain vs. square root of time plot, it showed primary consolidation was finished within 2 to 4 minutes after applying the new load. We now consider the additional strain after 4 minutes of elapsed time to be secondary consolidation. We were concerned that the large amount of secondary consolidation may be due to the inundation of the samples with distilled water. Therefore, another test was started (C-1/U-3C), the porous stones were moistened and the apparatus was wrapped in plastic to prevent loss of moisture. Since all load frames were occupied, test C-2/U-2E was stopped at 0.5 tsf and replaced with test C-1/U-3C. When test C-1/U-3C was completed, it was discovered that the sample had pulled moisture from the porous stones. A fifth test (C-1/U-2B) was conducted using dry porous stones with new loads added every 30 to 60 minutes.

The soil tested is a moderately to highly plastic, clayey silt, partially saturated. It appears to be alkaline since the conductivity of the distilled water inundating the samples was high (over 18,000 umho). Also, the soil reacts immediately to a 10% solution of hydrochloric acid. The stress vs. strain plots appear to show the maximum past pressure to be approximately 3 tsf. The secondary consolidation is significant after exceeding the maximum past pressure. The large secondary consolidation may be due to the deformation of a weakly cemented structure of the silt. The distilled water inundating the consolidation samples does not seem to have effected the results since the stress-strain curves are similar to those tests conducted without inundating the sample.

≠ test cancelled

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER ENGINEERING CORPORATION

SHEET 1/2

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.01	DATE	12/12/96				ACS	11/14/96
SITE: Skull Valley Goshute Indian Reservation		REVISED BY	ACS				APPROVED BY: NG	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
A1	S	7	30-31½					Y													changed gradation to % fines
A3	S	8	35-36½					Y													
A4	S	7	30-31½				Y														
B3	UD	2	10-12		Y	≠					≠			≠							unable to trim into ring; UU sample unsuitable
B4	U	3	10-12		Y	Y					Y			≠							unable to trim into ring
C2	U	2	10-12		Y	Y					Y			Y							
D1	S	4	15-16½		≠	≠															cancelled
D1	S	5	20-21½				Y														
D1	S	6	25-26½					Y													
D4	S	5	20-21½		≠	≠															cancelled
D4	S	6	25-26½				Y														
D4	S	7	30-31½					Y													

3 / TEST TO BE PERFORMED (NUMBERS USED TO INDICATE PRIORITIES)

3 Y TEST COMPLETED AND CHECKED BUT FINAL REPORT NOT COMPLETE

3 X TEST RESULTS REPORTED IN FINAL FORM (ALL LABORATORY WORK DONE)

~~Ⅎ~~ test ca[illegible]'ed

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER ENGINEERING CORPORATION

SHEET 2/2

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.01	DATE	12/12/96				ACS	11/14/96
SITE: Skull Valley Goshute Indian Reservation		REVISED BY	ACS				APPROVED BY: NTG	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
E3	S	3	10-11½		✓	✓															
E3	S	7	30-31½				←	✓													
E4	S	6	25-26½				✓														
AR3	S	2	5-6½		✓	✓															
AR4	S	2	5-6½		✓	✓															
C1	U	3	10-12		✓	✓			✓					✓							added due to unsuitable B3 & B4 soils
B2	U	1	8-10		X	≠								≠							soil unsuitable, fractured vertically

3/ TEST TO BE PERFORMED (NUMBERS

3✓ TEST COMPLETED AND CHECKED

3✓ TEST RESULTS REPORTED IN FINAL

JO 05996.01
PRIVATE FUEL STORAGE, LLC
SKULL VALLEY

TABLE 1
LAB TEST RESULTS - % PASSING #200 SIEVE

BORING	SAMPLE	DEPTH (ft)	WATER CONTENT	% PASSING
A1	S7	30-31.5	1.1	4.3
A3	S8	35-36.5	1.3	4.1
A4	S7	30-31.5	3.8	13.9
D1	S5	20-21.5	20.7	91.1
D1	S6	25-26.5	2.9	5.7
D4	S6	25-26.5	18.0	84.2
D4	S7	30-31.5	1.1	3.8
E3	S7	30-31.5	1.5	3.2
E4	S6	25-26.5	20.5	93.9

JO 05996.01
PRIVATE FUEL STORAGE, LLC
SKULL VALLEY

TABLE 2
LAB TEST RESULTS - ATTERBERG LIMITS

BORING	SAMPLE	DEPTH (ft)	WATER CONTENT	LL	PL	PI
B4	U3D	10.4	27.4	42.5	24.7	17.8
C1	U3B	10.8	30.3	33.0	28.1	4.9
C1	U3C	11.2	38.9	47.8	34.6	13.2
C1	U3D	11.4	46.7	61.1	44.1	17.0
C2	U2C	10.9	27.6	34.6	26.9	7.7
C2	U2E	11.7	39.7	41.2	28.5	12.7
E3	S3	10-11.5	37.3	49.9	27.2	22.7
AR3	S2	5-6.5	16.7	29.3	20.3	9.0
AR4	S2	5-6.5	20.5	36.4	30.1	6.3

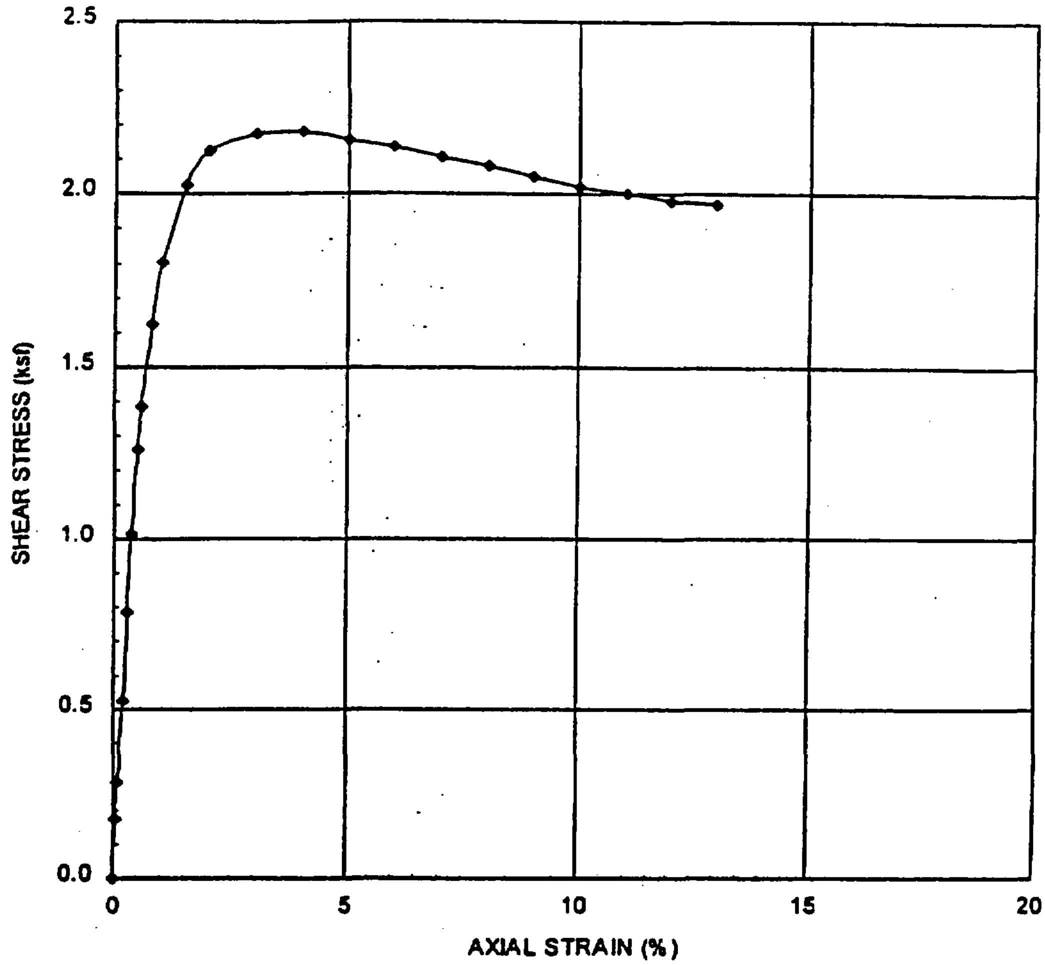
Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

ATTACHMENT 2

GEOTECHNICAL LABORATORY TESTING

TRIAXIAL TEST PLOTS and DATA



SAMPLE INFORMATION:

BORING:	B-4	DATE:	12/21/96
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	10.4 ft	CHECKED:	PJT
DESCRIPTION:	silty CLAY/clayey SILT		

SPECIMEN INFORMATION: (start of shear)

HEIGHT:	0.532 ft	WATER CONTENT:	27.4 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	67.1 pcf
AREA:	0.0443 ft^2		

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.6 %/min
CELL PRESSURE:	1.3 ksf		

UNDRAINED SHEAR STRENGTH:	2.18 ksf
COMPRESSIVE STRENGTH:	4.36 ksf
FAILURE STRAIN:	4.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	UNCONSOLIDATED UNDRAINED COMPRESSION TEST BORING B-4, SAMPLE U-3D	JO 05996.01 January 1997

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-4	DATE:	12/21/96
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	10.4 ft	CHECKED:	PJT
DESCRIPTION:	silty CLAY/clayey SILT		

SPECIMEN INFORMATION: (start of shear)

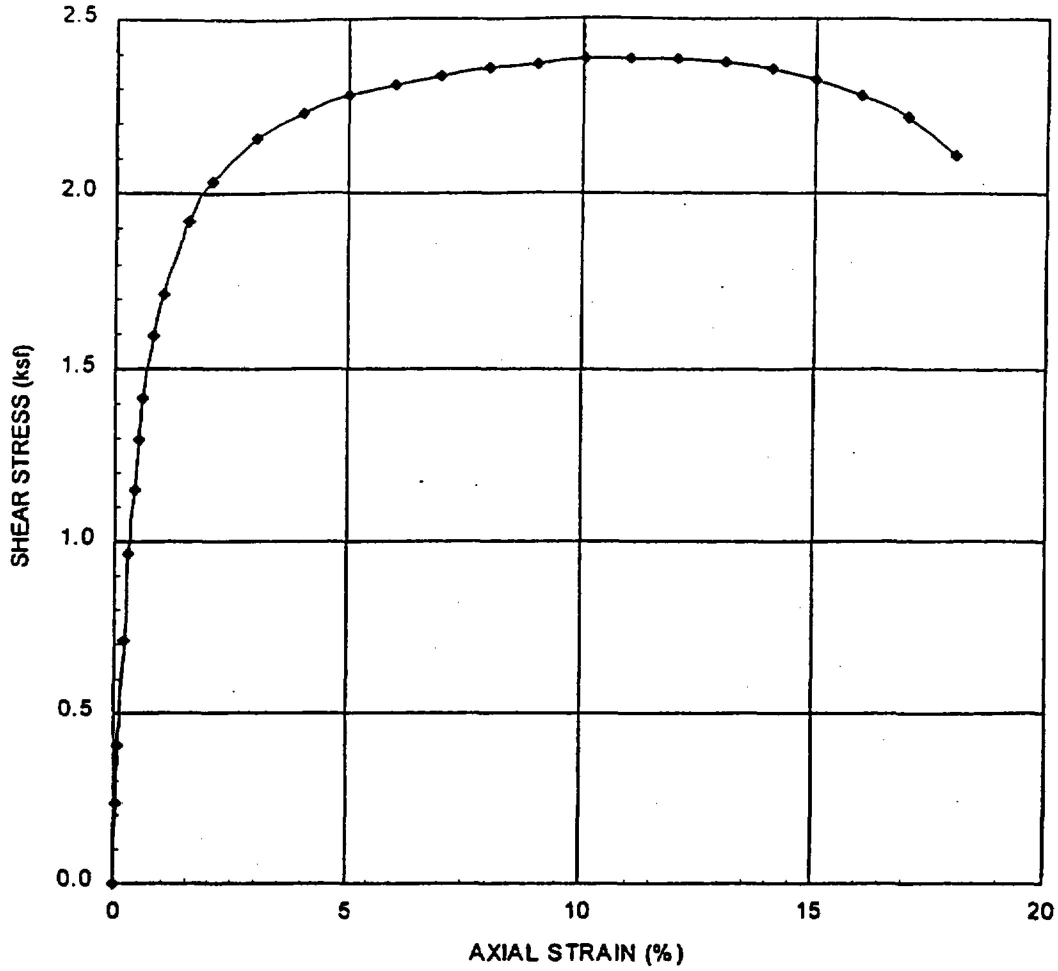
HEIGHT:	0.532 ft	WATER CONTENT:	27.4 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	67.1 pcf
AREA:	0.0443 ft^2		

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.6 %/min
CELL PRESSURE:	1.3 ksf		

UNDRAINED SHEAR STRENGTH:	2.18 ksf
COMPRESSIVE STRENGTH:	4.36 ksf
FAILURE STRAIN:	4.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.34	2.70	0.00	0.000	0.0443	0.00	0.00
0.42	5.38	0.05	0.016	0.0443	0.35	0.18
0.50	7.06	0.10	0.025	0.0443	0.57	0.29
0.66	10.69	0.20	0.047	0.0444	1.05	0.52
0.83	14.72	0.30	0.070	0.0444	1.57	0.79
0.99	18.23	0.40	0.090	0.0445	2.03	1.02
1.15	22.00	0.50	0.112	0.0445	2.52	1.26
1.31	23.88	0.60	0.123	0.0446	2.77	1.38
1.64	27.64	0.80	0.145	0.0447	3.25	1.63
1.96	30.45	1.00	0.162	0.0447	3.61	1.80
2.77	34.06	1.50	0.183	0.0450	4.06	2.03
3.58	35.70	2.00	0.192	0.0452	4.25	2.12
5.20	36.85	3.00	0.199	0.0457	4.35	2.18
6.83	37.27	4.00	0.201	0.0461	4.36	2.18
8.45	37.27	5.00	0.201	0.0466	4.31	2.16
10.07	37.30	6.00	0.201	0.0471	4.27	2.14
11.69	37.22	7.00	0.201	0.0476	4.22	2.11
13.31	37.13	8.00	0.200	0.0482	4.16	2.08
14.93	37.00	9.00	0.200	0.0487	4.10	2.05
16.56	36.88	10.00	0.199	0.0492	4.04	2.02
18.18	36.92	11.00	0.199	0.0498	4.00	2.00
19.80	37.03	12.00	0.200	0.0503	3.97	1.98
21.42	37.20	13.00	0.201	0.0509	3.94	1.97



SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/18/96
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	11.1 ft	CHECKED:	PJT
DESCRIPTION:	clayey SILT		

SPECIMEN INFORMATION: (start of shear)

HEIGHT:	0.553 ft	WATER CONTENT:	35.6 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	57.9 pcf
AREA:	0.0444 ft²		

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.6 %/min
CELL PRESSURE:	1.3 ksf		

UNDRAINED SHEAR STRENGTH:	2.39 ksf
COMPRESSIVE STRENGTH:	4.77 ksf
FAILURE STRAIN:	11.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	UNCONSOLIDATED UNDRAINED COMPRESSION TEST BORING C-2, SAMPLE U-2D	JO 05996.01 January 1997

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/18/96
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	11.1 ft	CHECKED:	PJT
DESCRIPTION:	clayey SILT		

SPECIMEN INFORMATION: (start of shear)

HEIGHT:	0.553 ft	WATER CONTENT:	35.6 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	57.9 pcf
AREA:	0.0444 ft^2		

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.6 %/min
CELL PRESSURE:	1.3 ksf		

UNDRAINED SHEAR STRENGTH:	2.39 ksf
COMPRESSIVE STRENGTH:	4.77 ksf
FAILURE STRAIN:	11.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.44	2.73	0.00	0.000	0.0444	0.00	0.00
0.52	6.30	0.05	0.021	0.0444	0.47	0.23
0.61	8.92	0.10	0.036	0.0444	0.81	0.41
0.78	13.65	0.20	0.064	0.0445	1.43	0.71
0.95	17.50	0.30	0.086	0.0445	1.93	0.97
1.12	20.34	0.40	0.102	0.0446	2.30	1.15
1.28	22.60	0.50	0.116	0.0446	2.59	1.30
1.45	24.42	0.60	0.126	0.0447	2.83	1.41
1.79	27.22	0.80	0.143	0.0447	3.19	1.59
2.13	29.13	1.00	0.154	0.0448	3.43	1.71
2.97	32.50	1.50	0.173	0.0451	3.84	1.92
3.82	34.45	2.01	0.185	0.0453	4.08	2.04
5.51	36.75	3.01	0.198	0.0458	4.33	2.16
7.20	38.21	4.01	0.206	0.0462	4.47	2.23
8.89	39.32	5.02	0.213	0.0467	4.56	2.28
10.57	40.24	6.01	0.218	0.0472	4.62	2.31
12.26	41.08	7.02	0.223	0.0477	4.68	2.34
13.95	41.85	8.02	0.228	0.0483	4.72	2.36
15.64	42.46	9.02	0.231	0.0488	4.74	2.37
17.33	43.13	10.03	0.235	0.0493	4.77	2.38
19.02	43.64	11.03	0.238	0.0499	4.77	2.39
20.71	44.05	12.03	0.240	0.0505	4.77	2.38
22.40	44.38	13.04	0.242	0.0510	4.75	2.37
24.09	44.54	14.04	0.243	0.0516	4.71	2.36
25.78	44.48	15.04	0.243	0.0522	4.65	2.33
27.46	44.20	16.04	0.241	0.0529	4.57	2.28
29.15	43.57	17.04	0.238	0.0535	4.44	2.22
30.84	42.00	18.05	0.229	0.0542	4.22	2.11

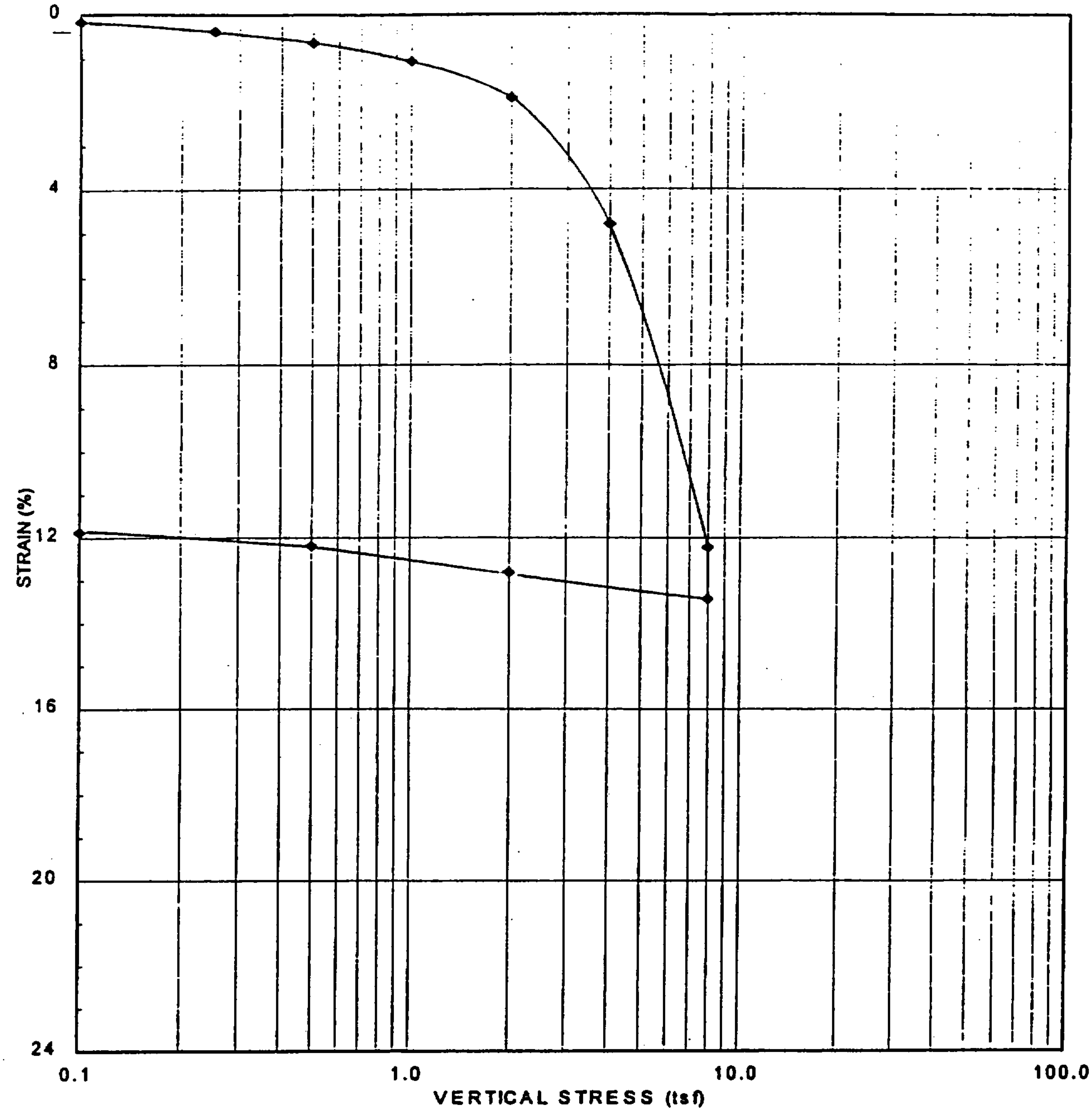
Report No. 05996.01-G(B)-2 Rev 1
SWEC Project No. 05996.01

GEOTECHNICAL DATA REPORT

ATTACHMENT 2

GEOTECHNICAL LABORATORY TESTING

CONSOLIDATION TEST PLOTS and DATA



SAMPLE INFORMATION:

BORING:	C-1	DATE:	1/9/97
SAMPLE:	U-3B	TESTED BY:	ACS
DEPTH:	10.8 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

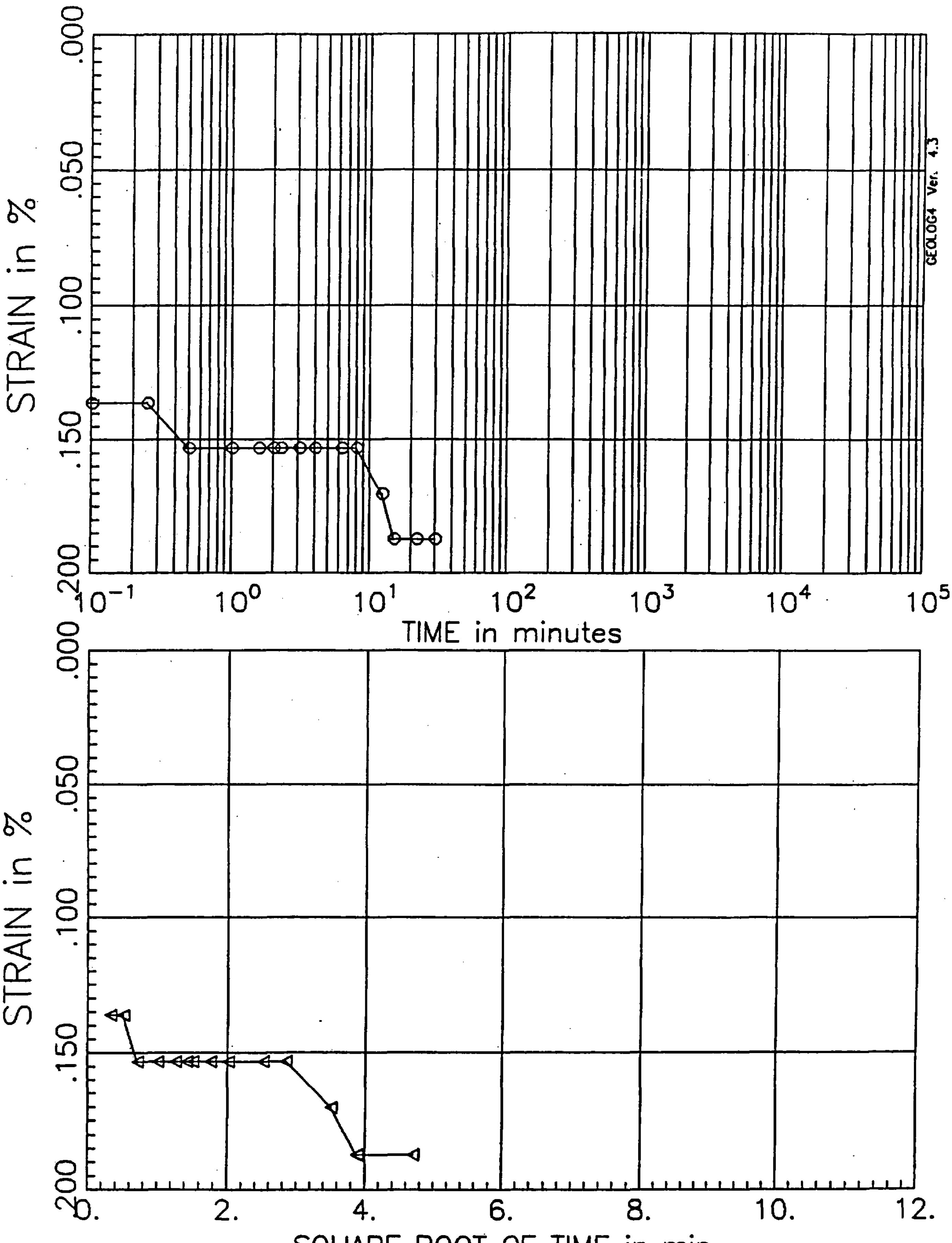
	INITIAL	FINAL
WATER CONTENT:	30.3 %	28.7 %
DRY UNIT WEIGHT:	64.7 pcf	73.4 pcf
VOID RATIO;	1.625	1.315
SATURATION:	50.7 %	59.3 %

SPECIFIC GRAVITY: 2.72 (est)

NOTE: Sample was not inundated and porous stones were dry

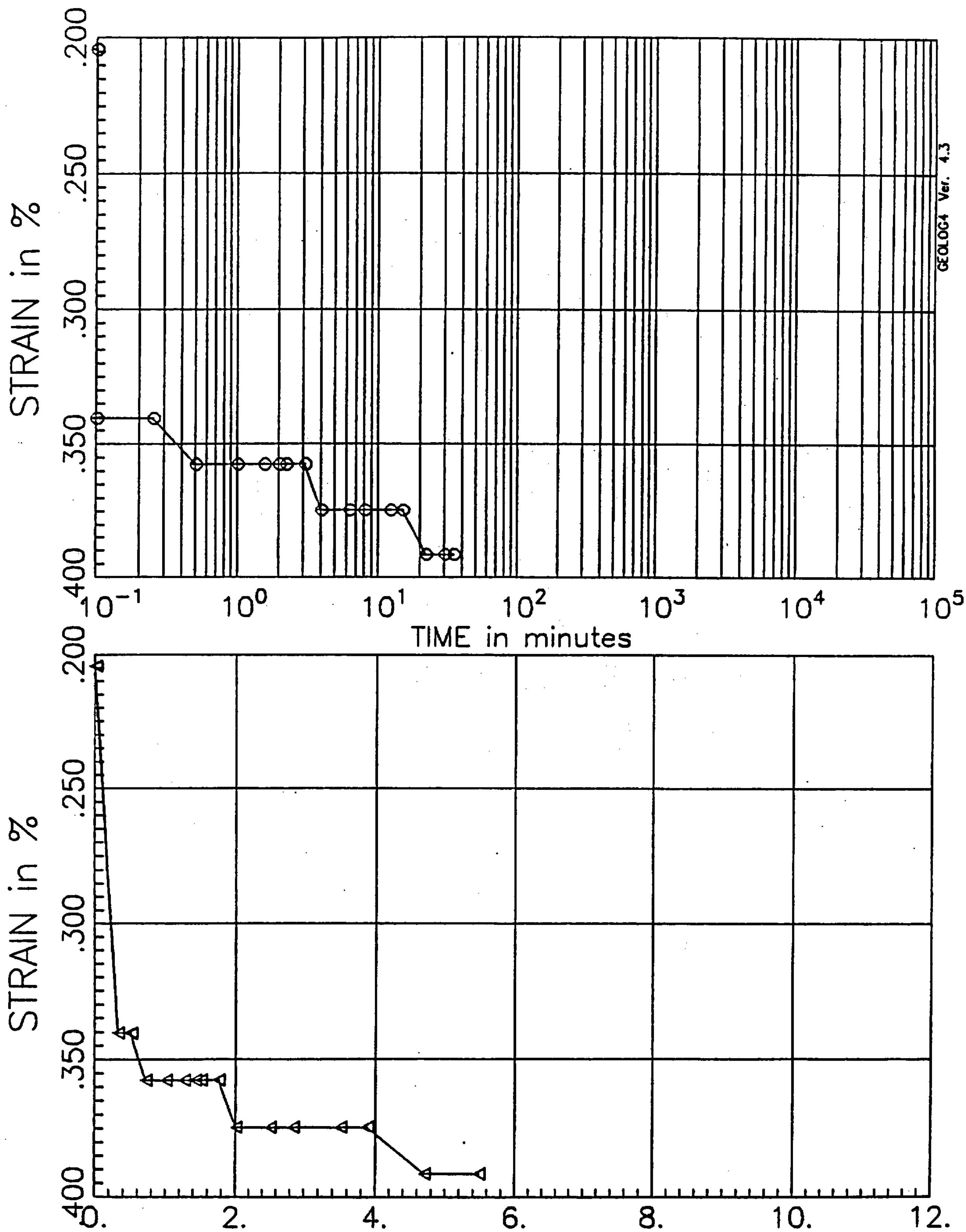
PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING C-1, SAMPLE U-3B	JO 05996.01 January 1997



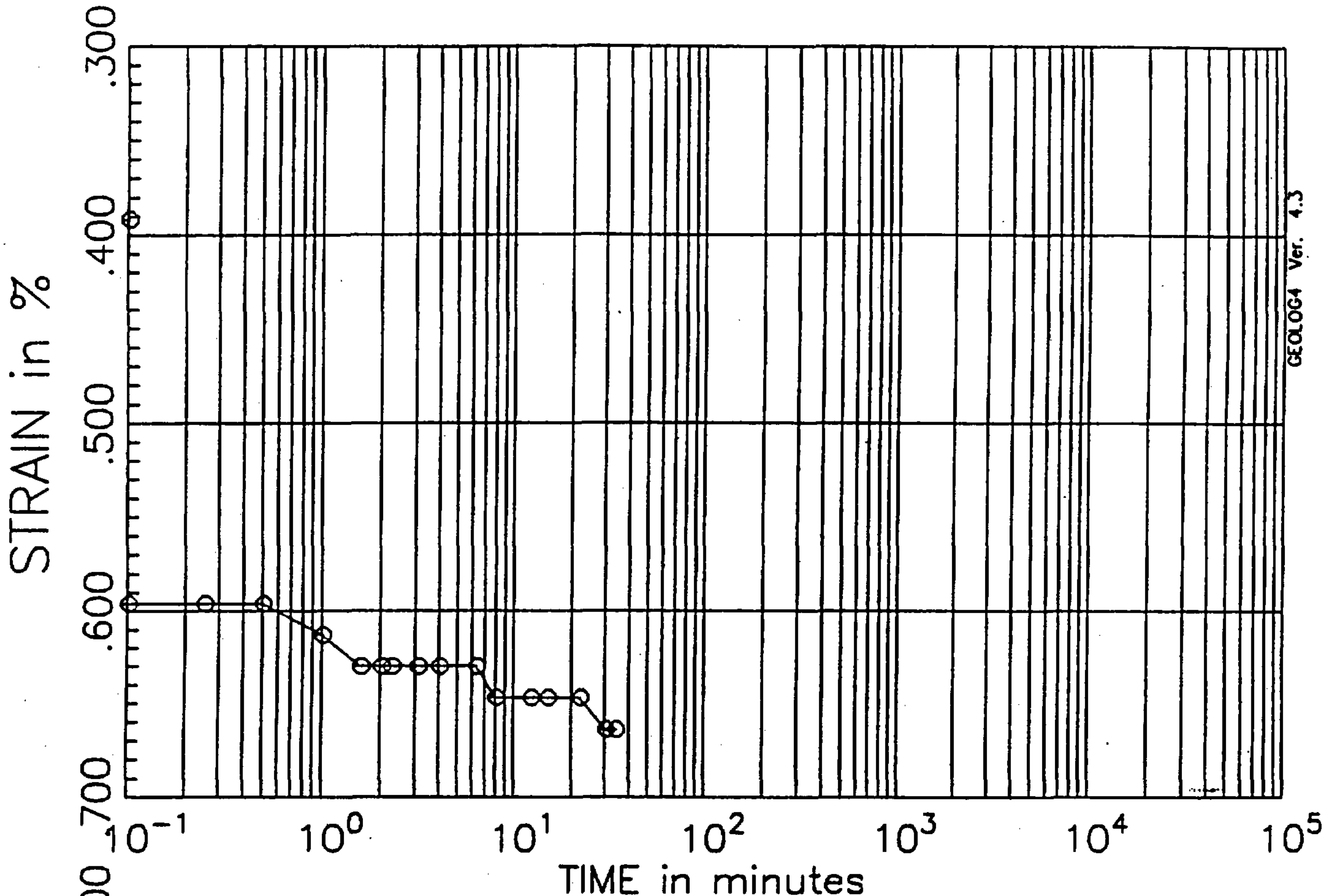
PRESSURE INCREMENT
from 0.00 tsf to 0.10 tsf

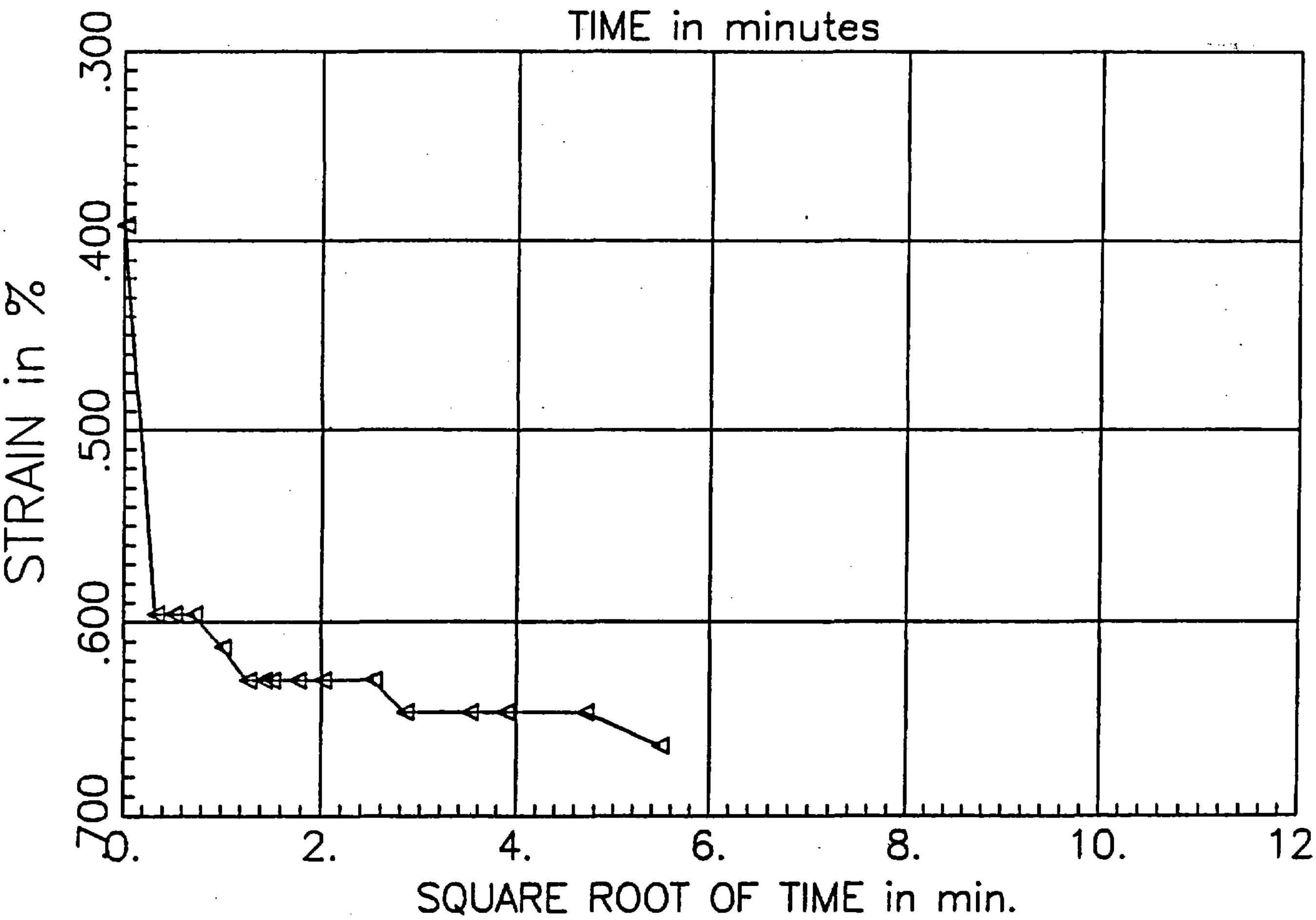
Test No: 5
Testname: C1–U3B



PRESSURE INCREMENT
from 0.10 tsf to 0.25 tsf

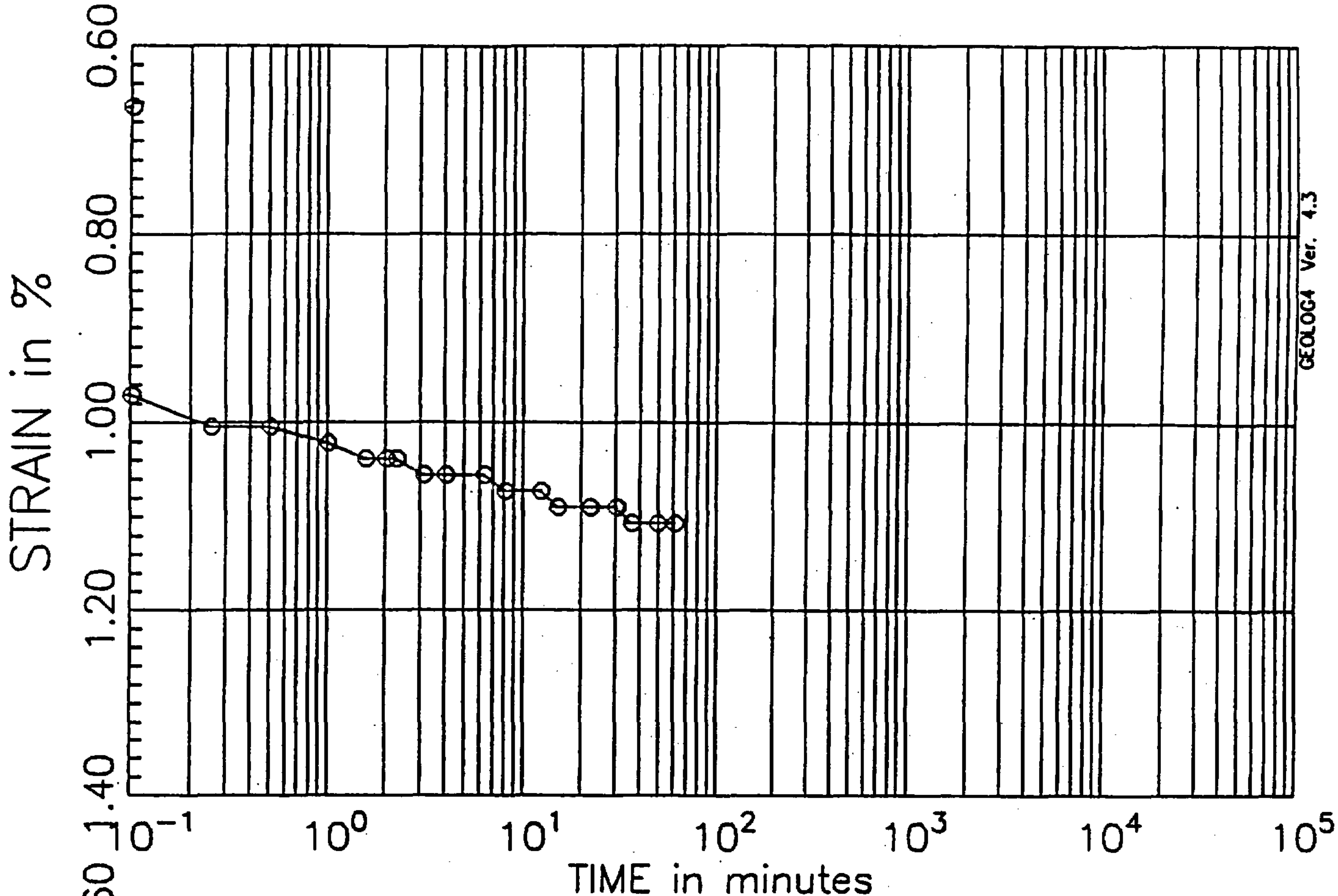
Test No: 5
Testname: C1-U3B

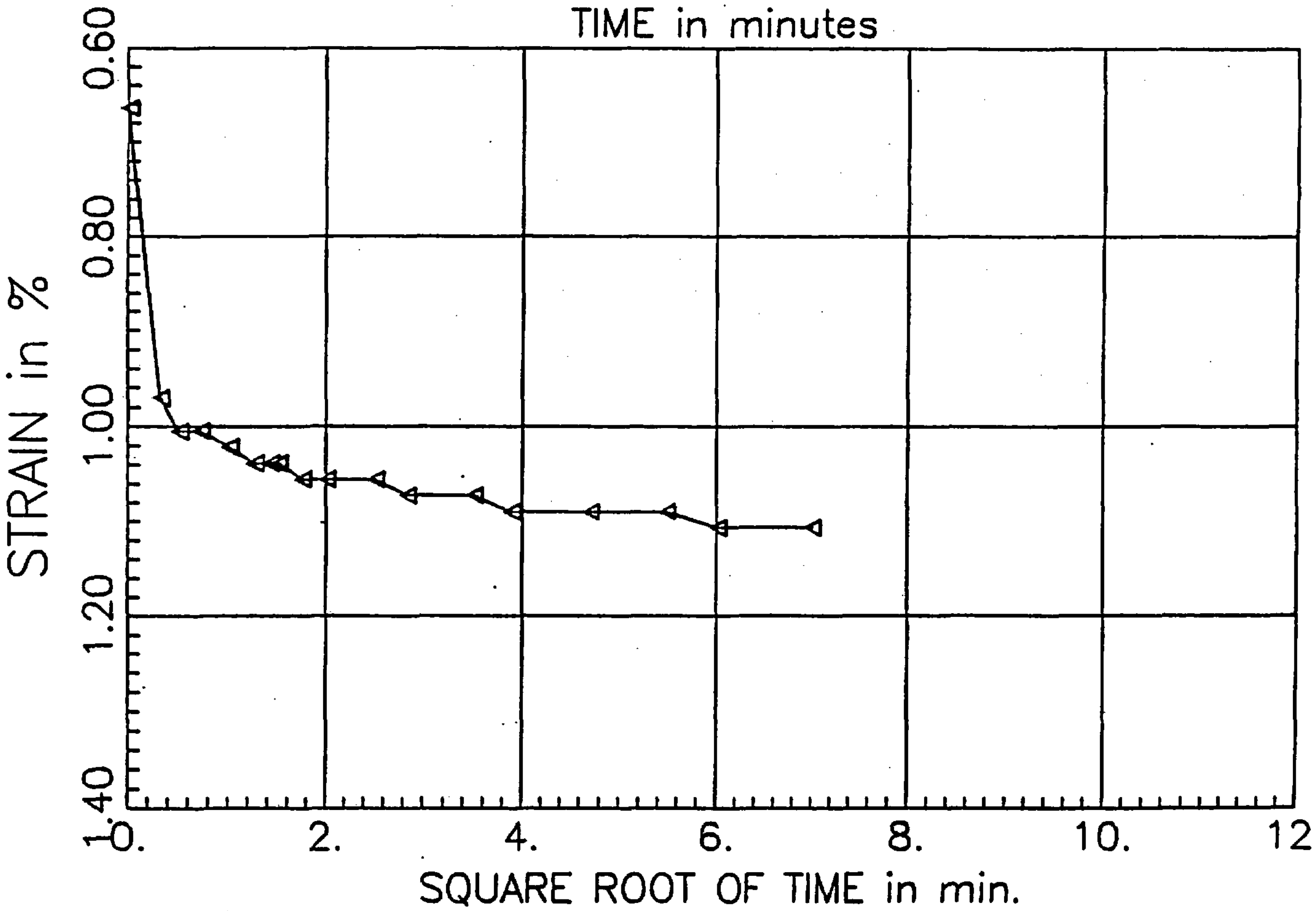




PRESSURE INCREMENT
from 0.25 tsf to 0.50 tsf

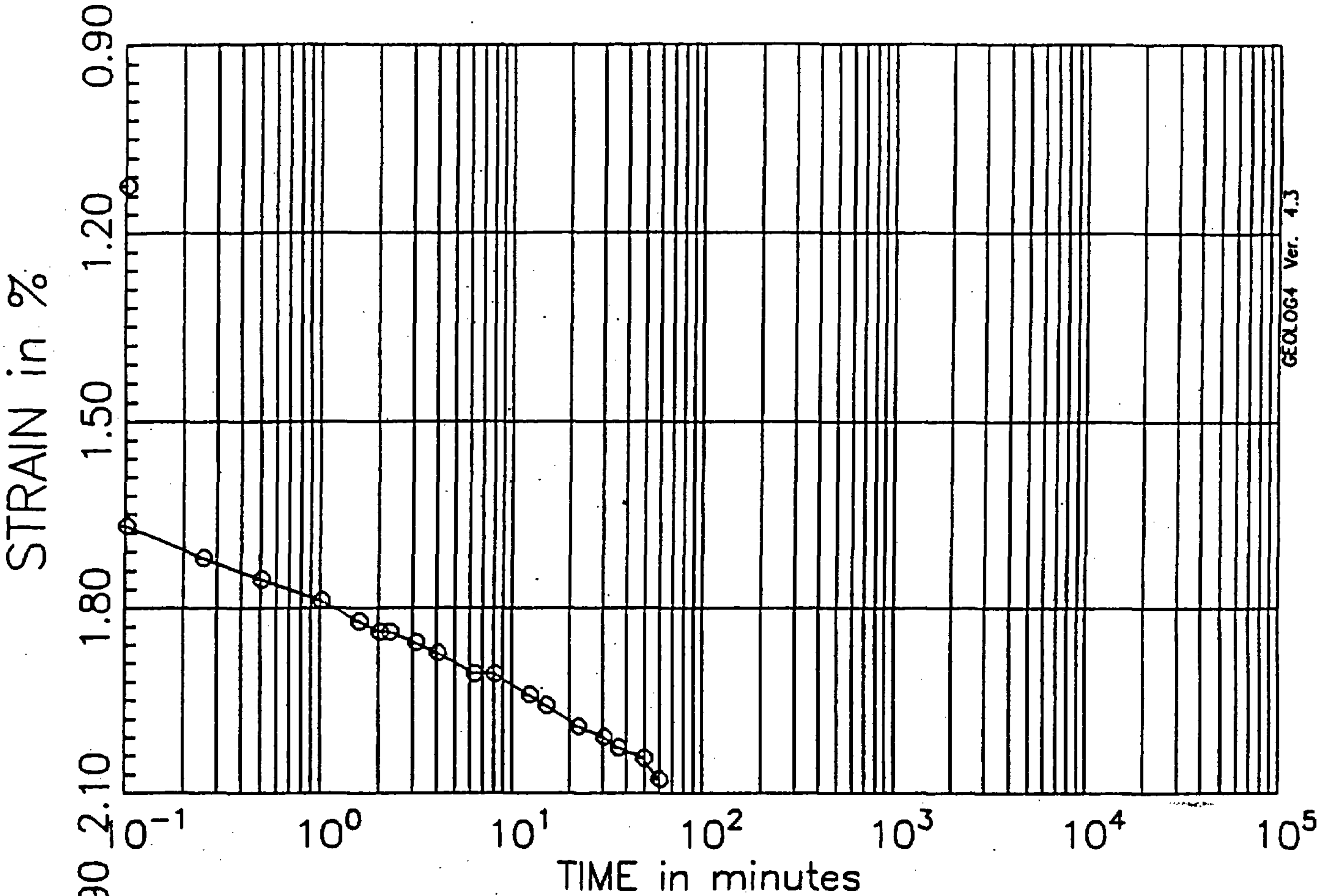
Test No: 5
Testname: C1–U3B

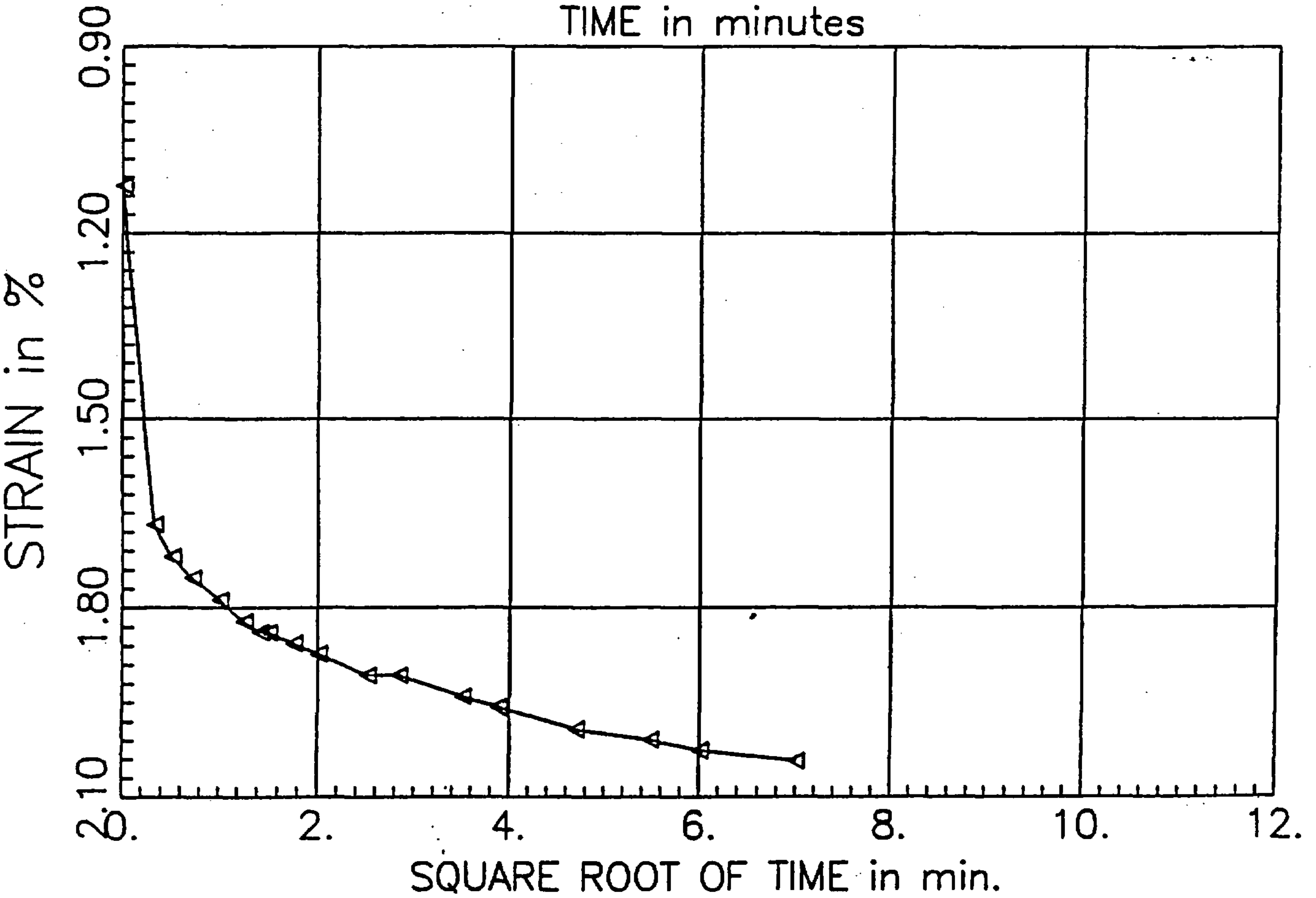




PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

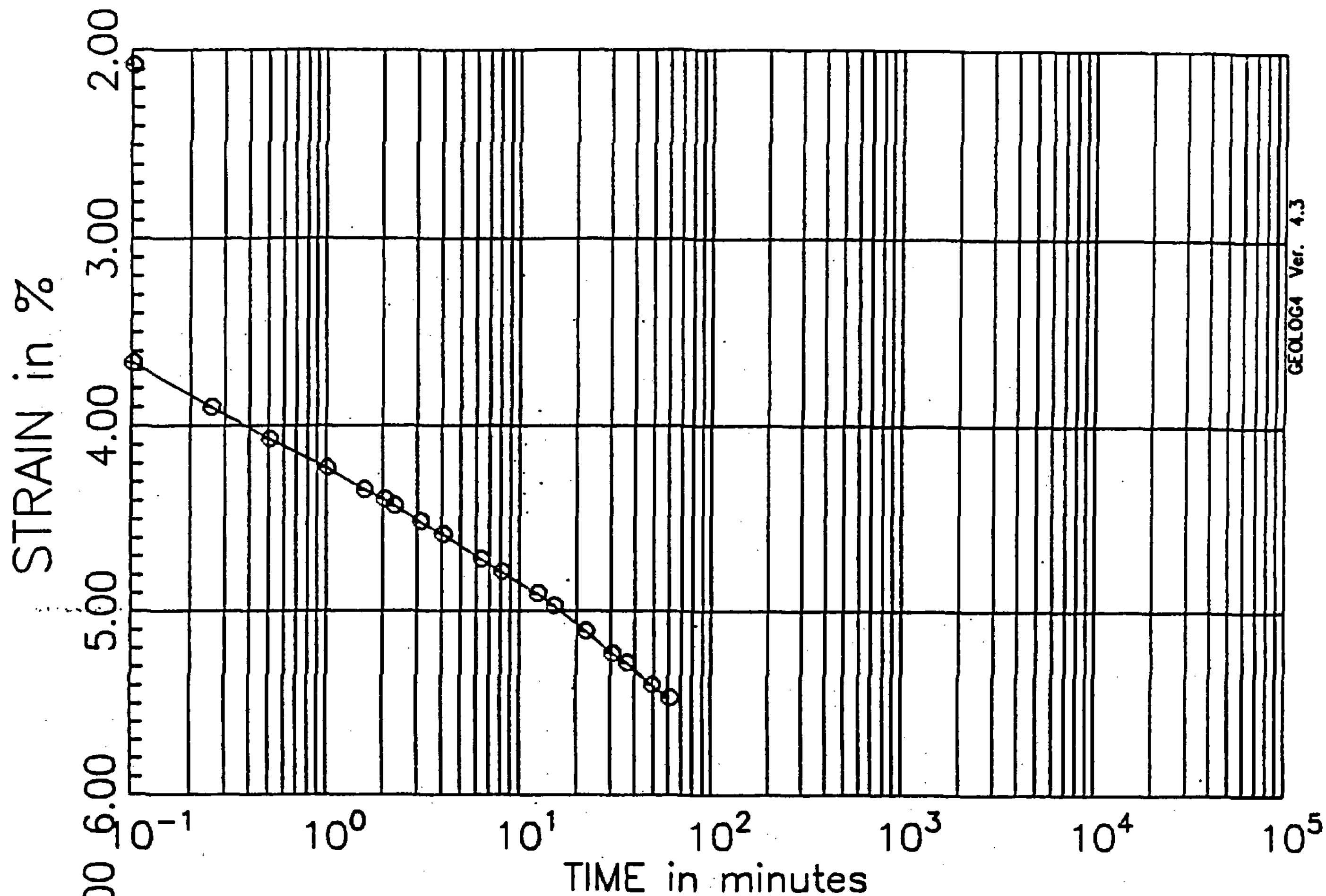
Test No: 5
Testname: C1-U3B

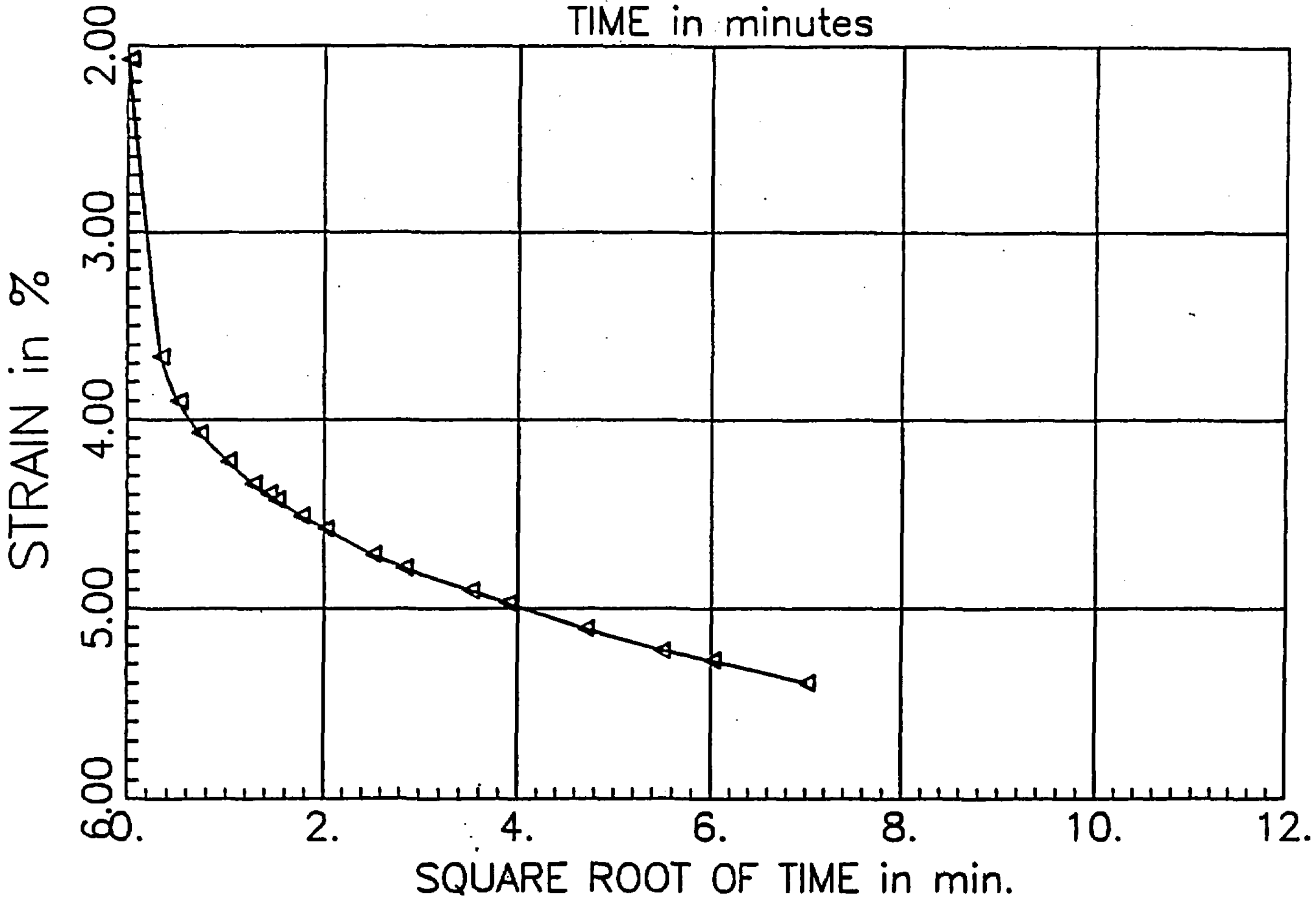




PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

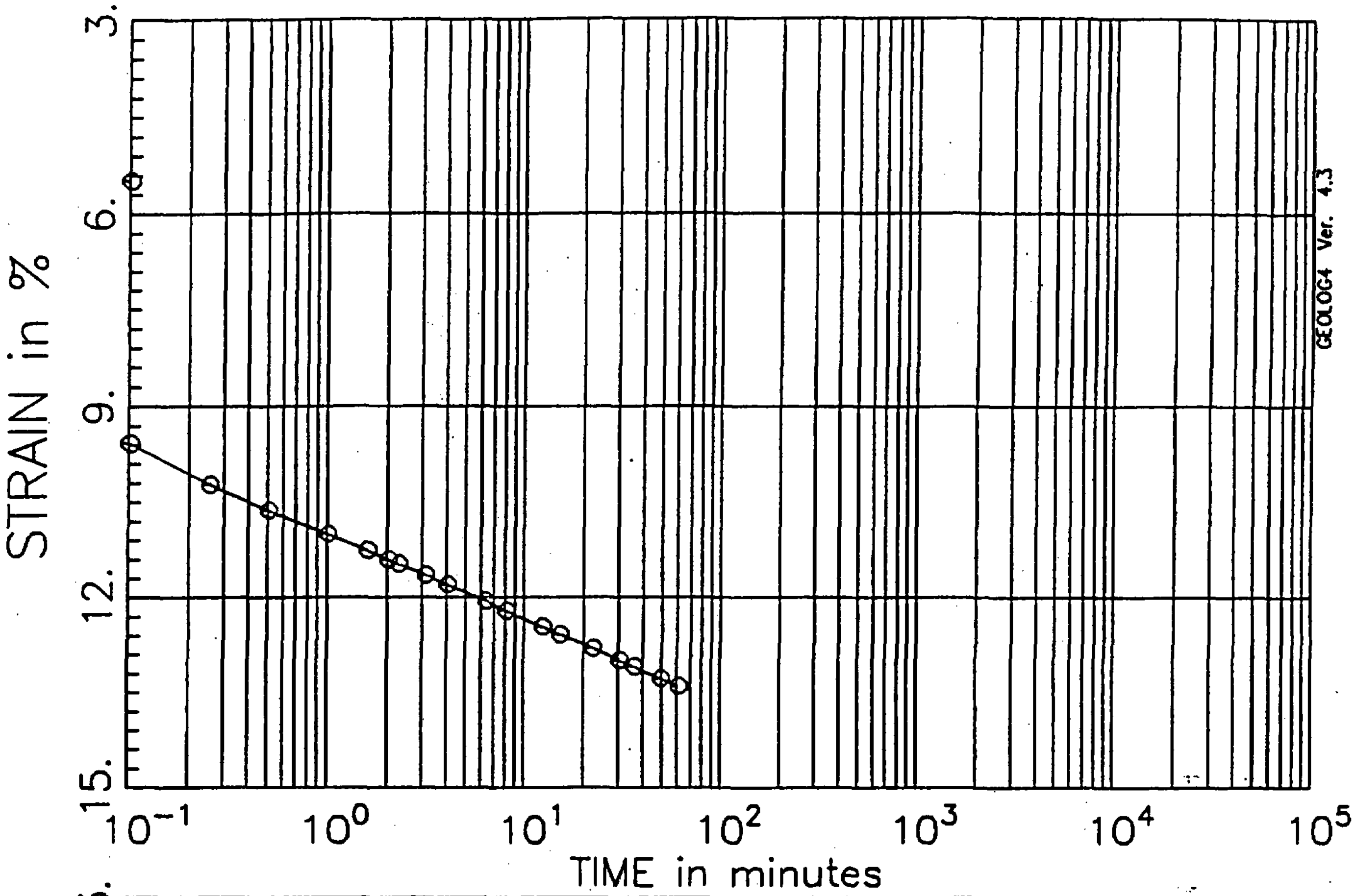
Test No: 5
Testname: C1–U3B

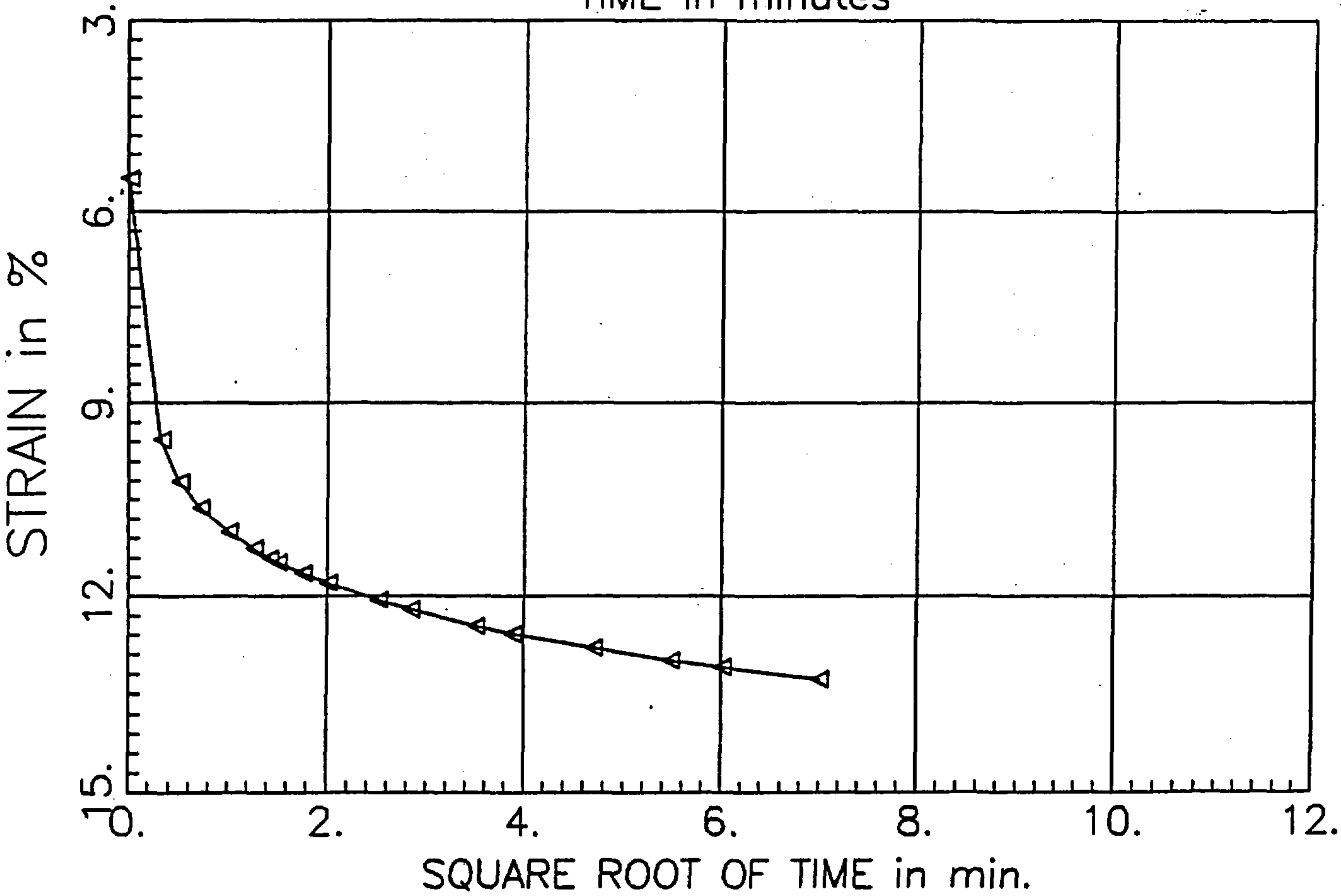




PRESSURE INCREMENT
from 2.00 tsf to 4.00 tsf

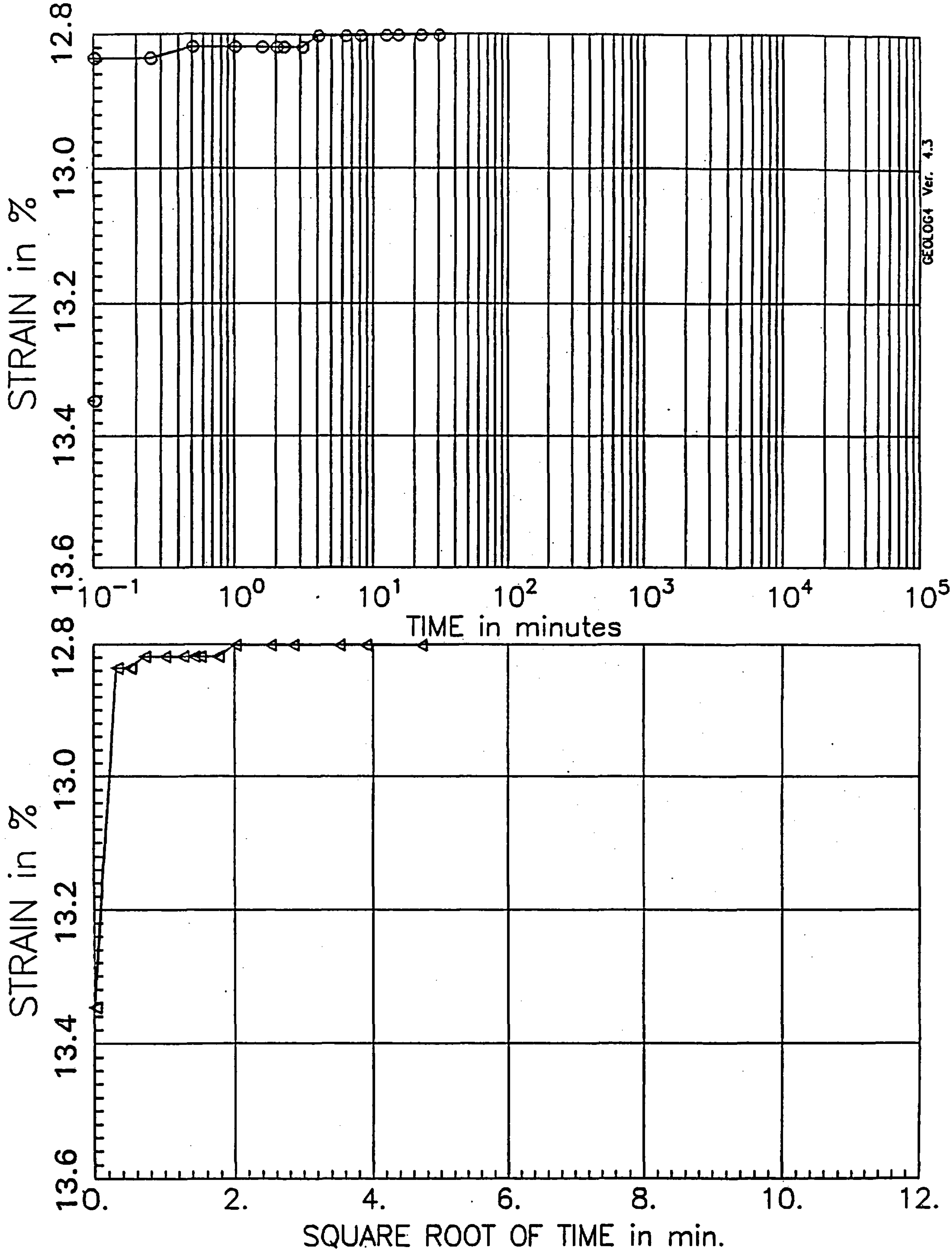
Test No: 5
Testname: C1-U3B





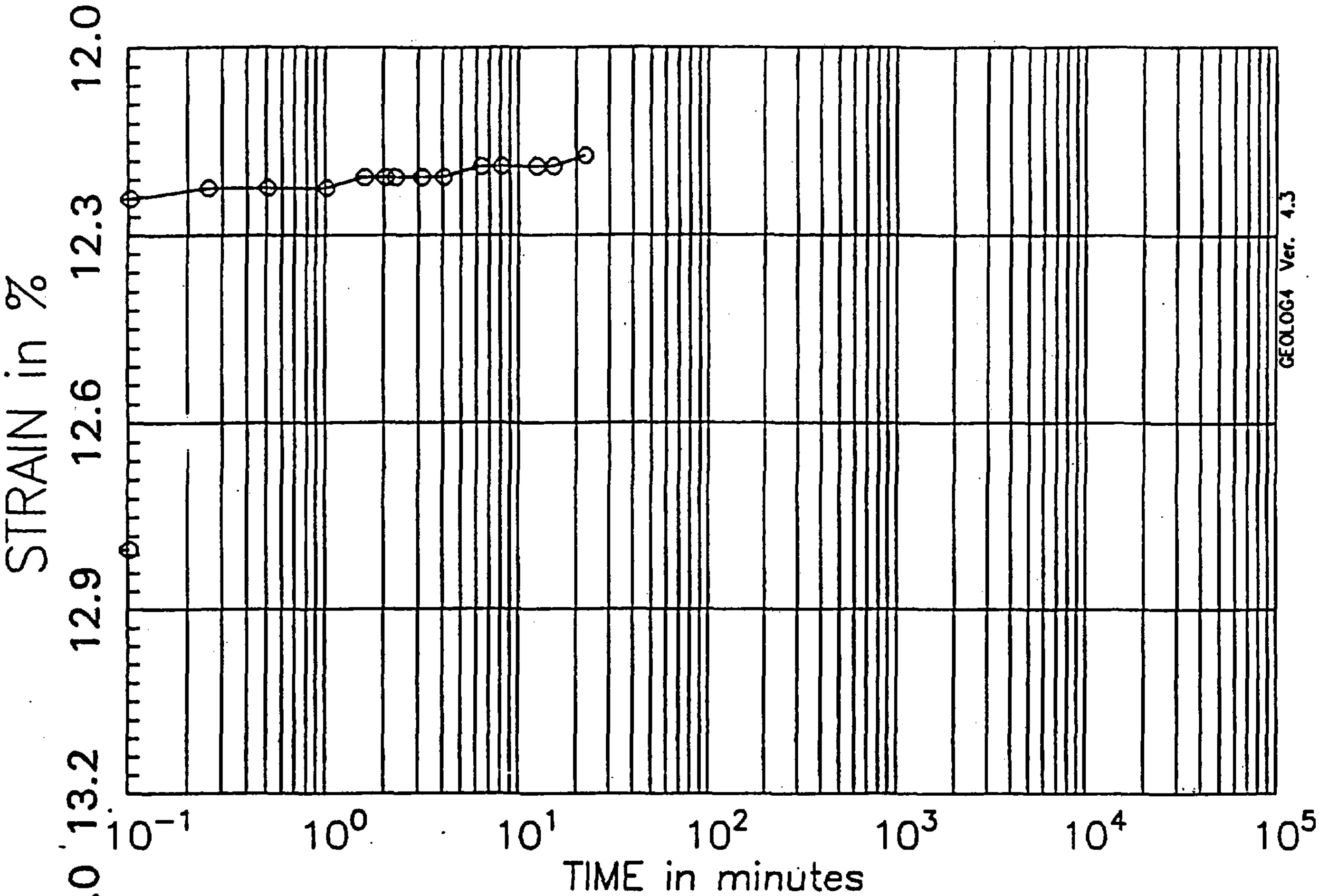
PRESSURE INCREMENT
from 4.00 tsf to 8.00 tsf

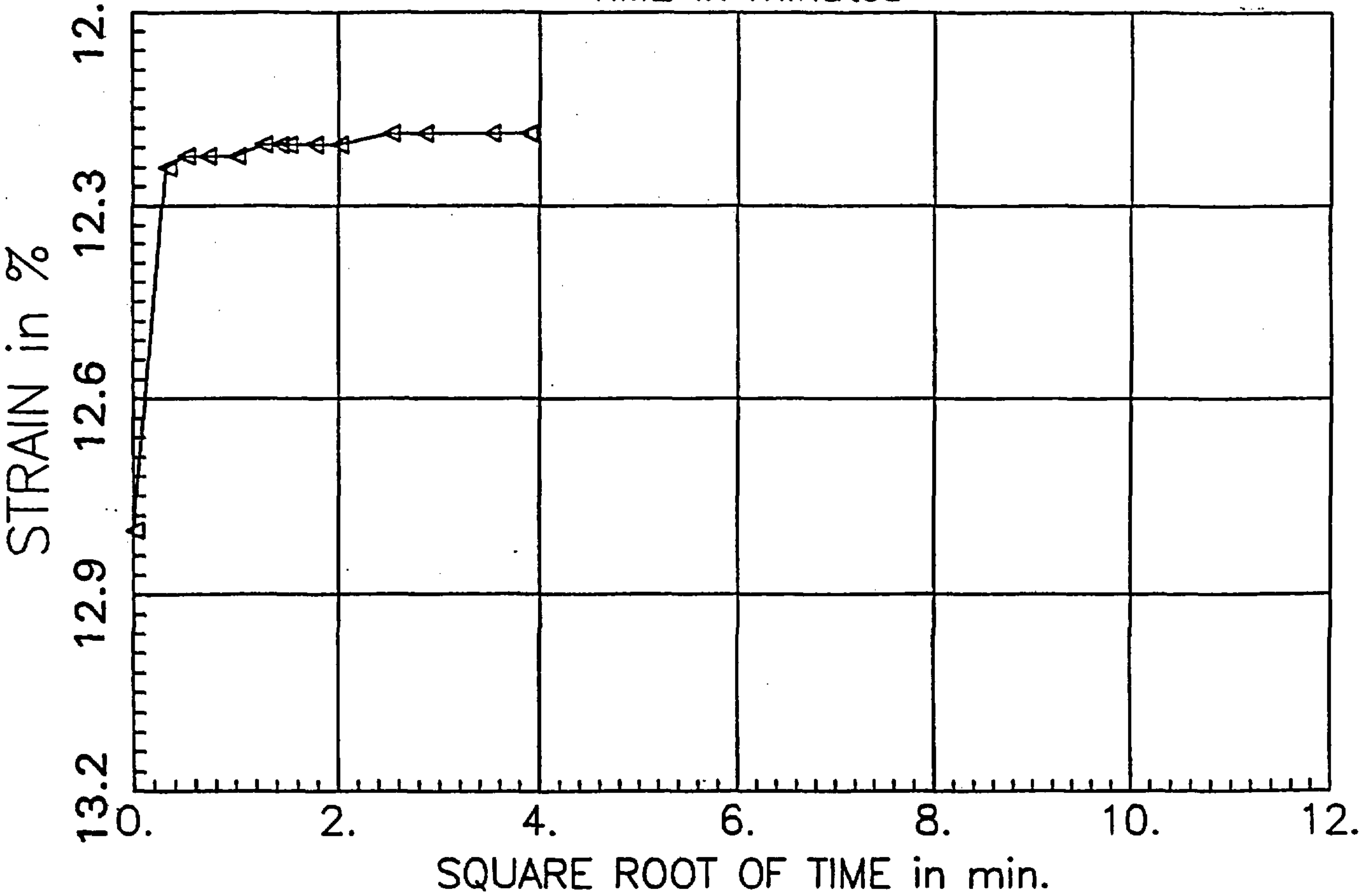
Test No: 5
Testname: C1-U3B



PRESSURE INCREMENT
from 8.00 tsf to 2.00 tsf

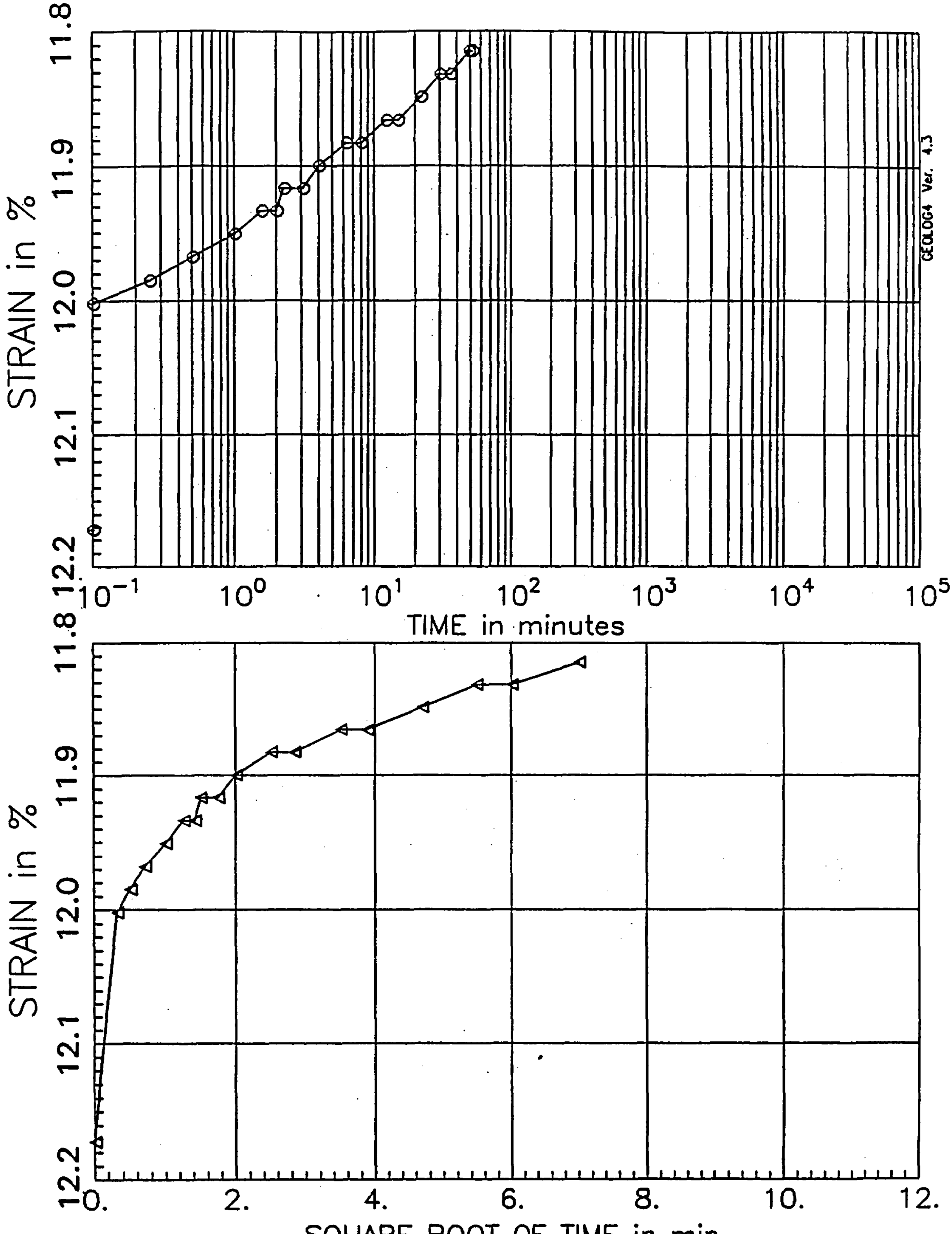
Test No: 5
Testname: C1–U3B





PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

Test No: 5
Testname: C1-U3B



PRESSURE INCREMENT
from 0.50 tsf to 0.10 tsf

Test No: 5
Testname: C1-U3B

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	C-1	DATE:	1/9/97
SAMPLE:	U-3B	TESTED BY:	ACS
DEPTH:	10.8 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	30.3 %	28.7 %
DRY UNIT WEIGHT:	64.7 pcf	73.4 pcf
VOID RATIO:	1.625	1.315
SATURATION:	50.7 %	59.3 %
HEIGHT:	1.893 cm	1.669 cm
AREA:	31.63 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE tsf	STRAIN %
0.10	0.15
0.25	0.37
0.50	0.65
1.00	1.07
2.00	1.91
4.00	4.78
8.00	12.21
8.00	13.40
2.00	12.80
0.50	12.19
0.10	11.88

LOAD INCREMENT DATA

TEST NAME: C1-U3B

TESTED BY: ACS

PAGE NO: 1

JO: 05996.01

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	09:36:19	0.10	0.32	0.0010	1.621	0.14
		0.25	0.50	0.0010	1.621	0.14
		0.50	0.71	0.0011	1.621	0.15
		1.00	1.00	0.0011	1.621	0.15
		1.57	1.25	0.0011	1.621	0.15
		2.00	1.41	0.0011	1.621	0.15
		2.25	1.50	0.0011	1.621	0.15
		3.07	1.75	0.0011	1.621	0.15
		4.00	2.00	0.0011	1.621	0.15
		6.25	2.50	0.0011	1.621	0.15
		8.00	2.83	0.0011	1.621	0.15
		12.25	3.50	0.0013	1.621	0.17
		15.00	3.87	0.0014	1.620	0.19
		22.00	4.69	0.0014	1.620	0.19
		30.00	5.48	0.0014	1.620	0.19

LOAD INCREMENT DATA

TEST NAME C1-U3B
TESTED BY: ACS

PAGE NO: 2
JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	10:09:47	0.00	0.00	0.0015	1.620	0.20
		0.10	0.32	0.0025	1.616	0.34
		0.25	0.50	0.0025	1.616	0.34
		0.50	0.71	0.0027	1.616	0.36
		1.00	1.00	0.0027	1.616	0.36
		1.57	1.25	0.0027	1.616	0.36
		2.00	1.41	0.0027	1.616	0.36
		2.25	1.50	0.0027	1.616	0.36
		3.07	1.75	0.0027	1.616	0.36
		4.00	2.00	0.0028	1.615	0.37
		6.25	2.50	0.0028	1.615	0.37
		8.00	2.83	0.0028	1.615	0.37
		12.25	3.50	0.0028	1.615	0.37
		15.00	3.87	0.0028	1.615	0.37
		22.00	4.69	0.0029	1.615	0.39
		30.00	5.48	0.0029	1.615	0.39
		35.00	5.92	0.0029	1.615	0.39

LOAD INCREMENT DATA

TEST NAME C1-U3B PAGE NO: 3

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	10:45:24	0.00	0.00	0.0029	1.615	0.39
		0.10	0.32	0.0044	1.609	0.60
		0.25	0.50	0.0044	1.609	0.60
		0.50	0.71	0.0044	1.609	0.60
		1.00	1.00	0.0046	1.609	0.61
		1.57	1.25	0.0047	1.608	0.63
		2.00	1.41	0.0047	1.608	0.63
		2.25	1.50	0.0047	1.608	0.63
		3.07	1.75	0.0047	1.608	0.63
		4.00	2.00	0.0047	1.608	0.63
		6.25	2.50	0.0047	1.608	0.63
		8.00	2.83	0.0048	1.608	0.65
		12.25	3.50	0.0048	1.608	0.65
		15.00	3.87	0.0048	1.608	0.65
		22.00	4.69	0.0048	1.608	0.65
		30.00	5.48	0.0049	1.608	0.66
		34.00	5.83	0.0049	1.608	0.66

LOAD INCREMENT DATA

TEST NAME C1-U3B

TESTED BY: ACS

PAGE NO: 4

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	11:20:05	0.00	0.00	0.0049	1.608	0.66
		0.10	0.32	0.0072	1.600	0.97
		0.25	0.50	0.0075	1.599	1.00
		0.50	0.71	0.0075	1.599	1.00
		1.00	1.00	0.0076	1.598	1.02
		1.57	1.25	0.0077	1.598	1.04
		2.00	1.41	0.0077	1.598	1.04
		2.25	1.50	0.0077	1.598	1.04
		3.07	1.75	0.0079	1.597	1.06
		4.00	2.00	0.0079	1.597	1.06
		6.25	2.50	0.0079	1.597	1.06
		8.00	2.83	0.0080	1.597	1.07
		12.25	3.50	0.0080	1.597	1.07
		15.00	3.87	0.0081	1.596	1.09
		22.00	4.69	0.0081	1.596	1.09
		30.00	5.48	0.0081	1.596	1.09
		36.00	6.00	0.0082	1.596	1.11
		49.00	7.00	0.0082	1.596	1.11
		60.00	7.75	0.0082	1.596	1.11

LOAD INCREMENT DATA

TEST NAME C1-U3B

TESTED BY: ACS

PAGE NO: 5

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	12:21:07	0.00	0.00	0.0084	1.596	1.12
		0.10	0.32	0.0124	1.581	1.67
		0.25	0.50	0.0128	1.580	1.72
		0.50	0.71	0.0131	1.579	1.75
		1.00	1.00	0.0133	1.578	1.79
		1.57	1.25	0.0136	1.577	1.82
		2.00	1.41	0.0137	1.577	1.84
		2.25	1.50	0.0137	1.577	1.84
		3.07	1.75	0.0138	1.576	1.86
		4.00	2.00	0.0140	1.576	1.87
		6.25	2.50	0.0142	1.575	1.91
		8.00	2.83	0.0142	1.575	1.91
		12.25	3.50	0.0145	1.574	1.94
		15.00	3.87	0.0146	1.574	1.96
		22.00	4.69	0.0148	1.573	1.99
		30.00	5.48	0.0150	1.572	2.01
		36.00	6.00	0.0151	1.572	2.03
		49.00	7.00	0.0152	1.571	2.04
		59.00	7.68	0.0155	1.570	2.08

LOAD INCREMENT DATA

TEST NAME C1-U3B

PAGE NO: 6

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	13:20:45	0.00	0.00	0.0155	1.570	2.08
		0.10	0.32	0.0273	1.529	3.66
		0.25	0.50	0.0291	1.523	3.90
		0.50	0.71	0.0303	1.518	4.07
		1.00	1.00	0.0315	1.514	4.22
		1.57	1.25	0.0324	1.511	4.34
		2.00	1.41	0.0327	1.510	4.39
		2.25	1.50	0.0330	1.509	4.43
		3.07	1.75	0.0336	1.507	4.51
		4.00	2.00	0.0341	1.505	4.58
		6.25	2.50	0.0351	1.501	4.72
		8.00	2.83	0.0357	1.499	4.78
		12.25	3.50	0.0365	1.496	4.90
		15.00	3.87	0.0370	1.495	4.97
		22.00	4.69	0.0381	1.491	5.11
		30.00	5.48	0.0390	1.488	5.23
		36.00	6.00	0.0393	1.486	5.28
		49.00	7.00	0.0402	1.483	5.40
		60.00	7.75	0.0407	1.482	5.46

LOAD INCREMENT DATA

TEST NAME C1-U3B

TESTED BY: ACS

PAGE NO: 7

JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	14:22:24	0.00	0.00	0.0409	1.481	5.48
		0.10	0.32	0.0713	1.374	9.57
		0.25	0.50	0.0761	1.357	10.21
		0.50	0.71	0.0792	1.346	10.62
		1.00	1.00	0.0820	1.336	11.00
		1.57	1.25	0.0839	1.330	11.25
		2.00	1.41	0.0850	1.326	11.41
		2.25	1.50	0.0855	1.324	11.47
		3.07	1.75	0.0868	1.319	11.64
		4.00	2.00	0.0879	1.315	11.80
		6.25	2.50	0.0898	1.309	12.05
		8.00	2.83	0.0910	1.305	12.21
		12.25	3.50	0.0929	1.298	12.46
		15.00	3.87	0.0938	1.295	12.58
		22.00	4.69	0.0954	1.289	12.80
		30.00	5.48	0.0968	1.284	12.99
		36.00	6.00	0.0976	1.281	13.09
		49.00	7.00	0.0990	1.276	13.28
		60.00	7.75	0.0999	1.273	13.40

LOAD INCREMENT DATA

TEST NAME C1-U3B
TESTED BY: ACS

PAGE NO: 8
JO: 05996.01

PRESSURE INCREMENT FROM 8.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	15:27:57	0.00	0.00	0.0995	1.275	13.35
		0.10	0.32	0.0957	1.288	12.84
		0.25	0.50	0.0957	1.288	12.84
		0.50	0.71	0.0955	1.289	12.82
		1.00	1.00	0.0955	1.289	12.82
		1.57	1.25	0.0955	1.289	12.82
		2.00	1.41	0.0955	1.289	12.82
		2.25	1.50	0.0955	1.289	12.82
		3.07	1.75	0.0955	1.289	12.82
		4.00	2.00	0.0954	1.289	12.80
		6.25	2.50	0.0954	1.289	12.80
		8.00	2.83	0.0954	1.289	12.80
		12.25	3.50	0.0954	1.289	12.80
		15.00	3.87	0.0954	1.289	12.80
		22.00	4.69	0.0954	1.289	12.80
		30.00	5.48	0.0954	1.289	12.80

LOAD INCREMENT DATA

TEST NAME C1-U3B
TESTED BY: ACS

PAGE NO: 9
JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

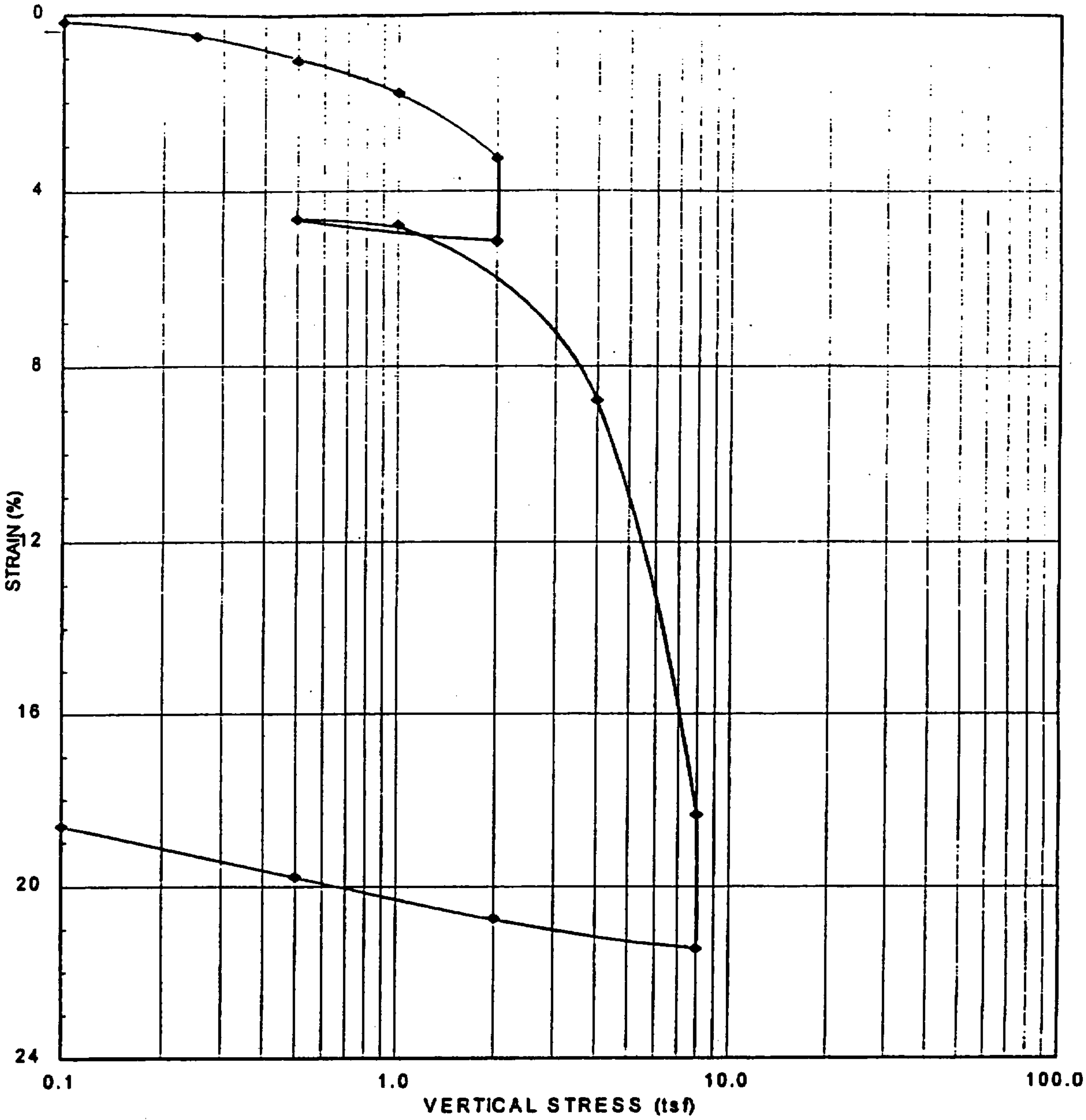
DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	16:00:47	0.00	0.00	0.0954	1.289	12.80
		0.10	0.32	0.0912	1.304	12.24
		0.25	0.50	0.0911	1.304	12.22
		0.50	0.71	0.0911	1.304	12.22
		1.00	1.00	0.0911	1.304	12.22
		1.57	1.25	0.0910	1.305	12.21
		2.00	1.41	0.0910	1.305	12.21
		2.25	1.50	0.0910	1.305	12.21
		3.07	1.75	0.0910	1.305	12.21
		4.00	2.00	0.0910	1.305	12.21
		6.25	2.50	0.0908	1.305	12.19
		8.00	2.83	0.0908	1.305	12.19
		12.25	3.50	0.0908	1.305	12.19
		15.00	3.87	0.0908	1.305	12.19
		22.00	4.69	0.0907	1.305	12.17

LOAD INCREMENT DATA

TEST NAME C1-U3B PAGE NO: 10
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-09-97	16:26:57	0.00	0.00	0.0907	1.305	12.17
		0.10	0.32	0.0894	1.310	12.00
		0.25	0.50	0.0893	1.310	11.98
		0.50	0.71	0.0892	1.311	11.97
		1.00	1.00	0.0891	1.311	11.95
		1.57	1.25	0.0889	1.312	11.93
		2.00	1.41	0.0889	1.312	11.93
		2.25	1.50	0.0888	1.312	11.92
		3.07	1.75	0.0888	1.312	11.92
		4.00	2.00	0.0887	1.313	11.90
		6.25	2.50	0.0886	1.313	11.88
		8.00	2.83	0.0886	1.313	11.88
		12.25	3.50	0.0884	1.314	11.87
		15.00	3.87	0.0884	1.314	11.87
		22.00	4.69	0.0883	1.314	11.85
		30.00	5.48	0.0882	1.314	11.83
		36.00	6.00	0.0882	1.314	11.83
		49.00	7.00	0.0881	1.315	11.81
		51.82	7.20	0.0881	1.315	11.81



SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/20/96
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	11.2 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	38.9 %	51.9 %	2.72
DRY UNIT WEIGHT:	55.8 pcf	68.4 pcf	
VOID RATIO:	2.041	1.484	
SATURATION:	51.8 %	95.2 %	

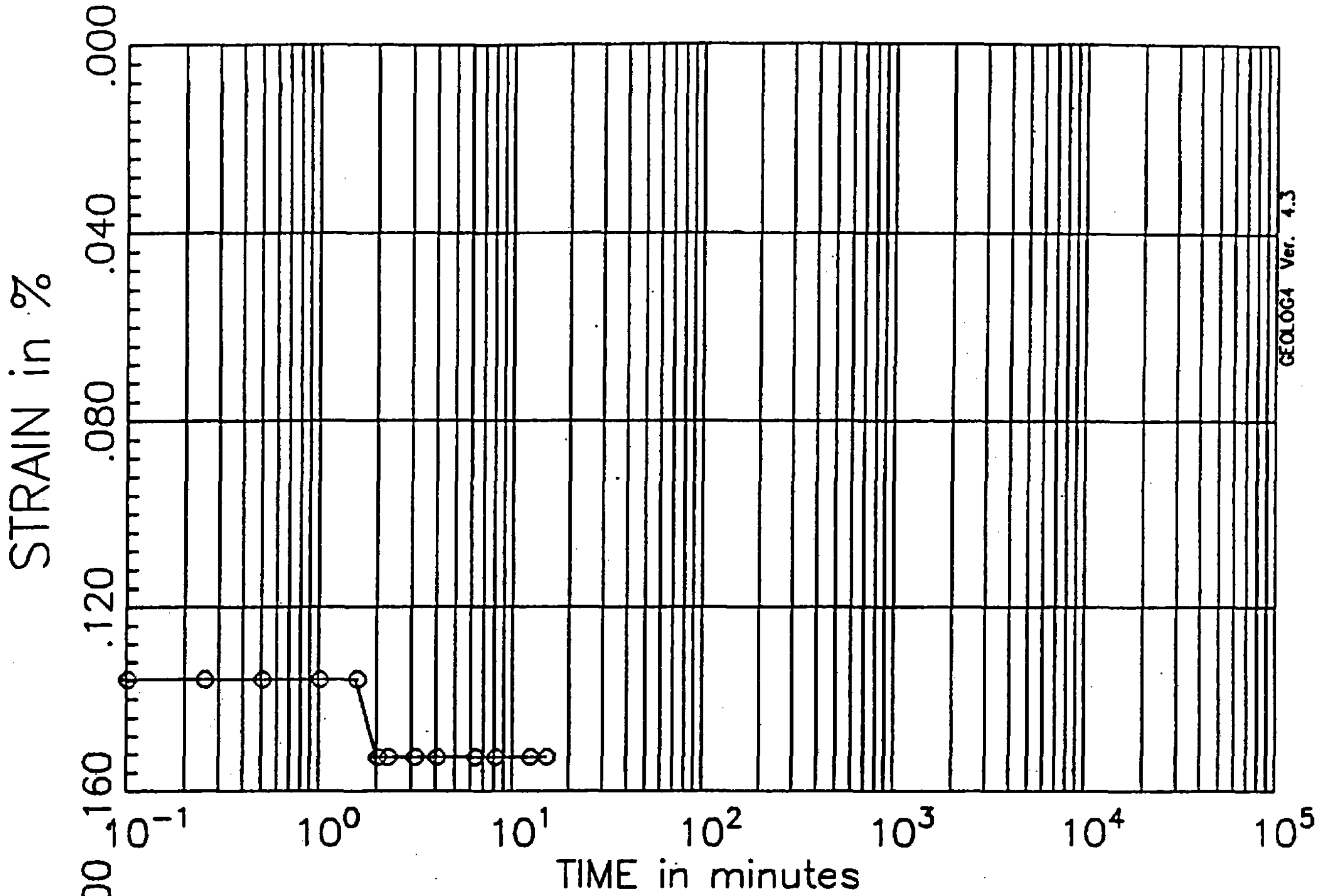
<u>NOTE:</u> Sample was not inundated and porous stones were moist

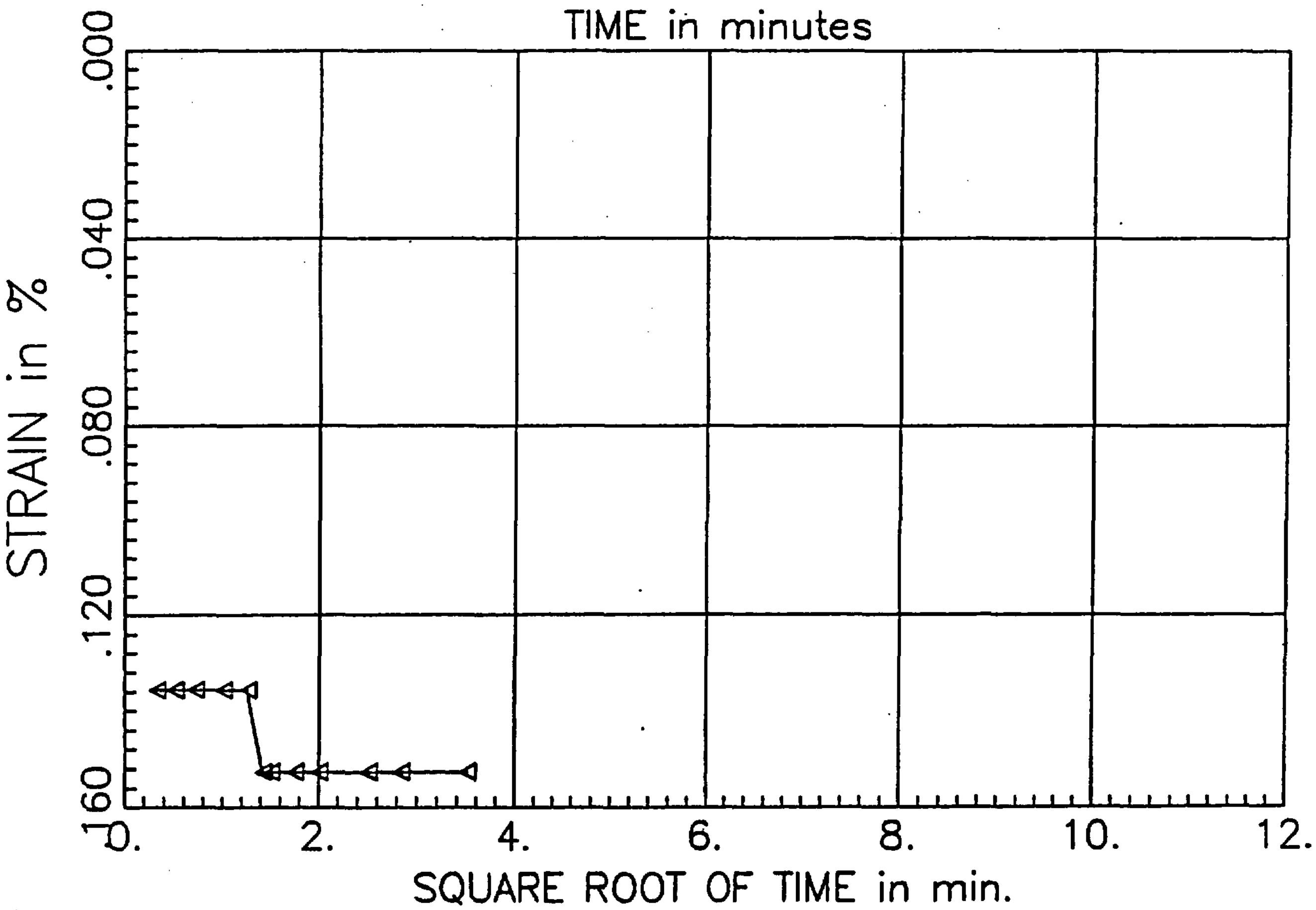
PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

CONSOLIDATION TEST RESULTS
BORING C-1, SAMPLE U-3C

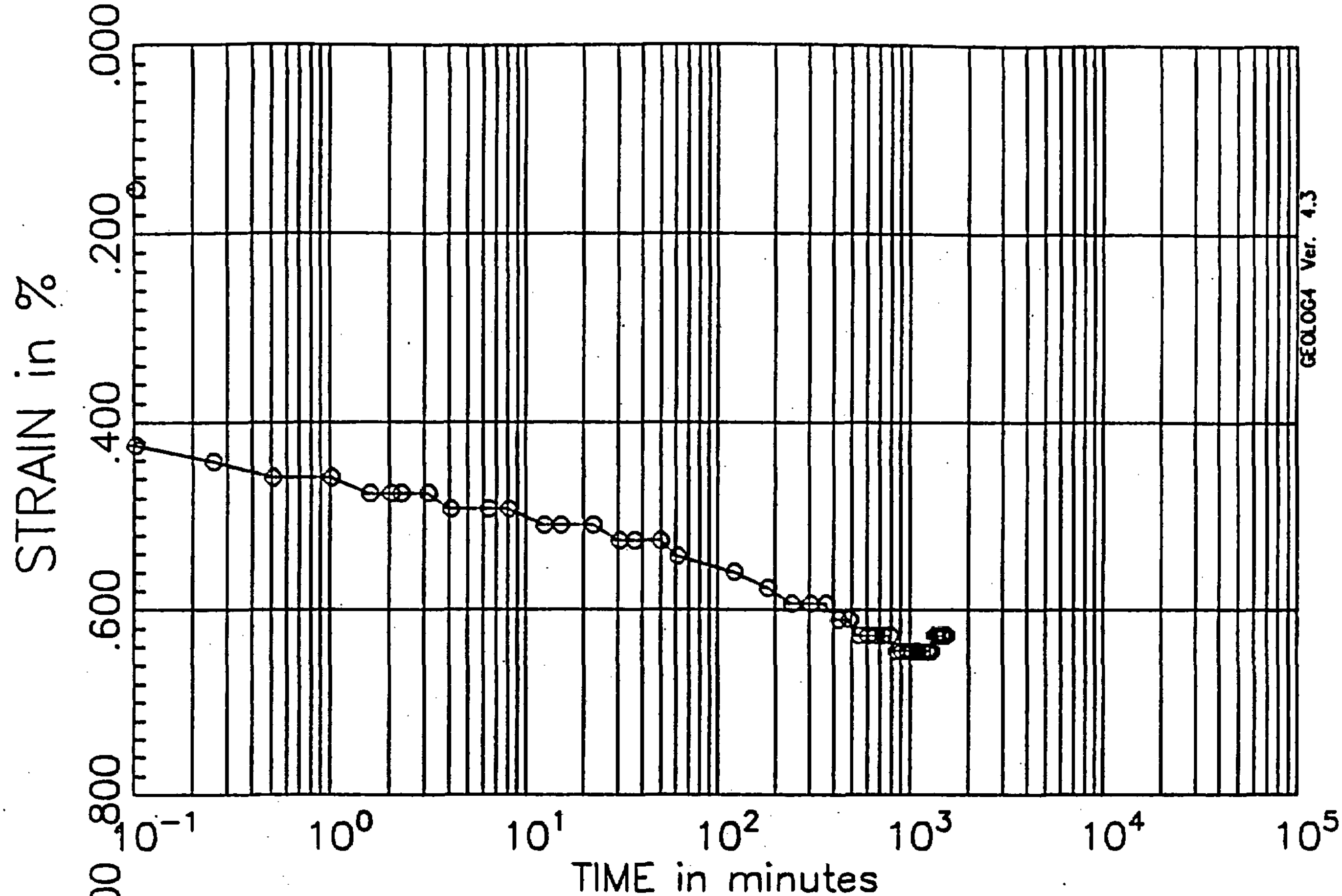
JO 05996.01
January 1997

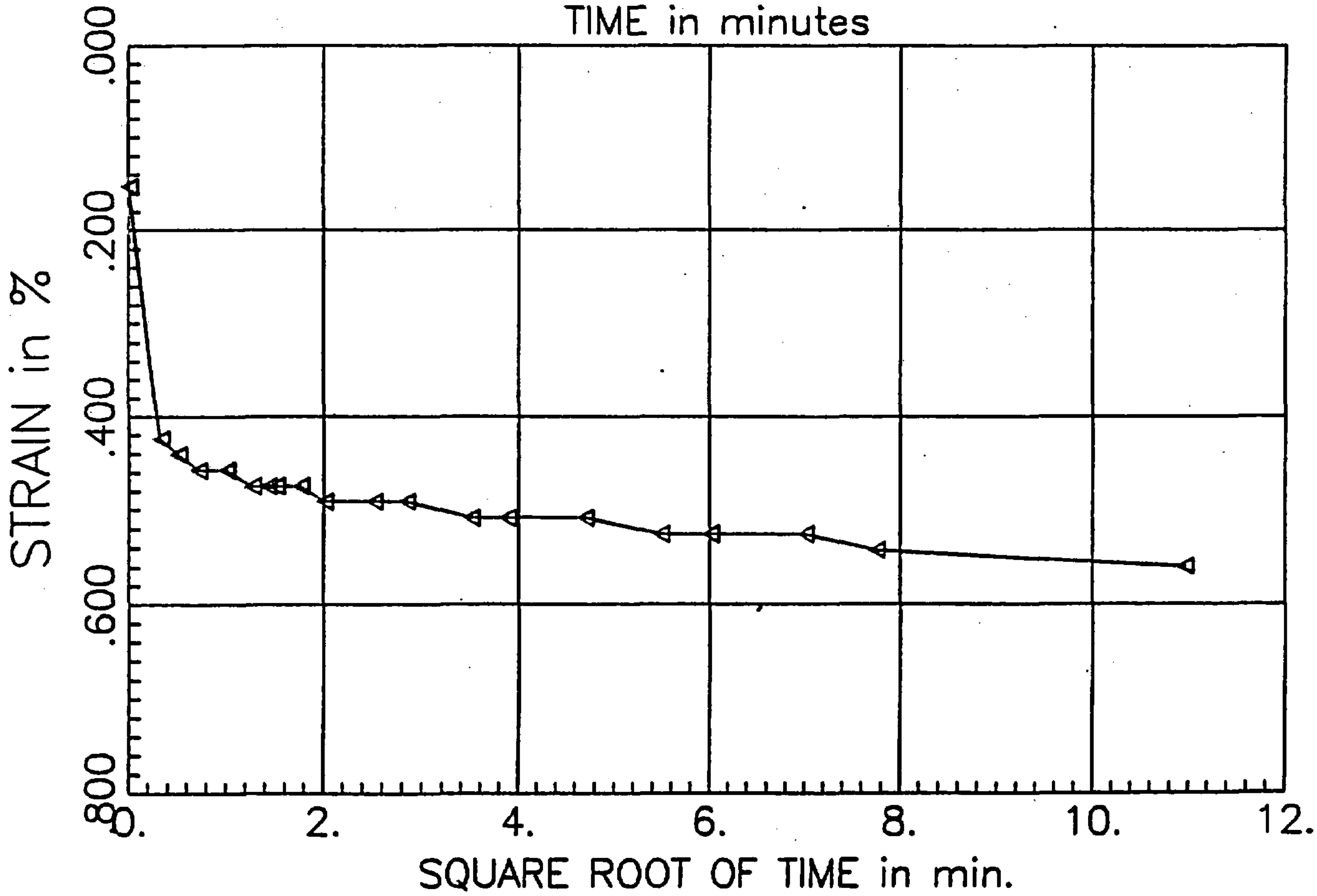




PRESSURE INCREMENT
from 0.00 tsf to 0.10 tsf

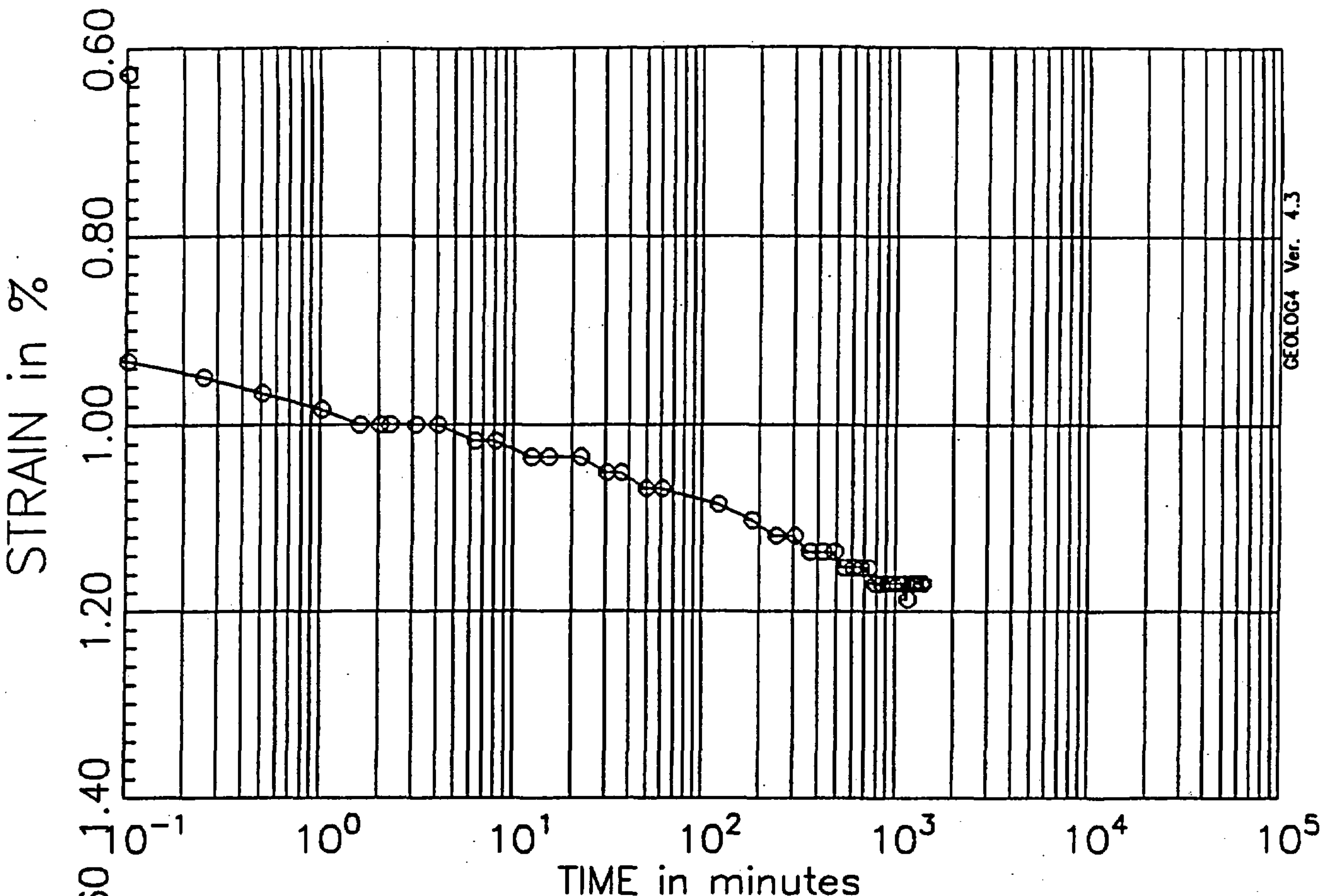
Test No: 4
Testname: C1-U3C

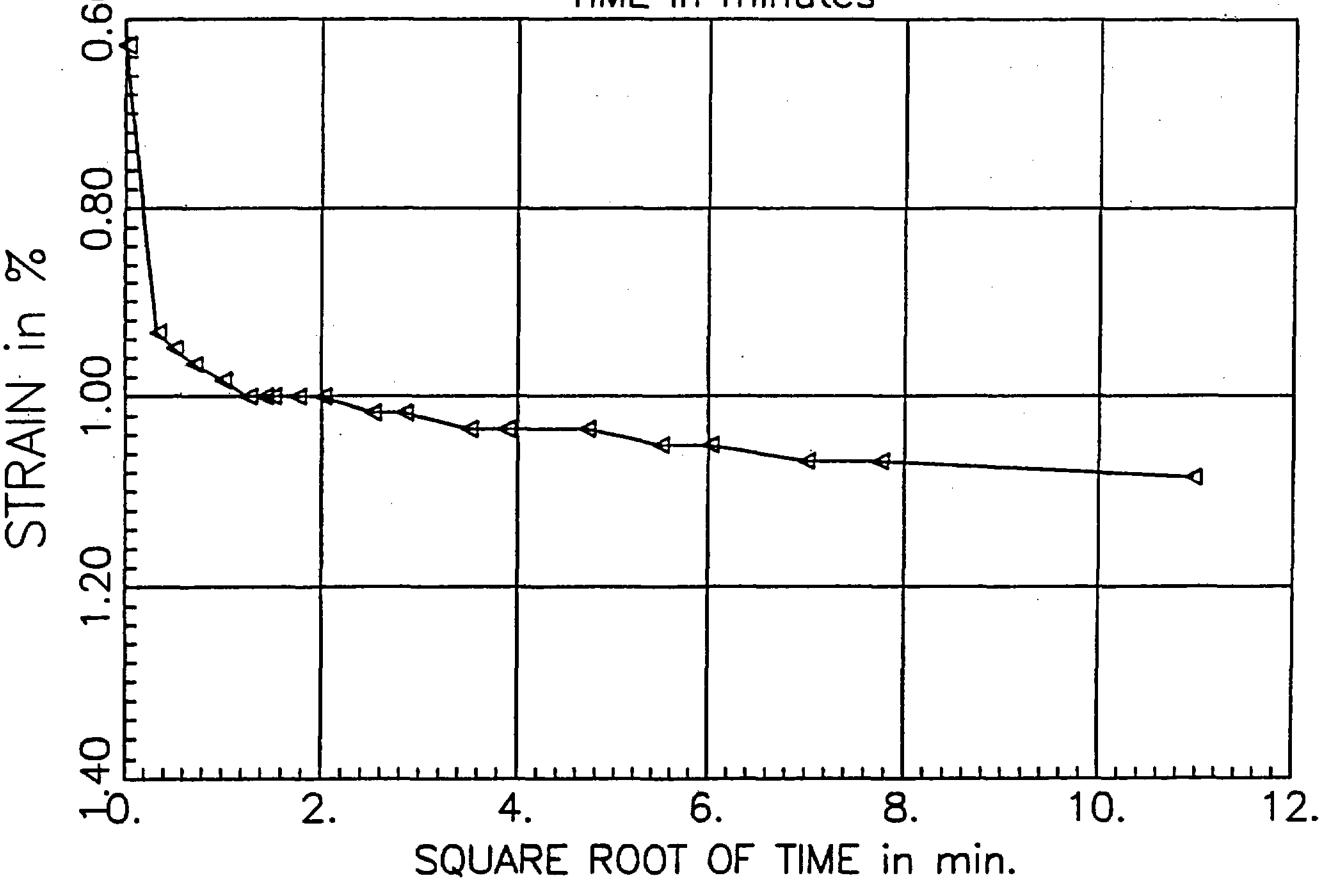




PRESSURE INCREMENT
from 0.10 tsf to 0.25 tsf

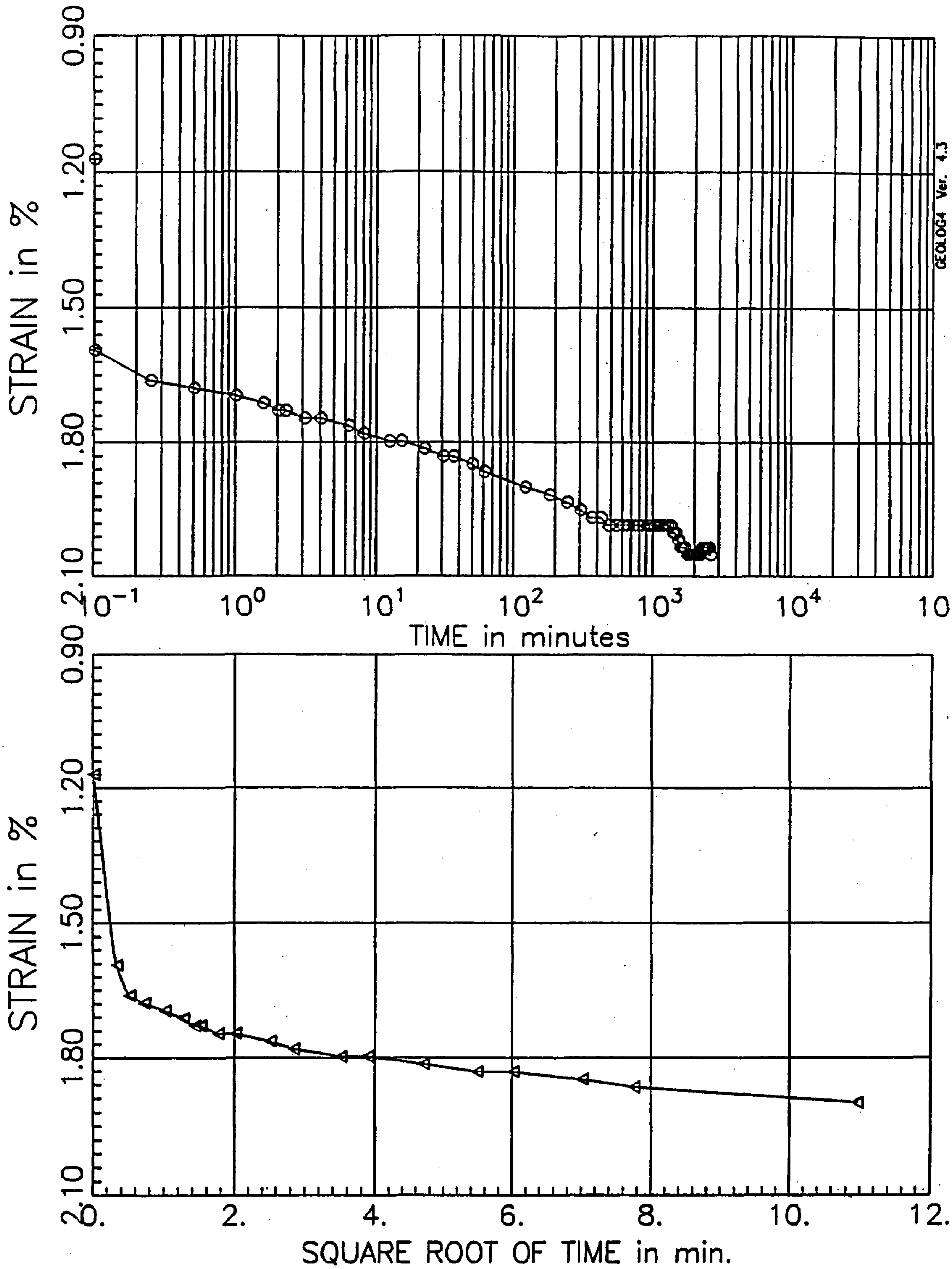
Test No: 4
Testname: C1-U3C





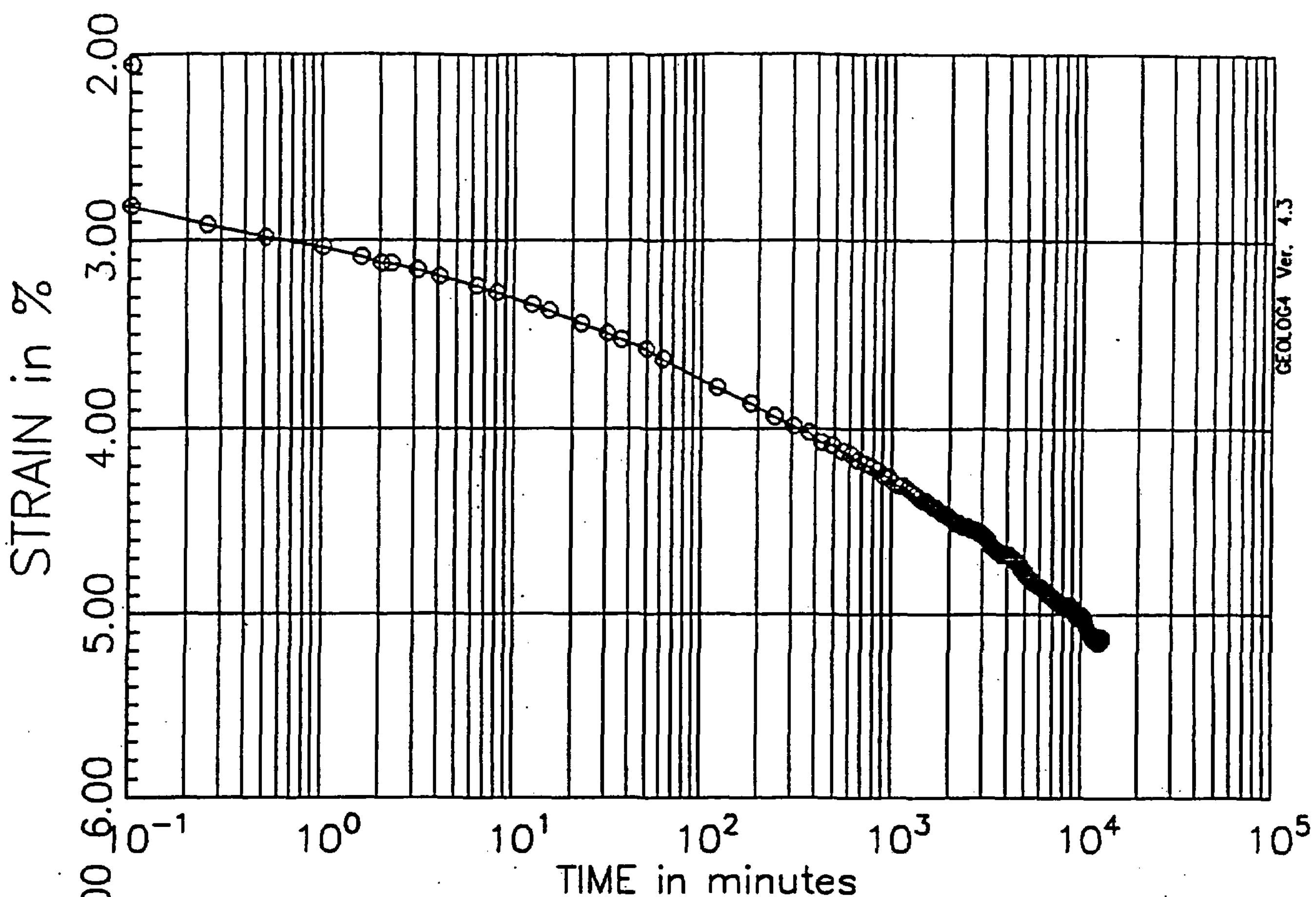
PRESSURE INCREMENT
from 0.25 tsf to 0.50 tsf

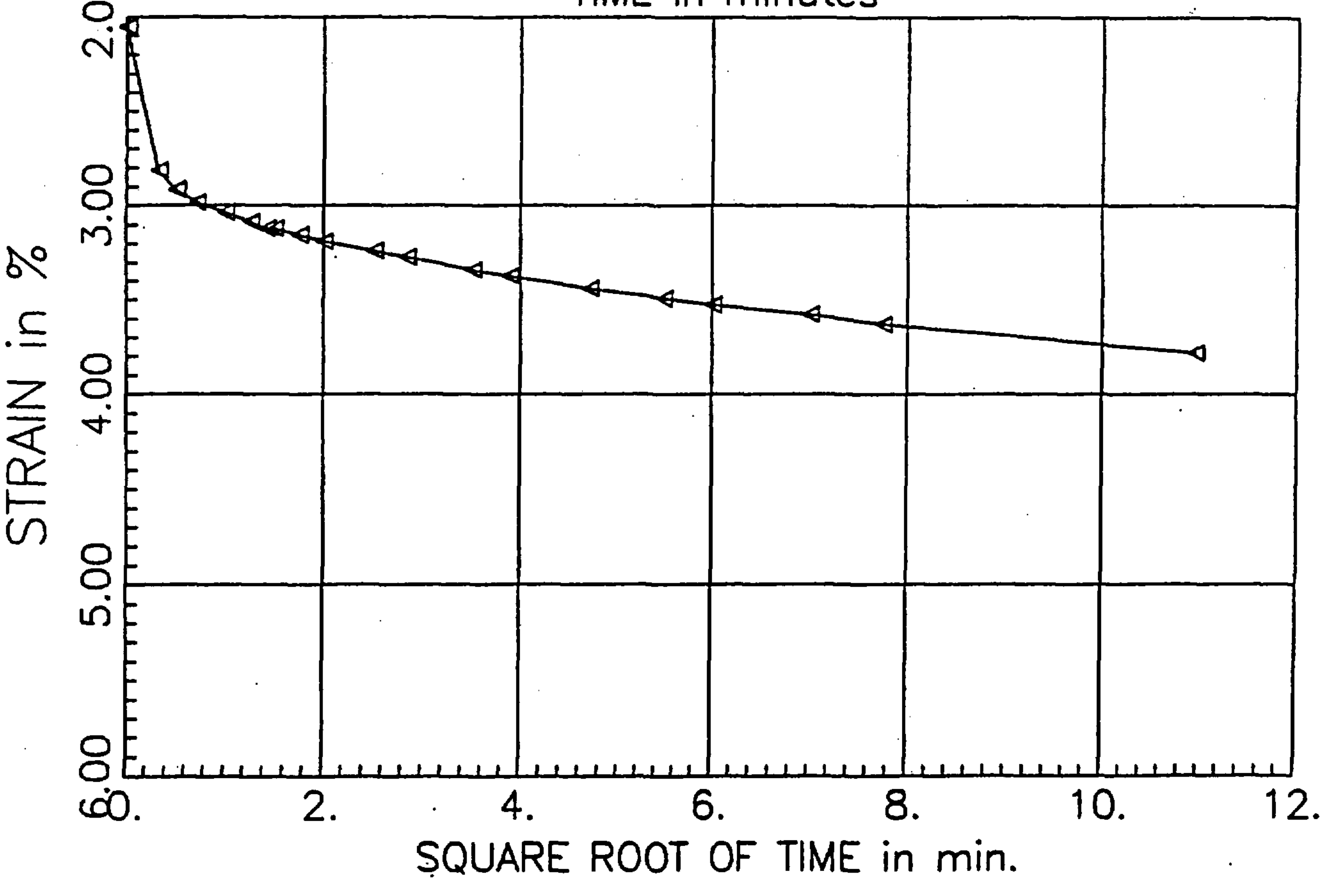
Test No: 4
Testname: C1-U3C



PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

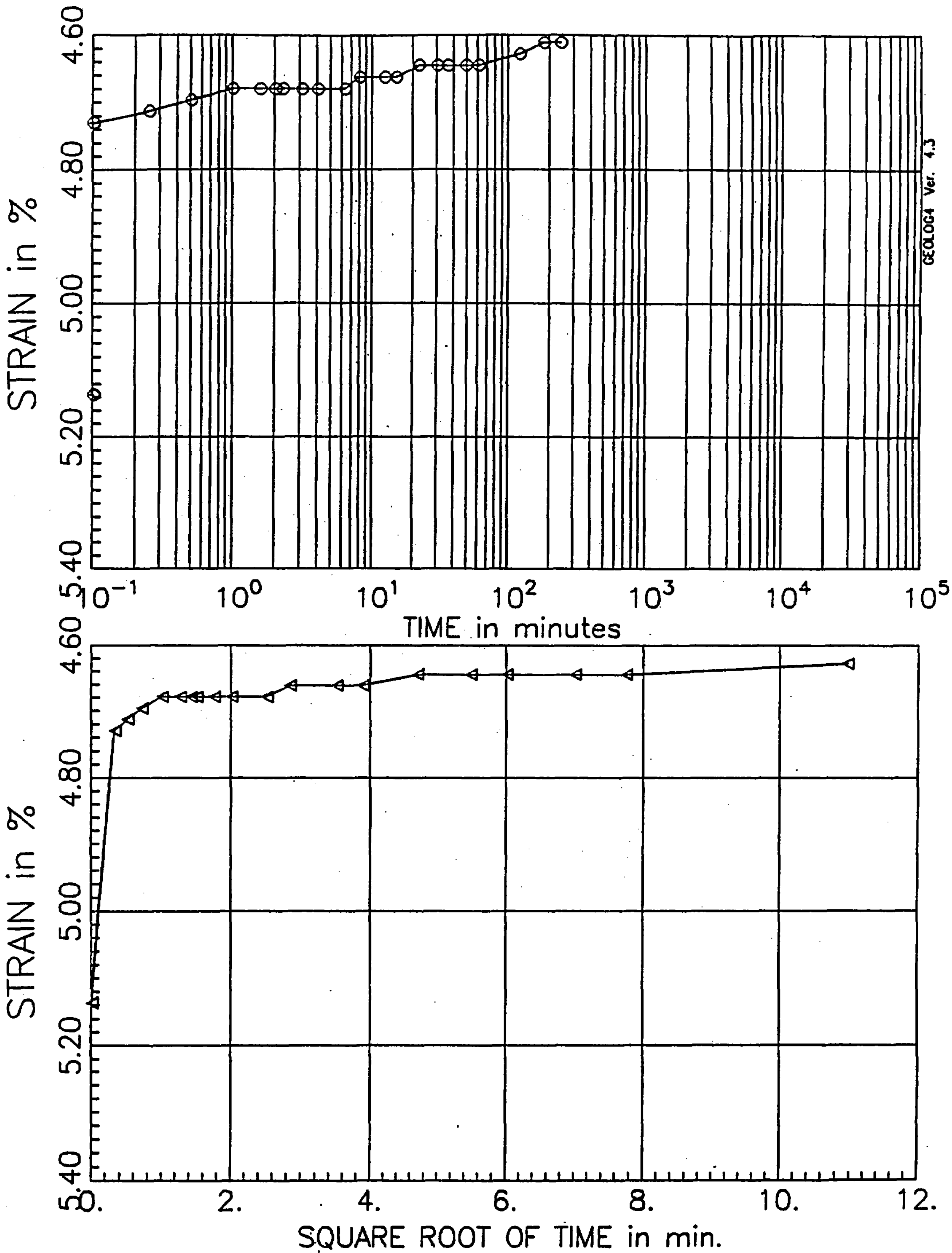
Test No: 4
Testname: C1-U3C





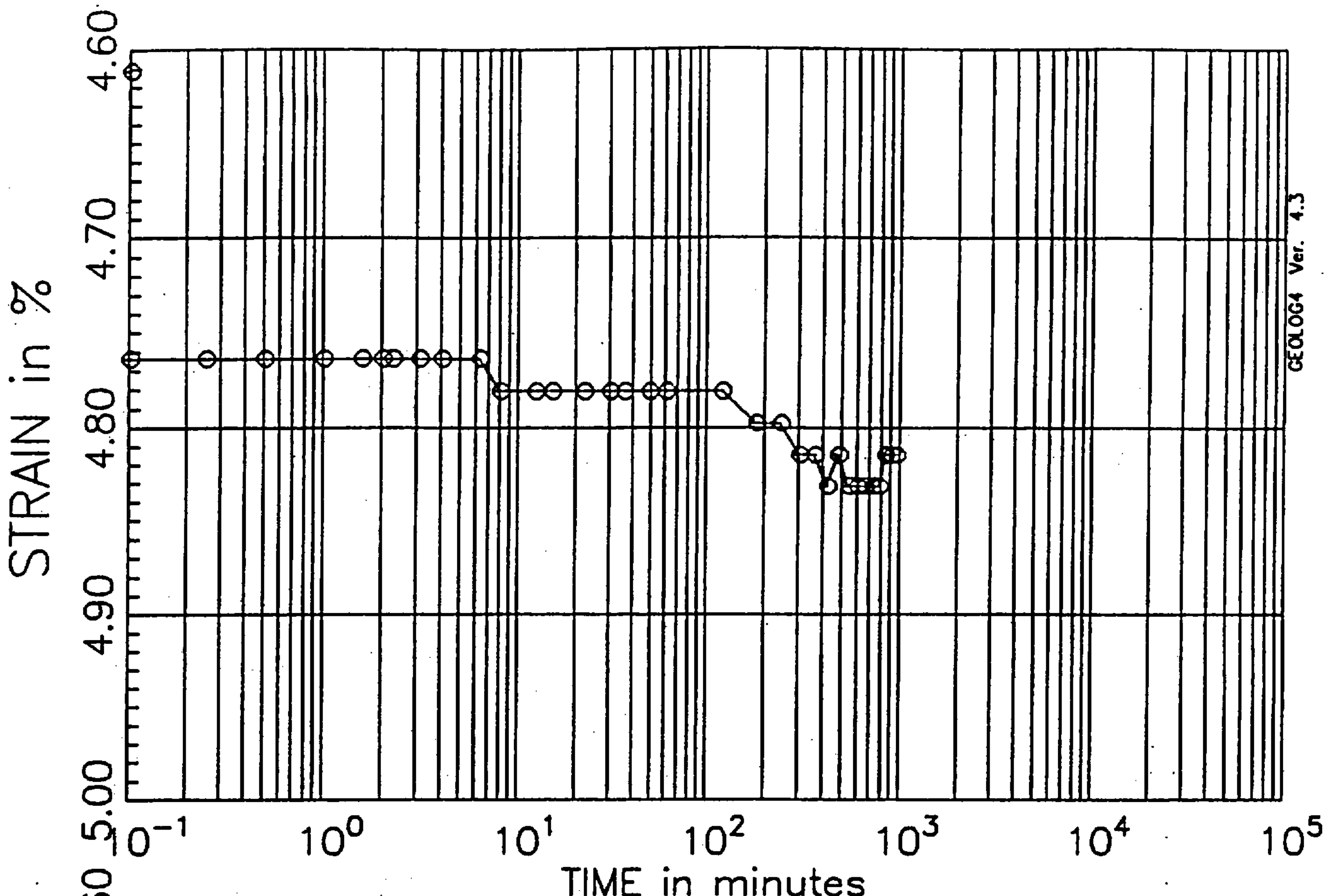
PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

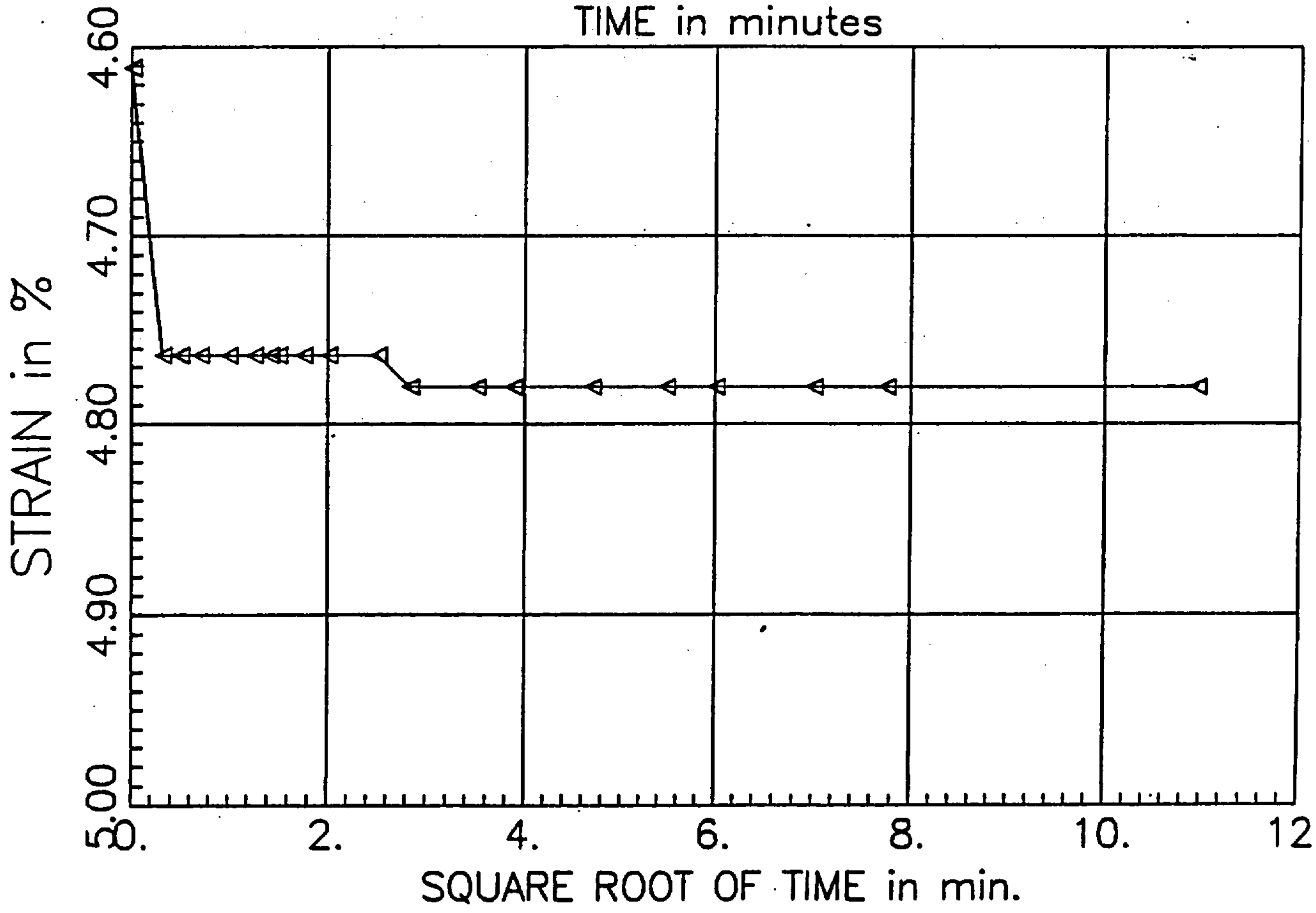
Test No: 4
Testname: C1-U3C



PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

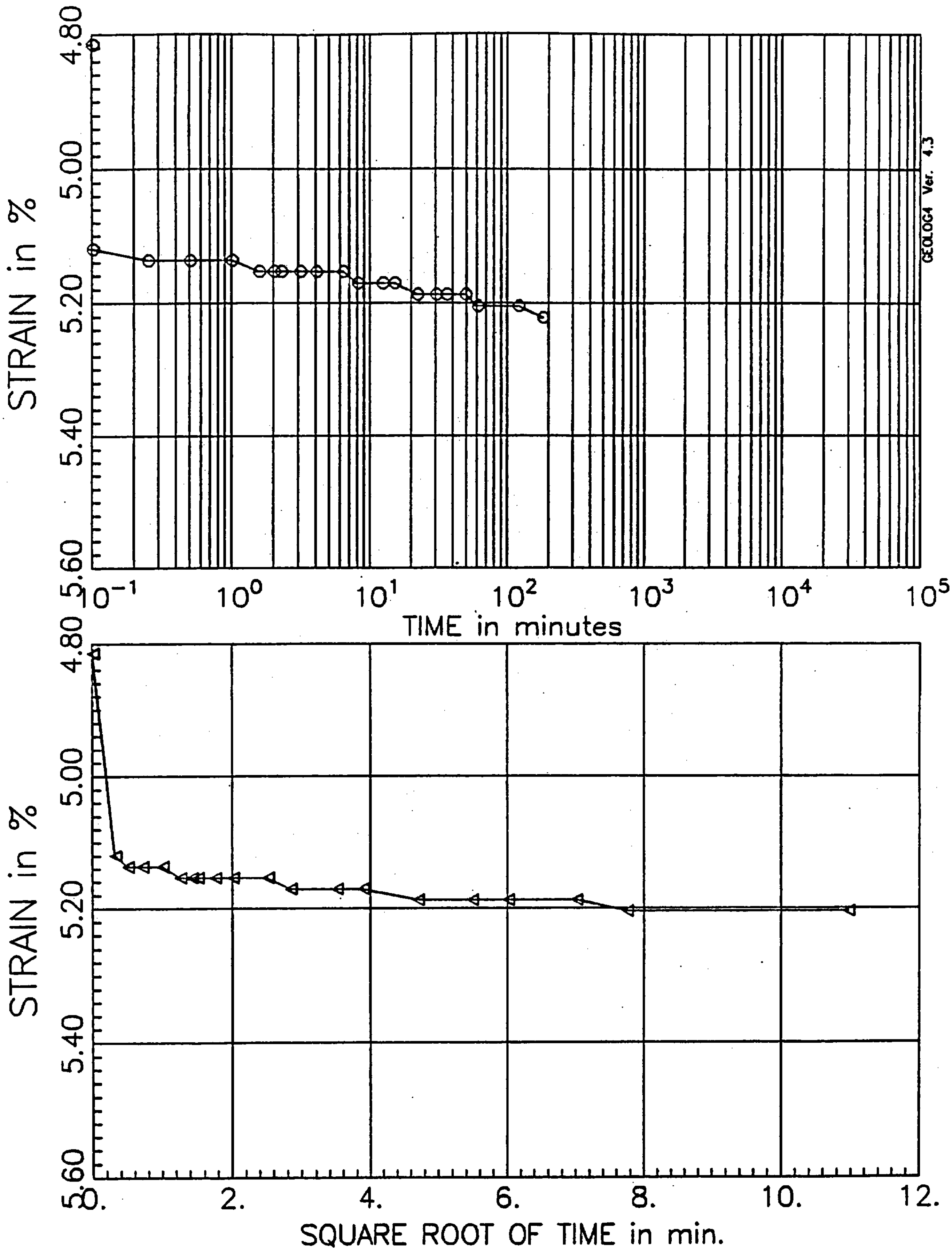
Test No: 4
Testname: C1–U3C





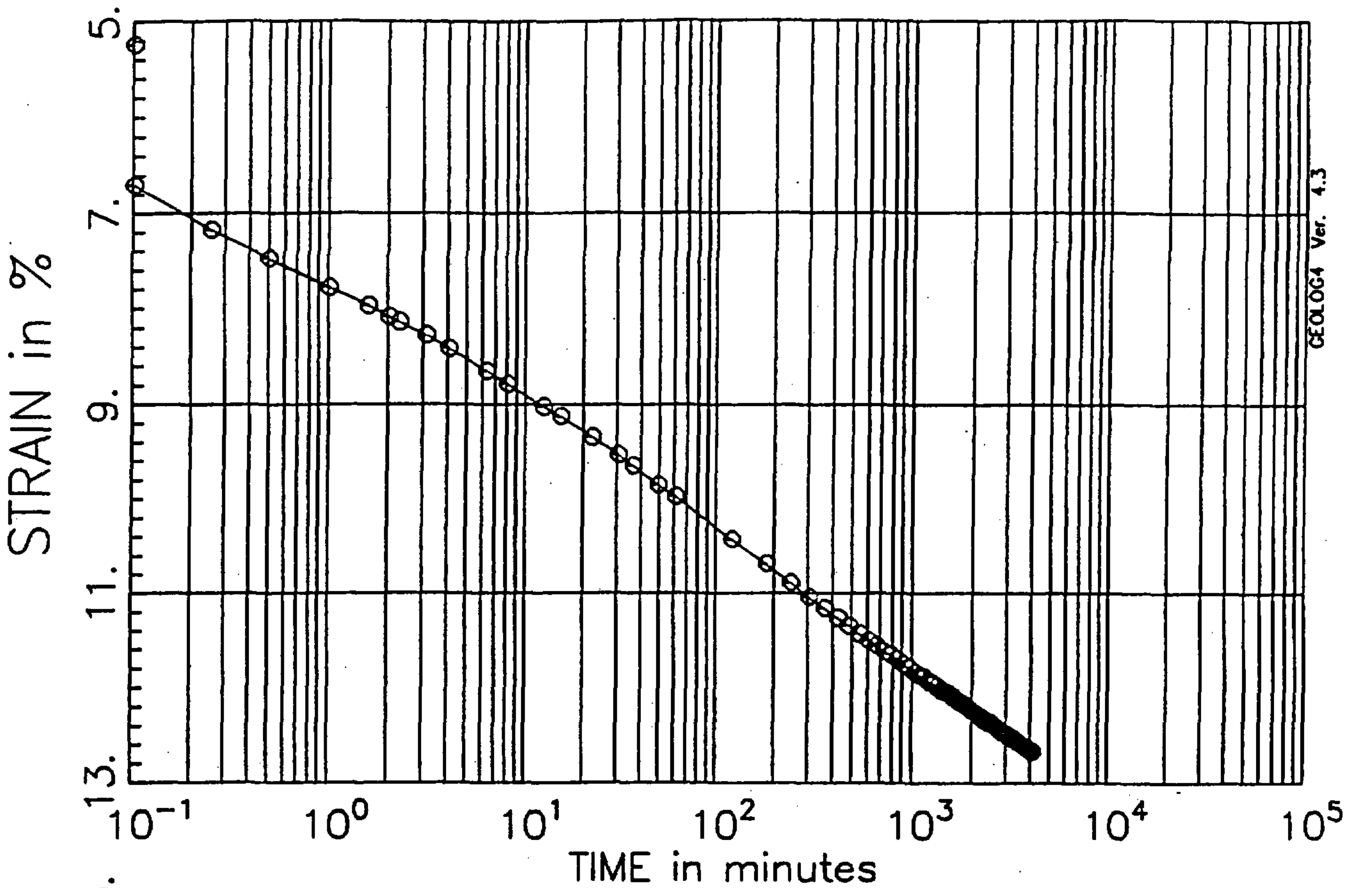
PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

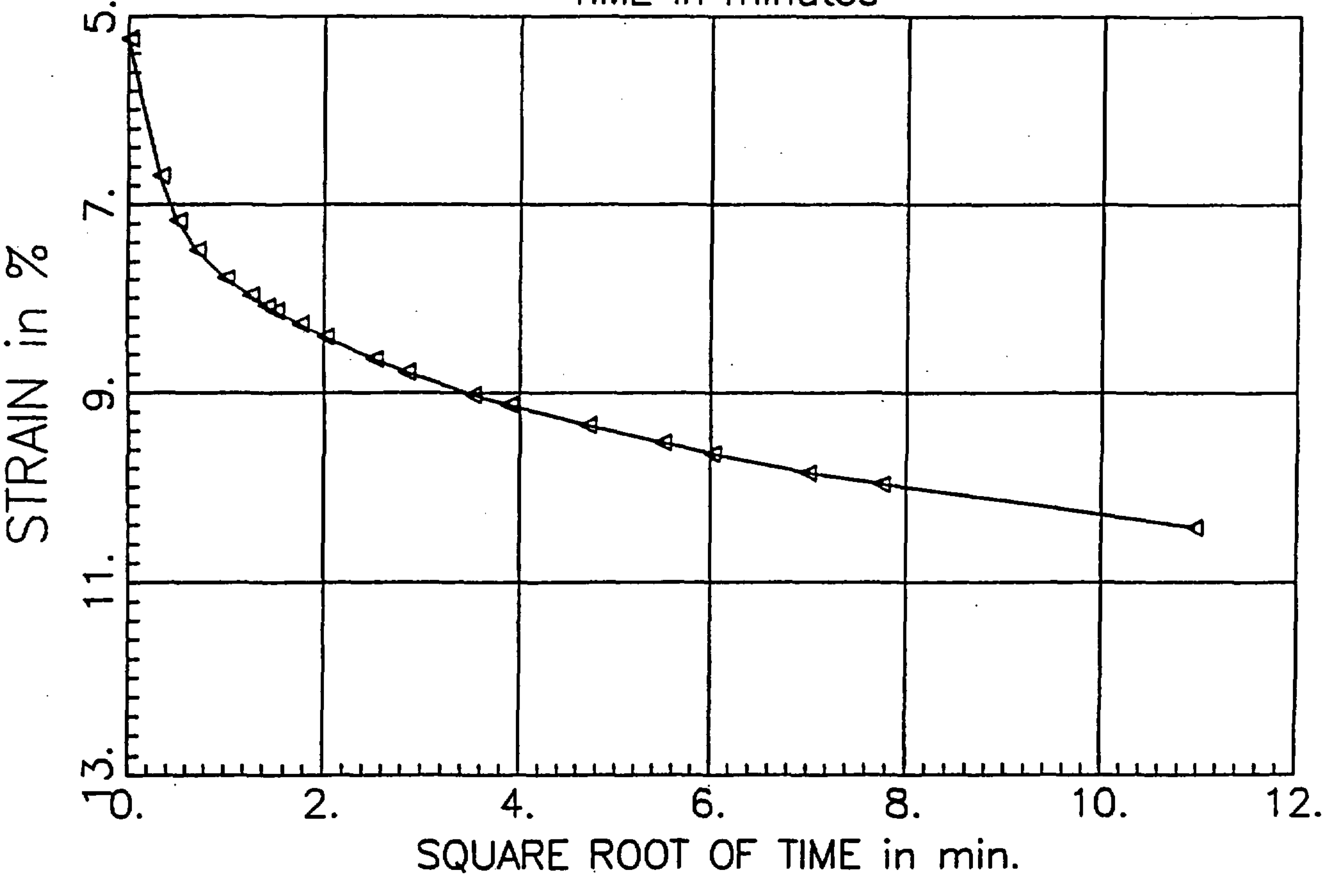
Test No: 4
Testname: C1–U3C



PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

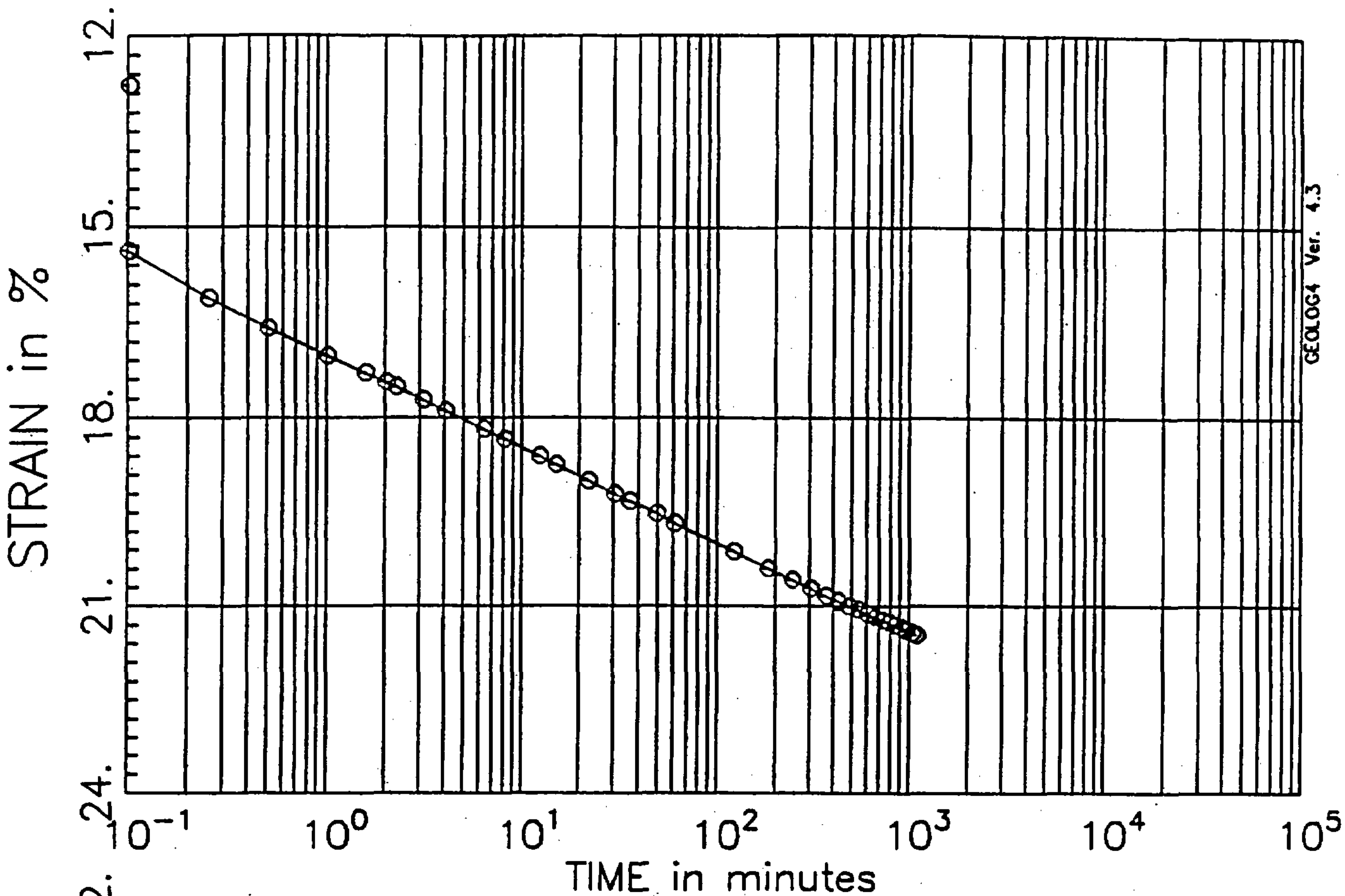
Test No: 4
Testname: C1–U3C

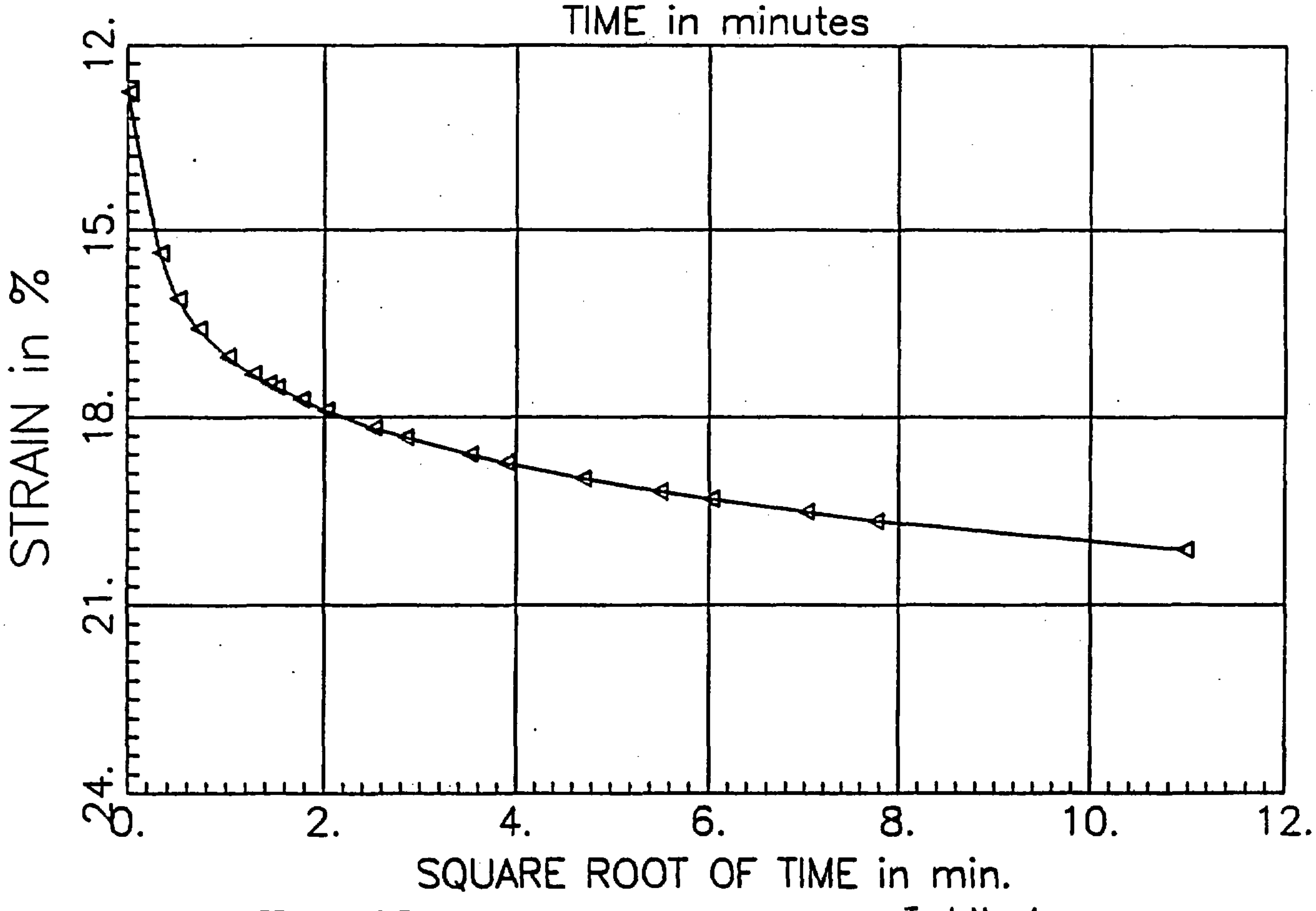




PRESSURE INCREMENT
from 2.00 tsf to 4.00 tsf

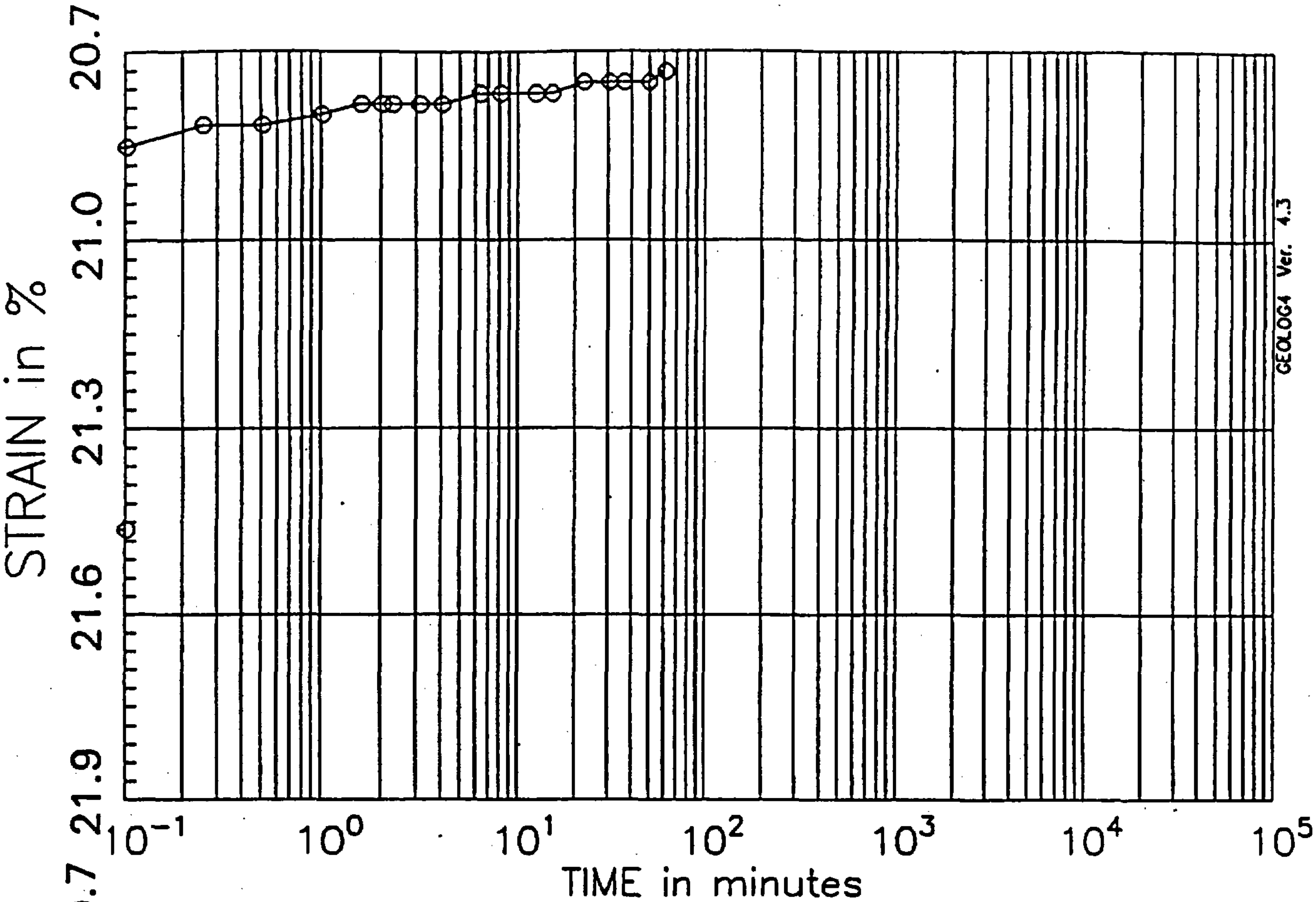
Test No: 4
Testname: C1-U3C

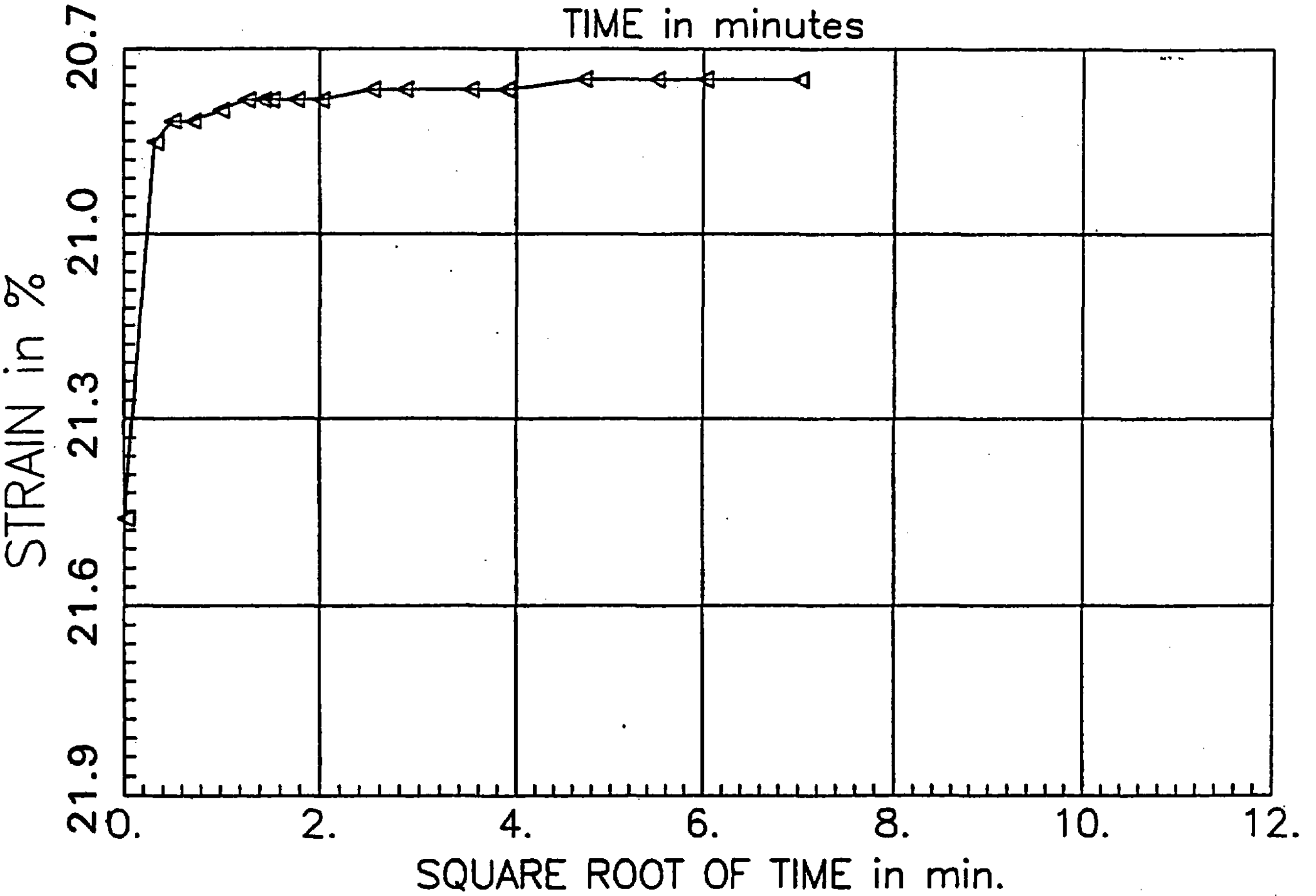




PRESSURE INCREMENT
from 4.00 tsf to 8.00 tsf

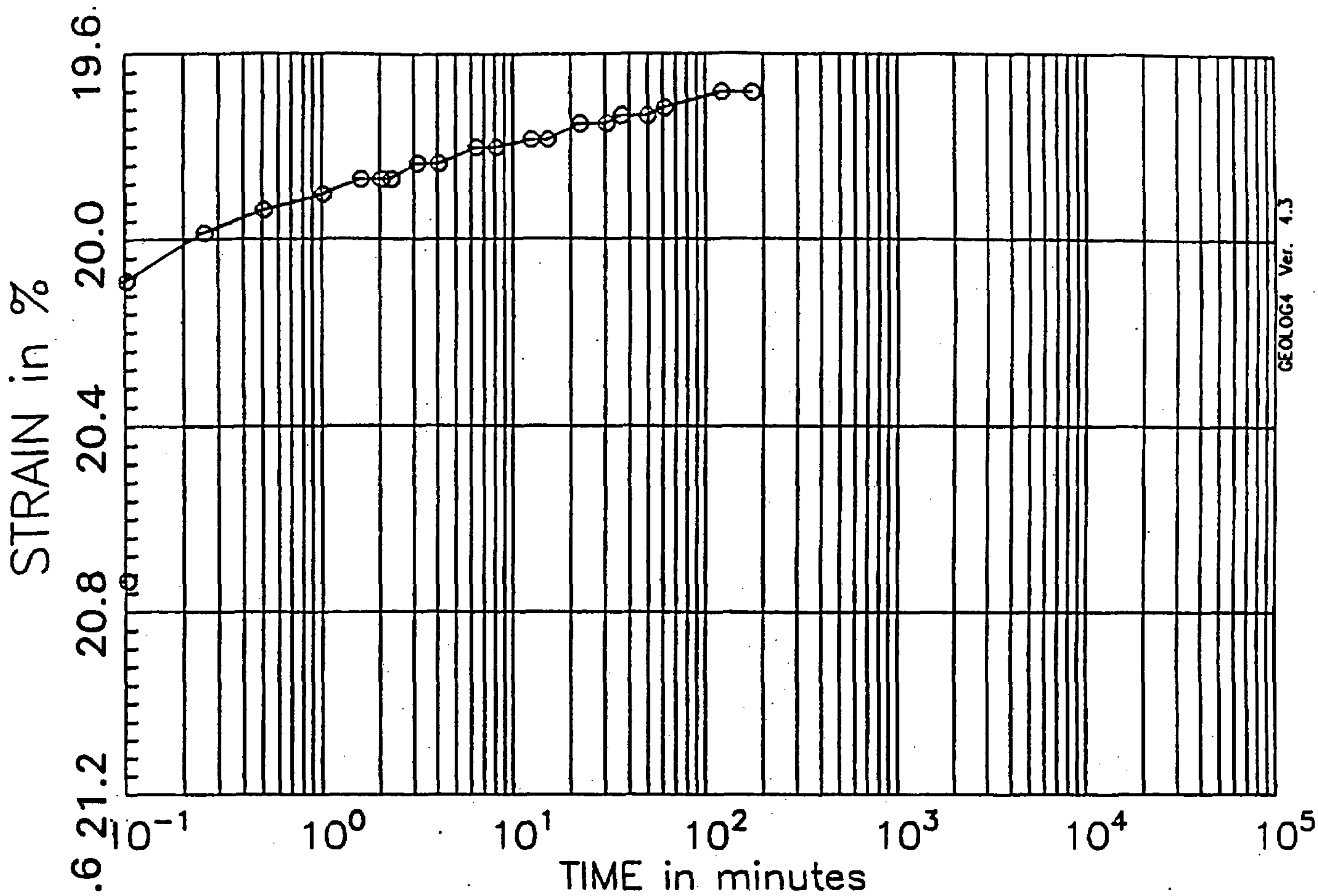
Test No: 4
Testname: C1-U3C

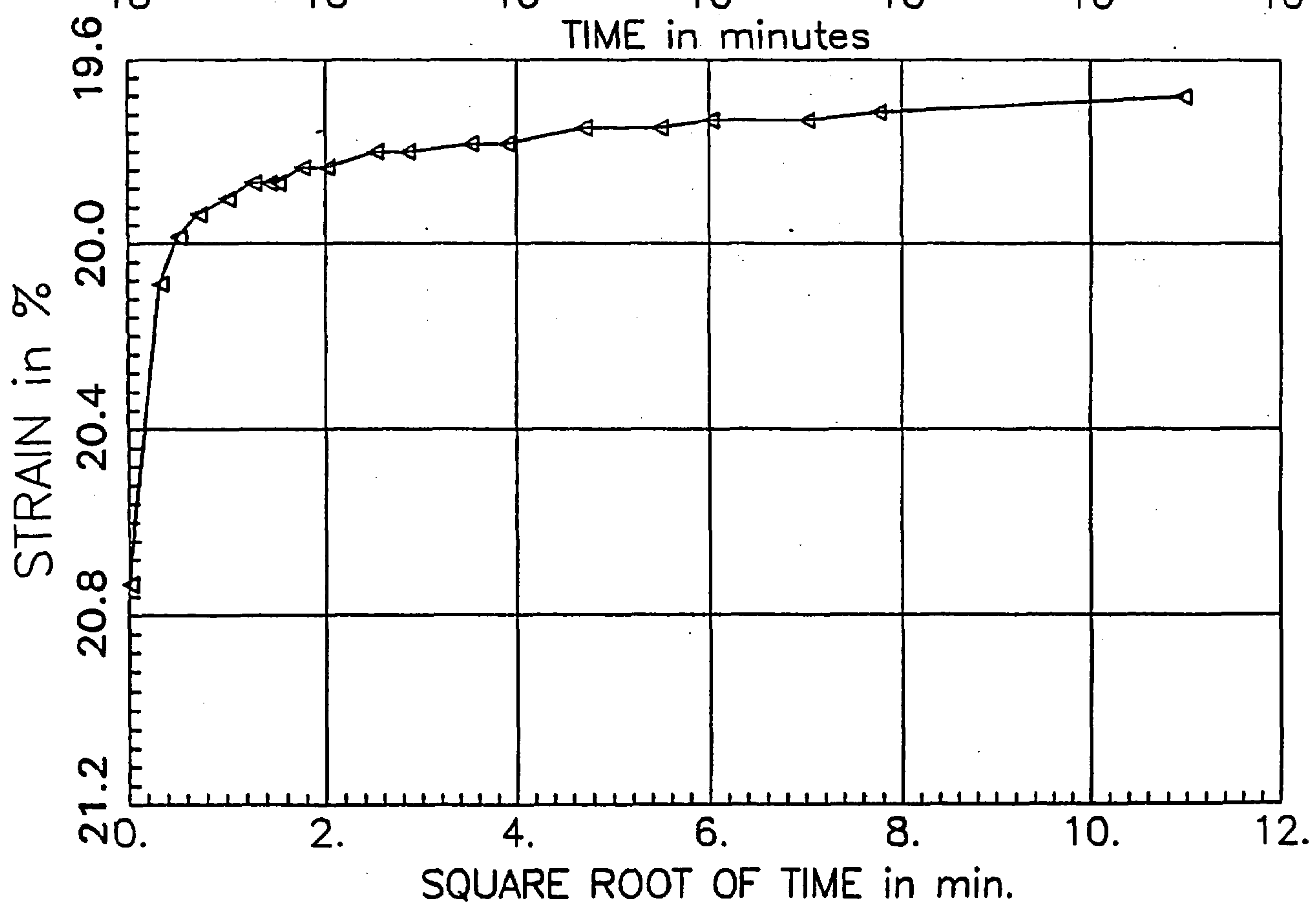




PRESSURE INCREMENT
from 8.00 tsf to 2.00 tsf

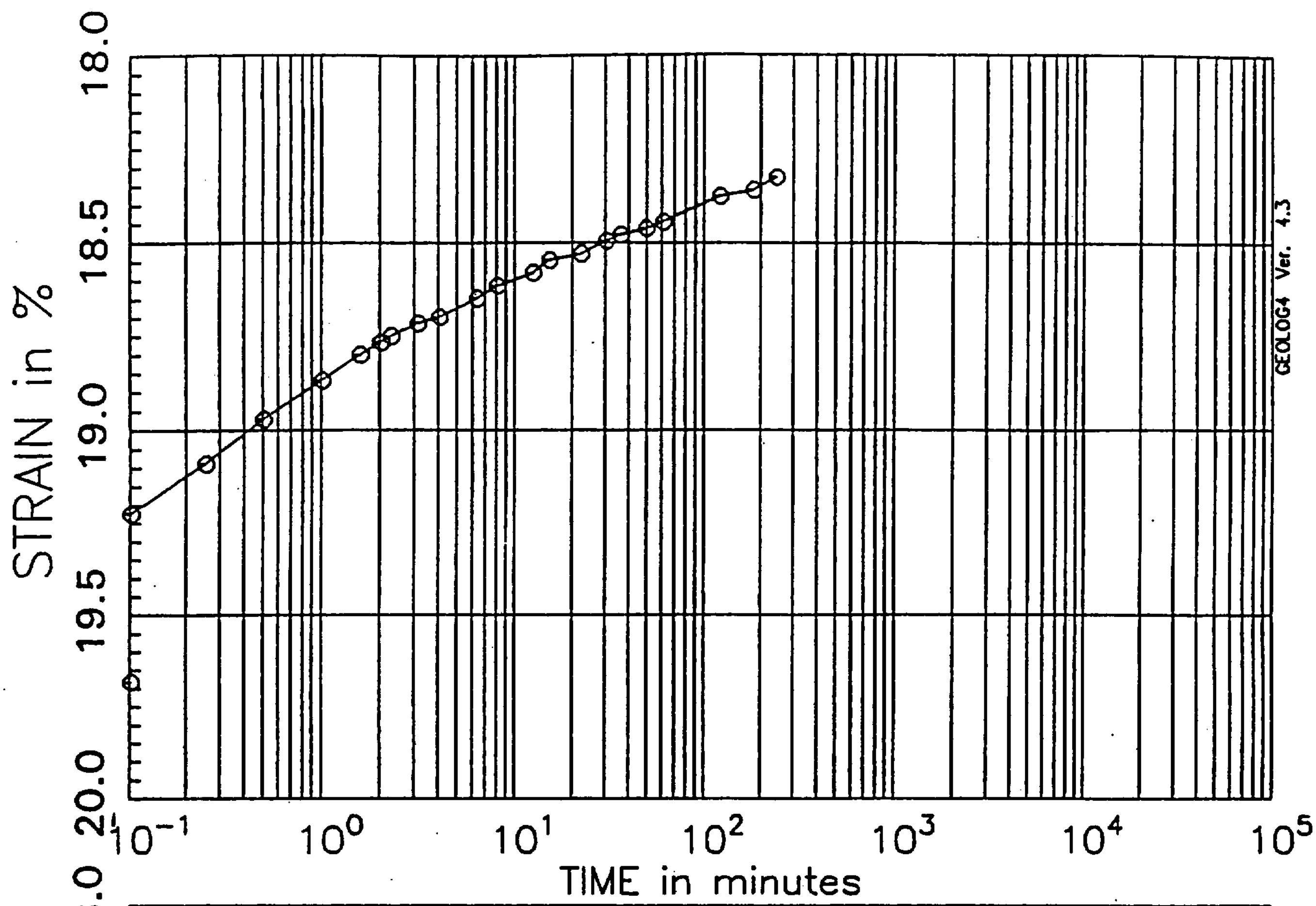
Test No: 4
Testname: C1-U3C

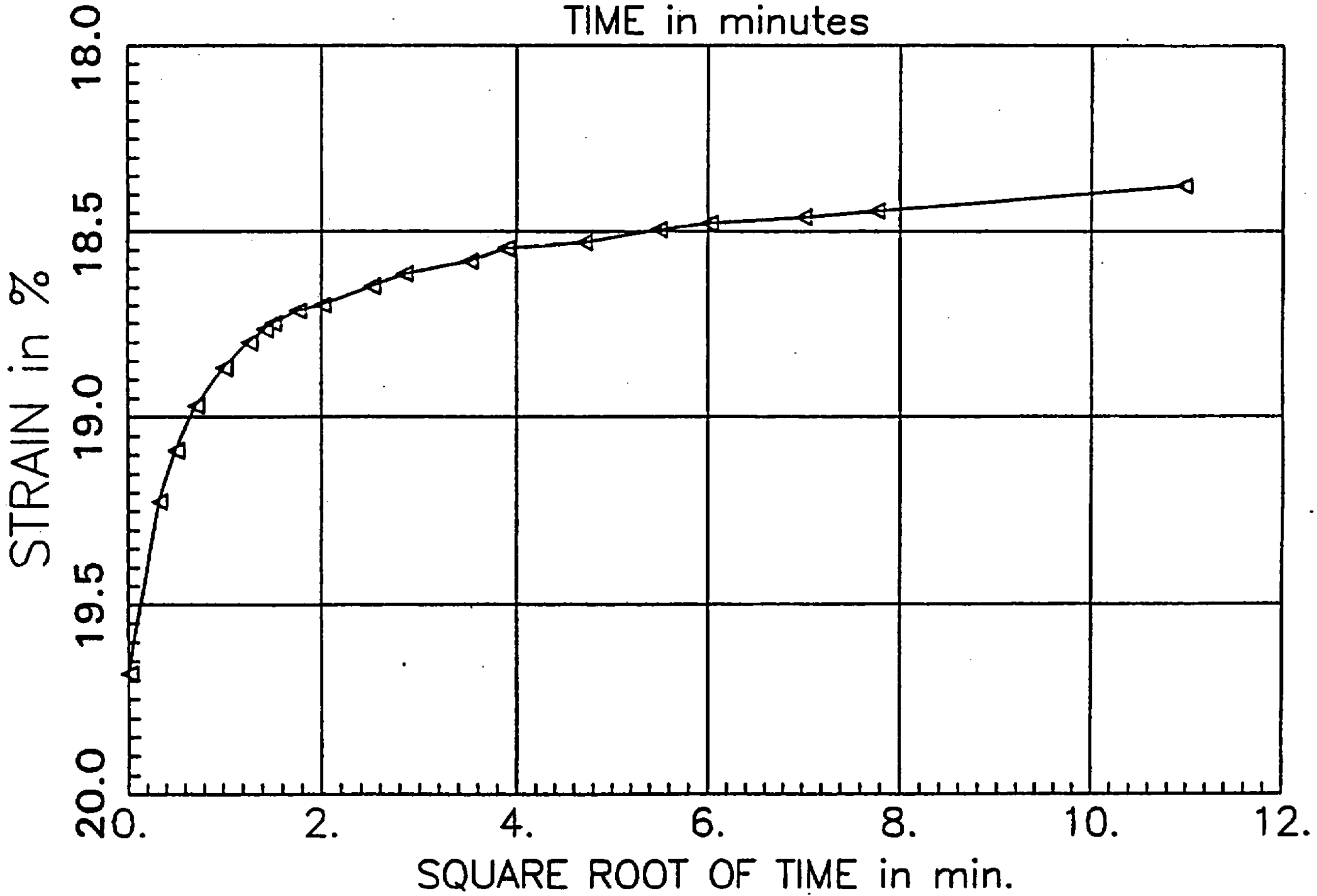




PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

Test No: 4
Testname: C1–U3C





PRESSURE INCREMENT
from 0.50 tsf to 0.10 tsf

Test No: 4
Testname: C1-U3C

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/20/96
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	11.2 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	38.9 %	51.9 %
DRY UNIT WEIGHT:	55.8 pcf	68.4 pcf
VOID RATIO:	2.041	1.484
SATURATION:	51.8 %	95.2 %
HEIGHT:	1.901 cm	1.553 cm
AREA:	31.61 sq cm	
SP. GRAVITY :	2.72	

TEST DATA:

APPLIED PRESSURE tsf	STRAIN %
0.10	0.15
0.25	0.49
0.50	1.02
1.00	1.78
2.00	3.27
2.00	5.15
0.50	4.66
1.00	4.78
2.00	5.17
4.00	8.78
8.00	18.33
8.00	21.46
2.00	20.77
0.50	19.80
0.10	18.61

LOAD INCREMENT DATA

TEST NAME C1-U3C

PAGE NO: 1

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-20-96	18:05:09	0.10	0.32	0.0010	2.037	0.14
		0.25	0.50	0.0010	2.037	0.14
		0.50	0.71	0.0010	2.037	0.14
		1.00	1.00	0.0010	2.037	0.14
		1.57	1.25	0.0010	2.037	0.14
		2.00	1.41	0.0011	2.036	0.15
		2.25	1.50	0.0011	2.036	0.15
		3.07	1.75	0.0011	2.036	0.15
		4.00	2.00	0.0011	2.036	0.15
		6.25	2.50	0.0011	2.036	0.15
		8.00	2.83	0.0011	2.036	0.15
		12.25	3.50	0.0011	2.036	0.15
		15.00	3.87	0.0011	2.036	0.15

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 2
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-20-96	18:24:30	0.00	0.00	0.0011	2.036	0.15
		0.10	0.32	0.0032	2.028	0.42
		0.25	0.50	0.0033	2.028	0.44
		0.50	0.71	0.0034	2.027	0.46
		1.00	1.00	0.0034	2.027	0.46
		1.57	1.25	0.0036	2.027	0.47
		2.00	1.41	0.0036	2.027	0.47
		2.25	1.50	0.0036	2.027	0.47
		3.07	1.75	0.0036	2.027	0.47
		4.00	2.00	0.0037	2.026	0.49
		6.25	2.50	0.0037	2.026	0.49
		8.00	2.83	0.0037	2.026	0.49
		12.25	3.50	0.0038	2.026	0.51
		15.00	3.87	0.0038	2.026	0.51
		22.00	4.69	0.0038	2.026	0.51
		30.00	5.48	0.0039	2.025	0.53
		36.00	6.00	0.0039	2.025	0.53
		49.00	7.00	0.0039	2.025	0.53
		60.00	7.75	0.0041	2.025	0.54
		120.00	10.95	0.0042	2.024	0.56
		180.00	13.42	0.0043	2.023	0.58
		240.00	15.49	0.0044	2.023	0.59
		300.00	17.32	0.0044	2.023	0.59
12-21-96	00:24:30	360.00	18.97	0.0044	2.023	0.59
		420.00	20.49	0.0046	2.022	0.61
		480.00	21.91	0.0046	2.022	0.61
		540.00	23.24	0.0047	2.022	0.63
		600.00	24.49	0.0047	2.022	0.63
		660.00	25.69	0.0047	2.022	0.63
		720.00	26.83	0.0047	2.022	0.63
		780.00	27.93	0.0047	2.022	0.63
		840.00	28.98	0.0048	2.021	0.64
		900.00	30.00	0.0048	2.021	0.64
		960.00	30.98	0.0048	2.021	0.64
		1020.00	31.94	0.0048	2.021	0.64
		1080.00	32.86	0.0048	2.021	0.64

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0048	2.021	0.64
		1200.00	34.64	0.0048	2.021	0.64
		1260.00	35.50	0.0048	2.021	0.64
		1320.00	36.33	0.0047	2.022	0.63
		1380.00	37.15	0.0047	2.022	0.63
		1440.00	37.95	0.0047	2.022	0.63
		1500.00	38.73	0.0047	2.022	0.63

LOAD INCREMENT DATA

TEST NAME C1-U3C
TESTED BY: ACS

PAGE NO: 4
JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-21-96	19:35:55	0.00	0.00	0.0047	2.022	0.63
		0.10	0.32	0.0070	2.013	0.93
		0.25	0.50	0.0071	2.012	0.95
		0.50	0.71	0.0072	2.012	0.97
		1.00	1.00	0.0074	2.011	0.98
		1,57	1.25	0.0075	2.011	1.00
		2.00	1.41	0.0075	2.011	1.00
		2.25	1.50	0.0075	2.011	1.00
		3.07	1.75	0.0075	2.011	1.00
		4.00	2.00	0.0075	2.011	1.00
		6.25	2.50	0.0076	2.010	1.02
		8.00	2.83	0.0076	2.010	1.02
		12.25	3.50	0.0077	2.010	1.03
		15.00	3.87	0.0077	2.010	1.03
		22.00	4.69	0.0077	2.010	1.03
		30.00	5.48	0.0079	2.009	1.05
		36.00	6.00	0.0079	2.009	1.05
		49.00	7.00	0.0080	2.009	1.07
		60.00	7.75	0.0080	2.009	1.07
		120.00	10.95	0.0081	2.008	1.08
		180.00	13.42	0.0082	2.007	1.10
		240.00	15.49	0.0084	2.007	1.12
12-22-96	00:35:55	300.00	17.32	0.0084	2.007	1.12
		360.00	18.97	0.0085	2.006	1.14
		420.00	20.49	0.0085	2.006	1.14
		480.00	21.91	0.0085	2.006	1.14
		540.00	23.24	0.0086	2.006	1.15
		600.00	24.49	0.0086	2.006	1.15
		660.00	25.69	0.0086	2.006	1.15
		720.00	26.83	0.0086	2.006	1.15
		780.00	27.93	0.0088	2.005	1.17
		840.00	28.98	0.0088	2.005	1.17
		900.00	30.00	0.0088	2.005	1.17
		960.00	30.98	0.0088	2.005	1.17
		1020.00	31.94	0.0088	2.005	1.17
		1080.00	32.86	0.0088	2.005	1.17
		1140.00	33.76	0.0089	2.005	1.19
		1200.00	34.64	0.0088	2.005	1.17
		1260.00	35.50	0.0088	2.005	1.17
		1320.00	36.33	0.0088	2.005	1.17
		1380.00	37.15	0.0088	2.005	1.17

LOAD INCREMENT DATA

TEST NAME C1-U3C
TESTED BY: ACS

PAGE NO: 5
JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-22-96	19:29:40	0.00	0.00	0.0088	2.005	1.17
		0.10	0.32	0.0119	1.993	1.59
		0.25	0.50	0.0124	1.990	1.66
		0.50	0.71	0.0126	1.990	1.68
		1.00	1.00	0.0127	1.989	1.70
		1.57	1.25	0.0128	1.989	1.71
		2.00	1.41	0.0129	1.988	1.73
		2.25	1.50	0.0129	1.988	1.73
		3.07	1.75	0.0131	1.988	1.75
		4.00	2.00	0.0131	1.988	1.75
		6.25	2.50	0.0132	1.987	1.76
		8.00	2.83	0.0133	1.987	1.78
		12.25	3.50	0.0134	1.986	1.80
		15.00	3.87	0.0134	1.986	1.80
		22.00	4.69	0.0136	1.986	1.81
		30.00	5.48	0.0137	1.985	1.83
		36.00	6.00	0.0137	1.985	1.83
		49.00	7.00	0.0138	1.985	1.85
		60.00	7.75	0.0140	1.984	1.86
		120.00	10.95	0.0142	1.983	1.90
		180.00	13.42	0.0143	1.983	1.92
		240.00	15.49	0.0145	1.982	1.93
12-23-96	00:29:40	300.00	17.32	0.0146	1.982	1.95
		360.00	18.97	0.0147	1.981	1.97
		420.00	20.49	0.0147	1.981	1.97
		480.00	21.91	0.0148	1.981	1.98
		540.00	23.24	0.0148	1.981	1.98
		600.00	24.49	0.0148	1.981	1.98
		660.00	25.69	0.0148	1.981	1.98
		720.00	26.83	0.0148	1.981	1.98
		780.00	27.93	0.0148	1.981	1.98
		840.00	28.98	0.0148	1.981	1.98
		900.00	30.00	0.0148	1.981	1.98
		960.00	30.98	0.0148	1.981	1.98
		1020.00	31.94	0.0148	1.981	1.98
		1080.00	32.86	0.0148	1.981	1.98

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 6

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0148	1.981	1.98
		1200.00	34.64	0.0148	1.981	1.98
		1260.00	35.50	0.0148	1.981	1.98
		1320.00	36.33	0.0148	1.981	1.98
		1380.00	37.15	0.0150	1.980	2.00
		1440.00	37.95	0.0150	1.980	2.00
		1500.00	38.73	0.0151	1.980	2.02
		1560.00	39.50	0.0152	1.979	2.03
		1620.00	40.25	0.0152	1.979	2.03
		1680.00	40.99	0.0152	1.979	2.03
12-24-96	00:29:40	1740.00	41.71	0.0154	1.979	2.05
		1800.00	42.43	0.0154	1.979	2.05
		1860.00	43.13	0.0154	1.979	2.05
		1920.00	43.82	0.0154	1.979	2.05
		1980.00	44.50	0.0154	1.979	2.05
		2040.00	45.17	0.0154	1.979	2.05
		2100.00	45.83	0.0154	1.979	2.05
		2160.00	46.48	0.0154	1.979	2.05
		2220.00	47.12	0.0152	1.979	2.03
		2280.00	47.75	0.0152	1.979	2.03
		2340.00	48.37	0.0152	1.979	2.03
		2400.00	48.99	0.0152	1.979	2.03
		2460.00	49.60	0.0152	1.979	2.03
		2520.00	50.20	0.0152	1.979	2.03
		2580.00	50.79	0.0154	1.979	2.05

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 7

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-24-96	14:45:04	0.00	0.00	0.0154	1.979	2.05
		0.10	0.32	0.0211	1.955	2.81
		0.25	0.50	0.0218	1.952	2.92
		0.50	0.71	0.0223	1.950	2.98
		1.00	1.00	0.0227	1.949	3.03
		1.57	1.25	0.0231	1.947	3.09
		2.00	1.41	0.0233	1.946	3.12
		2.25	1.50	0.0233	1.946	3.12
		3.07	1.75	0.0236	1.945	3.15
		4.00	2.00	0.0239	1.944	3.19
		6.25	2.50	0.0242	1.943	3.24
		8.00	2.83	0.0245	1.942	3.27
		12.25	3.50	0.0250	1.939	3.34
		15.00	3.87	0.0252	1.938	3.37
		22.00	4.69	0.0258	1.936	3.44
		30.00	5.48	0.0261	1.935	3.49
		36.00	6.00	0.0264	1.934	3.53
		49.00	7.00	0.0268	1.932	3.58
		60.00	7.75	0.0272	1.931	3.63
		120.00	10.95	0.0283	1.926	3.78
		180.00	13.42	0.0289	1.923	3.87
		240.00	15.49	0.0294	1.921	3.93
		300.00	17.32	0.0298	1.920	3.98
		360.00	18.97	0.0301	1.919	4.02
		420.00	20.49	0.0305	1.917	4.07
		480.00	21.91	0.0306	1.917	4.09
		540.00	23.24	0.0308	1.916	4.12
12-25-96	00:45:04	600.00	24.49	0.0310	1.915	4.14
		660.00	25.69	0.0312	1.914	4.17
		720.00	26.83	0.0313	1.914	4.19
		780.00	27.93	0.0315	1.913	4.20
		840.00	28.98	0.0316	1.913	4.22
		900.00	30.00	0.0318	1.912	4.26
		960.00	30.98	0.0318	1.912	4.26
		1020.00	31.94	0.0321	1.911	4.29
		1080.00	32.86	0.0322	1.910	4.31

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 8

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0322	1.910	4.31
		1200.00	34.64	0.0324	1.910	4.32
		1260.00	35.50	0.0325	1.909	4.34
		1320.00	36.33	0.0326	1.909	4.36
		1380.00	37.15	0.0327	1.908	4.37
		1440.00	37.95	0.0329	1.907	4.39
		1500.00	38.73	0.0329	1.907	4.39
		1560.00	39.50	0.0330	1.907	4.41
		1620.00	40.25	0.0331	1.906	4.42
		1680.00	40.99	0.0331	1.906	4.42
		1740.00	41.71	0.0332	1.906	4.44
		1800.00	42.43	0.0334	1.905	4.46
		1860.00	43.13	0.0334	1.905	4.46
		1920.00	43.82	0.0335	1.905	4.48
		1980.00	44.50	0.0335	1.905	4.48
12-26-96	00:45:04	2040.00	45.17	0.0336	1.904	4.49
		2100.00	45.83	0.0337	1.904	4.51
		2160.00	46.48	0.0337	1.904	4.51
		2220.00	47.12	0.0337	1.904	4.51
		2280.00	47.75	0.0339	1.903	4.53
		2340.00	48.37	0.0339	1.903	4.53
		2400.00	48.99	0.0339	1.903	4.53
		2460.00	49.60	0.0339	1.903	4.53
		2520.00	50.20	0.0340	1.903	4.54
		2580.00	50.79	0.0340	1.903	4.54
		2640.00	51.38	0.0340	1.903	4.54
		2700.00	51.96	0.0340	1.903	4.54
		2760.00	52.54	0.0341	1.902	4.56
		2820.00	53.10	0.0341	1.902	4.56
		2880.00	53.67	0.0341	1.902	4.56
		2940.00	54.22	0.0343	1.902	4.58
		3000.00	54.77	0.0343	1.902	4.58
		3060.00	55.32	0.0344	1.901	4.59
		3120.00	55.86	0.0344	1.901	4.59

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 9

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0345	1.901	4.61
		3240.00	56.92	0.0346	1.900	4.63
		3300.00	57.45	0.0346	1.900	4.63
		3360.00	57.97	0.0348	1.900	4.64
		3420.00	58.48	0.0348	1.900	4.64
12-27-96	00:45:04	3480.00	58.99	0.0349	1.899	4.66
		3540.00	59.50	0.0349	1.899	4.66
		3600.00	60.00	0.0349	1.899	4.66
		3660.00	60.50	0.0350	1.899	4.68
		3720.00	60.99	0.0350	1.899	4.68
		3780.00	61.48	0.0350	1.899	4.68
		3840.00	61.97	0.0350	1.899	4.68
		3900.00	62.45	0.0350	1.899	4.68
		3960.00	62.93	0.0350	1.899	4.68
		4020.00	63.40	0.0350	1.899	4.68
		4080.00	63.87	0.0350	1.899	4.68
		4140.00	64.34	0.0351	1.898	4.70
		4200.00	64.81	0.0351	1.898	4.70
		4260.00	65.27	0.0351	1.898	4.70
		4320.00	65.73	0.0353	1.898	4.71
		4380.00	66.18	0.0353	1.898	4.71
		4440.00	66.63	0.0353	1.898	4.71
		4500.00	67.08	0.0353	1.898	4.71
		4560.00	67.53	0.0354	1.897	4.73
		4620.00	67.97	0.0354	1.897	4.73
		4680.00	68.41	0.0355	1.897	4.75
		4740.00	68.85	0.0357	1.896	4.76
		4800.00	69.28	0.0357	1.896	4.76
		4860.00	69.71	0.0357	1.896	4.76
12-28-96	00:45:04	4920.00	70.14	0.0358	1.896	4.78
		4980.00	70.57	0.0358	1.896	4.78
		5040.00	70.99	0.0359	1.895	4.80
		5100.00	71.41	0.0359	1.895	4.80
		5160.00	71.83	0.0359	1.895	4.80

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 10

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		5220.00	72.25	0.0360	1.895	4.81
		5280.00	72.66	0.0360	1.895	4.81
		5340.00	73.08	0.0360	1.895	4.81
		5400.00	73.48	0.0360	1.895	4.81
		5460.00	73.89	0.0362	1.894	4.83
		5520.00	74.30	0.0362	1.894	4.83
		5580.00	74.70	0.0362	1.894	4.83
		5640.00	75.10	0.0362	1.894	4.83
		5700.00	75.50	0.0362	1.894	4.83
		5760.00	75.89	0.0363	1.894	4.85
		5820.00	76.29	0.0363	1.894	4.85
		5880.00	76.68	0.0363	1.894	4.85
		5940.00	77.07	0.0363	1.894	4.85
		6000.00	77.46	0.0363	1.894	4.85
		6060.00	77.85	0.0364	1.893	4.87
		6120.00	78.23	0.0364	1.893	4.87
		6180.00	78.61	0.0364	1.893	4.87
		6240.00	78.99	0.0364	1.893	4.87
		6300.00	79.37	0.0365	1.893	4.88
12-29-96	00:45:04	6360.00	79.75	0.0365	1.893	4.88
		6420.00	80.12	0.0365	1.893	4.88
		6480.00	80.50	0.0365	1.893	4.88
		6540.00	80.87	0.0365	1.893	4.88
		6600.00	81.24	0.0367	1.892	4.90
		6660.00	81.61	0.0367	1.892	4.90
		6720.00	81.98	0.0367	1.892	4.90
		6780.00	82.34	0.0367	1.892	4.90
		6840.00	82.70	0.0367	1.892	4.90
		6900.00	83.07	0.0368	1.891	4.92
		6960.00	83.43	0.0368	1.891	4.92
		7020.00	83.79	0.0368	1.891	4.92
		7080.00	84.14	0.0368	1.891	4.92
		7140.00	84.50	0.0368	1.891	4.92
		7200.00	84.85	0.0369	1.891	4.93

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 11

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		7260.00	85.21	0.0369	1.891	4.93
		7320.00	85.56	0.0369	1.891	4.93
		7380.00	85.91	0.0369	1.891	4.93
		7440.00	86.26	0.0369	1.891	4.93
		7500.00	86.60	0.0369	1.891	4.93
		7560.00	86.95	0.0369	1.891	4.93
		7620.00	87.29	0.0370	1.890	4.95
		7680.00	87.64	0.0370	1.890	4.95
		7740.00	87.98	0.0370	1.890	4.95
12-30-96	00:45:04	7800.00	88.32	0.0370	1.890	4.95
		7860.00	88.66	0.0370	1.890	4.95
		7920.00	88.99	0.0370	1.890	4.95
		7980.00	89.33	0.0370	1.890	4.95
		8040.00	89.67	0.0372	1.890	4.97
		8100.00	90.00	0.0372	1.890	4.97
		8160.00	90.33	0.0370	1.890	4.95
		8220.00	90.66	0.0370	1.890	4.95
		8280.00	90.99	0.0370	1.890	4.95
		8340.00	91.32	0.0370	1.890	4.95
		8400.00	91.65	0.0370	1.890	4.95
		8460.00	91.98	0.0370	1.890	4.95
		8520.00	92.30	0.0370	1.890	4.95
		8580.00	92.63	0.0370	1.890	4.95
		8640.00	92.95	0.0370	1.890	4.95
		8700.00	93.27	0.0370	1.890	4.95
		8760.00	93.59	0.0372	1.890	4.97
		8820.00	93.91	0.0372	1.890	4.97
		8880.00	94.23	0.0372	1.890	4.97
		8940.00	94.55	0.0373	1.889	4.98
		9000.00	94.87	0.0373	1.889	4.98
		9060.00	95.18	0.0373	1.889	4.98
		9120.00	95.50	0.0374	1.889	5.00
		9180.00	95.81	0.0374	1.889	5.00
12-31-96	00:45:04	9240.00	96.12	0.0374	1.889	5.00

LOAD INCREMENT DATA

TEST NAME C1-U3C
TESTED BY: ACS

JO: 05996.01

PRESSURE INC[illegible]ENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		9[illegible].00	96.44	0.0374	1.889	5.0[illegible]
		9[illegible].00	96.75	0.0376	1.888	5.02
		9[illegible].00	97.06	0.0376	1.888	5.02
		9[illegible].00	97.37	0.0376	1.888	5.02
		95[illegible].00	97.67	0.0376	1.888	5.02
		9[illegible].00	97.98	0.0376	1.888	5.02
		96[illegible].00	98.29	0.0376	1.888	5.02
		97[illegible].00	98.59	0.0376	1.888	5.02
		97[illegible].00	98.89	0.0374	1.889	5.00
		9[illegible].00	99.20	0.0374	1.889	5.00
		99[illegible].00	99.50	0.0374	1.889	5.0[illegible]
		9[illegible].00	99.80	0.0374	1.889	5.00
		10[illegible]00E+01	100.10	0.0374	1.889	5.[illegible]
		10[illegible]00E+01	100.40	0.0376	1.888	5.[illegible]
		10[illegible]00E+01	100.70	0.0376	1.888	5.[illegible]
		10[illegible]00E+01	101.00	0.0376	1.888	5.[illegible]
		10[illegible]00E+01	101.29	0.0377	1.888	5.[illegible]
		103[illegible]00E+01	101.59	0.0377	1.888	5.[illegible]
		10[illegible]00E+01	101.88	0.0378	1.887	5.[illegible]
		104[illegible]00E+01	102.18	0.0378	1.887	5.[illegible]
		105[illegible]0E+01	102.47	0.0379	1.887	5.[illegible]
		105[illegible]00E+01	102.76	0.0379	1.887	5.[illegible]
		10[illegible]0E+01	103.05	0.0379	1.887	5.0[illegible]
01-01-97	00:45:04	10[illegible]0E+01	103.34	0.0381	1.886	5.09
		10[illegible]0E+01	103.63	0.0381	1.886	5.09
		108[illegible]0E+01	103.92	0.0381	1.886	5.0[illegible]
		10[illegible]0E+01	104.21	0.0382	1.886	5.[illegible]
		109[illegible]0E+01	104.50	0.0382	1.886	5.[illegible]
		109[illegible]0E+01	104.79	0.0382	1.886	5.[illegible]
		110[illegible]0E+01	105.07	0.0382	1.886	5.[illegible]
		11[illegible]0E+01	105.36	0.0382	1.886	5.[illegible]
		11[illegible]0E+01	105.64	0.0382	1.886	5.[illegible]
		1122[illegible]0E+01	105.92	0.0383	1.885	5.12
		11[illegible]0E+01	106.21	0.0383	1.885	5.12

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 13
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1134.00E+01	106.49	0.0383	1.885	5.12
		1140.00E+01	106.77	0.0384	1.885	5.14
		1146.00E+01	107.05	0.0384	1.885	5.14
		1152.00E+01	107.33	0.0384	1.885	5.14
		1158.00E+01	107.61	0.0384	1.885	5.14
		1164.00E+01	107.89	0.0384	1.885	5.14
		1170.00E+01	108.17	0.0384	1.885	5.14
		1176.00E+01	108.44	0.0384	1.885	5.14
		1182.00E+01	108.72	0.0384	1.885	5.14
		1188.00E+01	109.00	0.0384	1.885	5.14
		1194.00E+01	109.27	0.0384	1.885	5.14
		1200.00E+01	109.54	0.0386	1.884	5.15
		1206.00E+01	109.82	0.0384	1.885	5.14
01-02-97	00:45:04	1212.00E+01	110.09	0.0386	1.884	5.15
		1218.00E+01	110.36	0.0386	1.884	5.15
		1224.00E+01	110.63	0.0386	1.884	5.15
		1230.00E+01	110.91	0.0386	1.884	5.15
		1236.00E+01	111.18	0.0386	1.884	5.15
		1242.00E+01	111.45	0.0386	1.884	5.15
		1248.00E+01	111.71	0.0384	1.885	5.14
		1254.00E+01	111.98	0.0384	1.885	5.14
		1260.00E+01	112.25	0.0384	1.885	5.14
		1266.00E+01	112.52	0.0383	1.885	5.12
		1272.00E+01	112.78	0.0384	1.885	5.14
		1278.00E+01	113.05	0.0384	1.885	5.14

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 14

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	11:56:36	0.00	0.00	0.0384	1.885	5.14
		0.10	0.32	0.0354	1.897	4.73
		0.25	0.50	0.0353	1.898	4.71
		0.50	0.71	0.0351	1.898	4.70
		1.00	1.00	0.0350	1.899	4.68
		1.57	1.25	0.0350	1.899	4.68
		2.00	1.41	0.0350	1.899	4.68
		2.25	1.50	0.0350	1.899	4.68
		3.07	1.75	0.0350	1.899	4.68
		4.00	2.00	0.0350	1.899	4.68
		6.25	2.50	0.0350	1.899	4.68
		8.00	2.83	0.0349	1.899	4.66
		12.25	3.50	0.0349	1.899	4.66
		15.00	3.87	0.0349	1.899	4.66
		22.00	4.69	0.0348	1.900	4.64
		30.00	5.48	0.0348	1.900	4.64
		36.00	6.00	0.0348	1.900	4.64
		49.00	7.00	0.0348	1.900	4.64
		60.00	7.75	0.0348	1.900	4.64
		120.00	10.95	0.0346	1.900	4.63
		180.00	13.42	0.0345	1.901	4.61
		240.00	15.49	0.0345	1.901	4.61

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 15

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	16:56:49	0.00	0.00	0.0345	1.901	4.61
		0.10	0.32	0.0357	1.896	4.76
		0.25	0.50	0.0357	1.896	4.76
		0.50	0.71	0.0357	1.896	4.76
		1.00	1.00	0.0357	1.896	4.76
		1.57	1.25	0.0357	1.896	4.76
		2.00	1.41	0.0357	1.896	4.76
		2.25	1.50	0.0357	1.896	4.76
		3.07	1.75	0.0357	1.896	4.76
		4.00	2.00	0.0357	1.896	4.76
		6.25	2.50	0.0357	1.896	4.76
		8.00	2.83	0.0358	1.896	4.78
		12.25	3.50	0.0358	1.896	4.78
		15.00	3.87	0.0358	1.896	4.78
		22.00	4.69	0.0358	1.896	4.78
		30.00	5.48	0.0358	1.896	4.78
		36.00	6.00	0.0358	1.896	4.78
		49.00	7.00	0.0358	1.896	4.78
		60.00	7.75	0.0358	1.896	4.78
		120.00	10.95	0.0358	1.896	4.78
		180.00	13.42	0.0359	1.895	4.80
		240.00	15.49	0.0359	1.895	4.80
		300.00	17.32	0.0360	1.895	4.81
		360.00	18.97	0.0360	1.895	4.81
		420.00	20.49	0.0362	1.894	4.83
01-03-97	00:56:49	480.00	21.91	0.0360	1.895	4.81
		540.00	23.24	0.0362	1.894	4.83
		600.00	24.49	0.0362	1.894	4.83
		660.00	25.69	0.0362	1.894	4.83
		720.00	26.83	0.0362	1.894	4.83
		780.00	27.93	0.0362	1.894	4.83
		840.00	28.98	0.0360	1.895	4.81
		900.00	30.00	0.0360	1.895	4.81
		960.00	30.98	0.0360	1.895	4.81

LOAD INCREMENT DATA

TEST NAME C1-U3C
TESTED BY: ACS

PAGE NO: 16
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	09:03:56	0.00	0.00	0.0360	1.895	4.81
		0.10	0.32	0.0383	1.885	5.12
		0.25	0.50	0.0384	1.885	5.14
		0.50	0.71	0.0384	1.885	5.14
		1.00	1.00	0.0384	1.885	5.14
		1.57	1.25	0.0386	1.884	5.15
		2.00	1.41	0.0386	1.884	5.15
		2.25	1.50	0.0386	1.884	5.15
		3.07	1.75	0.0386	1.884	5.15
		4.00	2.00	0.0386	1.884	5.15
		6.25	2.50	0.0386	1.884	5.15
		8.00	2.83	0.0387	1.884	5.17
		12.25	3.50	0.0387	1.884	5.17
		15.00	3.87	0.0387	1.884	5.17
		22.00	4.69	0.0388	1.883	5.19
		30.00	5.48	0.0388	1.883	5.19
		36.00	6.00	0.0388	1.883	5.19
		49.00	7.00	0.0388	1.883	5.19
		60.00	7.75	0.0390	1.883	5.20
		120.00	10.95	0.0390	1.883	5.20
		180.00	13.42	0.0391	1.882	5.22

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 17

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	12:22:29	0.00	0.00	0.0392	1.882	5.24
		0.10	0.32	0.0501	1.837	6.70
		0.25	0.50	0.0537	1.823	7.17
		0.50	0.71	0.0560	1.814	7.48
		1.00	1.00	0.0582	1.804	7.78
		1.57	1.25	0.0596	1.799	7.97
		2.00	1.41	0.0605	1.795	8.09
		2.25	1.50	0.0609	1.794	8.14
		3.07	1.75	0.0619	1.789	8.27
		4.00	2.00	0.0629	1.785	8.41
		6.25	2.50	0.0647	1.778	8.65
		8.00	2.83	0.0657	1.774	8.78
		12.25	3.50	0.0675	1.767	9.02
		15.00	3.87	0.0683	1.764	9.12
		22.00	4.69	0.0699	1.757	9.34
		30.00	5.48	0.0713	1.751	9.53
		36.00	6.00	0.0722	1.748	9.65
		49.00	7.00	0.0737	1.741	9.85
		60.00	7.75	0.0746	1.738	9.97
		120.00	10.95	0.0780	1.724	10.43
		180.00	13.42	0.0799	1.716	10.68
		240.00	15.49	0.0815	1.710	10.88
		300.00	17.32	0.0826	1.705	11.04
		360.00	18.97	0.0835	1.702	11.15
		420.00	20.49	0.0842	1.699	11.26
		480.00	21.91	0.0849	1.696	11.34
		540.00	23.24	0.0855	1.694	11.43
		600.00	24.49	0.0860	1.691	11.49
		660.00	25.69	0.0864	1.690	11.54
01-04-97	00:22:29	720.00	26.83	0.0869	1.688	11.61
		780.00	27.93	0.0872	1.687	11.65
		840.00	28.98	0.0875	1.685	11.70
		900.00	30.00	0.0879	1.684	11.75
		960.00	30.98	0.0882	1.683	11.78
		1020.00	31.94	0.0886	1.681	11.83
		1080.00	32.86	0.0888	1.680	11.87

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 18
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0889	1.680	11.88
		1200.00	34.64	0.0893	1.678	11.93
		1260.00	35.50	0.0894	1.678	11.95
		1320.00	36.33	0.0897	1.677	11.99
		1380.00	37.15	0.0900	1.675	12.02
		1440.00	37.95	0.0901	1.675	12.04
		1500.00	38.73	0.0902	1.674	12.05
		1560.00	39.50	0.0905	1.673	12.09
		1620.00	40.25	0.0906	1.673	12.10
		1680.00	40.99	0.0908	1.672	12.14
		1740.00	41.71	0.0910	1.671	12.15
		1800.00	42.43	0.0911	1.671	12.17
		1860.00	43.13	0.0912	1.670	12.19
		1920.00	43.82	0.0915	1.669	12.22
		1980.00	44.50	0.0916	1.669	12.24
		2040.00	45.17	0.0917	1.668	12.26
		2100.00	45.83	0.0919	1.668	12.27
01-05-97	00:22:29	2160.00	46.48	0.0920	1.667	12.29
		2220.00	47.12	0.0921	1.667	12.31
		2280.00	47.75	0.0922	1.666	12.32
		2340.00	48.37	0.0924	1.666	12.34
		2400.00	48.99	0.0925	1.665	12.36
		2460.00	49.60	0.0925	1.665	12.36
		2520.00	50.20	0.0926	1.665	12.38
		2580.00	50.79	0.0927	1.664	12.39
		2640.00	51.38	0.0929	1.664	12.41
		2700.00	51.96	0.0930	1.663	12.43
		2760.00	52.54	0.0931	1.663	12.44
		2820.00	53.10	0.0933	1.662	12.46
		2880.00	53.67	0.0933	1.662	12.46
		2940.00	54.22	0.0934	1.662	12.48
		3000.00	54.77	0.0935	1.661	12.49
		3060.00	55.32	0.0935	1.661	12.49
		3120.00	55.86	0.0936	1.661	12.51

LOAD INCREMENT DATA

TEST NAME C1-U3C PAGE NO: 19
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0938	1.660	12.53
		3240.00	56.92	0.0938	1.660	12.53
		3300.00	57.45	0.0939	1.660	12.54
		3360.00	57.97	0.0940	1.659	12.56
		3420.00	58.48	0.0940	1.659	12.56
		3480.00	58.99	0.0941	1.658	12.58
		3540.00	59.50	0.0943	1.658	12.60
01-06-97	00:22:29	3600.00	60.00	0.0943	1.658	12.60
		3660.00	60.50	0.0944	1.657	12.61
		3720.00	60.99	0.0944	1.657	12.61
		3780.00	61.48	0.0945	1.657	12.63
		3840.00	61.97	0.0945	1.657	12.63
		3900.00	62.45	0.0946	1.656	12.65
		3960.00	62.93	0.0948	1.656	12.66
		4020.00	63.40	0.0948	1.656	12.66
		4080.00	63.87	0.0948	1.656	12.66
		4320.27	65.73	0.0950	1.655	12.70

LOAD INCREMENT DATA

TEST NAME C1-U3C | PAGE NO: 20
TESTED BY: ACS | JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-06-97	13:45:09	0.00	0.00	0.0955	1.653	12.77
		0.10	0.32	0.1151	1.573	15.38
		0.25	0.50	0.1205	1.551	16.10
		0.50	0.71	0.1241	1.537	16.58
		1.00	1.00	0.1274	1.523	17.02
		1.57	1.25	0.1294	1.515	17.29
		2.00	1.41	0.1306	1.511	17.44
		2.25	1.50	0.1311	1.508	17.51
		3.07	1.75	0.1326	1.502	17.72
		4.00	2.00	0.1339	1.497	17.88
		6.25	2.50	0.1360	1.488	18.17
		8.00	2.83	0.1372	1.484	18.33
		12.25	3.50	0.1392	1.475	18.60
		15.00	3.87	0.1402	1.471	18.73
		22.00	4.69	0.1421	1.464	18.99
		30.00	5.48	0.1436	1.457	19.19
		36.00	6.00	0.1445	1.454	19.31
		49.00	7.00	0.1460	1.448	19.51
		60.00	7.75	0.1472	1.443	19.66
		120.00	10.95	0.1506	1.429	20.12
		180.00	13.42	0.1526	1.421	20.39
		240.00	15.49	0.1540	1.415	20.58
		300.00	17.32	0.1550	1.411	20.72
		360.00	18.97	0.1559	1.407	20.83
		420.00	20.49	0.1566	1.405	20.92
		480.00	21.91	0.1572	1.402	21.00
		540.00	23.24	0.1576	1.401	21.05
		600.00	24.49	0.1581	1.399	21.12
01-07-97	00:45:09	660.00	25.69	0.1585	1.397	21.17
		720.00	26.83	0.1588	1.396	21.22
		780.00	27.93	0.1591	1.395	21.26
		840.00	28.98	0.1595	1.393	21.31
		900.00	30.00	0.1597	1.392	21.34
		960.00	30.98	0.1601	1.390	21.39
		1020.00	31.94	0.1604	1.389	21.43
		1080.00	32.86	0.1606	1.388	21.46

LOAD INCREMENT DATA

TEST NAME C1-U3C
TESTED BY: ACS

PAGE NO: 21
JO: 05996.01

PRESSURE INCREMENT FROM 8.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	08:03:59	0.00	0.00	0.1606	1.388	21.46
		0.10	0.32	0.1561	1.407	20.85
		0.25	0.50	0.1558	1.408	20.82
		0.50	0.71	0.1558	1.408	20.82
		1.00	1.00	0.1557	1.408	20.80
		1.57	1.25	0.1556	1.409	20.78
		2.00	1.41	0.1556	1.409	20.78
		2.25	1.50	0.1556	1.409	20.78
		3.07	1.75	0.1556	1.409	20.78
		4.00	2.00	0.1556	1.409	20.78
		6.25	2.50	0.1554	1.409	20.77
		8.00	2.83	0.1554	1.409	20.77
		12.25	3.50	0.1554	1.409	20.77
		15.00	3.87	0.1554	1.409	20.77
		22.00	4.69	0.1553	1.410	20.75
		30.00	5.48	0.1553	1.410	20.75
		36.00	6.00	0.1553	1.410	20.75
		49.00	7.00	0.1553	1.410	20.75
		60.00	7.75	0.1552	1.411	20.73

LOAD INCREMENT DATA

TEST NAME C1-U3C

TESTED BY: ACS

PAGE NO: 22

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	09:18:11	0.00	0.00	0.1552	1.411	20.73
		0.10	0.32	0.1503	1.430	20.09
		0.25	0.50	0.1496	1.433	19.99
		0.50	0.71	0.1492	1.435	19.94
		1.00	1.00	0.1490	1.436	19.90
		1.57	1.25	0.1487	1.437	19.87
		2.00	1.41	0.1487	1.437	19.87
		2.25	1.50	0.1487	1.437	19.87
		3.07	1.75	0.1484	1.438	19.83
		4.00	2.00	0.1484	1.438	19.83
		6.25	2.50	0.1482	1.439	19.80
		8.00	2.83	0.1482	1.439	19.80
		12.25	3.50	0.1481	1.439	19.78
		15.00	3.87	0.1481	1.439	19.78
		22.00	4.69	0.1478	1.440	19.75
		30.00	5.48	0.1478	1.440	19.75
		36.00	6.00	0.1477	1.441	19.73
		49.00	7.00	0.1477	1.441	19.73
		60.00	7.75	0.1476	1.441	19.72
		120.00	10.95	0.1473	1.442	19.68

LOAD INCREMENT DATA

TEST NAME C1-U3C

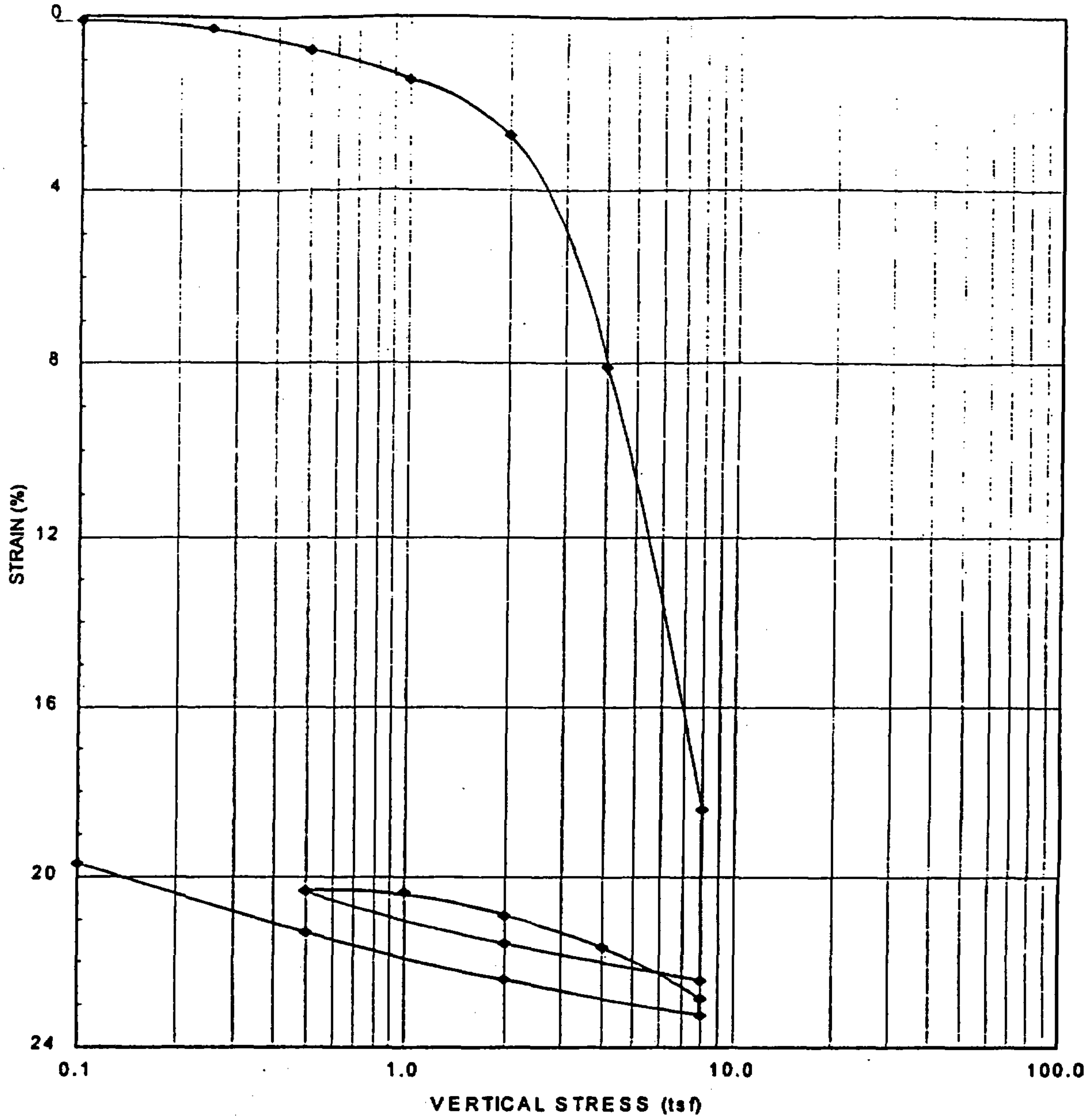
TESTED BY: ACS

PAGE NO: 23

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	12:11:24	0.00	0.00	0.1473	1.442	19.68
		0.10	0.32	0.1439	1.456	19.22
		0.25	0.50	0.1429	1.461	19.09
		0.50	0.71	0.1420	1.464	18.97
		1.00	1.00	0.1412	1.467	18.87
		1.57	1.25	0.1407	1.469	18.80
		2.00	1.41	0.1405	1.470	18.77
		2.25	1.50	0.1403	1.471	18.75
		3.07	1.75	0.1401	1.472	18.72
		4.00	2.00	0.1399	1.472	18.70
		6.25	2.50	0.1396	1.474	18.65
		8.00	2.83	0.1393	1.475	18.61
		12.25	3.50	0.1391	1.476	18.58
		15.00	3.87	0.1388	1.477	18.55
		22.00	4.69	0.1387	1.478	18.53
		30.00	5.48	0.1384	1.479	18.50
		36.00	6.00	0.1383	1.479	18.48
		49.00	7.00	0.1382	1.480	18.46
		60.00	7.75	0.1380	1.480	18.44
		120.00	10.95	0.1375	1.482	18.38
		180.00	13.42	0.1374	1.483	18.36
		240.00	15.49	0.1372	1.484	18.33



SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/12/96
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	11.4 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

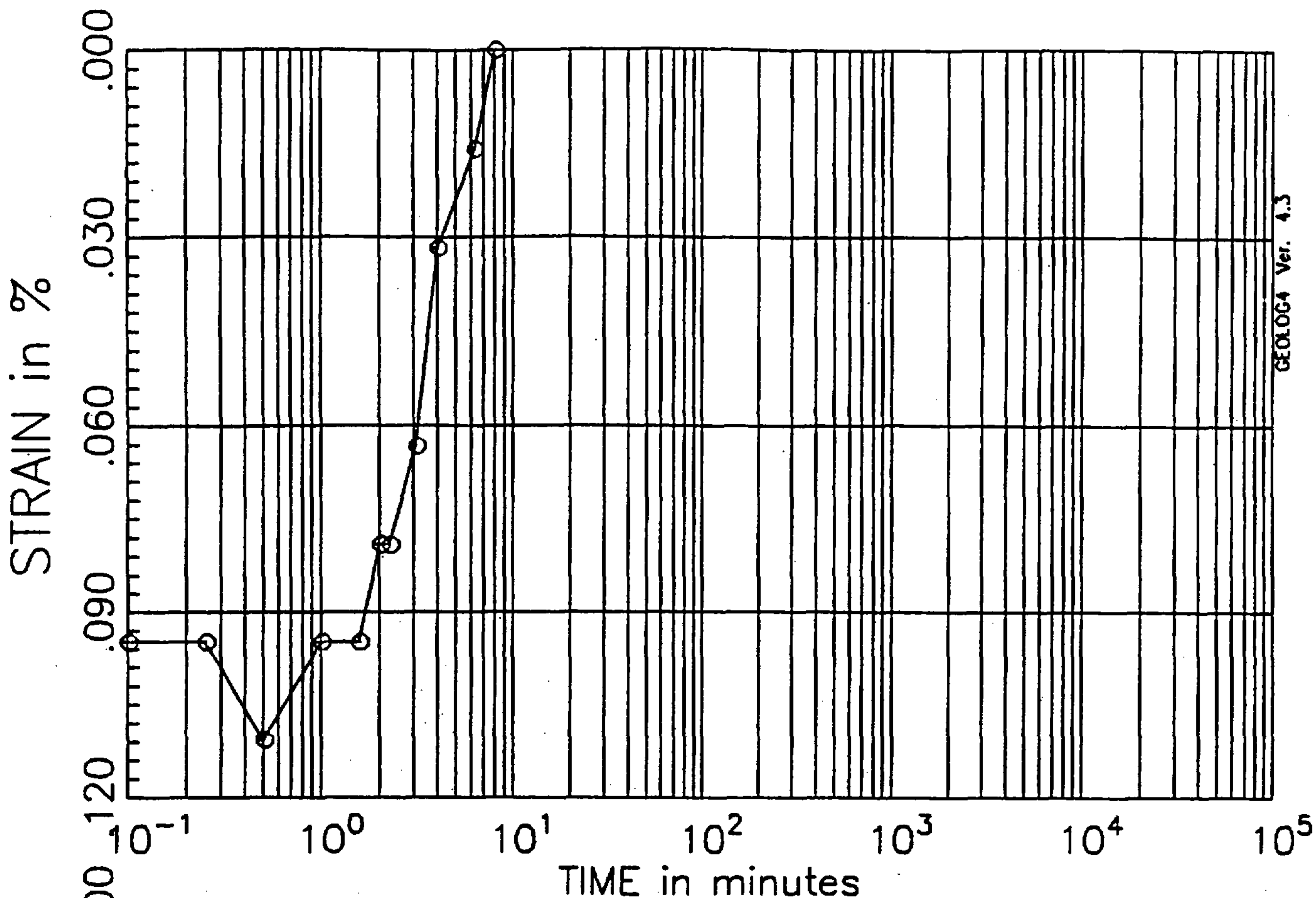
SPECIMEN INFORMATION:

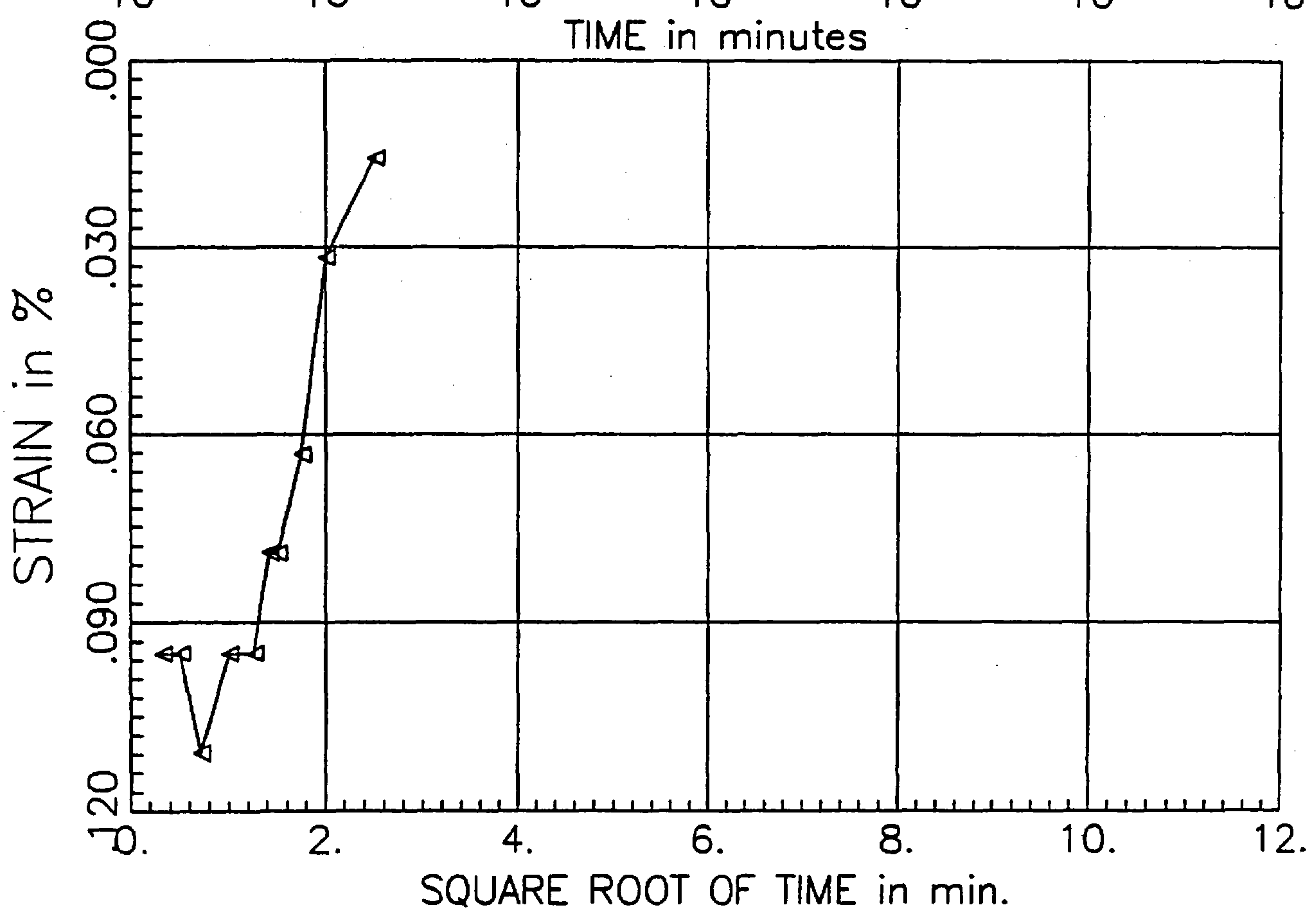
	INITIAL	FINAL	
WATER CONTENT:	46.7 %	62.4 %	SPECIFIC GRAVITY:
DRY UNIT WEIGHT:	51.7 pcf	64.1 pcf	2.72
VOID RATIO:	2.285	1.649	
SATURATION:	55.6 %	103.0 %	

NOTE: Sample was inundated when the applied pressure was 0.5 tsf.

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

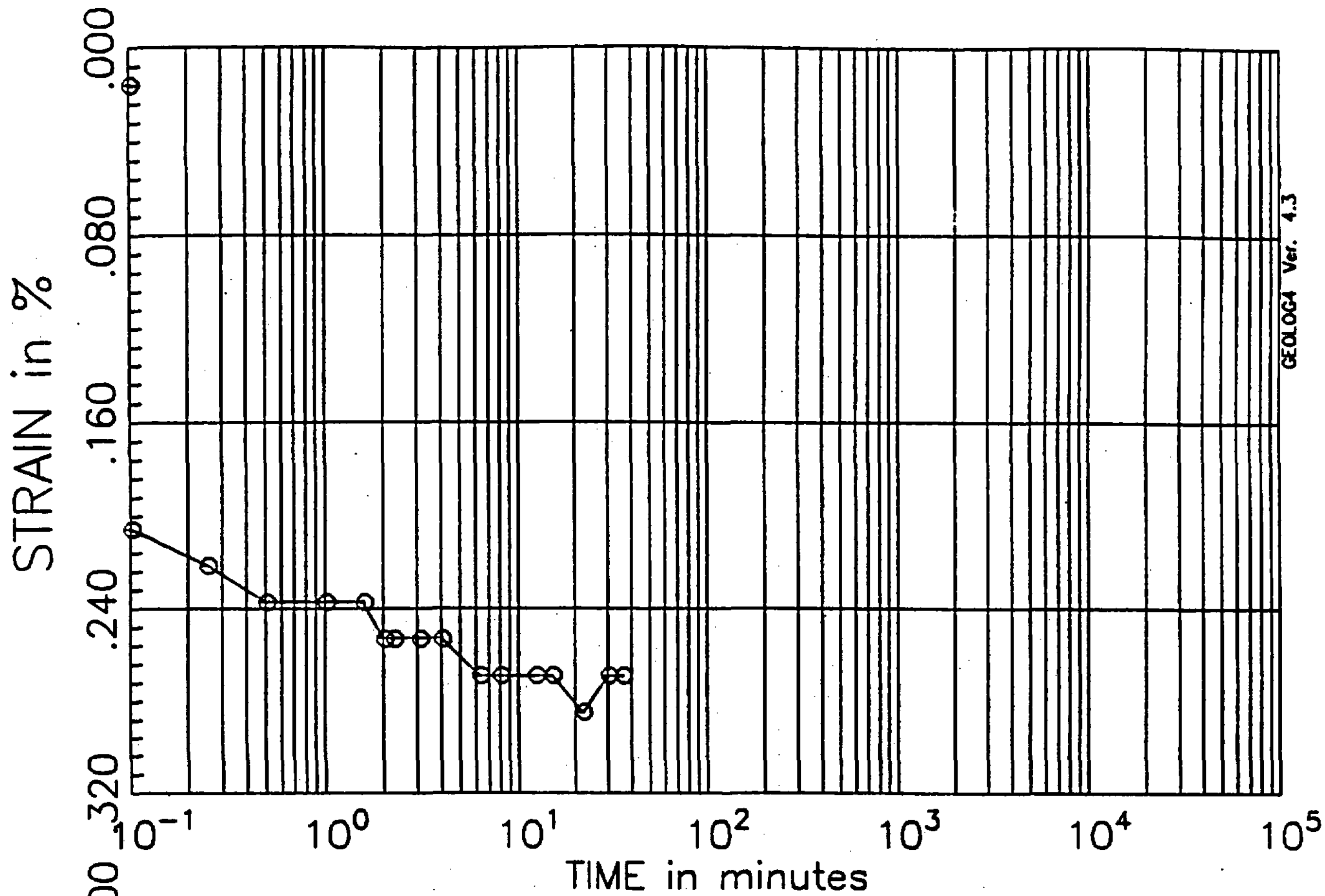
STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING C-1, SAMPLE U-3D	JO 05996.01 January 1997

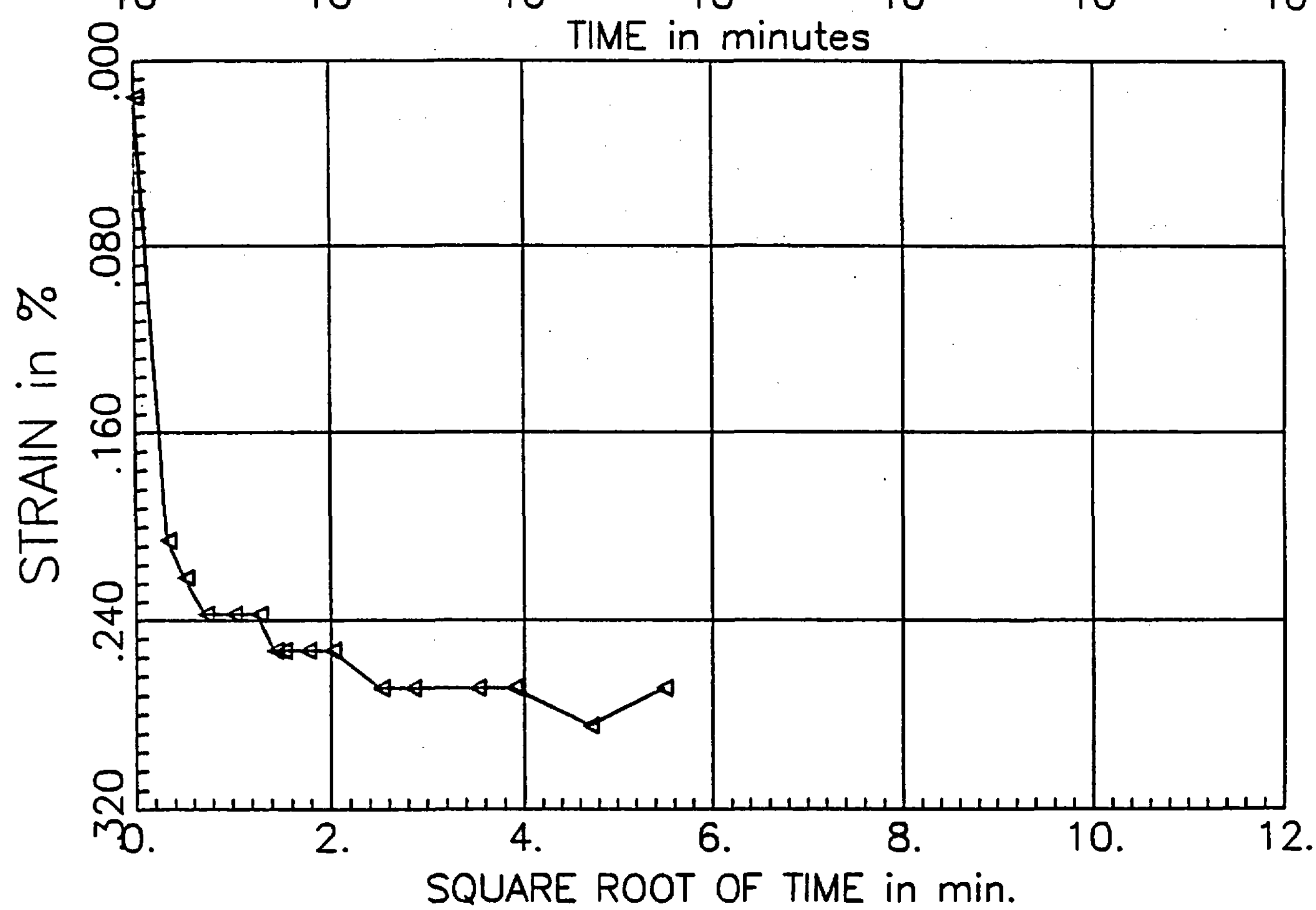




PRESSURE INCREMENT
from 0.00 tsf to 0.10 tsf

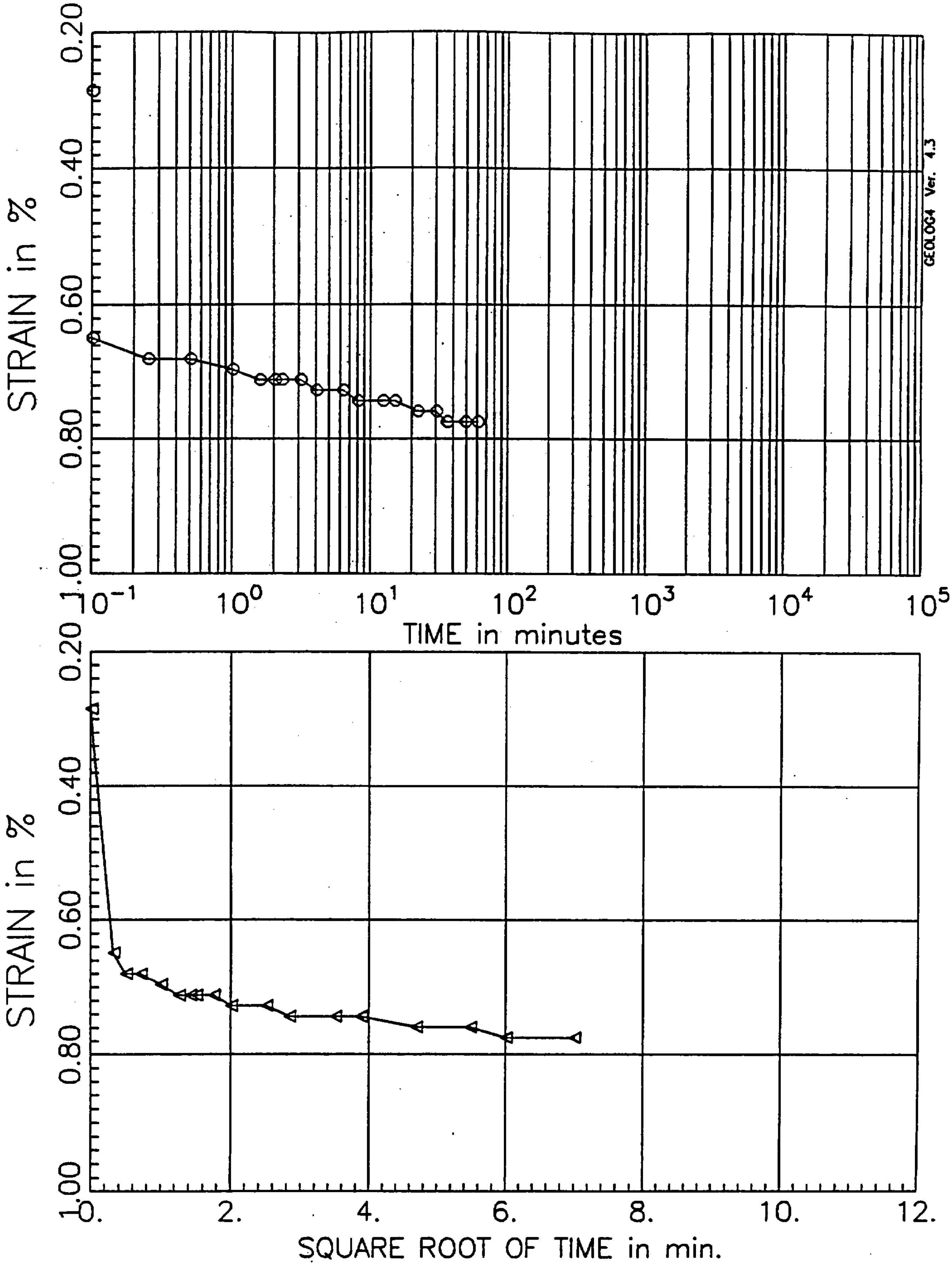
Test No: 2
Testname: C1–U3D





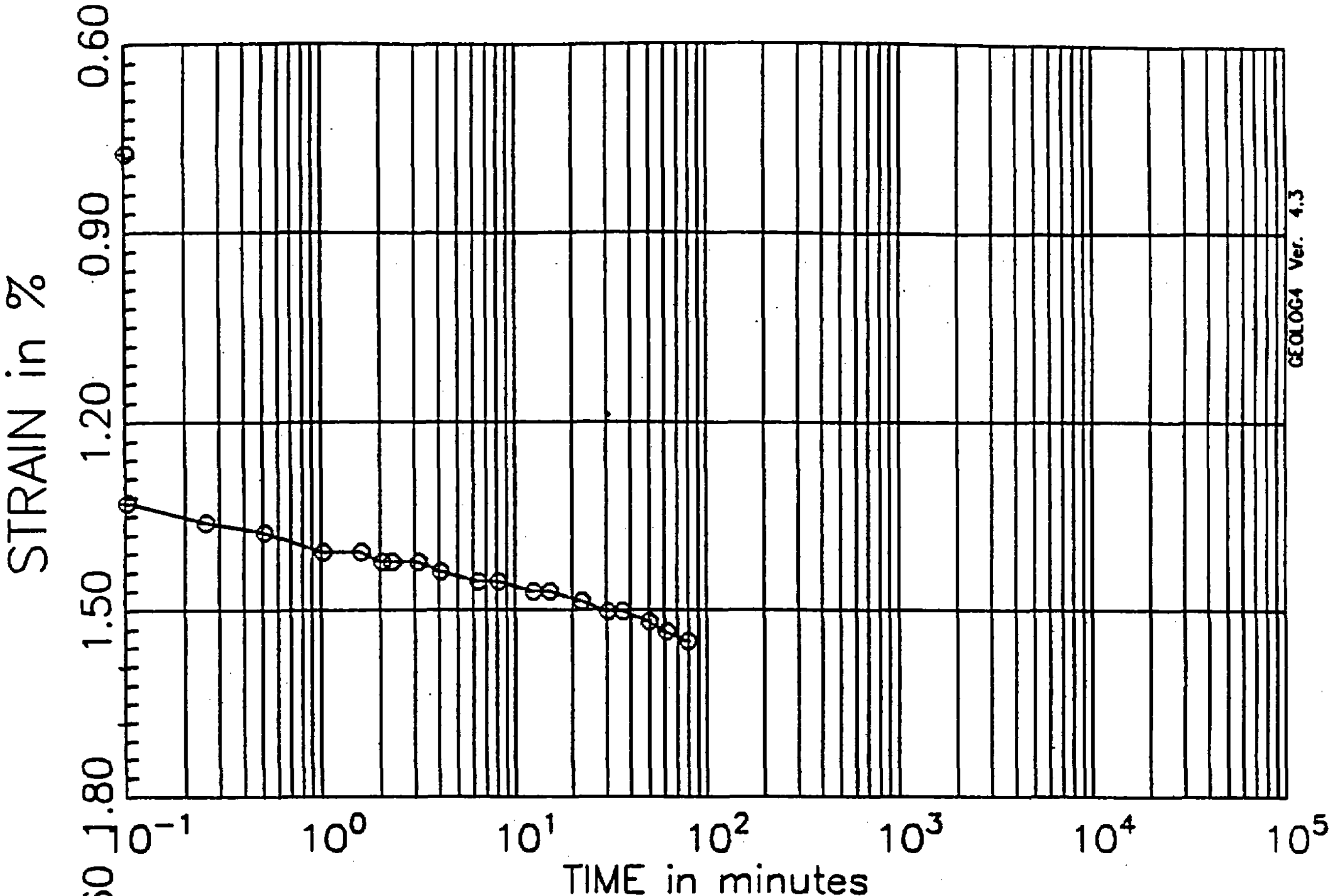
PRESSURE INCREMENT
from 0.10 tsf to 0.25 tsf

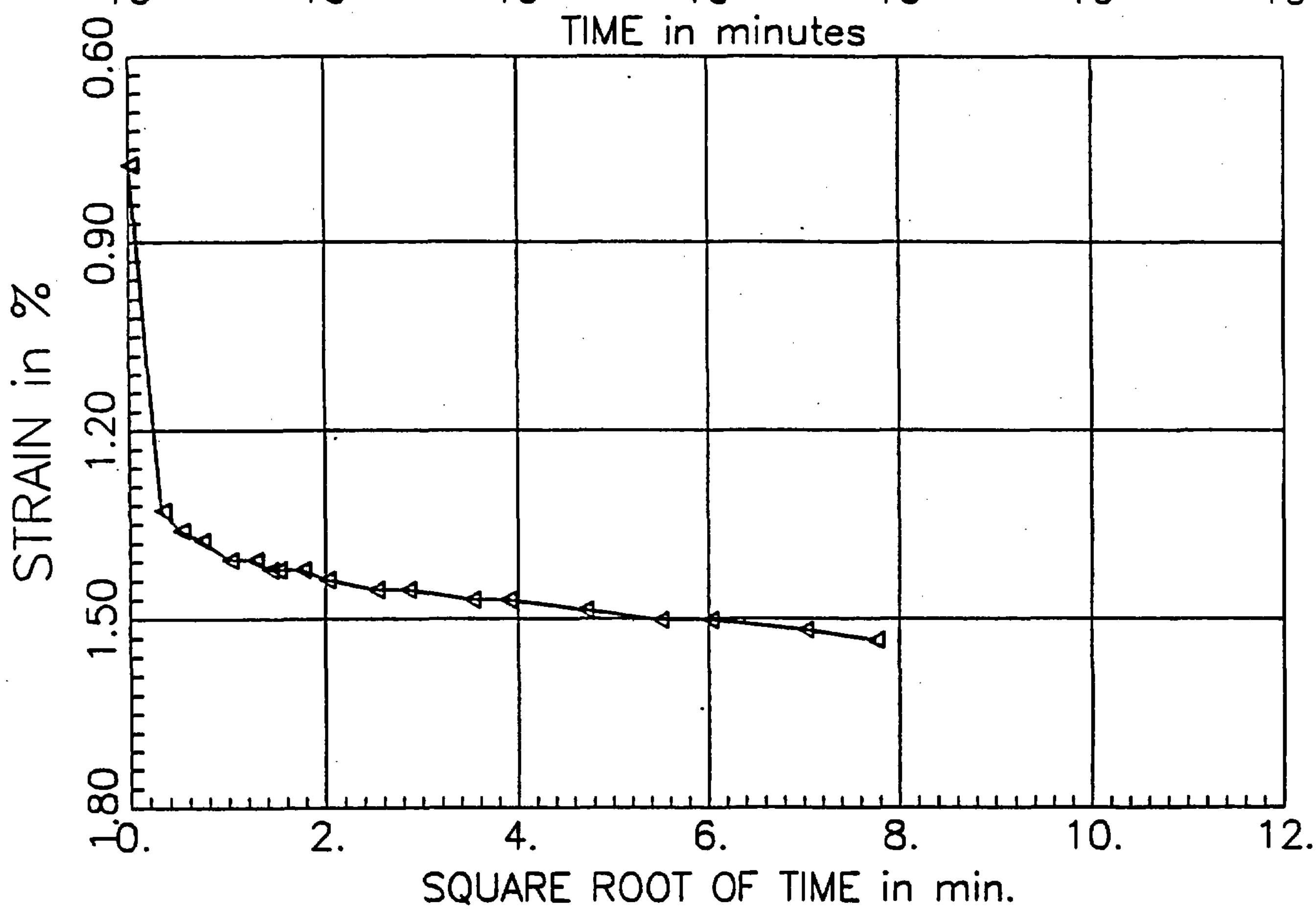
Test No: 2
Testname: C1-U3D



PRESSURE INCREMENT
from 0.25 tsf to 0.50 tsf

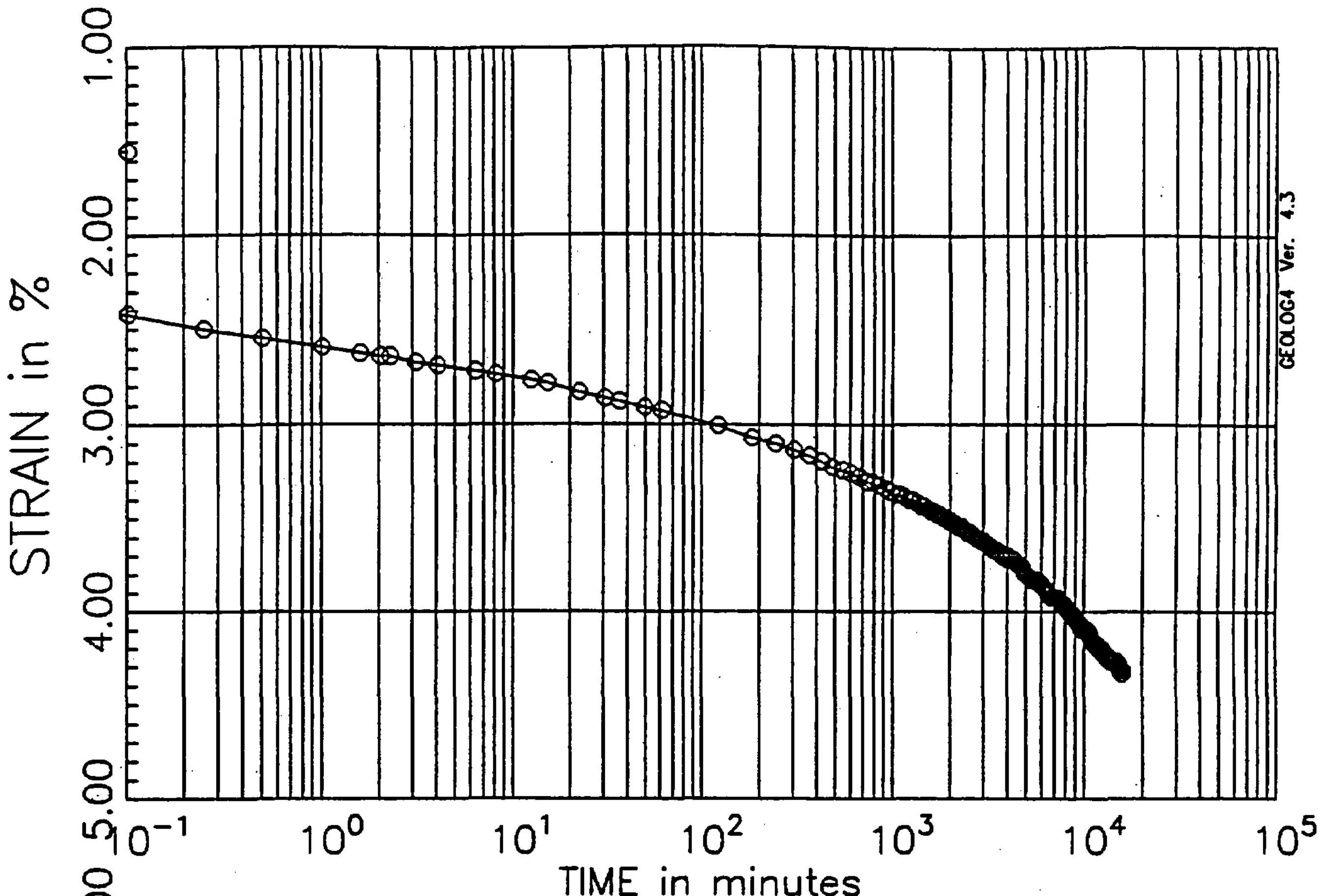
Test No: 2
Testname: C1–U3D

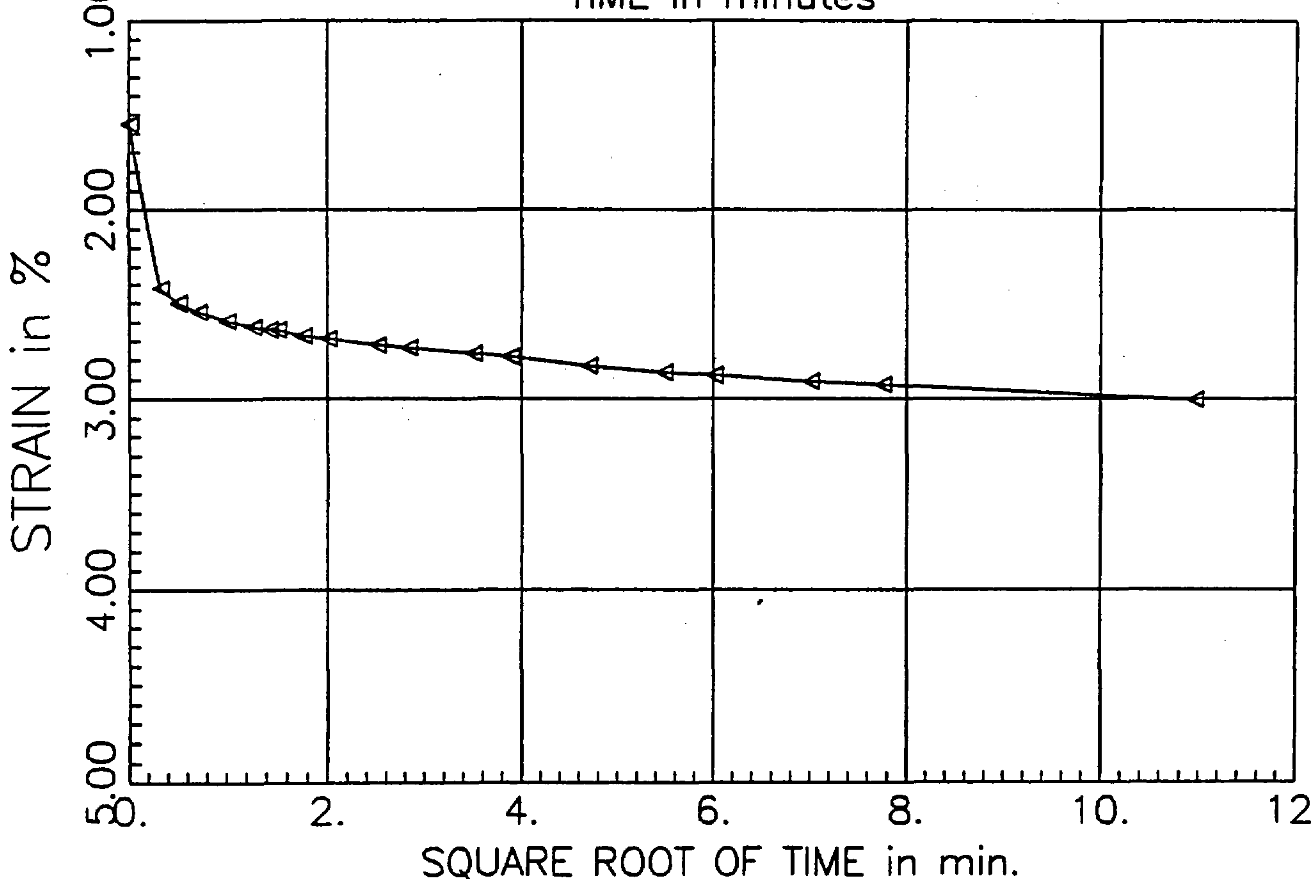




PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

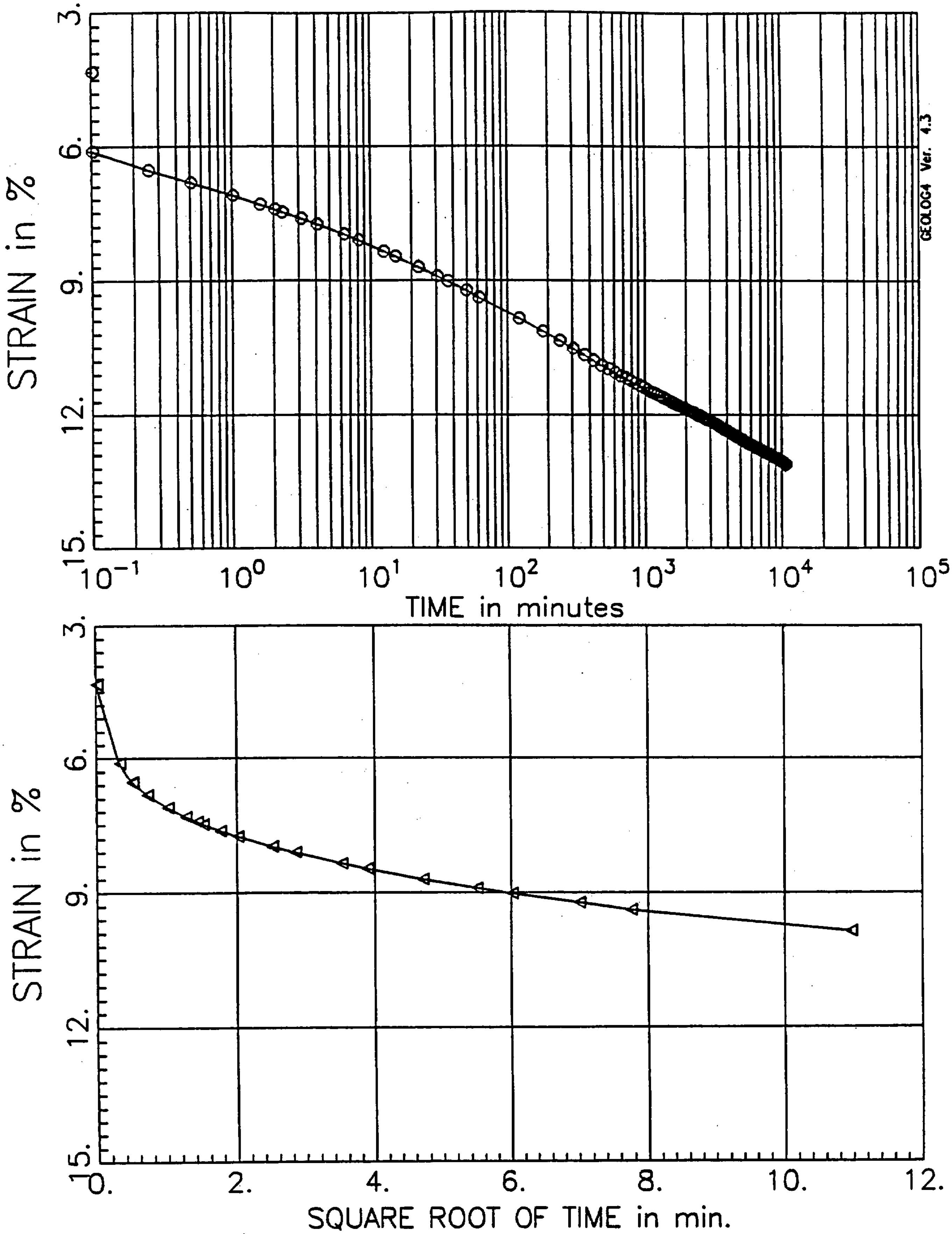
Test No: 2
Testname: C1-U3D





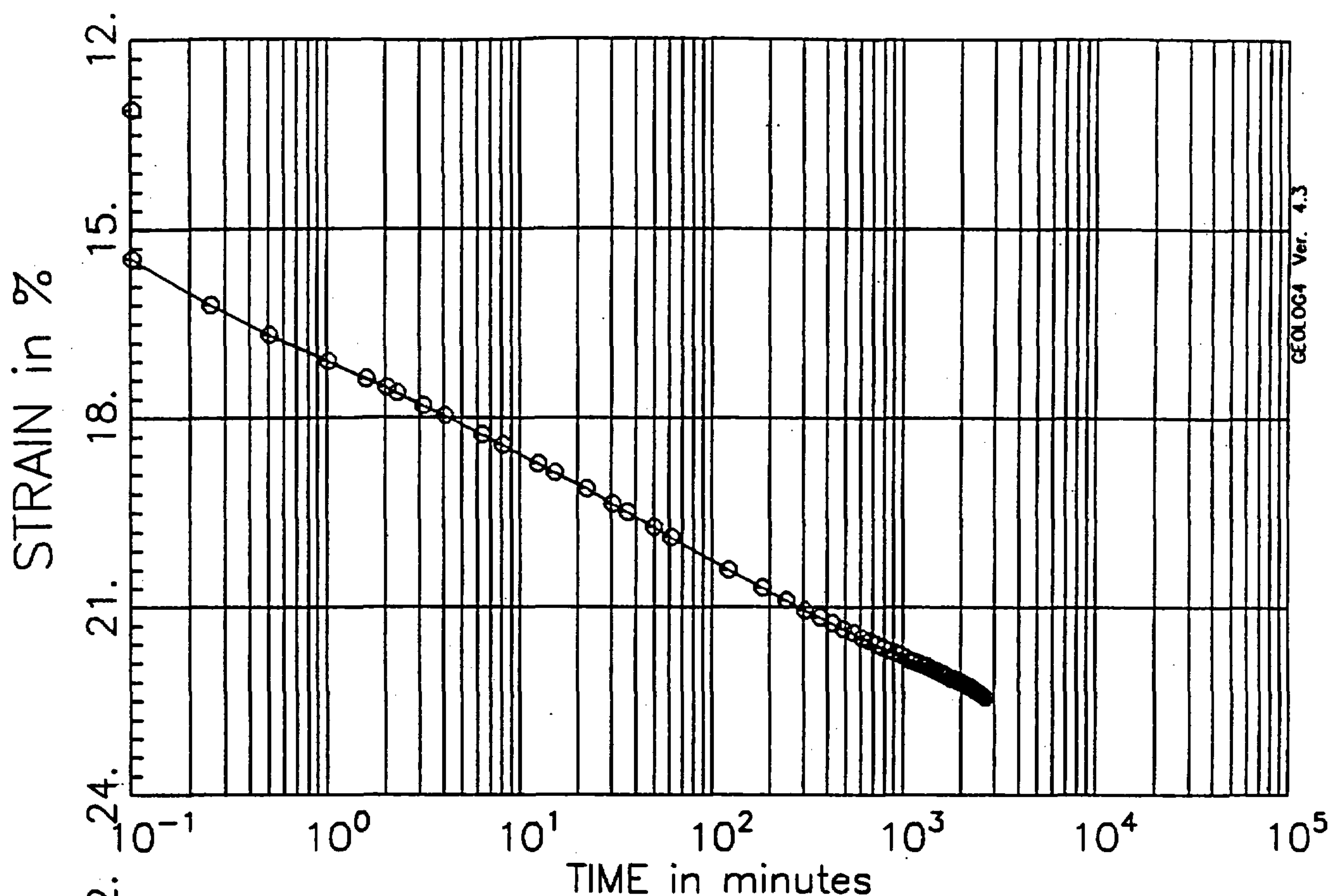
PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

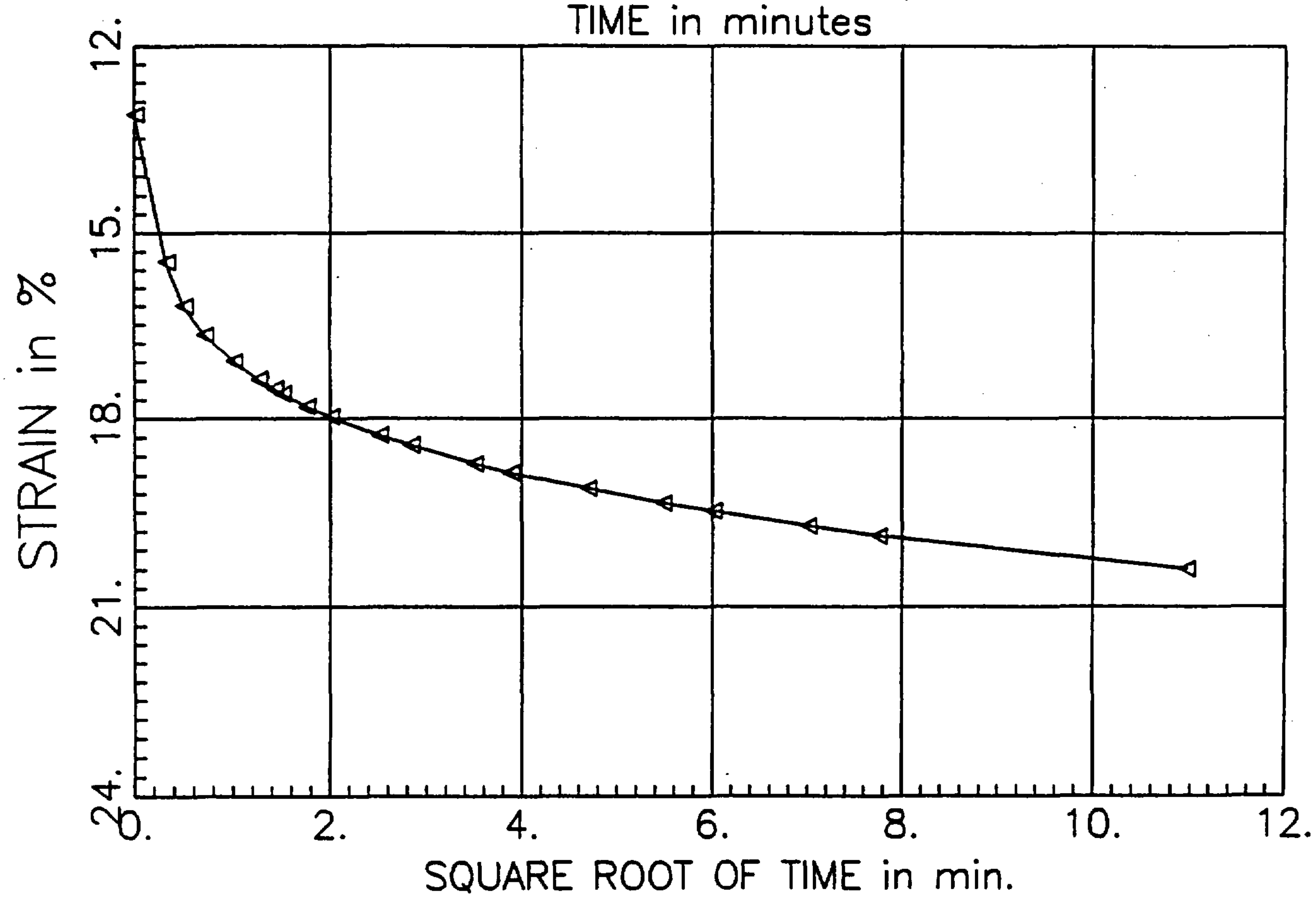
Test No: 2
Testname: C1–U3D



PRESSURE INCREMENT
from 2.00 tsf to 4.00 tsf

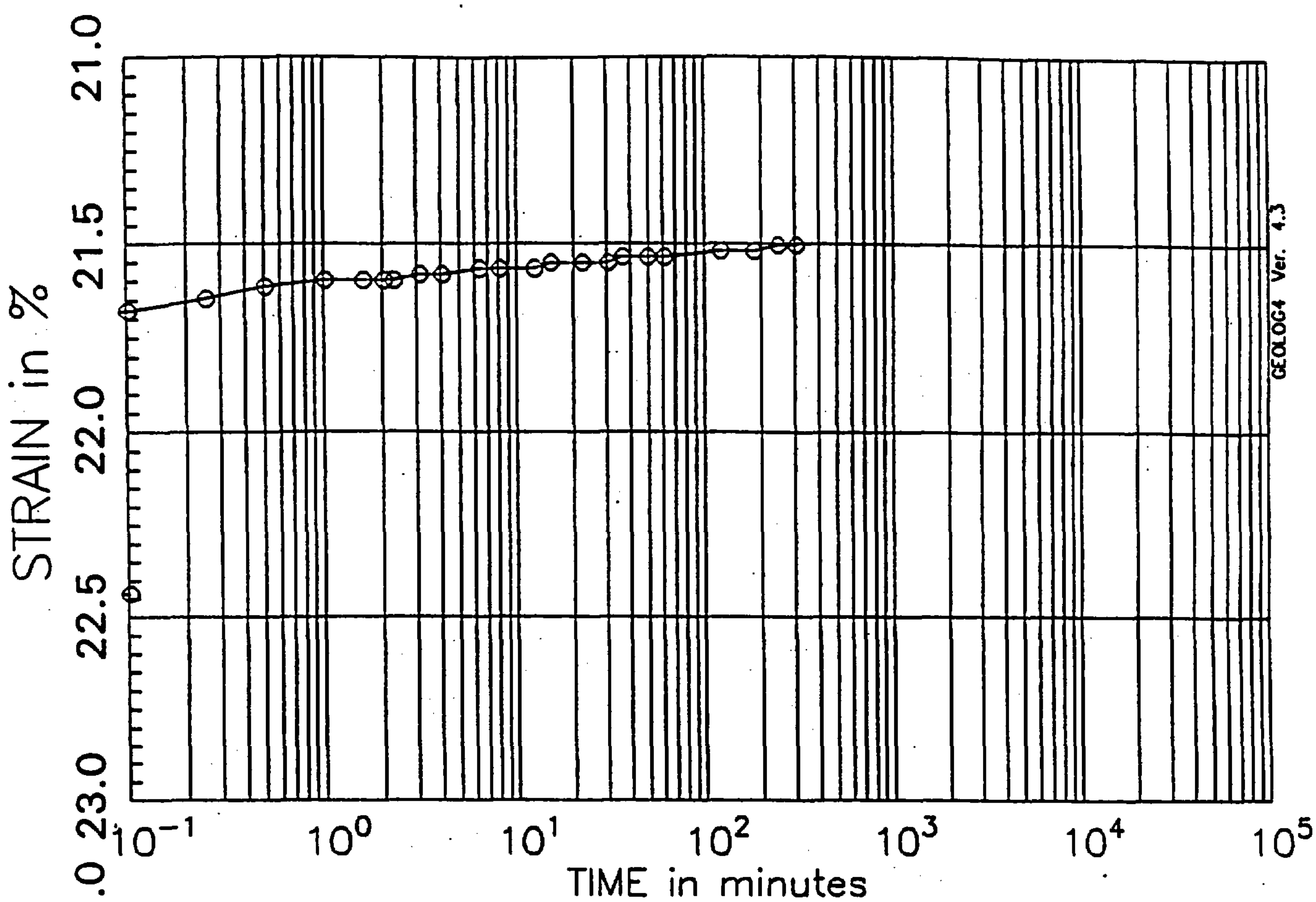
Test No: 2
Testname: C1-U3D

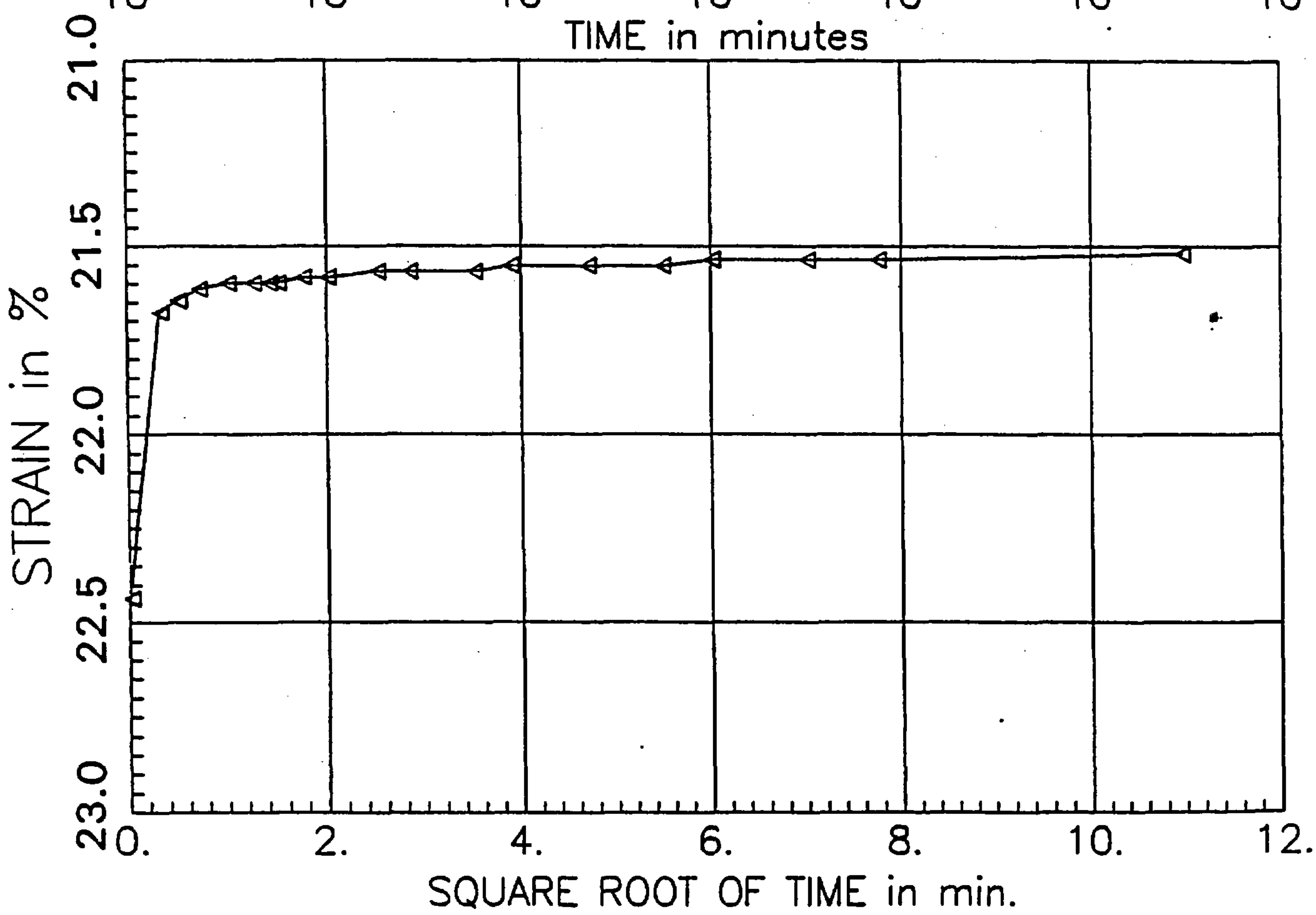




PRESSURE INCREMENT
from 4.00 tsf to 8.00 tsf

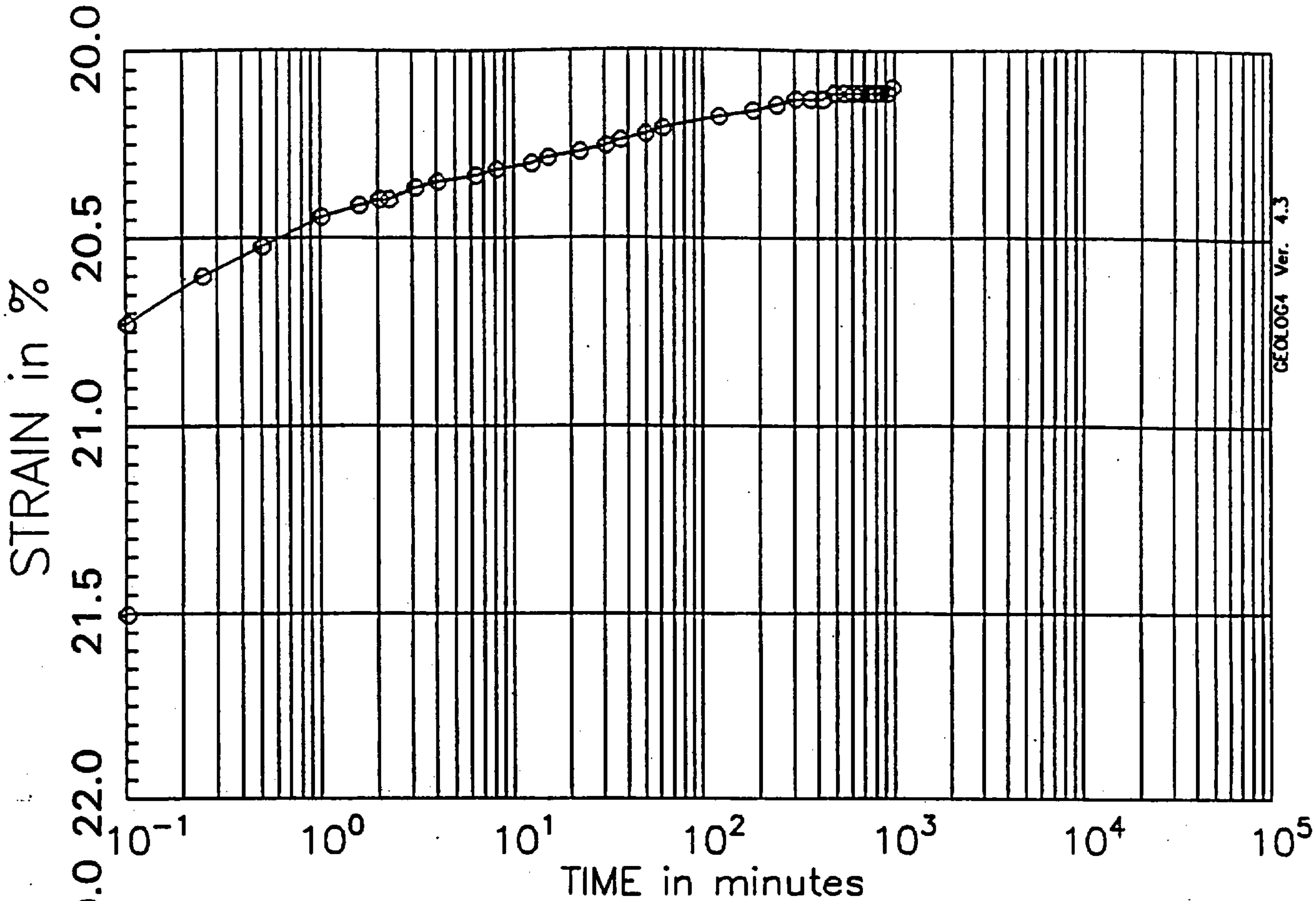
Test No: 2
Testname: C1–U3D

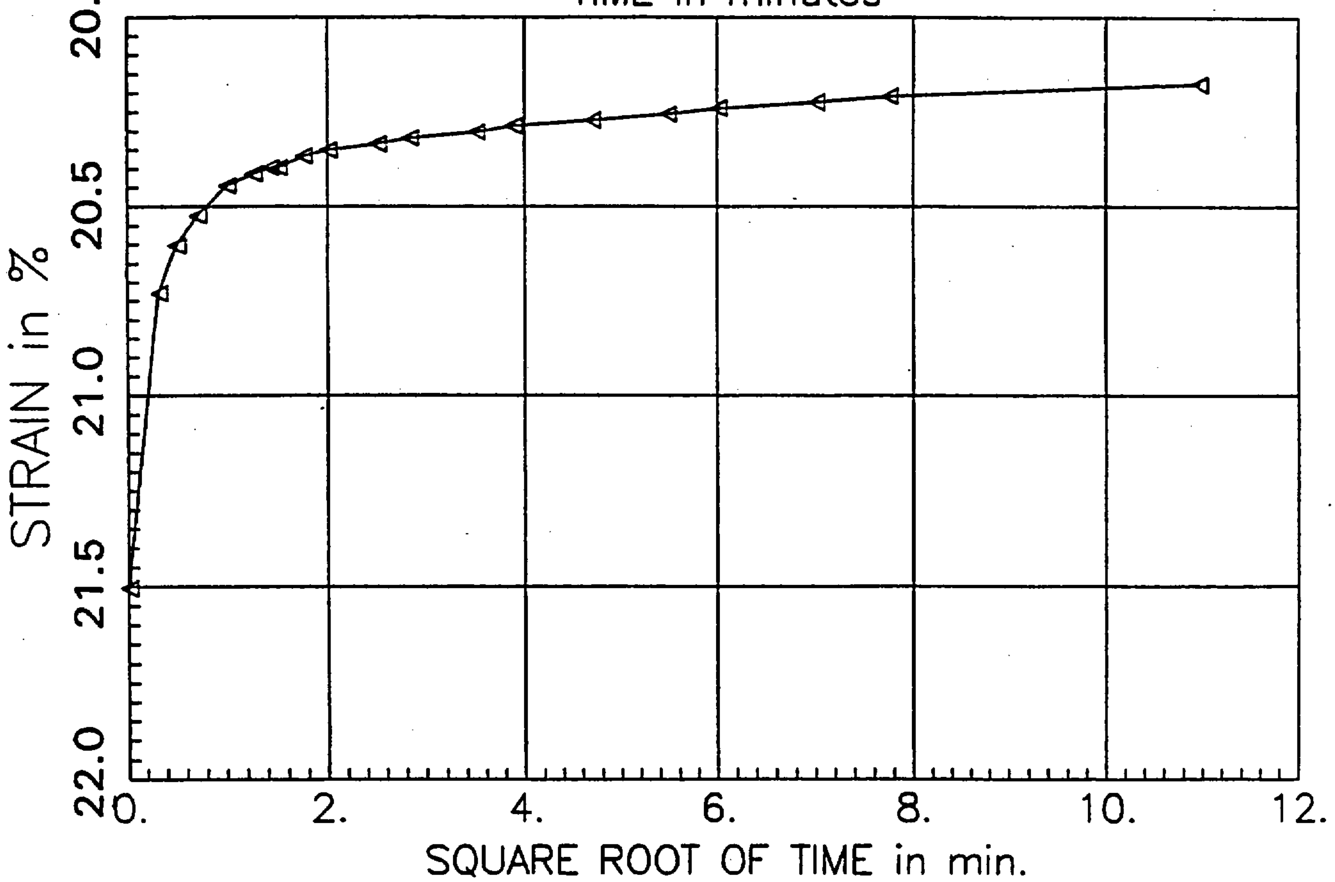




PRESSURE INCREMENT
from 8.00 tsf to 2.00 tsf

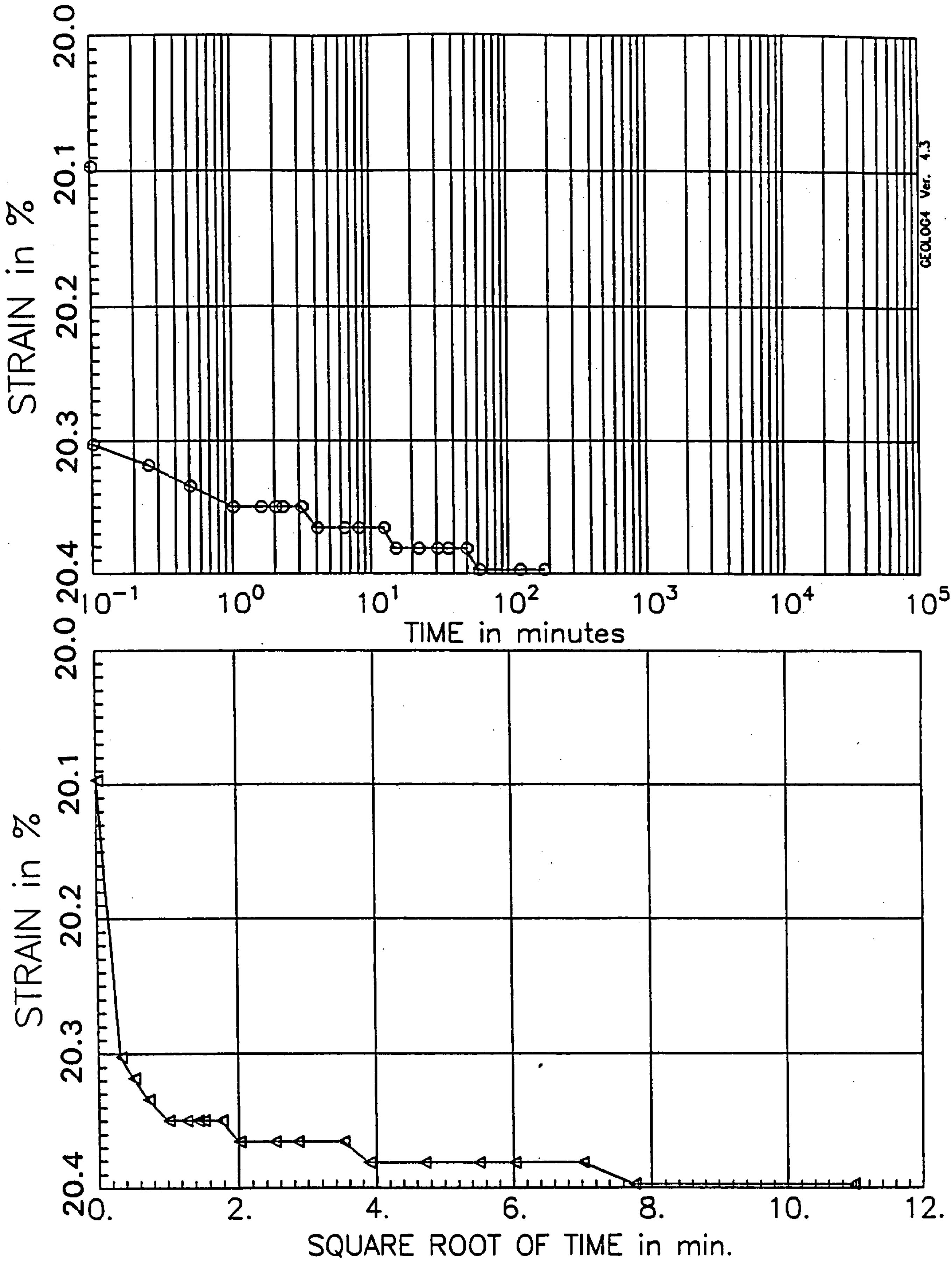
Test No: 2
Testname: C1-U3D





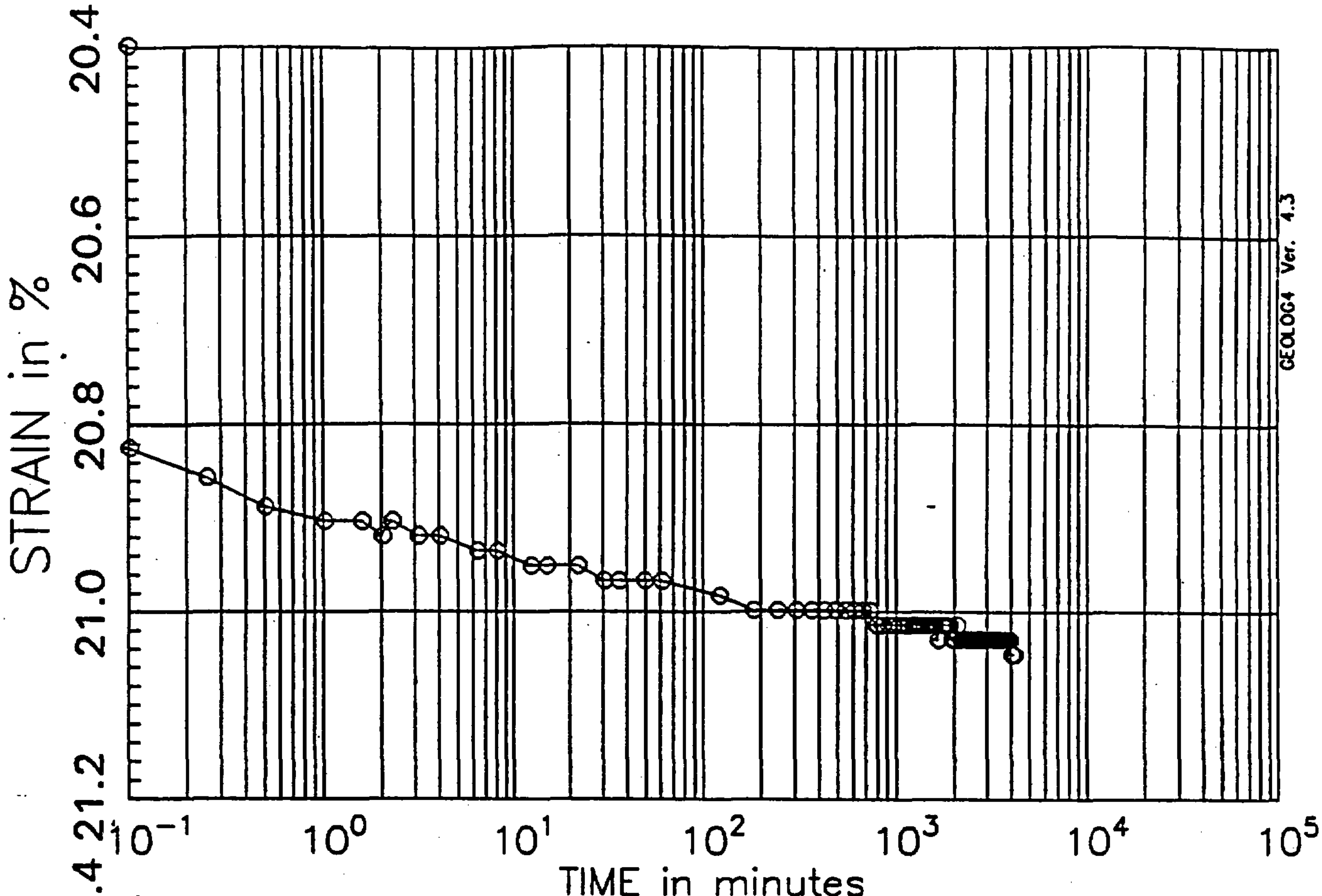
PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

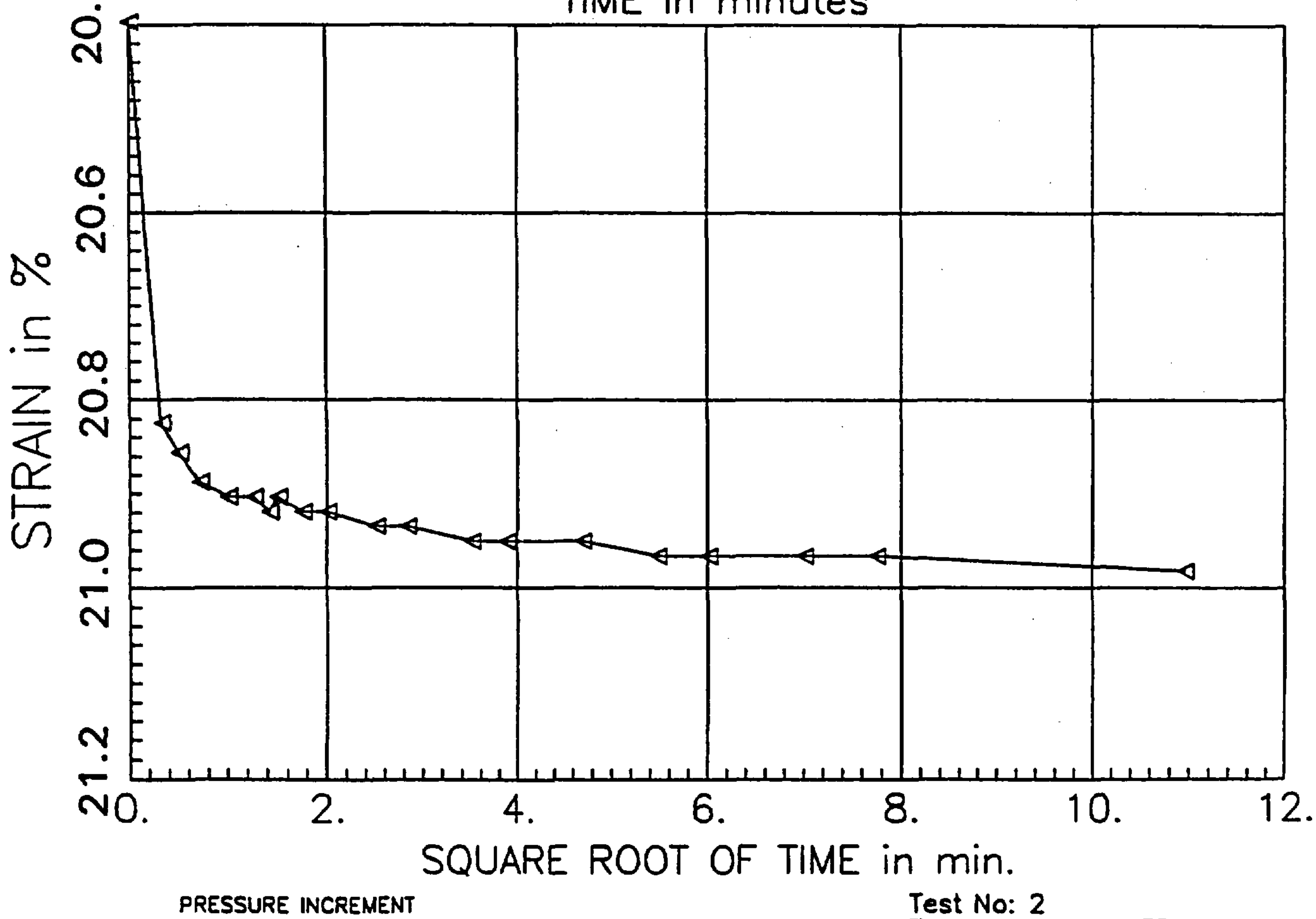
Test No: 2
Testname: C1-U3D



PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

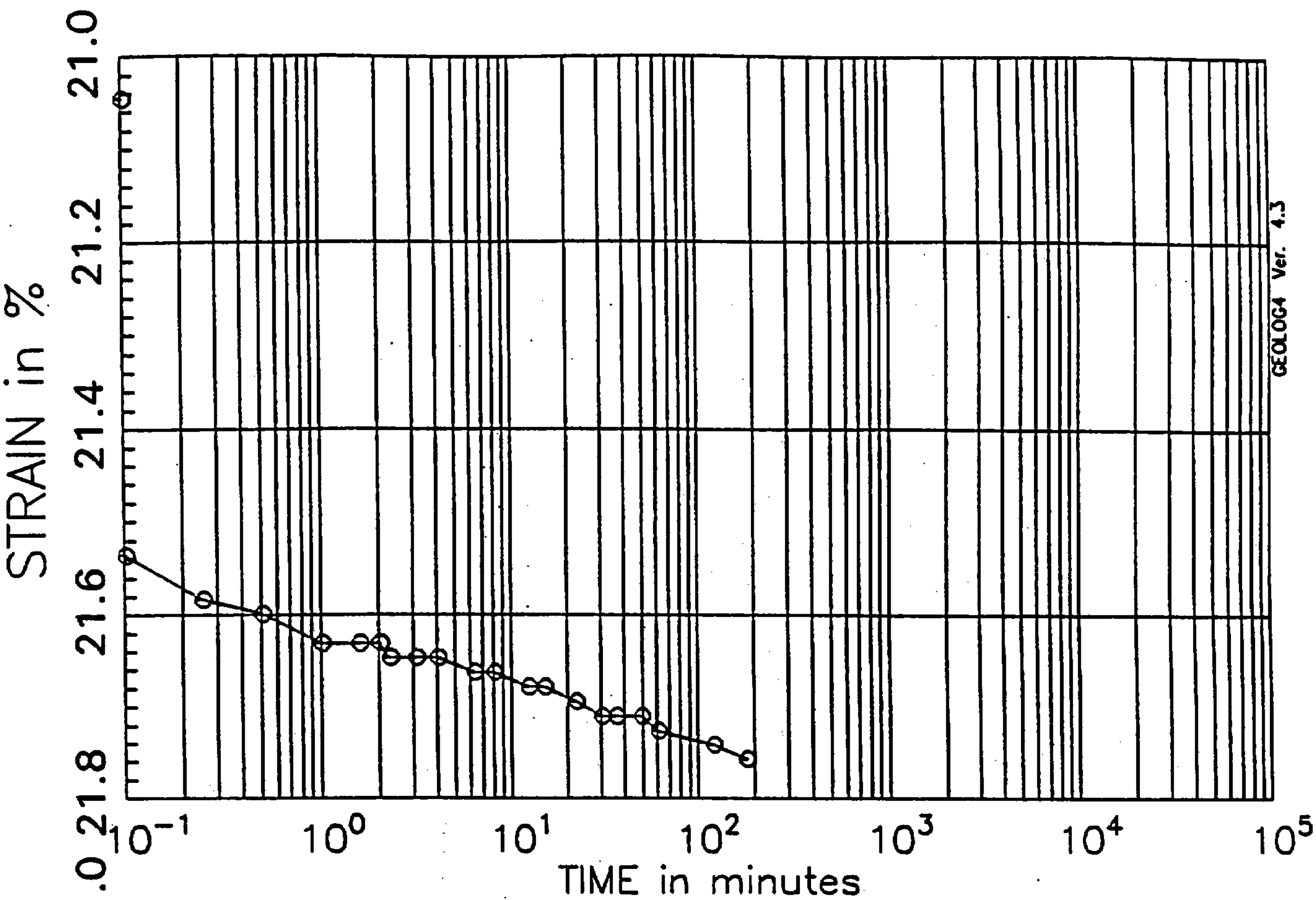
Test No: 2
Testname: C1–U3D

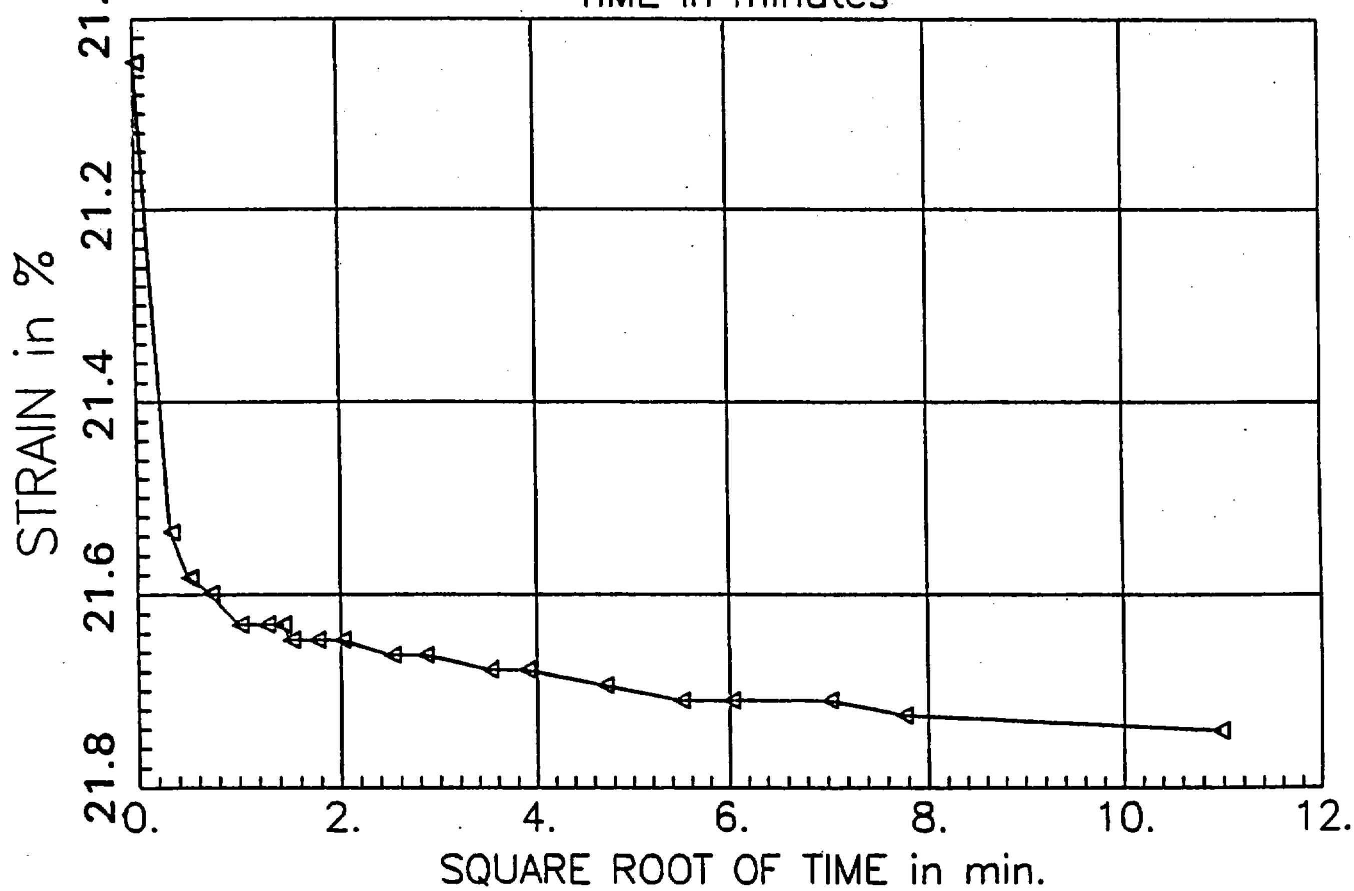




PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

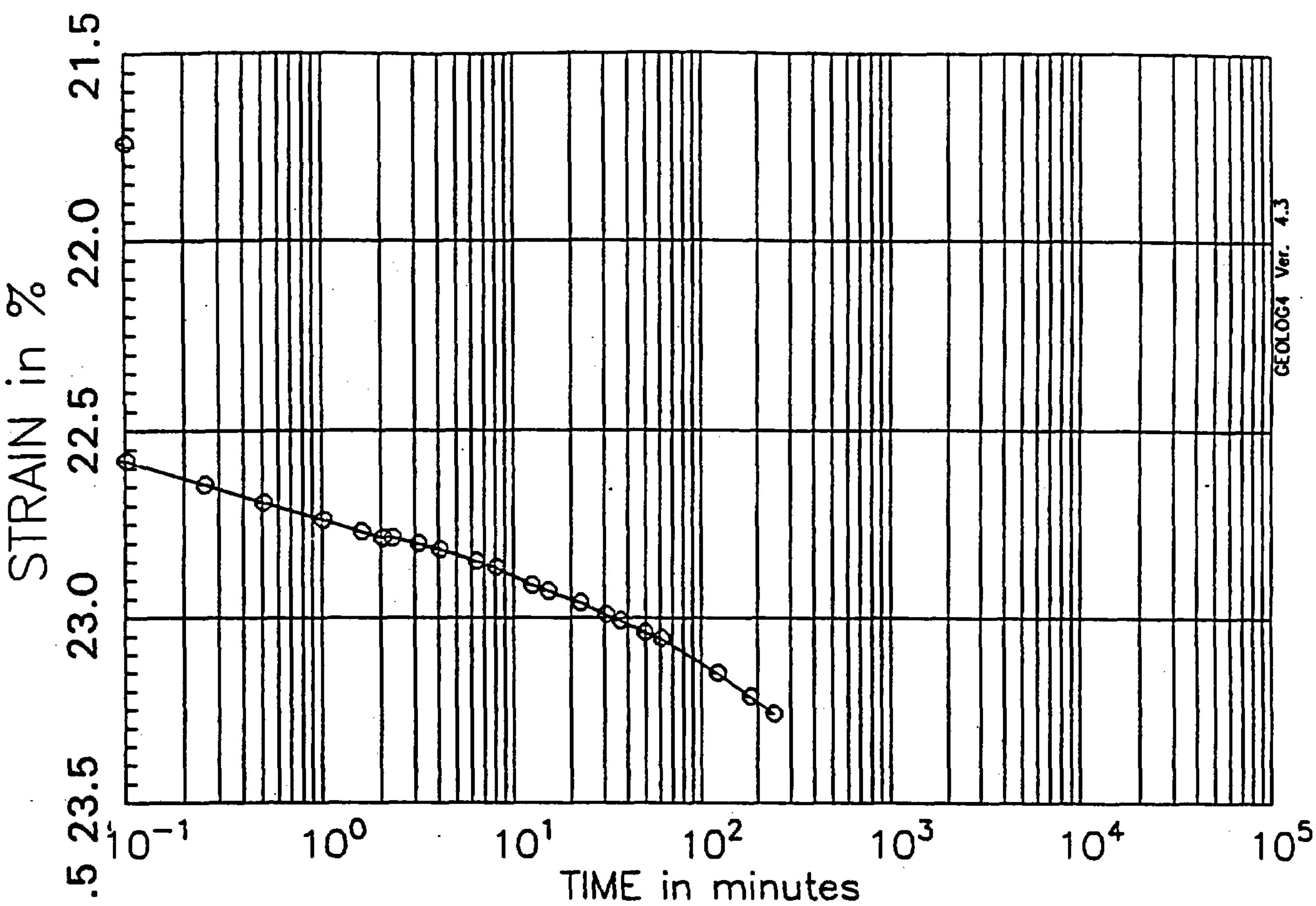
Test No: 2
Testname: C1-U3D

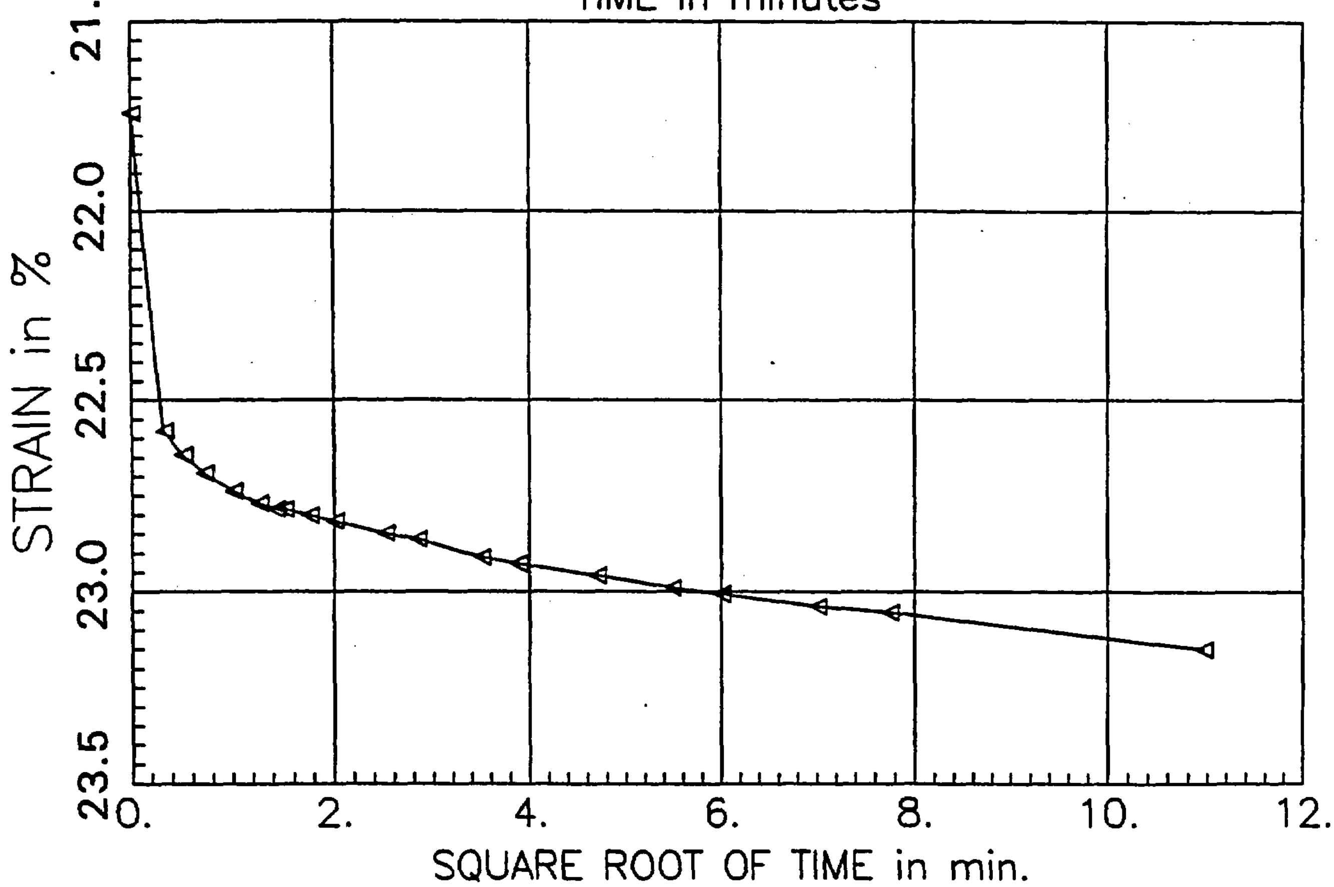




PRESSURE INCREMENT
from 2.00 tsf to 4.00 tsf

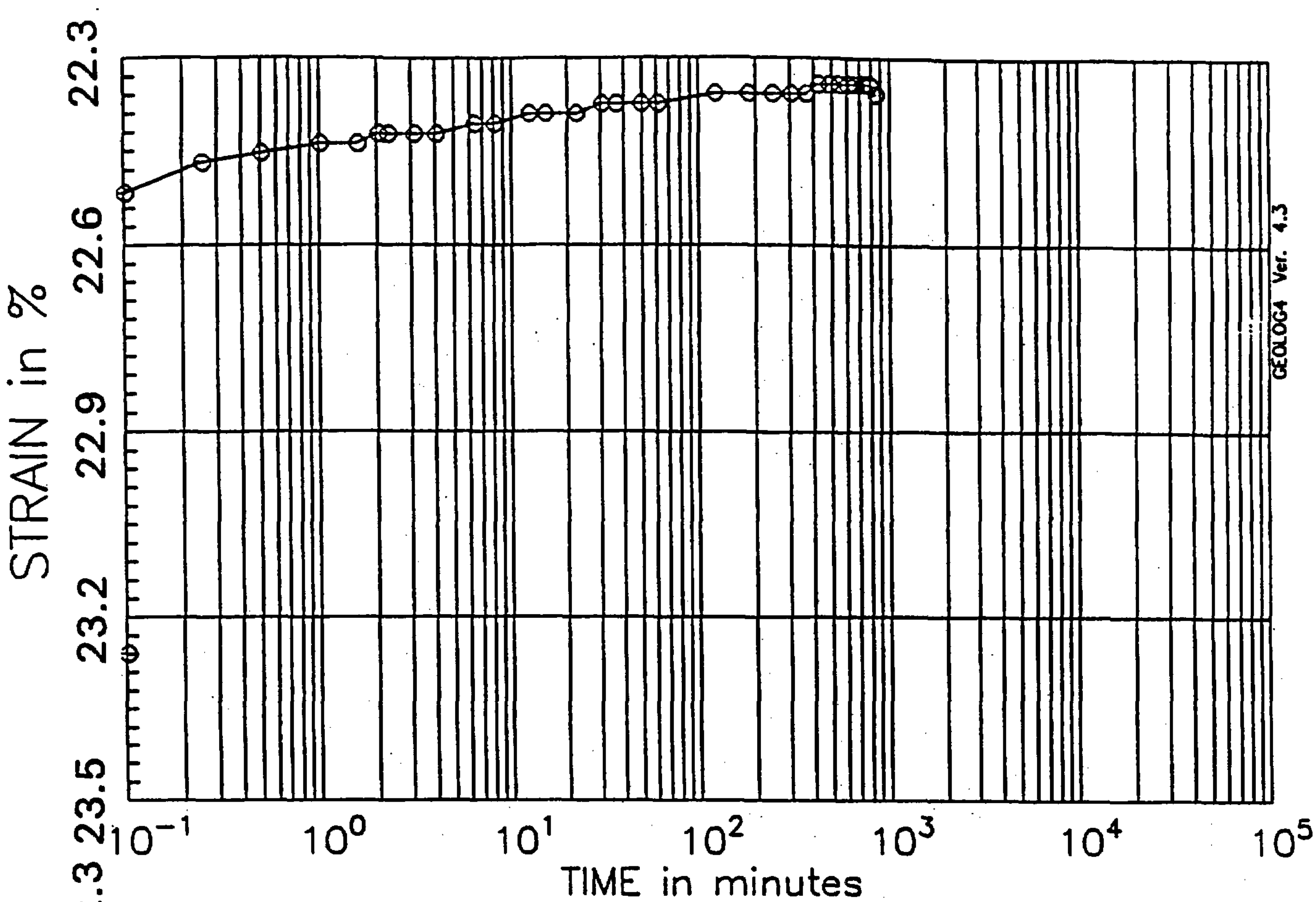
Test No: 2
Testname: C1-U3D

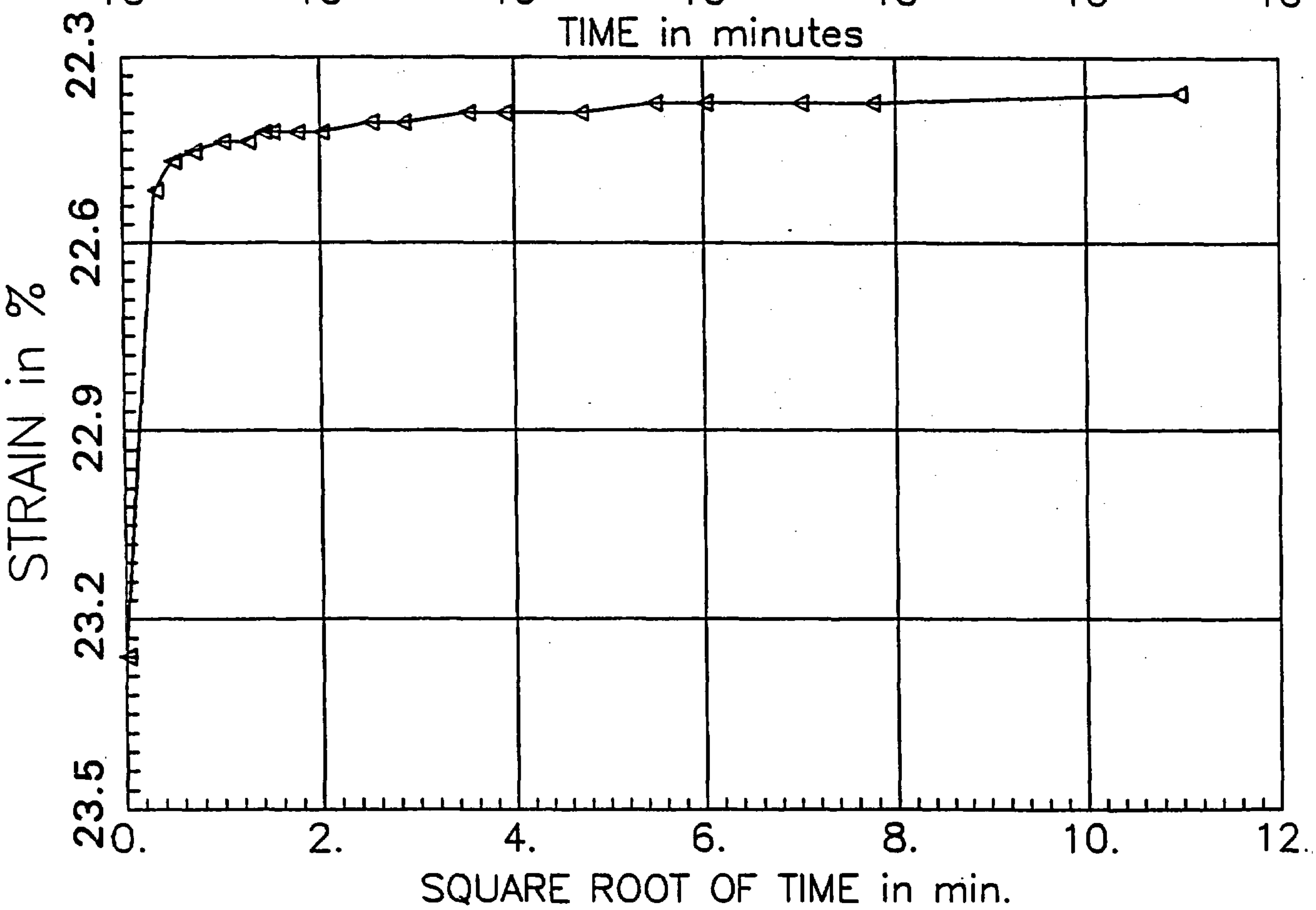




PRESSURE INCREMENT
from 4.00 tsf to 8.00 tsf

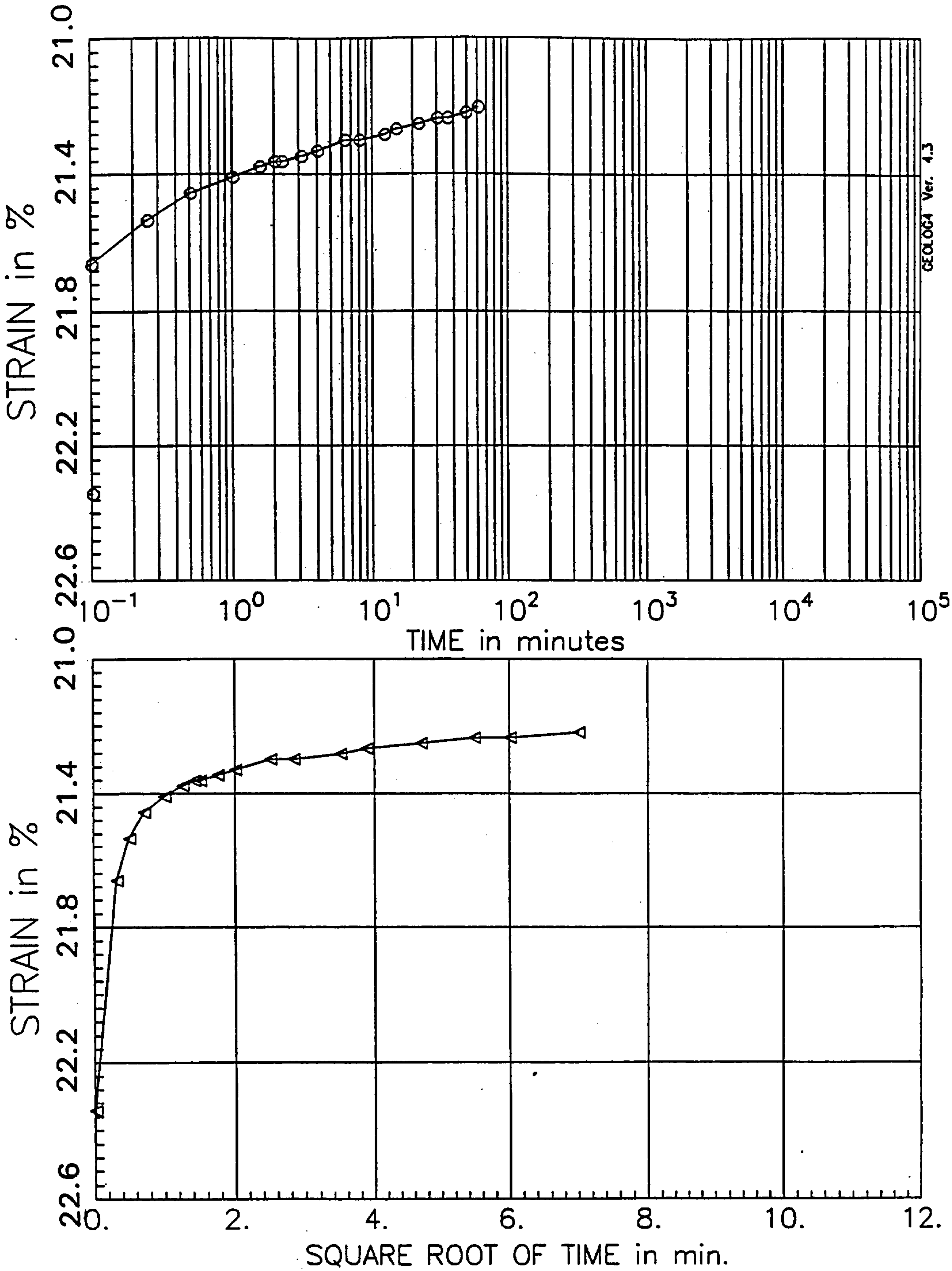
Test No: 2
Testname: C1–U3D





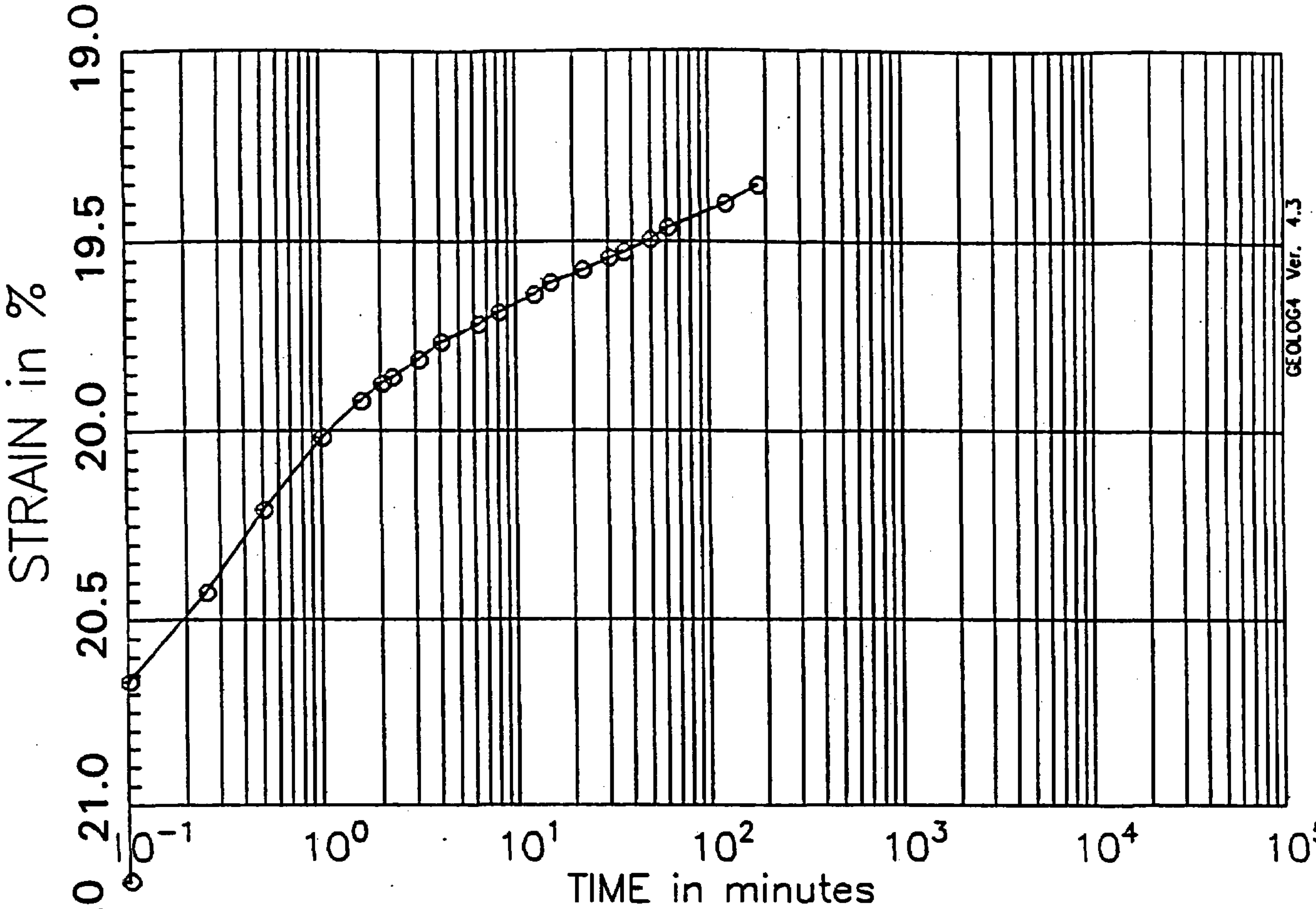
PRESSURE INCREMENT
from 8.00 tsf to 2.00 tsf

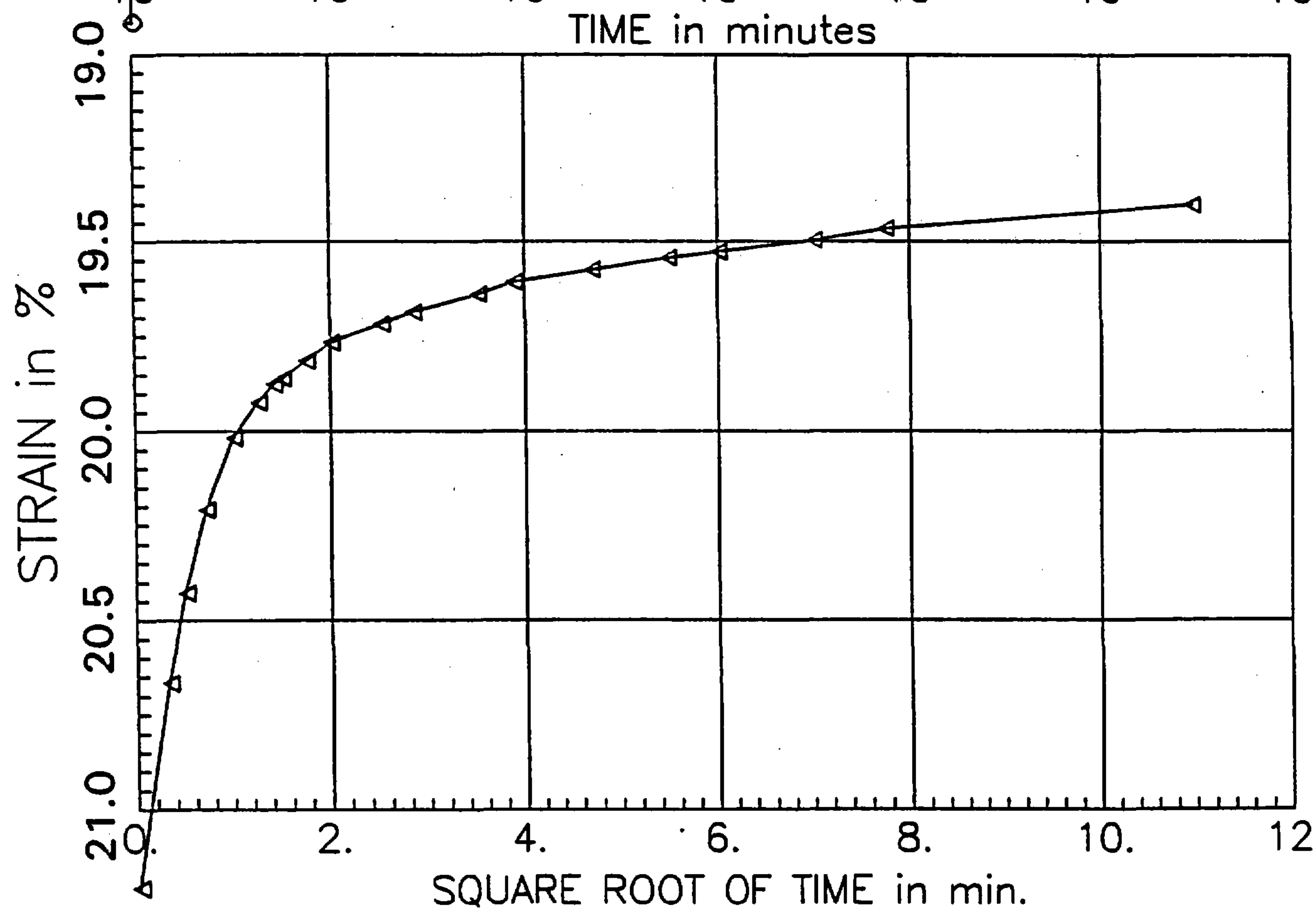
Test No: 2
Testname: C1-U3D



PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

Test No: 2
Testname: C1–U3D





PRESSURE INCREMENT
from 0.50 tsf to 0.10 tsf

Test No: 2
Testname: C1–U3D

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/12/96
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	11.4 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	46.7 %	62.4 %
DRY UNIT WEIGHT:	51.7 pcf	64.1 pcf
VOID RATIO:	2.285	1.649
SATURATION:	55.6 %	103 %
HEIGHT:	1.893 cm	1.527 cm
AREA:	31.63 sq cm	
SP. GRAVITY :	2.72	

TEST DATA:

APPLIED PRESSURE tsf	STRAIN %
0.10	0.11
0.25	0.27
0.50	0.74
1.00	1.45
2.00	2.74
4.00	8.10
8.00	18.42
8.00	22.44
2.00	21.57
0.50	20.32
1.00	20.37
2.00	20.93
4.00	21.66
8.00	22.86
8.00	23.26
2.00	22.41
0.50	21.30
0.10	19.69

LOAD INCREMENT DATA

TEST NAME: C1-U3D

TESTED BY: ACS

PAGE NO: 1

JO: 05996.01

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	09:31:06	0.10	0.32	0.0007	2.282	0.09
		0.25	0.50	0.0007	2.282	0.09
		0.50	0.71	0.0008	2.281	0.11
		1.00	1.00	0.0007	2.282	0.09
		1.57	1.25	0.0007	2.282	0.09
		2.00	1.41	0.0006	2.282	0.08
		2.25	1.50	0.0006	2.282	0.08
		3.07	1.75	0.0005	2.283	0.06
		4.00	2.00	0.0002	2.284	0.03
		6.25	2.50	0.0001	2.284	0.02
		8.00	2.83	0.0000	2.285	0.00

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 2

JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	09:40:45	0.00	0.00	0.0001	2.284	0.02
		0.10	0.32	0.0015	2.278	0.21
		0.25	0.50	0.0016	2.278	0.22
		0.50	0.71	0.0018	2.277	0.24
		1.00	1.00	0.0018	2.277	0.24
		1.57	1.25	0.0018	2.277	0.24
		2.00	1.41	0.0019	2.277	0.25
		2.25	1.50	0.0019	2.277	0.25
		3.07	1.75	0.0019	2.277	0.25
		4.00	2.00	0.0019	2.277	0.25
		6.25	2.50	0.0020	2.276	0.27
		8.00	2.83	0.0020	2.276	0.27
		12.25	3.50	0.0020	2.276	0.27
		15.00	3.87	0.0020	2.276	0.27
		22.00	4.69	0.0021	2.276	0.28
		30.00	5.48	0.0020	2.276	0.27
		36.00	6.00	0.0020	2.276	0.27

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 3

JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	10:17:48	0.00	0.00	0.0021	2.276	0.28
		0.10	0.32	0.0048	2.264	0.65
		0.25	0.50	0.0051	2.263	0.68
		0.50	0.71	0.0051	2.263	0.68
		1.00	1.00	0.0052	2.262	0.70
		1.57	1.25	0.0053	2.262	0.71
		2.00	1.41	0.0053	2.262	0.71
		2.25	1.50	0.0053	2.262	0.71
		3.07	1.75	0.0053	2.262	0.71
		4.00	2.00	0.0054	2.261	0.73
		6.25	2.50	0.0054	2.261	0.73
		8.00	2.83	0.0055	2.261	0.74
		12.25	3.50	0.0055	2.261	0.74
		15.00	3.87	0.0055	2.261	0.74
		22.00	4.69	0.0057	2.260	0.76
		30.00	5.48	0.0057	2.260	0.76
		36.00	6.00	0.0058	2.260	0.77
		49.00	7.00	0.0058	2.260	0.77
		60.00	7.75	0.0058	2.260	0.77

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 4

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	11:27:07	0.00	0.00	0.0058	2.260	0.77
		0.10	0.32	0.0099	2.241	1.33
		0.25	0.50	0.0101	2.240	1.36
		0.50	0.71	0.0103	2.240	1.38
		1.00	1.00	0.0105	2.239	1.41
		1.57	1.25	0.0105	2.239	1.41
		2.00	1.41	0.0106	2.238	1.42
		2.25	1.50	0.0106	2.238	1.42
		3.07	1.75	0.0106	2.238	1.42
		4.00	2.00	0.0107	2.238	1.44
		6.25	2.50	0.0108	2.237	1.45
		8.00	2.83	0.0108	2.237	1.45
		12.25	3.50	0.0110	2.237	1.47
		15.00	3.87	0.0110	2.237	1.47
		22.00	4.69	0.0111	2.236	1.49
		30.00	5.48	0.0112	2.236	1.50
		36.00	6.00	0.0112	2.236	1.50
		49.00	7.00	0.0113	2.235	1.52
		60.00	7.75	0.0114	2.235	1.53
		77.68	8.81	0.0115	2.234	1.55

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 5

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	12:44:49	0.00	0.00	0.0115	2.234	1.55
		0.10	0.32	0.0180	2.206	2.42
		0.25	0.50	0.0186	2.203	2.50
		0.50	0.71	0.0190	2.201	2.55
		1.00	1.00	0.0193	2.200	2.59
		1.57	1.25	0.0196	2.199	2.62
		2.00	1.41	0.0197	2.198	2.64
		2.25	1.50	0.0197	2.198	2.64
		3.07	1.75	0.0199	2.197	2.67
		4.00	2.00	0.0200	2.197	2.69
		6.25	2.50	0.0203	2.196	2.72
		8.00	2.83	0.0204	2.195	2.74
		12.25	3.50	0.0206	2.194	2.77
		15.00	3.87	0.0207	2.194	2.78
		22.00	4.69	0.0211	2.192	2.83
		30.00	5.48	0.0213	2.191	2.86
		36.00	6.00	0.0214	2.190	2.88
		49.00	7.00	0.0217	2.189	2.91
		60.00	7.75	0.0218	2.189	2.93
		120.00	10.95	0.0224	2.186	3.00
		180.00	13.42	0.0229	2.184	3.07
		240.00	15.49	0.0231	2.183	3.10
		300.00	17.32	0.0233	2.182	3.13
		360.00	18.97	0.0236	2.181	3.16
		420.00	20.49	0.0238	2.180	3.19
		480.00	21.91	0.0240	2.179	3.23
		540.00	23.24	0.0242	2.179	3.24
		600.00	24.49	0.0243	2.178	3.26
		660.00	25.69	0.0244	2.177	3.27
12-14-96	00:44:49	720.00	26.83	0.0246	2.176	3.30
		780.00	27.93	0.0246	2.176	3.30
		840.00	28.98	0.0247	2.176	3.32
		900.00	30.00	0.0249	2.175	3.34
		960.00	30.98	0.0250	2.175	3.35
		1020.00	31.94	0.0251	2.174	3.37
		1080.00	32.86	0.0251	2.174	3.37

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 6

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0252	2.174	3.38
		1200.00	34.64	0.0253	2.173	3.40
		1260.00	35.50	0.0253	2.173	3.40
		1320.00	36.33	0.0255	2.173	3.42
		1380.00	37.15	0.0256	2.172	3.43
		1440.00	37.95	0.0256	2.172	3.43
		1500.00	38.73	0.0257	2.172	3.45
		1560.00	39.50	0.0258	2.171	3.46
		1620.00	40.25	0.0258	2.171	3.46
		1680.00	40.99	0.0259	2.171	3.48
		1740.00	41.71	0.0259	2.171	3.48
		1800.00	42.43	0.0260	2.170	3.49
		1860.00	43.13	0.0260	2.170	3.49
		1920.00	43.82	0.0262	2.170	3.51
		1980.00	44.50	0.0262	2.170	3.51
		2040.00	45.17	0.0263	2.169	3.53
		2100.00	45.83	0.0263	2.169	3.53
12-15-96	00:44:49	2160.00	46.48	0.0264	2.169	3.54
		2220.00	47.12	0.0264	2.169	3.54
		2280.00	47.75	0.0264	2.169	3.54
		2340.00	48.37	0.0265	2.168	3.56
		2400.00	48.99	0.0266	2.168	3.57
		2460.00	49.60	0.0266	2.168	3.57
		2520.00	50.20	0.0266	2.168	3.57
		2580.00	50.79	0.0267	2.167	3.59
		2640.00	51.38	0.0267	2.167	3.59
		2700.00	51.96	0.0269	2.167	3.61
		2760.00	52.54	0.0269	2.167	3.61
		2820.00	53.10	0.0269	2.167	3.61
		2880.00	53.67	0.0270	2.166	3.62
		2940.00	54.22	0.0270	2.166	3.62
		3000.00	54.77	0.0270	2.166	3.62
		3060.00	55.32	0.0271	2.166	3.64
		3120.00	55.86	0.0271	2.166	3.64

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 7
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0272	2.165	3.65
		3240.00	56.92	0.0272	2.165	3.65
		3300.00	57.45	0.0272	2.165	3.65
		3360.00	57.97	0.0273	2.164	3.67
		3420.00	58.48	0.0273	2.164	3.67
		3480.00	58.99	0.0273	2.164	3.67
		3540.00	59.50	0.0275	2.164	3.68
12-16-96	00:44:49	3600.00	60.00	0.0275	2.164	3.68
		3660.00	60.50	0.0275	2.164	3.68
		3720.00	60.99	0.0276	2.163	3.70
		3780.00	61.48	0.0276	2.163	3.70
		3840.00	61.97	0.0276	2.163	3.70
		3900.00	62.45	0.0277	2.163	3.72
		3960.00	62.93	0.0277	2.163	3.72
		4020.00	63.40	0.0276	2.163	3.70
		4080.00	63.87	0.0277	2.163	3.72
		4140.00	64.34	0.0277	2.163	3.72
		4200.00	64.81	0.0277	2.163	3.72
		4260.00	65.27	0.0277	2.163	3.72
		4320.00	65.73	0.0278	2.162	3.73
		4380.00	66.18	0.0278	2.162	3.73
		4440.00	66.63	0.0278	2.162	3.73
		4500.00	67.08	0.0279	2.162	3.75
		4560.00	67.53	0.0279	2.162	3.75
		4620.00	67.97	0.0279	2.162	3.75
		4680.00	68.41	0.0280	2.161	3.76
		4740.00	68.85	0.0280	2.161	3.76
		4800.00	69.28	0.0282	2.161	3.78
		4860.00	69.71	0.0283	2.160	3.79
		4920.00	70.14	0.0283	2.160	3.79
		4980.00	70.57	0.0283	2.160	3.79
12-17-96	00:44:49	5040.00	70.99	0.0284	2.160	3.81
		5100.00	71.41	0.0284	2.160	3.81
		5160.00	71.83	0.0284	2.160	3.81

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 8
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		5220.00	72.25	0.0285	2.159	3.83
		5280.00	72.66	0.0285	2.159	3.83
		5340.00	73.08	0.0285	2.159	3.83
		5400.00	73.48	0.0285	2.159	3.83
		5460.00	73.89	0.0285	2.159	3.83
		5520.00	74.30	0.0285	2.159	3.83
		5580.00	74.70	0.0285	2.159	3.83
		5640.00	75.10	0.0285	2.159	3.83
		5700.00	75.50	0.0286	2.159	3.84
		5760.00	75.89	0.0286	2.159	3.84
		5820.00	76.29	0.0286	2.159	3.84
		5880.00	76.68	0.0286	2.159	3.84
		5940.00	77.07	0.0288	2.158	3.86
		6000.00	77.46	0.0288	2.158	3.86
		6060.00	77.85	0.0288	2.158	3.86
		6120.00	78.23	0.0289	2.158	3.87
		6180.00	78.61	0.0289	2.158	3.87
		6240.00	78.99	0.0289	2.158	3.87
		6300.00	79.37	0.0290	2.157	3.89
		6360.00	79.75	0.0290	2.157	3.89
		6420.00	80.12	0.0291	2.157	3.91
12-18-96	00:44:49	6480.00	80.50	0.0291	2.157	3.91
		6540.00	80.87	0.0291	2.157	3.91
		6600.00	81.24	0.0292	2.156	3.92
		6660.00	81.61	0.0292	2.156	3.92
		6720.00	81.98	0.0292	2.156	3.92
		6780.00	82.34	0.0292	2.156	3.92
		6840.00	82.70	0.0292	2.156	3.92
		6900.00	83.07	0.0292	2.156	3.92
		6960.00	83.43	0.0292	2.156	3.92
		7020.00	83.79	0.0292	2.156	3.92
		7080.00	84.14	0.0292	2.156	3.92
		7140.00	84.50	0.0292	2.156	3.92
		7200.00	84.85	0.0293	2.156	3.94

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 9
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		7260.00	85.21	0.0293	2.156	3.94
		7320.00	85.56	0.0293	2.156	3.94
		7380.00	85.91	0.0293	2.156	3.94
		7440.00	86.26	0.0295	2.155	3.95
		7500.00	86.60	0.0295	2.155	3.95
		7560.00	86.95	0.0295	2.155	3.95
		7620.00	87.29	0.0295	2.155	3.95
		7680.00	87.64	0.0295	2.155	3.95
		7740.00	87.98	0.0296	2.155	3.97
		7800.00	88.32	0.0296	2.155	3.97
		7860.00	88.66	0.0296	2.155	3.97
12-19-96	00:44:49	7920.00	88.99	0.0297	2.154	3.98
		7980.00	89.33	0.0297	2.154	3.98
		8040.00	89.67	0.0297	2.154	3.98
		8100.00	90.00	0.0297	2.154	3.98
		8160.00	90.33	0.0298	2.154	4.00
		8220.00	90.66	0.0298	2.154	4.00
		8280.00	90.99	0.0298	2.154	4.00
		8340.00	91.32	0.0298	2.154	4.00
		8400.00	91.65	0.0298	2.154	4.00
		8460.00	91.98	0.0299	2.153	4.02
		8520.00	92.30	0.0299	2.153	4.02
		8580.00	92.63	0.0299	2.153	4.02
		8640.00	92.95	0.0299	2.153	4.02
		8700.00	93.27	0.0299	2.153	4.02
		8760.00	93.59	0.0300	2.153	4.03
		8820.00	93.91	0.0300	2.153	4.03
		8880.00	94.23	0.0300	2.153	4.03
		8940.00	94.55	0.0302	2.152	4.05
		9000.00	94.87	0.0302	2.152	4.05
		9060.00	95.18	0.0302	2.152	4.05
		9120.00	95.50	0.0302	2.152	4.05
		9180.00	95.81	0.0303	2.152	4.06
		9240.00	96.12	0.0303	2.152	4.06

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		9300.00	96.44	0.0303	2.152	4.06
12-20-96	00:44:49	9360.00	96.75	0.0303	2.152	4.06
		9420.00	97.06	0.0304	2.151	4.08
		9480.00	97.37	0.0304	2.151	4.08
		9540.00	97.67	0.0304	2.151	4.08
		9600.00	97.98	0.0305	2.150	4.10
		9660.00	98.29	0.0305	2.150	4.10
		9720.00	98.59	0.0305	2.150	4.10
		9780.00	98.89	0.0305	2.150	4.10
		9840.00	99.20	0.0305	2.150	4.10
		9900.00	99.50	0.0305	2.150	4.10
		9960.00	99.80	0.0305	2.150	4.10
		1002.00E+01	100.10	0.0305	2.150	4.10
		1008.00E+01	100.40	0.0305	2.150	4.10
		1014.00E+01	100.70	0.0305	2.150	4.10
		1020.00E+01	101.00	0.0305	2.150	4.10
		1026.00E+01	101.29	0.0306	2.150	4.11
		1032.00E+01	101.59	0.0306	2.150	4.11
		1038.00E+01	101.88	0.0306	2.150	4.11
		1044.00E+01	102.18	0.0306	2.150	4.11
		1050.00E+01	102.47	0.0308	2.149	4.13
		1056.00E+01	102.76	0.0308	2.149	4.13
		1062.00E+01	103.05	0.0308	2.149	4.13
		1068.00E+01	103.34	0.0309	2.149	4.14
		1074.00E+01	103.63	0.0309	2.149	4.14
12-21-96	00:44:49	1080.00E+01	103.92	0.0310	2.148	4.16
		1086.00E+01	104.21	0.0310	2.148	4.16
		1092.00E+01	104.50	0.0310	2.148	4.16
		1098.00E+01	104.79	0.0310	2.148	4.16
		1104.00E+01	105.07	0.0310	2.148	4.16
		1110.00E+01	105.36	0.0311	2.148	4.17
		1116.00E+01	105.64	0.0311	2.148	4.17
		1122.00E+01	105.92	0.0311	2.148	4.17
		1128.00E+01	106.21	0.0311	2.148	4.17

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 11
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1134.00E+01	106.49	0.0311	2.148	4.17
		1140.00E+01	106.77	0.0311	2.148	4.17
		1146.00E+01	107.05	0.0312	2.147	4.19
		1152.00E+01	107.33	0.0312	2.147	4.19
		1158.00E+01	107.61	0.0312	2.147	4.19
		1164.00E+01	107.89	0.0312	2.147	4.19
		1170.00E+01	108.17	0.0312	2.147	4.19
		1176.00E+01	108.44	0.0312	2.147	4.19
		1182.00E+01	108.72	0.0312	2.147	4.19
		1188.00E+01	109.00	0.0312	2.147	4.19
		1194.00E+01	109.27	0.0312	2.147	4.19
		1200.00E+01	109.54	0.0312	2.147	4.19
		1206.00E+01	109.82	0.0313	2.147	4.21
		1212.00E+01	110.09	0.0313	2.147	4.21
		1218.00E+01	110.36	0.0313	2.147	4.21
12-22-96	00:44:49	1224.00E+01	110.63	0.0313	2.147	4.21
		1230.00E+01	110.91	0.0315	2.146	4.22
		1236.00E+01	111.18	0.0315	2.146	4.22
		1242.00E+01	111.45	0.0315	2.146	4.22
		1248.00E+01	111.71	0.0315	2.146	4.22
		1254.00E+01	111.98	0.0315	2.146	4.22
		1260.00E+01	112.25	0.0315	2.146	4.22
		1266.00E+01	112.52	0.0316	2.146	4.24
		1272.00E+01	112.78	0.0316	2.146	4.24
		1278.00E+01	113.05	0.0316	2.146	4.24
		1284.00E+01	113.31	0.0316	2.146	4.24
		1290.00E+01	113.58	0.0316	2.146	4.24
		1296.00E+01	113.84	0.0316	2.146	4.24
		1302.00E+01	114.11	0.0317	2.145	4.25
		1308.00E+01	114.37	0.0316	2.146	4.24
		1314.00E+01	114.63	0.0317	2.145	4.25
		1320.00E+01	114.89	0.0317	2.145	4.25
		1326.00E+01	115.15	0.0317	2.145	4.25
		1332.00E+01	115.41	0.0317	2.145	4.25

LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 12
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1338.00E+01	115.67	0.0317	2.145	4.25
		1344.00E+01	115.93	0.0317	2.145	4.25
		1350.00E+01	116.19	0.0317	2.145	4.25
		1356.00E+01	116.45	0.0317	2.145	4.25
		1362.00E+01	116.70	0.0318	2.145	4.27
12-23-96	00:44:49	1368.00E+01	116.96	0.0318	2.145	4.27
		1374.00E+01	117.22	0.0318	2.145	4.27
		1380.00E+01	117.47	0.0318	2.145	4.27
		1386.00E+01	117.73	0.0318	2.145	4.27
		1392.00E+01	117.98	0.0318	2.145	4.27
		1398.00E+01	118.24	0.0318	2.145	4.27
		1404.00E+01	118.49	0.0318	2.145	4.27
		1410.00E+01	118.74	0.0318	2.145	4.27
		1416.00E+01	119.00	0.0318	2.145	4.27
		1422.00E+01	119.25	0.0318	2.145	4.27
		1428.00E+01	119.50	0.0318	2.145	4.27
		1434.00E+01	119.75	0.0318	2.145	4.27
		1440.00E+01	120.00	0.0318	2.145	4.27
		1446.00E+01	120.25	0.0318	2.145	4.27
		1452.00E+01	120.50	0.0318	2.145	4.27
		1458.00E+01	120.75	0.0318	2.145	4.27
		1464.00E+01	121.00	0.0318	2.145	4.27
		1470.00E+01	121.24	0.0318	2.145	4.27
		1476.00E+01	121.49	0.0319	2.144	4.28
		1482.00E+01	121.74	0.0319	2.144	4.28
		1488.00E+01	121.98	0.0319	2.144	4.28
		1494.00E+01	122.23	0.0321	2.144	4.30
		1500.00E+01	122.47	0.0321	2.144	4.30
		1506.00E+01	122.72	0.0321	2.144	4.30
12-24-96	00:44:49	1512.00E+01	122.96	0.0321	2.144	4.30
		1518.00E+01	123.21	0.0322	2.143	4.32
		1524.00E+01	123.45	0.0322	2.143	4.32
		1530.00E+01	123.69	0.0322	2.143	4.32
		1536.00E+01	123.94	0.0322	2.143	4.32
		1542.00E+01	124.18	0.0322	2.143	4.32
		1548.00E+01	124.42	0.0323	2.143	4.33
		1554.00E+01	124.66	0.0322	2.143	4.32

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 13
JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-24-96	08:09:25	0.00	0.00	0.0322	2.143	4.32
		0.10	0.32	0.0455	2.085	6.10
		0.25	0.50	0.0487	2.070	6.53
		0.50	0.71	0.0508	2.061	6.81
		1.00	1.00	0.0529	2.052	7.10
		1.57	1.25	0.0544	2.045	7.31
		2.00	1.41	0.0553	2.041	7.42
		2.25	1.50	0.0557	2.039	7.48
		3.07	1.75	0.0568	2.035	7.62
		4.00	2.00	0.0577	2.030	7.75
		6.25	2.50	0.0594	2.023	7.97
		8.00	2.83	0.0603	2.019	8.10
		12.25	3.50	0.0622	2.011	8.35
		15.00	3.87	0.0630	2.007	8.46
		22.00	4.69	0.0648	1.999	8.70
		30.00	5.48	0.0663	1.993	8.90
		36.00	6.00	0.0672	1.989	9.01
		49.00	7.00	0.0687	1.982	9.22
		60.00	7.75	0.0699	1.977	9.38
		120.00	10.95	0.0734	1.961	9.85
		180.00	13.42	0.0755	1.952	10.14
		240.00	15.49	0.0771	1.945	10.34
		300.00	17.32	0.0784	1.940	10.51
		360.00	18.97	0.0794	1.935	10.66
		420.00	20.49	0.0804	1.931	10.78
		480.00	21.91	0.0812	1.927	10.89
		540.00	23.24	0.0818	1.925	10.97
		600.00	24.49	0.0824	1.922	11.05
		660.00	25.69	0.0830	1.919	11.13
		720.00	26.83	0.0833	1.918	11.18
		780.00	27.93	0.0838	1.916	11.24
		840.00	28.98	0.0841	1.914	11.29
		900.00	30.00	0.0845	1.913	11.34
12-25-96	00:09:25	960.00	30.98	0.0848	1.911	11.38
		1020.00	31.94	0.0852	1.909	11.43
		1080.00	32.86	0.0856	1.908	11.48

LOAD INCREMENT DATA

TEST NAME. C1-U3D

TESTED BY: ACS

PAGE NO: 14

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0858	1.907	11.51
		1200.00	34.64	0.0860	1.906	11.54
		1260.00	35.50	0.0863	1.905	11.57
		1320.00	36.33	0.0865	1.904	11.61
		1380.00	37.15	0.0867	1.903	11.64
		1440.00	37.95	0.0870	1.902	11.67
		1500.00	38.73	0.0872	1.901	11.70
		1560.00	39.50	0.0874	1.900	11.73
		1620.00	40.25	0.0876	1.899	11.75
		1680.00	40.99	0.0878	1.898	11.78
		1740.00	41.71	0.0879	1.898	11.80
		1800.00	42.43	0.0881	1.896	11.83
		1860.00	43.13	0.0883	1.896	11.84
		1920.00	43.82	0.0884	1.895	11.86
		1980.00	44.50	0.0886	1.894	11.89
		2040.00	45.17	0.0887	1.894	11.91
		2100.00	45.83	0.0889	1.893	11.92
		2160.00	46.48	0.0890	1.893	11.94
		2220.00	47.12	0.0891	1.892	11.95
		2280.00	47.75	0.0892	1.892	11.97
		2340.00	48.37	0.0894	1.891	12.00
12-26-96	00:09:25	2400.00	48.99	0.0896	1.890	12.02
		2460.00	49.60	0.0896	1.890	12.02
		2520.00	50.20	0.0898	1.889	12.05
		2580.00	50.79	0.0898	1.889	12.05
		2640.00	51.38	0.0900	1.888	12.08
		2700.00	51.96	0.0900	1.888	12.08
		2760.00	52.54	0.0903	1.887	12.11
		2820.00	53.10	0.0903	1.887	12.11
		2880.00	53.67	0.0904	1.887	12.13
		2940.00	54.22	0.0905	1.886	12.14
		3000.00	54.77	0.0906	1.886	12.16
		3060.00	55.32	0.0906	1.886	12.16
		3120.00	55.86	0.0907	1.885	12.18

LOAD INCREMENT DATA

TEST NAME C1-U3D   PAGE NO: 15
TESTED BY: ACS   JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0909	1.885	12.19
		3240.00	56.92	0.0910	1.884	12.21
		3300.00	57.45	0.0911	1.883	12.22
		3360.00	57.97	0.0912	1.883	12.24
		3420.00	58.48	0.0913	1.882	12.25
		3480.00	58.99	0.0914	1.882	12.27
		3540.00	59.50	0.0916	1.881	12.29
		3600.00	60.00	0.0917	1.881	12.30
		3660.00	60.50	0.0918	1.880	12.32
		3720.00	60.99	0.0919	1.880	12.33
		3780.00	61.48	0.0919	1.880	12.33
12-27-96	00:09:25	3840.00	61.97	0.0920	1.879	12.35
		3900.00	62.45	0.0922	1.879	12.36
		3960.00	62.93	0.0923	1.878	12.38
		4020.00	63.40	0.0923	1.878	12.38
		4080.00	63.87	0.0924	1.878	12.40
		4140.00	64.34	0.0925	1.877	12.41
		4200.00	64.81	0.0926	1.877	12.43
		4260.00	65.27	0.0926	1.877	12.43
		4320.00	65.73	0.0927	1.876	12.44
		4380.00	66.18	0.0929	1.876	12.46
		4440.00	66.63	0.0929	1.876	12.46
		4500.00	67.08	0.0930	1.875	12.48
		4560.00	67.53	0.0930	1.875	12.48
		4620.00	67.97	0.0931	1.875	12.49
		4680.00	68.41	0.0932	1.874	12.51
		4740.00	68.85	0.0932	1.874	12.51
		4800.00	69.28	0.0933	1.874	12.52
		4860.00	69.71	0.0934	1.873	12.54
		4920.00	70.14	0.0936	1.873	12.55
		4980.00	70.57	0.0936	1.873	12.55
		5040.00	70.99	0.0937	1.872	12.57
		5100.00	71.41	0.0937	1.872	12.57
		5160.00	71.83	0.0938	1.872	12.59

LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 16
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		5220.00	72.25	0.0938	1.872	12.59
12-28-96	00:09:25	5280.00	72.66	0.0939	1.871	12.60
		5340.00	73.08	0.0939	1.871	12.60
		5400.00	73.48	0.0940	1.871	12.62
		5460.00	73.89	0.0942	1.870	12.63
		5520.00	74.30	0.0942	1.870	12.63
		5580.00	74.70	0.0942	1.870	12.63
		5640.00	75.10	0.0943	1.869	12.65
		5700.00	75.50	0.0944	1.869	12.67
		5760.00	75.89	0.0944	1.869	12.67
		5820.00	76.29	0.0944	1.869	12.67
		5880.00	76.68	0.0945	1.868	12.68
		5940.00	77.07	0.0945	1.868	12.68
		6000.00	77.46	0.0946	1.868	12.70
		6060.00	77.85	0.0946	1.868	12.70
		6120.00	78.23	0.0946	1.868	12.70
		6180.00	78.61	0.0947	1.867	12.71
		6240.00	78.99	0.0947	1.867	12.71
		6300.00	79.37	0.0949	1.867	12.73
		6360.00	79.75	0.0949	1.867	12.73
		6420.00	80.12	0.0949	1.867	12.73
		6480.00	80.50	0.0950	1.866	12.74
		6540.00	80.87	0.0950	1.866	12.74
		6600.00	81.24	0.0951	1.866	12.76
		6660.00	81.61	0.0951	1.866	12.76
12-29-96	00:09:25	6720.00	81.98	0.0951	1.866	12.76
		6780.00	82.34	0.0952	1.865	12.78
		6840.00	82.70	0.0952	1.865	12.78
		6900.00	83.07	0.0953	1.865	12.79
		6960.00	83.43	0.0953	1.865	12.79
		7020.00	83.79	0.0955	1.864	12.81
		7080.00	84.14	0.0955	1.864	12.81
		7140.00	84.50	0.0955	1.864	12.81
		7200.00	84.85	0.0956	1.864	12.82

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 17

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		7260.00	85.21	0.0956	1.864	12.82
		7320.00	85.56	0.0957	1.863	12.84
		7380.00	85.91	0.0957	1.863	12.84
		7440.00	86.26	0.0957	1.863	12.84
		7500.00	86.60	0.0957	1.863	12.84
		7560.00	86.95	0.0958	1.863	12.85
		7620.00	87.29	0.0958	1.863	12.85
		7680.00	87.64	0.0958	1.863	12.85
		7740.00	87.98	0.0959	1.862	12.87
		7800.00	88.32	0.0959	1.862	12.87
		7860.00	88.66	0.0959	1.862	12.87
		7920.00	88.99	0.0960	1.862	12.89
		7980.00	89.33	0.0960	1.862	12.89
		8040.00	89.67	0.0960	1.862	12.89
		8100.00	90.00	0.0960	1.862	12.89
12-30-96	01:09:25	8220.00	90.66	0.0962	1.861	12.90
		8340.00	91.32	0.0963	1.861	12.92
		8460.00	91.98	0.0964	1.860	12.93
		8580.00	92.63	0.0964	1.860	12.93
		8700.00	93.27	0.0965	1.860	12.95
		8820.00	93.91	0.0966	1.859	12.97
		8940.00	94.55	0.0966	1.859	12.97
		9060.00	95.18	0.0967	1.859	12.98
		9180.00	95.81	0.0967	1.859	12.98
		9300.00	96.44	0.0969	1.858	13.00
		9420.00	97.06	0.0970	1.858	13.01
		9480.00	97.37	0.0970	1.858	13.01
		9540.00	97.67	0.0971	1.857	13.03
12-31-96	01:09:25	9660.00	98.29	0.0971	1.857	13.03
		9780.00	98.89	0.0972	1.856	13.04
		9900.00	99.50	0.0972	1.856	13.04
		1002.00E+01	100.10	0.0973	1.856	13.06
		1014.00E+01	100.70	0.0975	1.855	13.08
		1026.00E+01	101.29	0.0975	1.855	13.08
		1038.00E+01	101.88	0.0976	1.855	13.09
		1050.00E+01	102.47	0.0977	1.854	13.11

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 18
JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-31-96	15:32:16	0.00	0.00	0.0977	1.854	13.11
		0.10	0.32	0.1152	1.777	15.46
		0.25	0.50	0.1207	1.753	16.19
		0.50	0.71	0.1242	1.738	16.67
		1.00	1.00	0.1274	1.724	17.09
		1.57	1.25	0.1295	1.714	17.38
		2.00	1.41	0.1306	1.709	17.52
		2.25	1.50	0.1312	1.707	17.60
		3.07	1.75	0.1327	1.700	17.80
		4.00	2.00	0.1339	1.695	17.96
		6.25	2.50	0.1361	1.685	18.26
		8.00	2.83	0.1373	1.680	18.42
		12.25	3.50	0.1395	1.670	18.72
		15.00	3.87	0.1406	1.665	18.86
		22.00	4.69	0.1425	1.657	19.12
		30.00	5.48	0.1442	1.649	19.35
		36.00	6.00	0.1452	1.645	19.48
		49.00	7.00	0.1469	1.637	19.72
		60.00	7.75	0.1481	1.632	19.88
		120.00	10.95	0.1520	1.615	20.40
		180.00	13.42	0.1541	1.606	20.68
		240.00	15.49	0.1557	1.599	20.89
		300.00	17.32	0.1568	1.594	21.05
		360.00	18.97	0.1577	1.590	21.16
		420.00	20.49	0.1584	1.587	21.25
		480.00	21.91	0.1591	1.584	21.35
01-01-97	00:32:16	540.00	23.24	0.1596	1.582	21.41
		600.00	24.49	0.1601	1.579	21.49
		660.00	25.69	0.1605	1.578	21.54
		720.00	26.83	0.1610	1.575	21.60
		780.00	27.93	0.1613	1.574	21.65
		840.00	28.98	0.1617	1.572	21.69
		900.00	30.00	0.1619	1.571	21.73
		960.00	30.98	0.1621	1.570	21.76
		1020.00	31.94	0.1625	1.569	21.80
		1080.00	32.86	0.1629	1.567	21.85

LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 19

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.1630	1.567	21.87
		1200.00	34.64	0.1632	1.566	21.90
		1260.00	35.50	0.1634	1.565	21.93
		1320.00	36.33	0.1636	1.564	21.95
		1380.00	37.15	0.1638	1.563	21.98
		1440.00	37.95	0.1640	1.562	22.01
		1500.00	38.73	0.1642	1.561	22.03
		1560.00	39.50	0.1644	1.560	22.06
		1620.00	40.25	0.1645	1.560	22.07
		1680.00	40.99	0.1647	1.559	22.10
		1740.00	41.71	0.1650	1.558	22.14
		1800.00	42.43	0.1650	1.558	22.14
		1860.00	43.13	0.1652	1.557	22.17
		1920.00	43.82	0.1653	1.556	22.18
01-02-97	00:32:16	1980.00	44.50	0.1654	1.556	22.20
		2040.00	45.17	0.1656	1.555	22.22
		2100.00	45.83	0.1658	1.554	22.25
		2160.00	46.48	0.1659	1.554	22.26
		2220.00	47.12	0.1660	1.553	22.28
		2280.00	47.75	0.1663	1.552	22.31
		2340.00	48.37	0.1664	1.552	22.33
		2400.00	48.99	0.1666	1.551	22.36
		2460.00	49.60	0.1667	1.550	22.37
		2520.00	50.20	0.1669	1.550	22.39
		2580.00	50.79	0.1671	1.548	22.42
		2640.00	51.38	0.1672	1.548	22.44

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 8.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	11:37:38	0.00	0.00	0.1672	1.548	22.44
		0.10	0.32	0.1616	1.573	21.68
		0.25	0.50	0.1613	1.574	21.65
		0.50	0.71	0.1611	1.575	21.61
		1.00	1.00	0.1610	1.575	21.60
		1.57	1.25	0.1610	1.575	21.60
		2.00	1.41	0.1610	1.575	21.60
		2.25	1.50	0.1610	1.575	21.60
		3.07	1.75	0.1609	1.576	21.58
		4.00	2.00	0.1609	1.576	21.58
		6.25	2.50	0.1607	1.577	21.57
		8.00	2.83	0.1607	1.577	21.57
		12.25	3.50	0.1607	1.577	21.57
		15.00	3.87	0.1606	1.577	21.55
		22.00	4.69	0.1606	1.577	21.55
		30.00	5.48	0.1606	1.577	21.55
		36.00	6.00	0.1605	1.578	21.54
		49.00	7.00	0.1605	1.578	21.54
		60.00	7.75	0.1605	1.578	21.54
		120.00	10.95	0.1604	1.578	21.52
		180.00	13.42	0.1604	1.578	21.52
		240.00	15.49	0.1603	1.579	21.50
		300.00	17.32	0.1603	1.579	21.50

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 21

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	16:55:06	0.00	0.00	0.1603	1.579	21.50
		0.10	0.32	0.1545	1.604	20.73
		0.25	0.50	0.1535	1.608	20.60
		0.50	0.71	0.1530	1.611	20.52
		1.00	1.00	0.1524	1.613	20.44
		1.57	1.25	0.1521	1.614	20.41
		2.00	1.41	0.1520	1.615	20.40
		2.25	1.50	0.1520	1.615	20.40
		3.07	1.75	0.1518	1.616	20.37
		4.00	2.00	0.1517	1.617	20.35
		6.25	2.50	0.1515	1.617	20.33
		8.00	2.83	0.1514	1.618	20.32
		12.25	3.50	0.1513	1.618	20.30
		15.00	3.87	0.1512	1.619	20.29
		22.00	4.69	0.1511	1.619	20.27
		30.00	5.48	0.1510	1.620	20.25
		36.00	6.00	0.1508	1.620	20.24
		49.00	7.00	0.1507	1.621	20.22
		60.00	7.75	0.1506	1.621	20.21
		120.00	10.95	0.1504	1.622	20.18
		180.00	13.42	0.1502	1.623	20.16
		240.00	15.49	0.1501	1.623	20.14
		300.00	17.32	0.1500	1.624	20.13
		360.00	18.97	0.1500	1.624	20.13
		420.00	20.49	0.1500	1.624	20.13
01-03-97	00:55:06	480.00	21.91	0.1499	1.624	20.11
		540.00	23.24	0.1499	1.624	20.11
		600.00	24.49	0.1499	1.624	20.11
		660.00	25.69	0.1499	1.624	20.11
		720.00	26.83	0.1499	1.624	20.11
		780.00	27.93	0.1499	1.624	20.11
		840.00	28.98	0.1499	1.624	20.11
		900.00	30.00	0.1499	1.624	20.11
		960.00	30.98	0.1498	1.625	20.10

LOAD INCREMENT DATA

TEST NAME C1-U3D

PAGE NO: 22

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	09:02:18	0.00	0.00	0.1498	1.625	20.10
		0.10	0.32	0.1513	1.618	20.30
		0.25	0.50	0.1514	1.618	20.32
		0.50	0.71	0.1515	1.617	20.33
		1.00	1.00	0.1517	1.617	20.35
		1.57	1.25	0.1517	1.617	20.35
		2.00	1.41	0.1517	1.617	20.35
		2.25	1.50	0.1517	1.617	20.35
		3.07	1.75	0.1517	1.617	20.35
		4.00	2.00	0.1518	1.616	20.37
		6.25	2.50	0.1518	1.616	20.37
		8.00	2.83	0.1518	1.616	20.37
		12.25	3.50	0.1518	1.616	20.37
		15.00	3.87	0.1519	1.615	20.38
		22.00	4.69	0.1519	1.615	20.38
		30.00	5.48	0.1519	1.615	20.38
		36.00	6.00	0.1519	1.615	20.38
		49.00	7.00	0.1519	1.615	20.38
		60.00	7.75	0.1520	1.615	20.40
		120.00	10.95	0.1520	1.615	20.40
		180.00	13.42	0.1520	1.615	20.40

LOAD INCREMENT DATA

TEST NAME C1-U3D
TESTED BY: ACS

PAGE NO: 23
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	12:20:30	0.00	0.00	0.1520	1.615	20.40
		0.10	0.32	0.1552	1.601	20.82
		0.25	0.50	0.1554	1.600	20.86
		0.50	0.71	0.1557	1.599	20.89
		1.00	1.00	0.1558	1.598	20.90
		1.57	1.25	0.1558	1.598	20.90
		2.00	1.41	0.1559	1.598	20.92
		2.25	1.50	0.1558	1.598	20.90
		3.07	1.75	0.1559	1.598	20.92
		4.00	2.00	0.1559	1.598	20.92
		6.25	2.50	0.1560	1.597	20.93
		8.00	2.83	0.1560	1.597	20.93
		12.25	3.50	0.1561	1.597	20.95
		15.00	3.87	0.1561	1.597	20.95
		22.00	4.69	0.1561	1.597	20.95
		30.00	5.48	0.1563	1.596	20.97
		36.00	6.00	0.1563	1.596	20.97
		49.00	7.00	0.1563	1.596	20.97
		60.00	7.75	0.1563	1.596	20.97
		120.00	10.95	0.1564	1.596	20.98
		180.00	13.42	0.1565	1.595	21.00
		240.00	15.49	0.1565	1.595	21.00
		300.00	17.32	0.1565	1.595	21.00
		360.00	18.97	0.1565	1.595	21.00
		420.00	20.49	0.1565	1.595	21.00
		480.00	21.91	0.1565	1.595	21.00
		540.00	23.24	0.1565	1.595	21.00
		600.00	24.49	0.1565	1.595	21.00
		660.00	25.69	0.1565	1.595	21.00
01-04-97	00:20:30	720.00	26.83	0.1565	1.595	21.00
		780.00	27.93	0.1566	1.595	21.01
		840.00	28.98	0.1566	1.595	21.01
		900.00	30.00	0.1566	1.595	21.01
		960.00	30.98	0.1566	1.595	21.01
		1020.00	31.94	0.1566	1.595	21.01
		1080.00	32.86	0.1566	1.595	21.01

LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 24

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.1566	1.595	21.01
		1200.00	34.64	0.1566	1.595	21.01
		1260.00	35.50	0.1566	1.595	21.01
		1320.00	36.33	0.1566	1.595	21.01
		1380.00	37.15	0.1566	1.595	21.01
		1440.00	37.95	0.1566	1.595	21.01
		1500.00	38.73	0.1566	1.595	21.01
		1560.00	39.50	0.1566	1.595	21.01
		1620.00	40.25	0.1567	1.594	21.03
		1680.00	40.99	0.1566	1.595	21.01
		1740.00	41.71	0.1566	1.595	21.01
		1800.00	42.43	0.1566	1.595	21.01
		1860.00	43.13	0.1566	1.595	21.01
		1920.00	43.82	0.1567	1.594	21.03
		1980.00	44.50	0.1567	1.594	21.03
		2040.00	45.17	0.1566	1.595	21.01
		2100.00	45.83	0.1567	1.594	21.03
01-05-97	00:20:30	2160.00	46.48	0.1567	1.594	21.03
		2220.00	47.12	0.1567	1.594	21.03
		2280.00	47.75	0.1567	1.594	21.03
		2340.00	48.37	0.1567	1.594	21.03
		2400.00	48.99	0.1567	1.594	21.03
		2460.00	49.60	0.1567	1.594	21.03
		2520.00	50.20	0.1567	1.594	21.03
		2580.00	50.79	0.1567	1.594	21.03
		2640.00	51.38	0.1567	1.594	21.03
		2700.00	51.96	0.1567	1.594	21.03
		2760.00	52.54	0.1567	1.594	21.03
		2820.00	53.10	0.1567	1.594	21.03
		2880.00	53.67	0.1567	1.594	21.03
		2940.00	54.22	0.1567	1.594	21.03
		3000.00	54.77	0.1567	1.594	21.03
		3060.00	55.32	0.1567	1.594	21.03
		3120.00	55.86	0.1567	1.594	21.03

LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 25
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.1567	1.594	21.03
		3240.00	56.92	0.1567	1.594	21.03
		3300.00	57.45	0.1567	1.594	21.03
		3360.00	57.97	0.1567	1.594	21.03
		3420.00	58.48	0.1567	1.594	21.03
		3480.00	58.99	0.1567	1.594	21.03
		3540.00	59.50	0.1567	1.594	21.03
01-06-97	00:20:30	3600.00	60.00	0.1567	1.594	21.03
		3660.00	60.50	0.1567	1.594	21.03
		3720.00	60.99	0.1567	1.594	21.03
		3780.00	61.48	0.1567	1.594	21.03
		3840.00	61.97	0.1567	1.594	21.03
		3900.00	62.45	0.1567	1.594	21.03
		3960.00	62.93	0.1568	1.594	21.05
		4020.00	63.40	0.1568	1.594	21.05
		4080.00	63.87	0.1568	1.594	21.05

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 26

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-06-97	08:46:30	0.00	0.00	0.1568	1.594	21.05
		0.10	0.32	0.1605	1.578	21.54
		0.25	0.50	0.1609	1.576	21.58
		0.50	0.71	0.1610	1.575	21.60
		1.00	1.00	0.1612	1.574	21.63
		1.57	1.25	0.1612	1.574	21.63
		2.00	1.41	0.1612	1.574	21.63
		2.25	1.50	0.1613	1.574	21.65
		3.07	1.75	0.1613	1.574	21.65
		4.00	2.00	0.1613	1.574	21.65
		6.25	2.50	0.1614	1.573	21.66
		8.00	2.83	0.1614	1.573	21.66
		12.25	3.50	0.1616	1.573	21.68
		15.00	3.87	0.1616	1.573	21.68
		22.00	4.69	0.1617	1.572	21.69
		30.00	5.48	0.1618	1.572	21.71
		36.00	6.00	0.1618	1.572	21.71
		49.00	7.00	0.1618	1.572	21.71
		60.00	7.75	0.1619	1.571	21.73
		120.00	10.95	0.1620	1.571	21.74
		180.00	13.42	0.1621	1.570	21.76

LOAD INCREMENT DATA

TEST NAME C1-U3D

PAGE NO: 27

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-06-97	12:18:50	0.00	0.00	0.1620	1.571	21.74
		0.10	0.32	0.1683	1.543	22.58
		0.25	0.50	0.1687	1.541	22.64
		0.50	0.71	0.1691	1.540	22.69
		1.00	1.00	0.1695	1.538	22.74
		1.57	1.25	0.1697	1.537	22.77
		2.00	1.41	0.1698	1.537	22.78
		2.25	1.50	0.1698	1.537	22.78
		3.07	1.75	0.1699	1.536	22.80
		4.00	2.00	0.1700	1.535	22.82
		6.25	2.50	0.1703	1.534	22.85
		8.00	2.83	0.1704	1.534	22.86
		12.25	3.50	0.1708	1.532	22.91
		15.00	3.87	0.1709	1.532	22.93
		22.00	4.69	0.1711	1.531	22.96
		30.00	5.48	0.1713	1.530	22.99
		36.00	6.00	0.1715	1.529	23.01
		49.00	7.00	0.1717	1.528	23.04
		60.00	7.75	0.1718	1.528	23.05
		120.00	10.95	0.1725	1.525	23.15
		180.00	13.42	0.1730	1.522	23.21
		240.00	15.49	0.1733	1.521	23.26

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 28

JO: 05996.01

PRESSURE INCREMENT FROM 8.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-06-97	16:36:27	0.00	0.00	0.1733	1.521	23.26
		0.10	0.32	0.1678	1.545	22.52
		0.25	0.50	0.1675	1.547	22.47
		0.50	0.71	0.1673	1.547	22.45
		1.00	1.00	0.1672	1.548	22.44
		1.57	1.25	0.1672	1.548	22.44
		2.00	1.41	0.1671	1.548	22.42
		2.25	1.50	0.1671	1.548	22.42
		3.07	1.75	0.1671	1.548	22.42
		4.00	2.00	0.1671	1.548	22.42
		6.25	2.50	0.1670	1.549	22.41
		8.00	2.83	0.1670	1.549	22.41
		12.25	3.50	0.1669	1.550	22.39
		15.00	3.87	0.1669	1.550	22.39
		22.00	4.69	0.1669	1.550	22.39
		30.00	5.48	0.1667	1.550	22.37
		36.00	6.00	0.1667	1.550	22.37
		49.00	7.00	0.1667	1.550	22.37
		60.00	7.75	0.1667	1.550	22.37
		120.00	10.95	0.1666	1.551	22.36
		180.00	13.42	0.1666	1.551	22.36
		240.00	15.49	0.1666	1.551	22.36
		300.00	17.32	0.1666	1.551	22.36
		360.00	18.97	0.1666	1.551	22.36
		420.00	20.49	0.1665	1.551	22.34
01-07-97	00:36:27	480.00	21.91	0.1665	1.551	22.34
		540.00	23.24	0.1665	1.551	22.34
		600.00	24.49	0.1665	1.551	22.34
		660.00	25.69	0.1665	1.551	22.34
		720.00	26.83	0.1665	1.551	22.34
		780.00	27.93	0.1665	1.551	22.34
		840.00	28.98	0.1666	1.551	22.36
		900.00	30.00	0.1666	1.551	22.36

LOAD INCREMENT DATA

TEST NAME C1-U3D

TESTED BY: ACS

PAGE NO: 29

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	08:02:44	0.00	0.00	0.1665	1.551	22.34
		0.10	0.32	0.1614	1.573	21.66
		0.25	0.50	0.1605	1.578	21.54
		0.50	0.71	0.1599	1.580	21.46
		1.00	1.00	0.1596	1.582	21.41
		1.57	1.25	0.1593	1.583	21.38
		2.00	1.41	0.1592	1.583	21.36
		2.25	1.50	0.1592	1.583	21.36
		3.07	1.75	0.1591	1.584	21.35
		4.00	2.00	0.1590	1.584	21.33
		6.25	2.50	0.1587	1.585	21.30
		8.00	2.83	0.1587	1.585	21.30
		12.25	3.50	0.1586	1.586	21.28
		15.00	3.87	0.1585	1.586	21.27
		22.00	4.69	0.1584	1.587	21.25
		30.00	5.48	0.1583	1.587	21.24
		36.00	6.00	0.1583	1.587	21.24
		49.00	7.00	0.1581	1.588	21.22
		60.00	7.75	0.1580	1.588	21.20

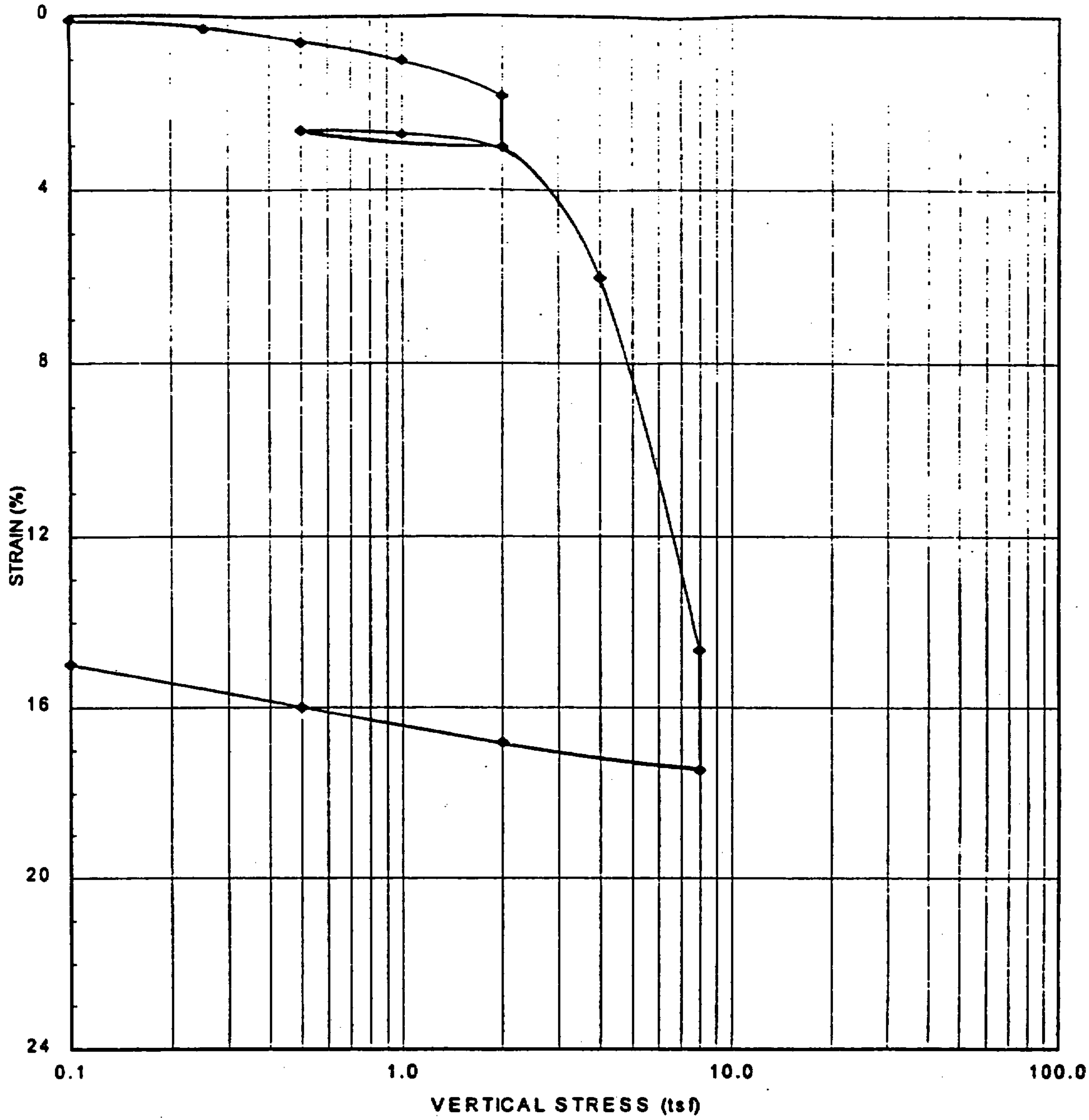
LOAD INCREMENT DATA

TEST NAME C1-U3D PAGE NO: 30

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	09:16:44	0.00	0.00	0.1580	1.588	21.20
		0.10	0.32	0.1540	1.606	20.67
		0.25	0.50	0.1523	1.614	20.43
		0.50	0.71	0.1506	1.621	20.21
		1.00	1.00	0.1492	1.627	20.02
		1.57	1.25	0.1485	1.631	19.92
		2.00	1.41	0.1481	1.632	19.88
		2.25	1.50	0.1480	1.633	19.86
		3.07	1.75	0.1477	1.634	19.81
		4.00	2.00	0.1473	1.636	19.76
		6.25	2.50	0.1469	1.637	19.72
		8.00	2.83	0.1467	1.638	19.69
		12.25	3.50	0.1464	1.640	19.64
		15.00	3.87	0.1461	1.641	19.61
		22.00	4.69	0.1459	1.642	19.57
		30.00	5.48	0.1457	1.643	19.54
		36.00	6.00	0.1455	1.644	19.53
		49.00	7.00	0.1453	1.645	19.50
		60.00	7.75	0.1451	1.646	19.46
		120.00	10.95	0.1446	1.648	19.40
		180.00	13.42	0.1442	1.649	19.35



SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/17/96
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	10.9 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

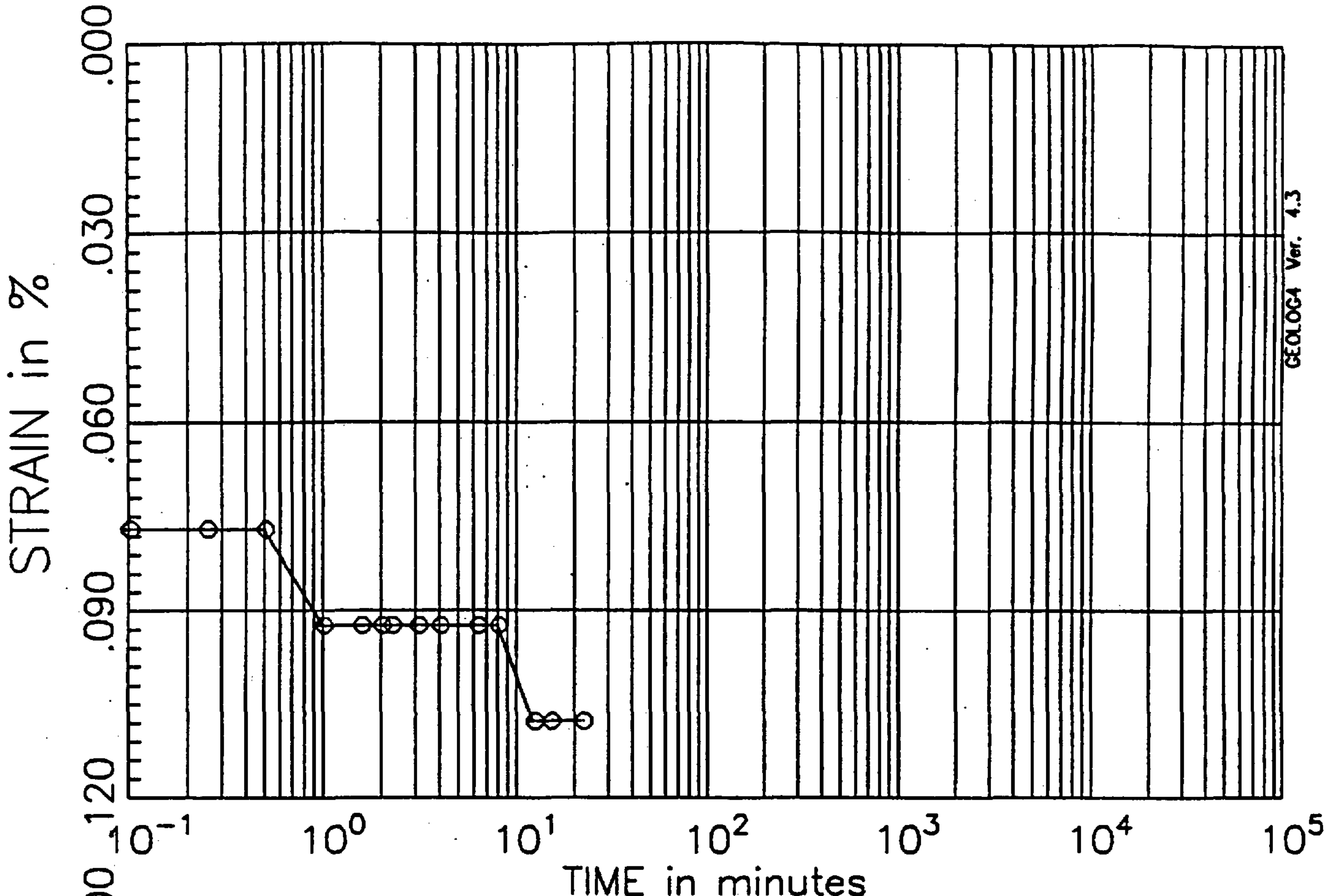
SPECIMEN INFORMATION:

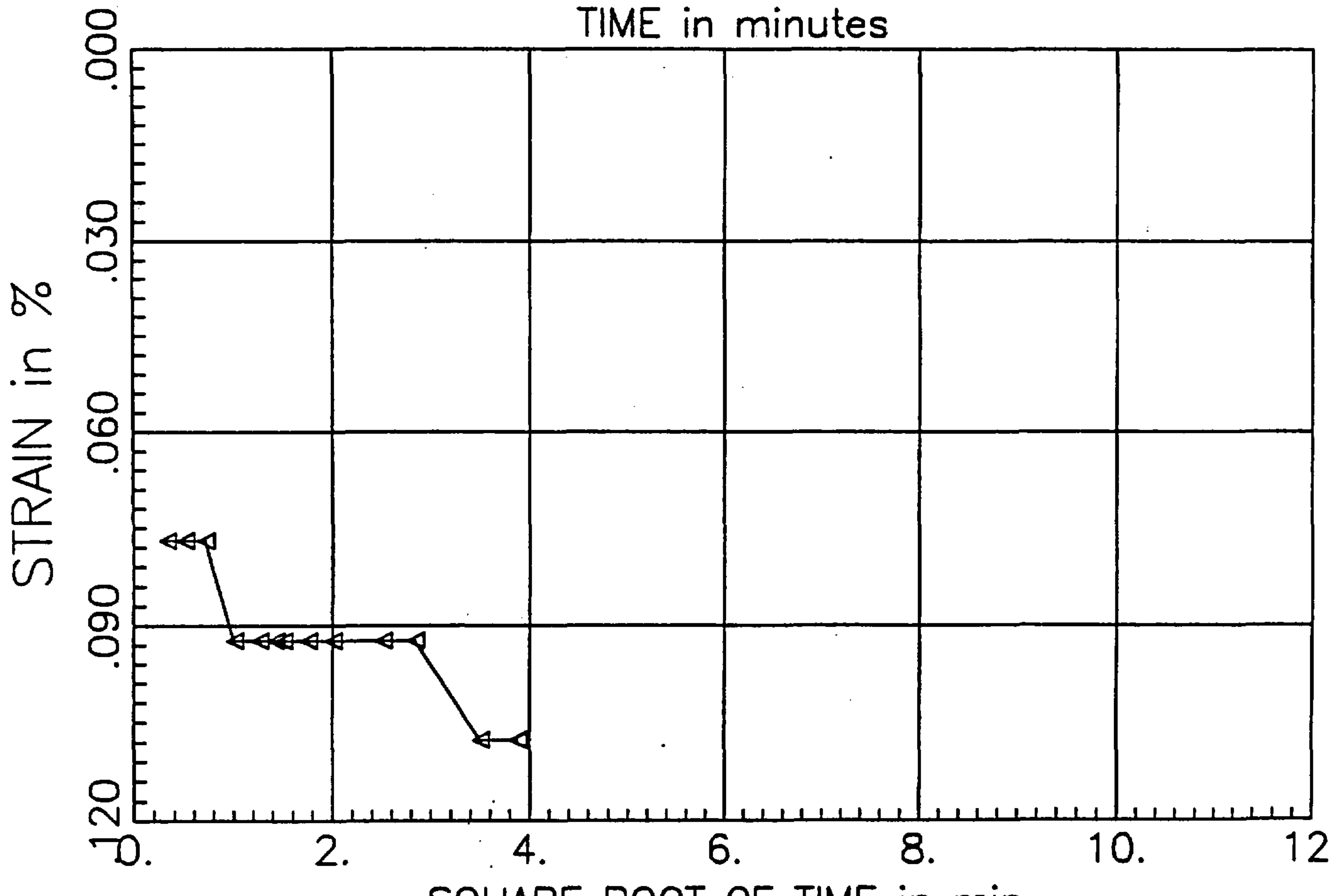
	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	27.6 %	44.2 %	2.72 (est)
DRY UNIT WEIGHT:	64.9 pcf	76.2 pcf	
VOID RATIO:	1.615	1.230	
SATURATION:	46.4 %	97.7 %	

NOTE: Sample was inundated when the applied pressure was 0.5 tsf.

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

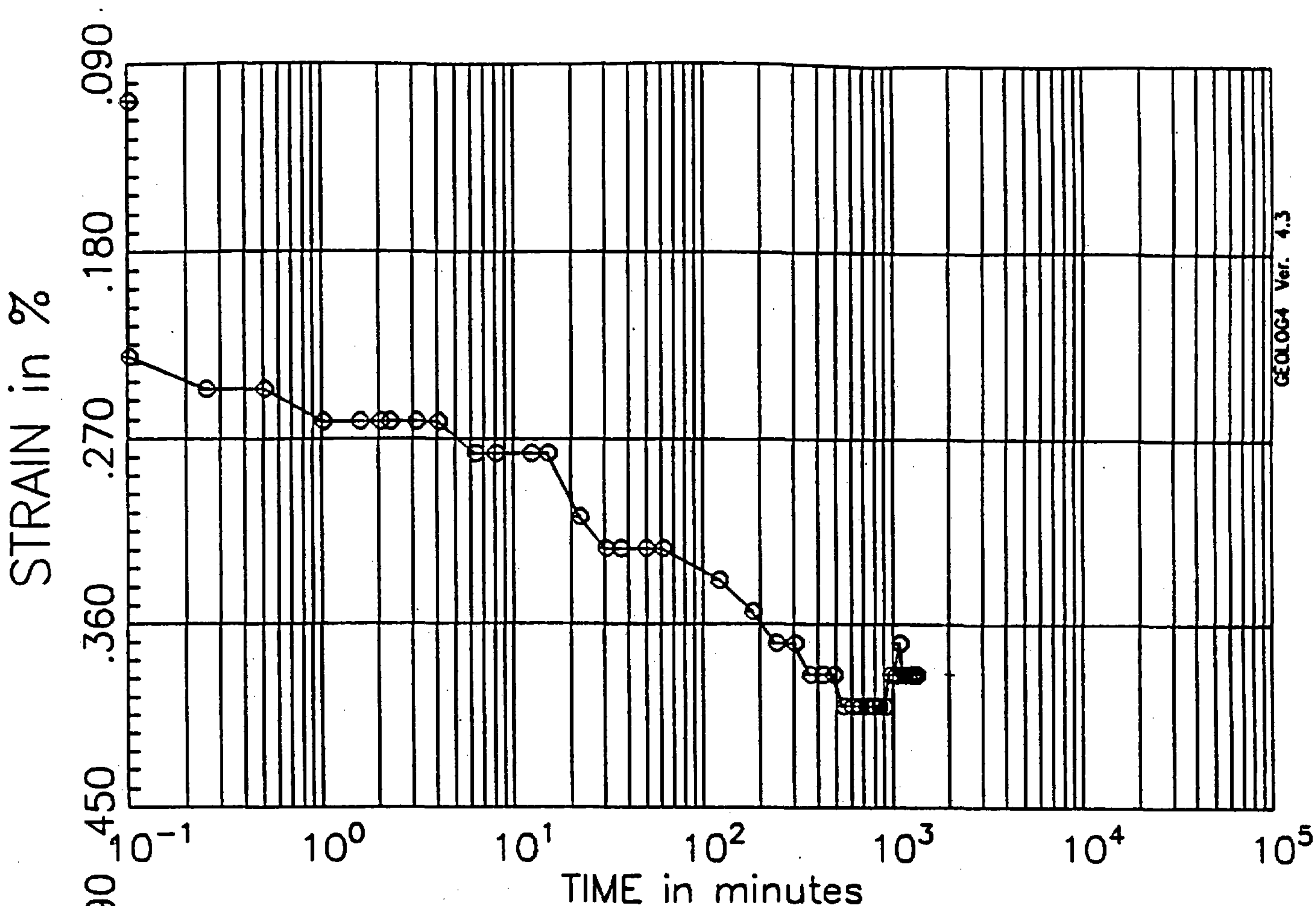
STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING C-2, SAMPLE U-2C	JO 05996.01 January 1997

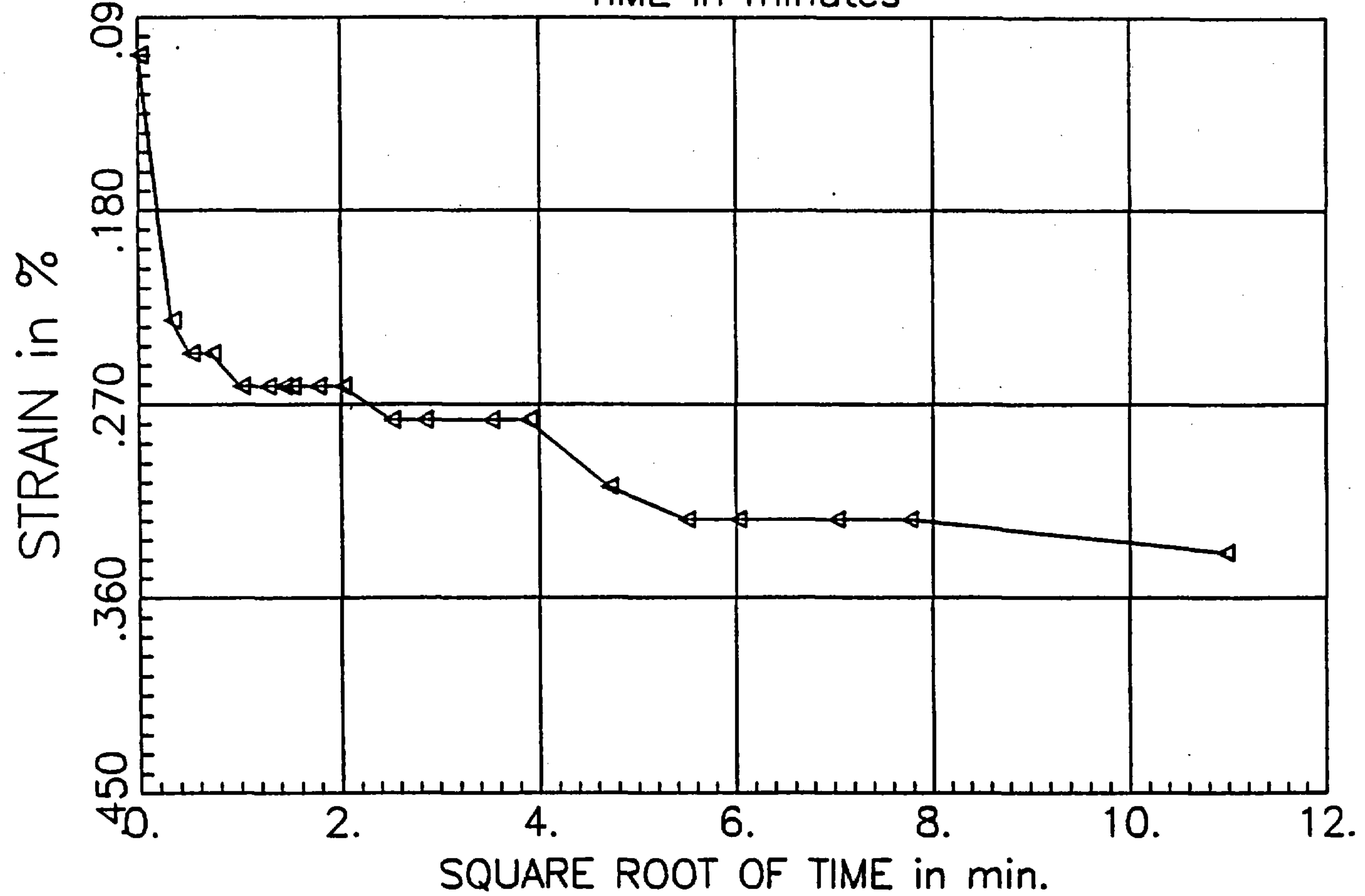




PRESSURE INCREMENT
from 0.00 tsf to 0.10 tsf

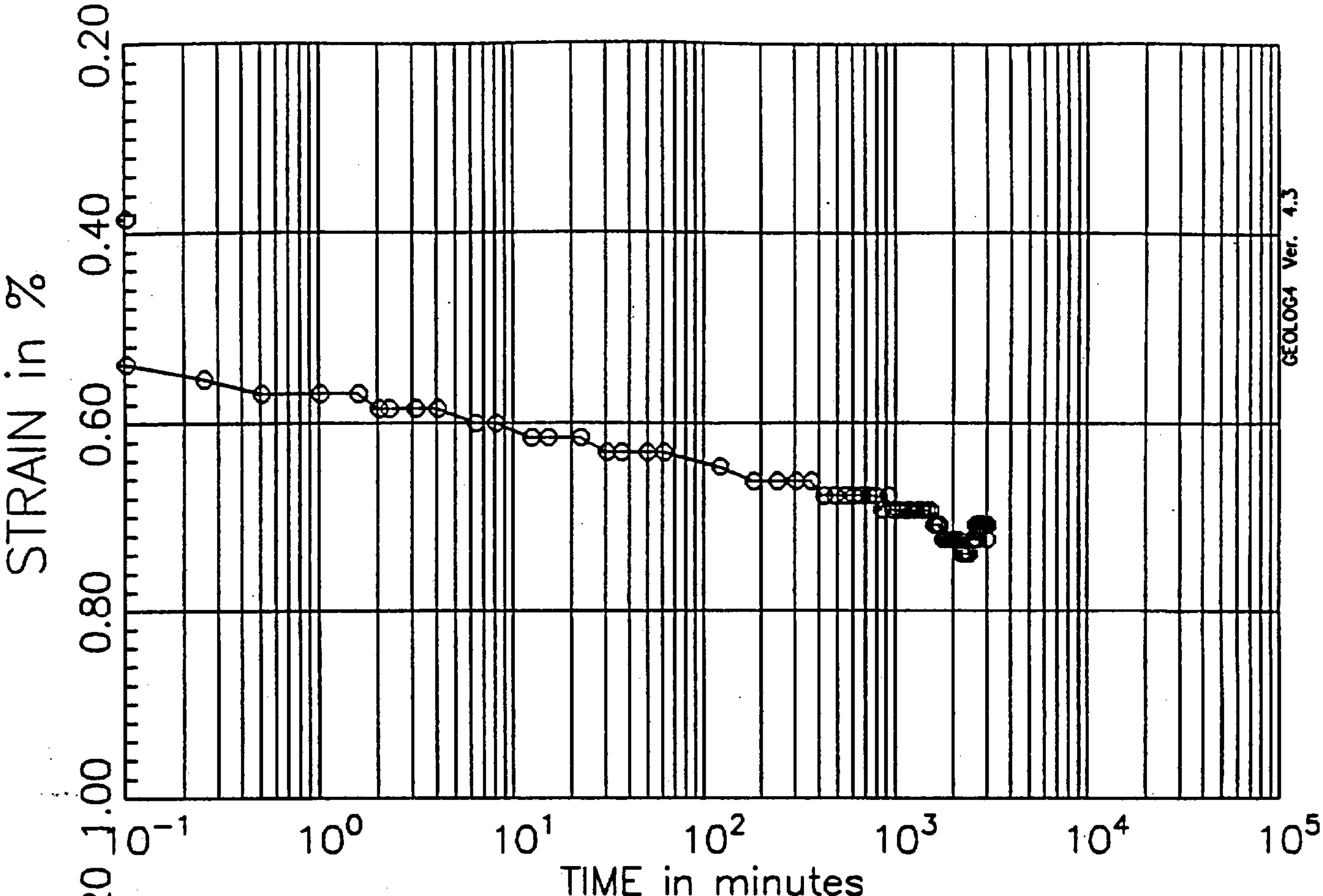
Test No: 3
Testname: C2–U2C

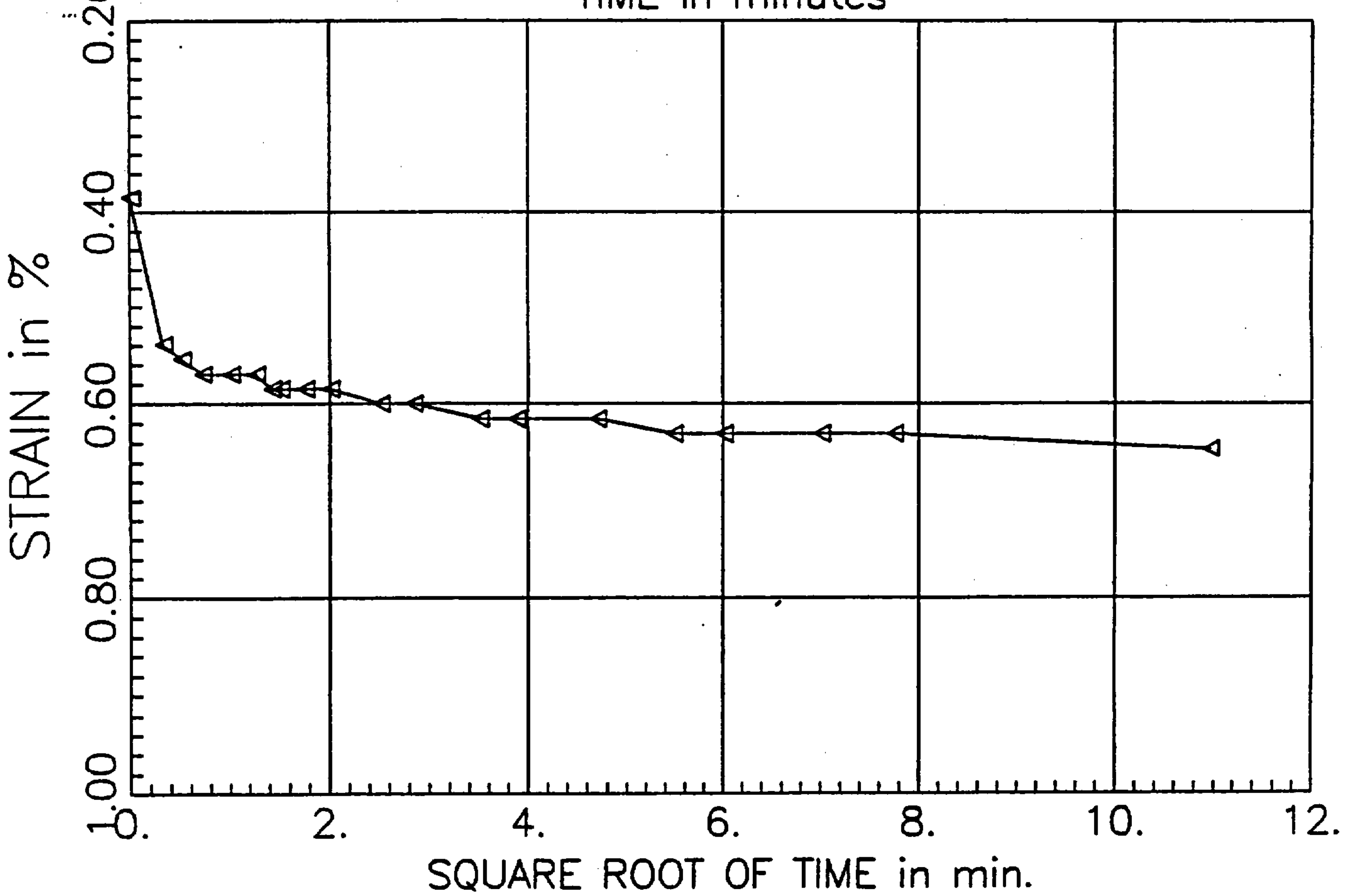




PRESSURE INCREMENT
from 0.10 tsf to 0.25 tsf

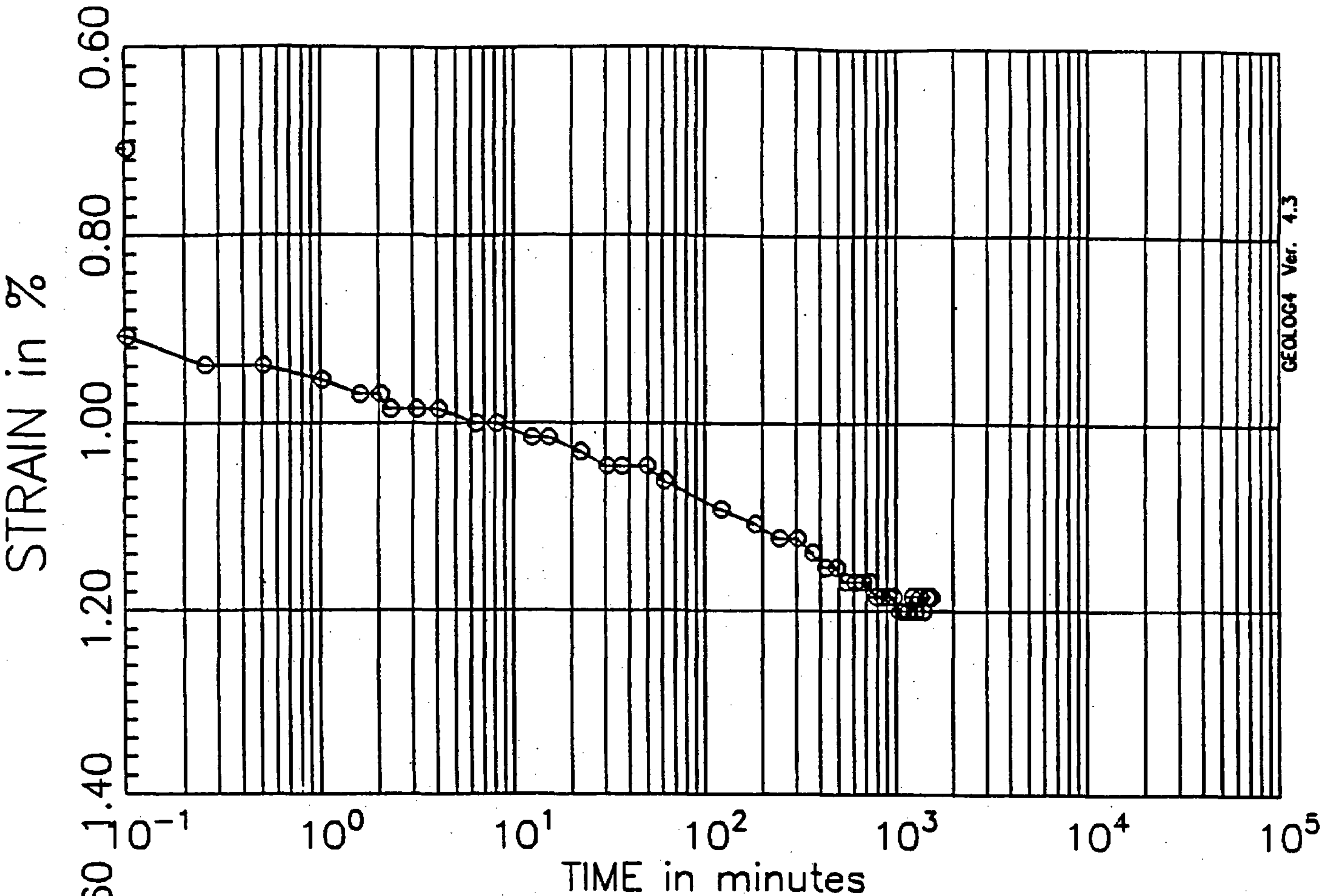
Test No: 3
Testname: C2-U2C

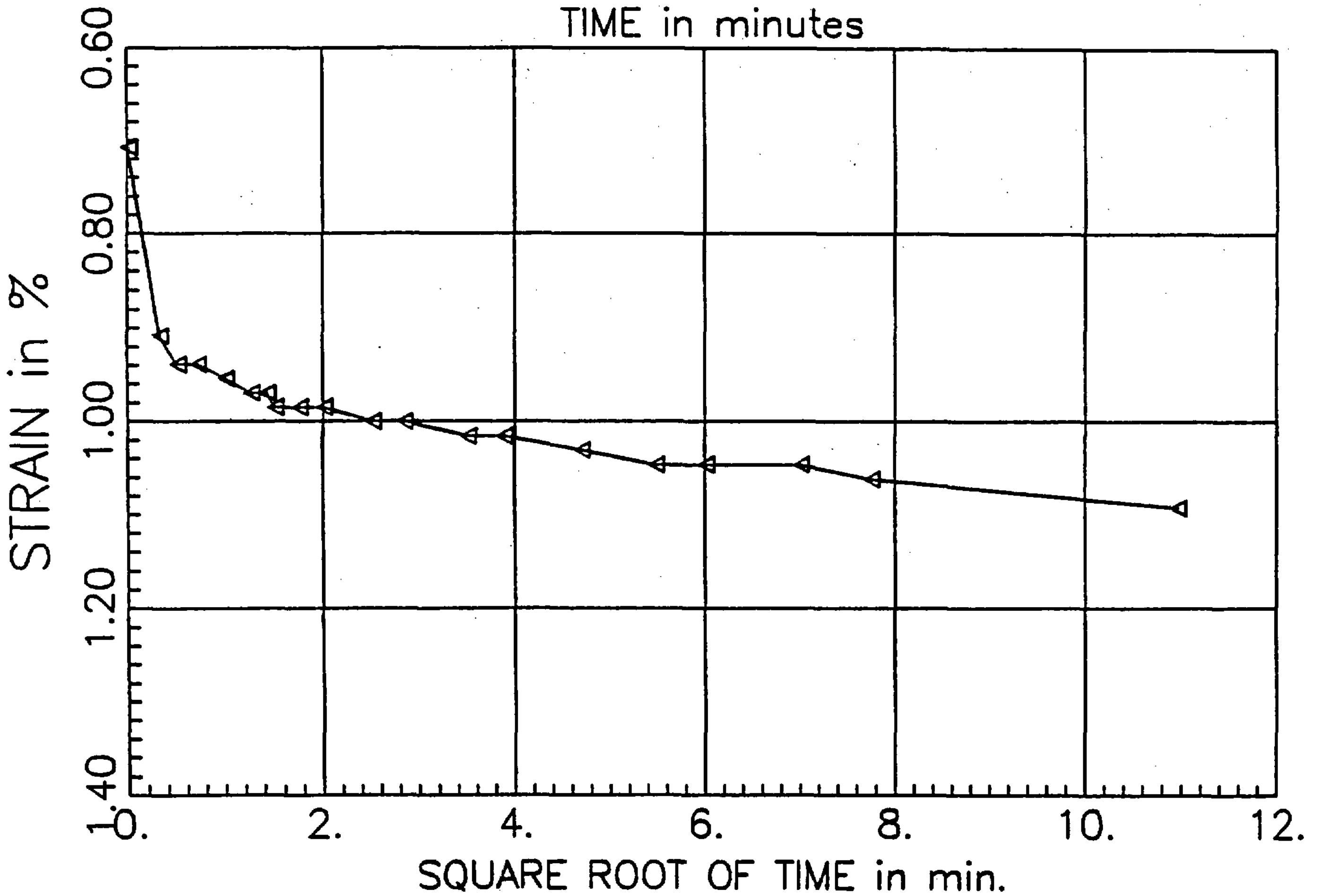




PRESSURE INCREMENT
from 0.25 tsf to 0.50 tsf

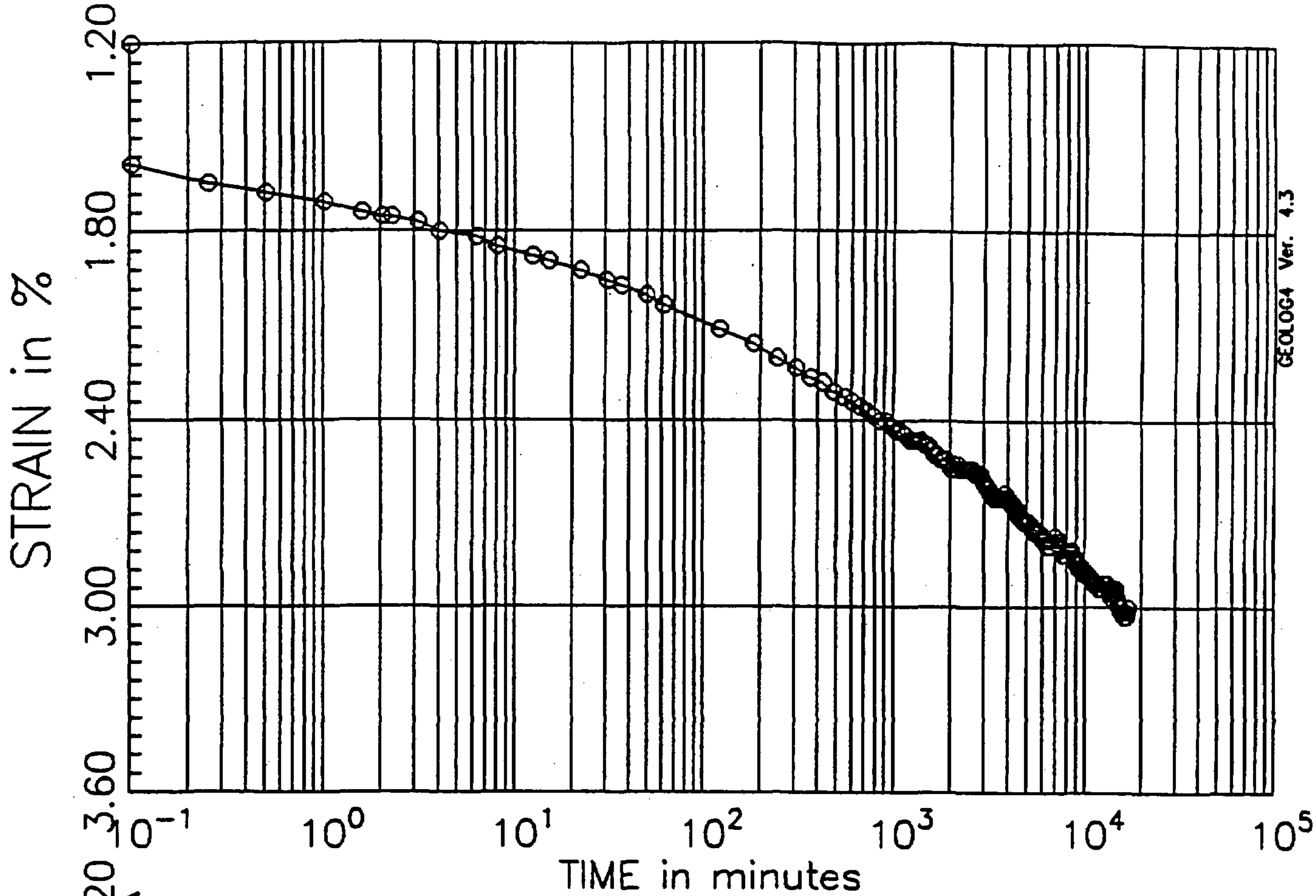
Test No: 3
Testname: C2-U2C

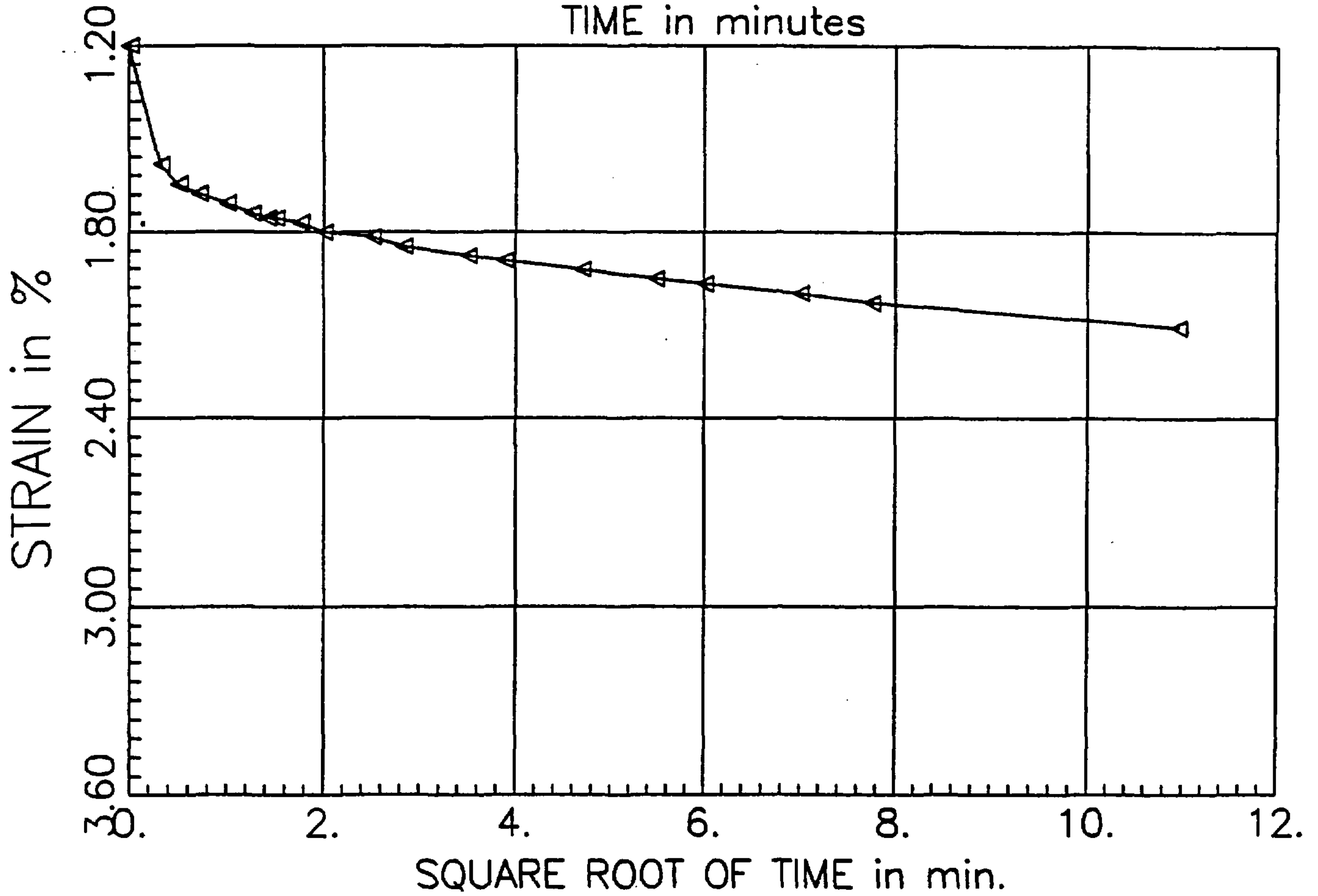




PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

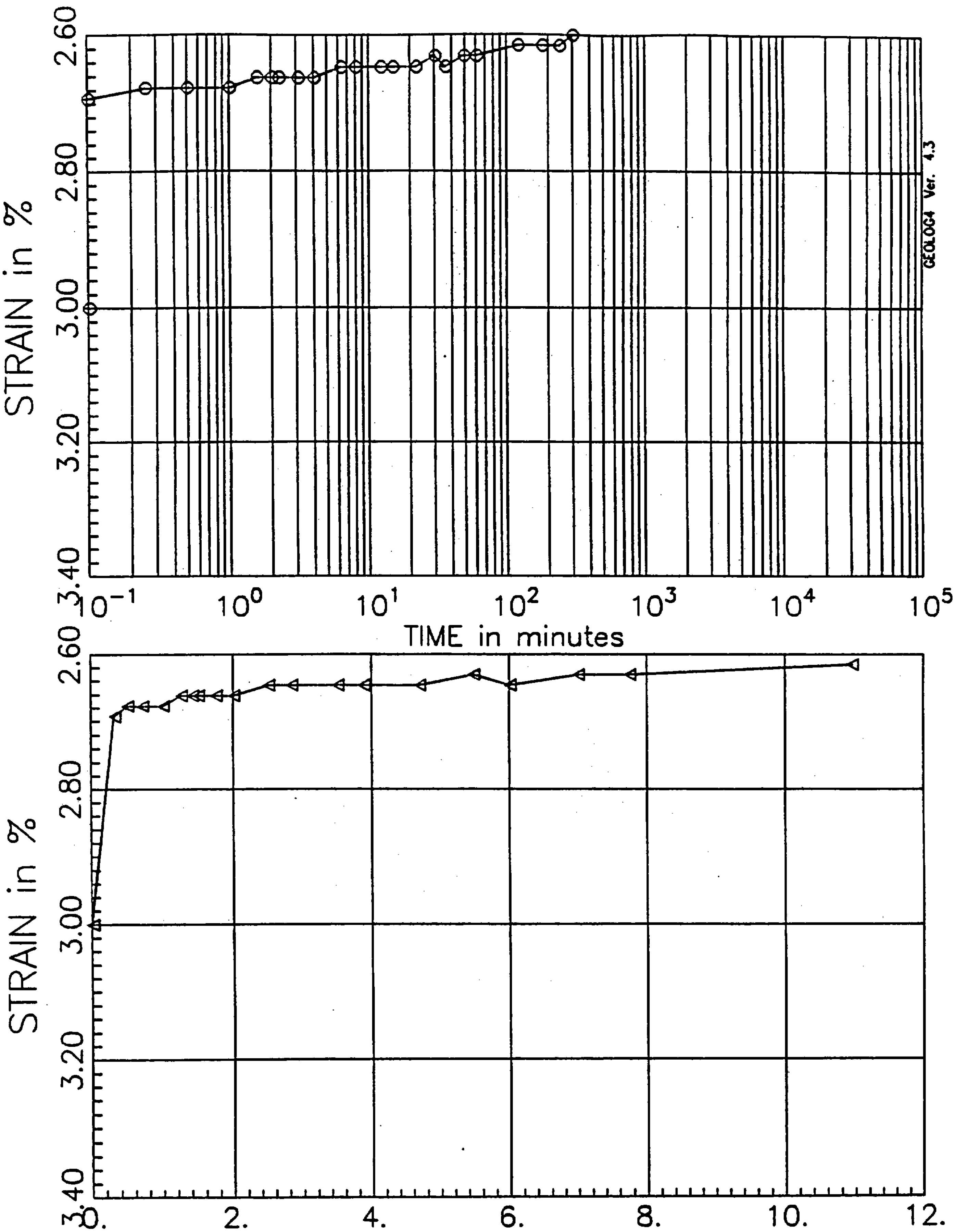
Test No: 3
Testname: C2-U2C





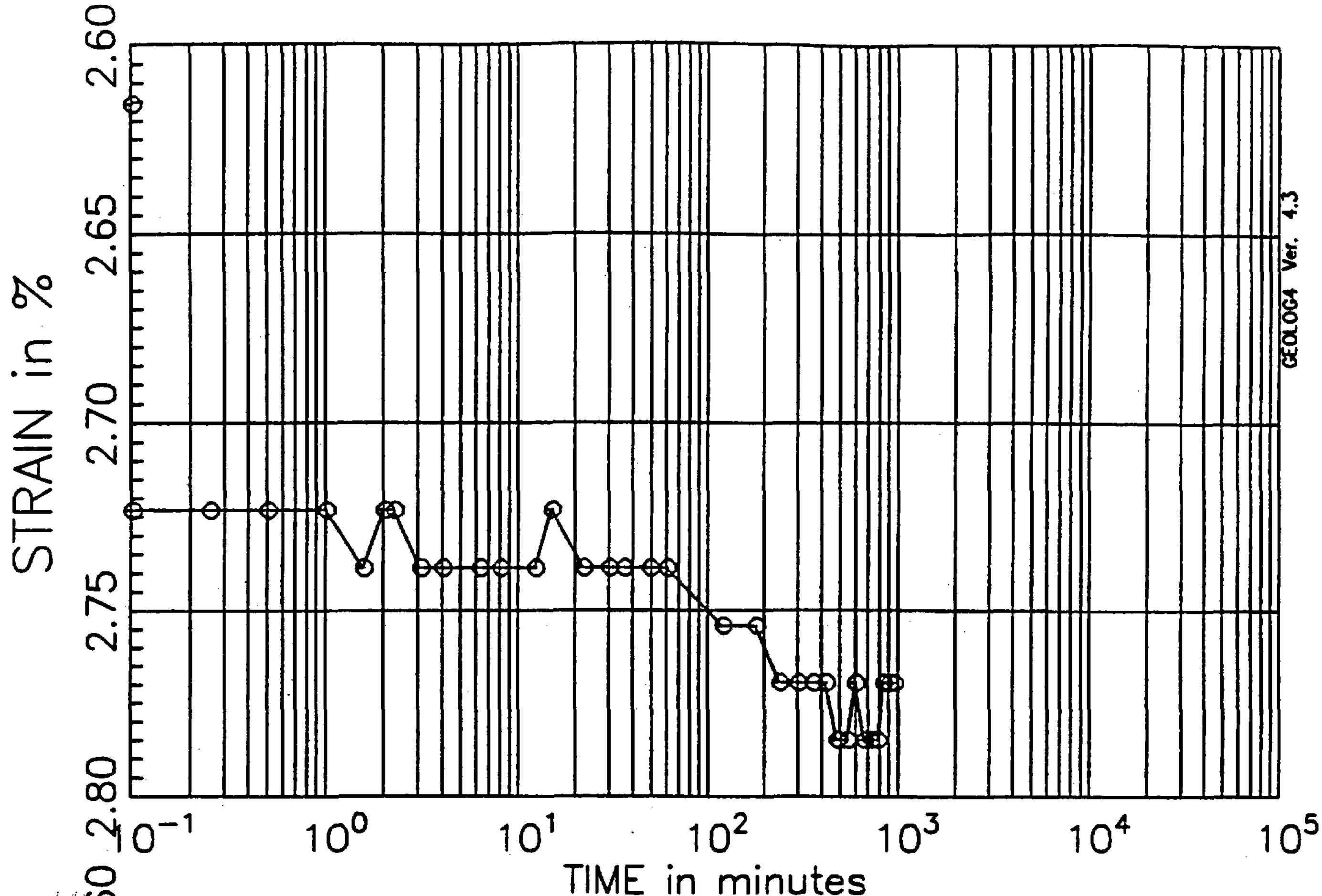
PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

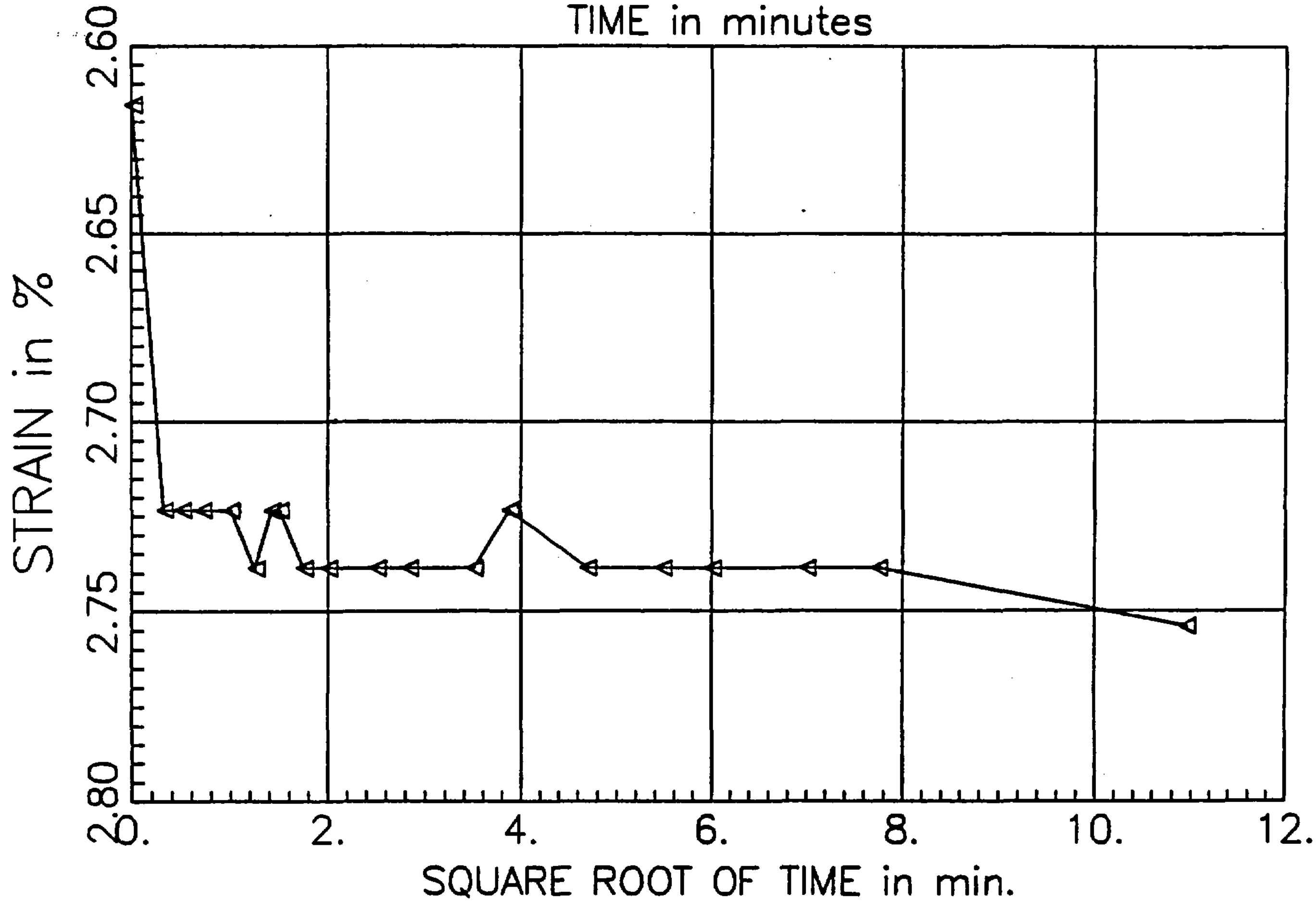
Test No: 3
Testname: C2–U2C



PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

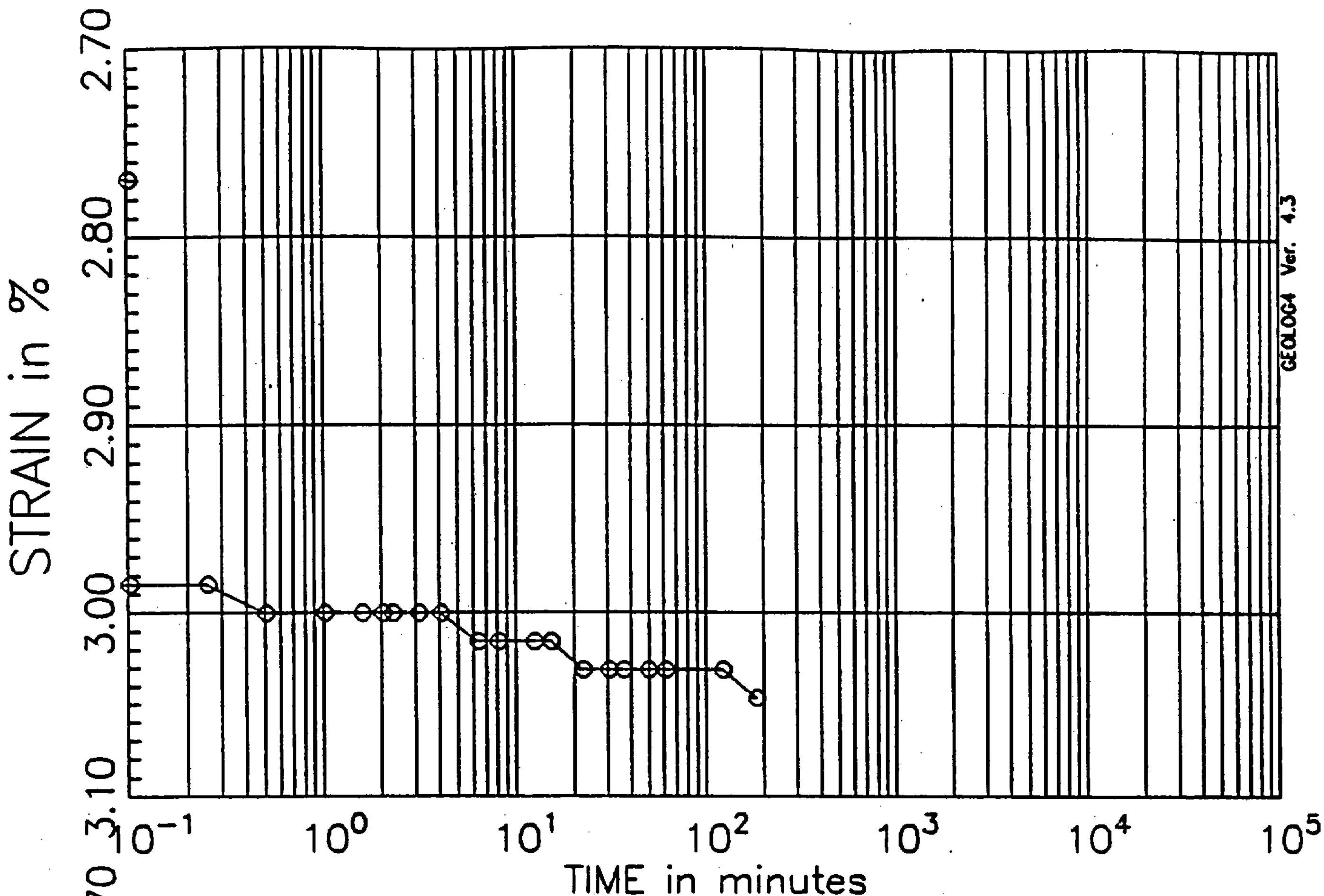
Test No: 3
Testname: C2–U2C

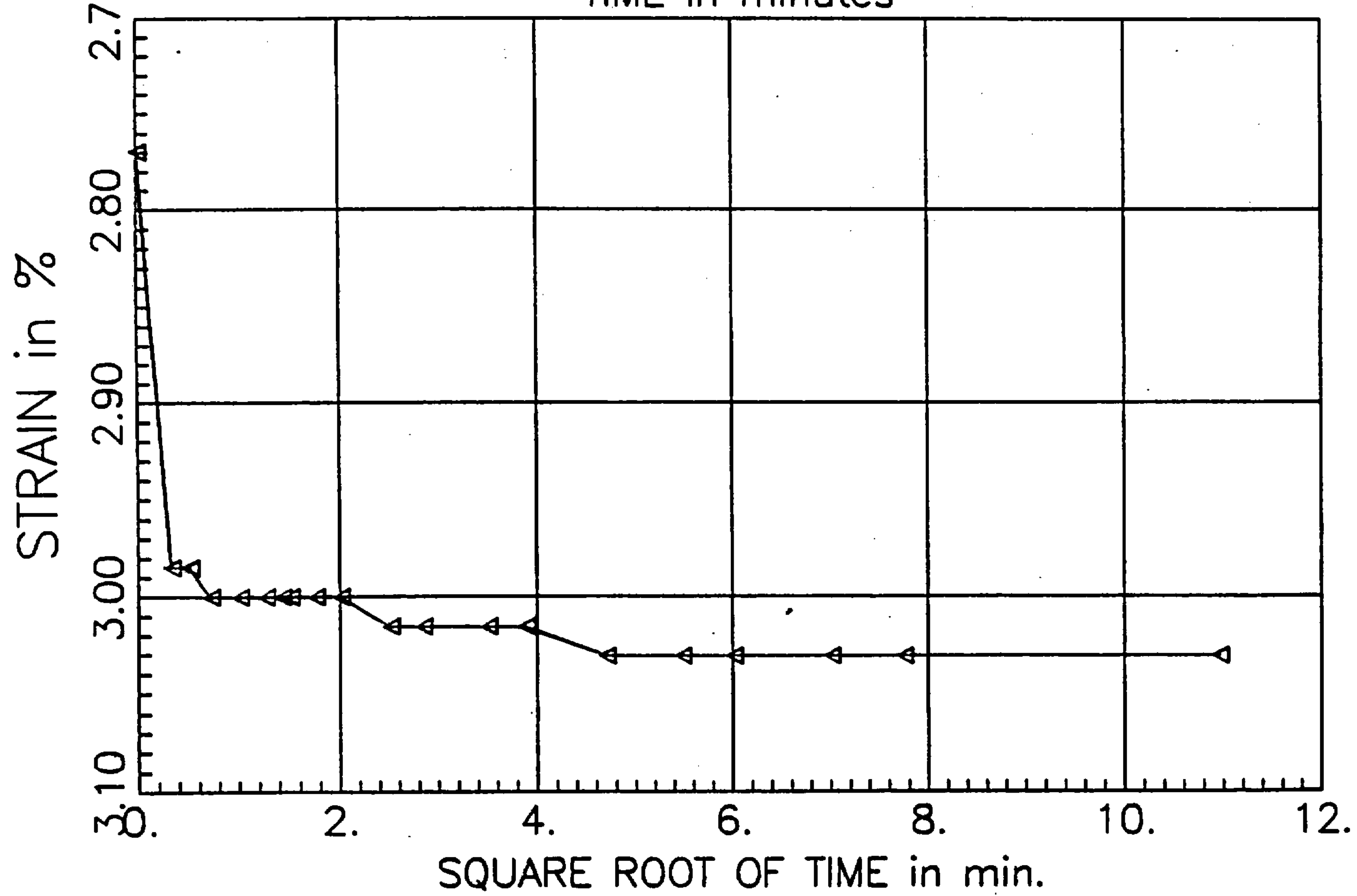




PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

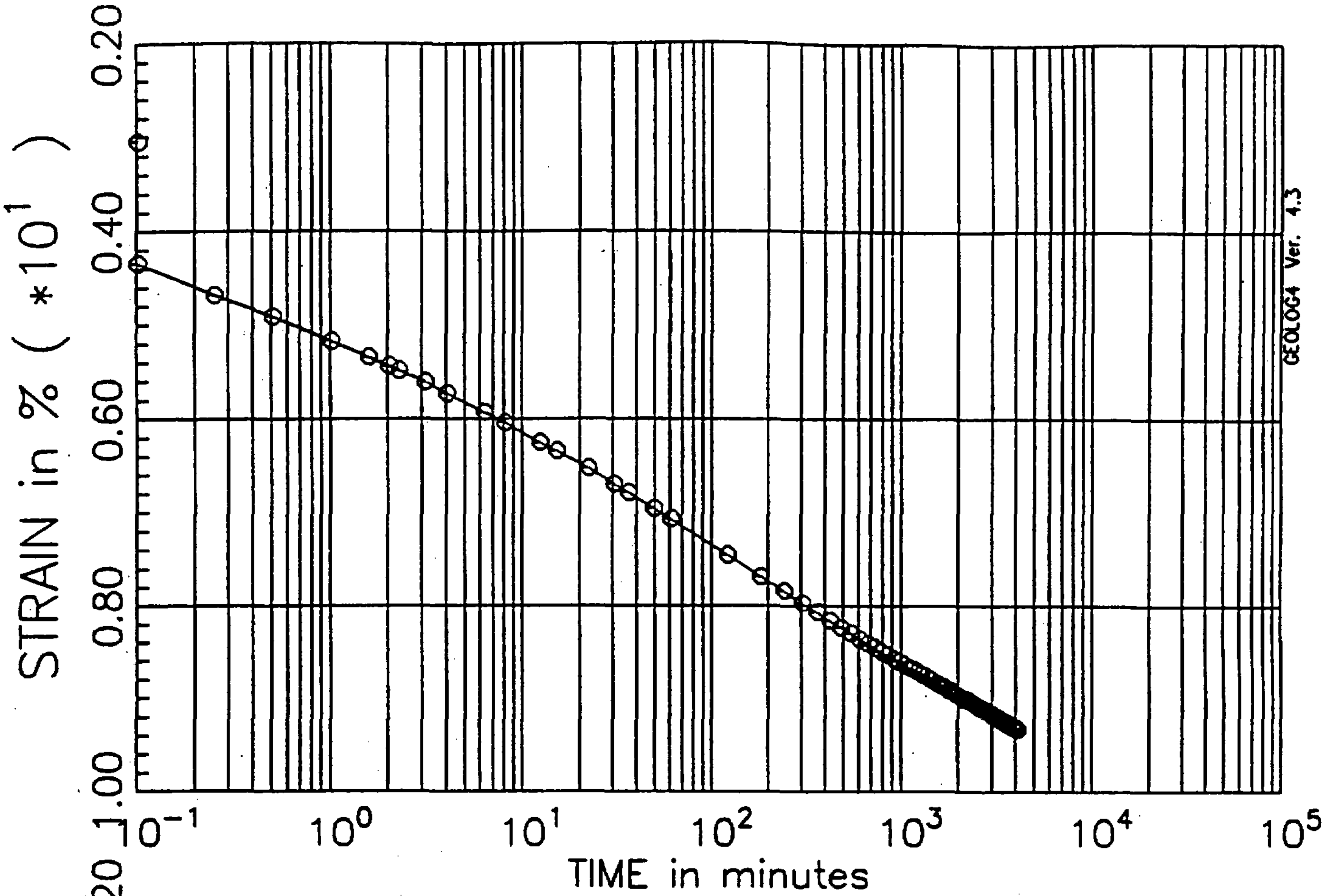
Test No: 3
Testname: C2-U2C

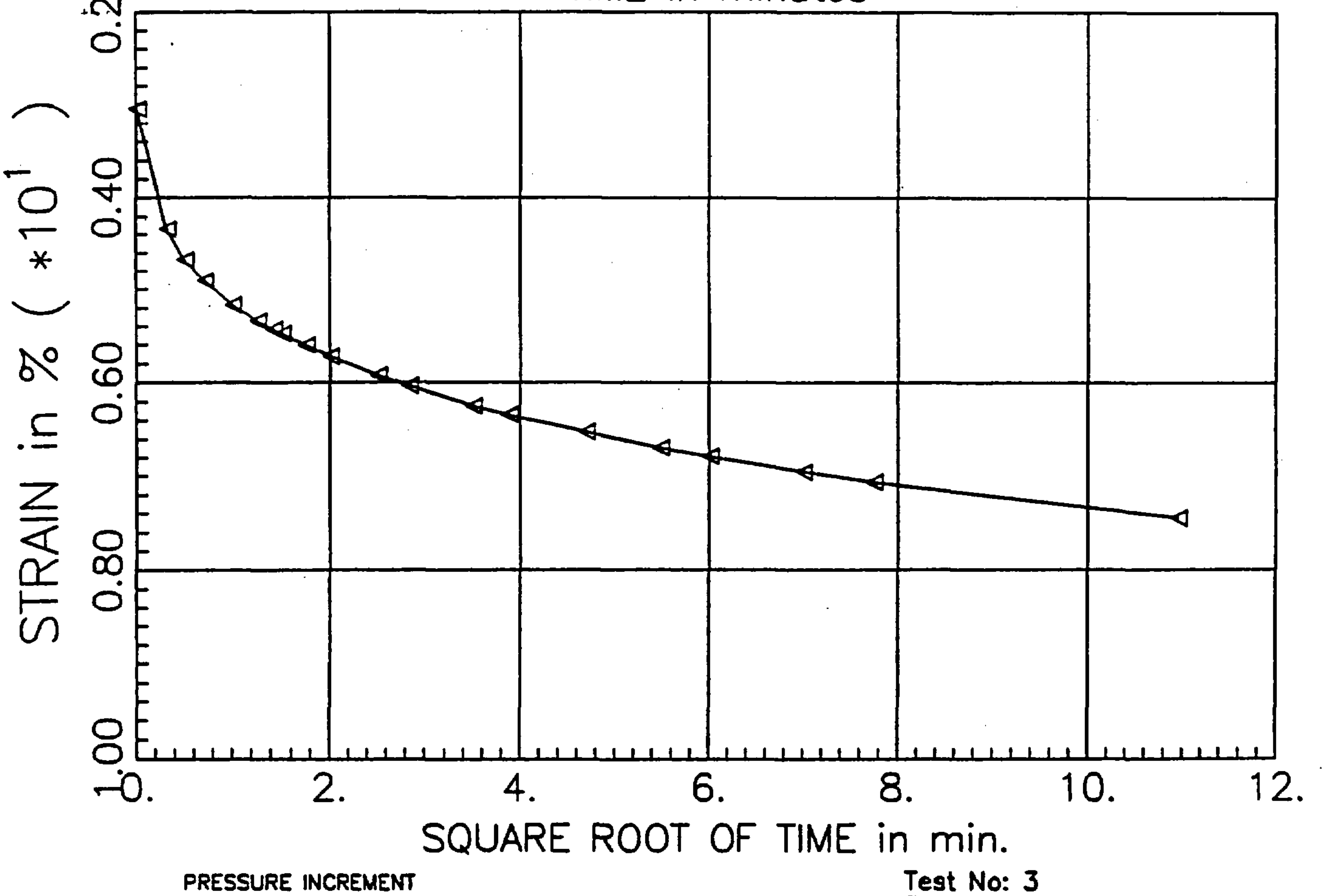




PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

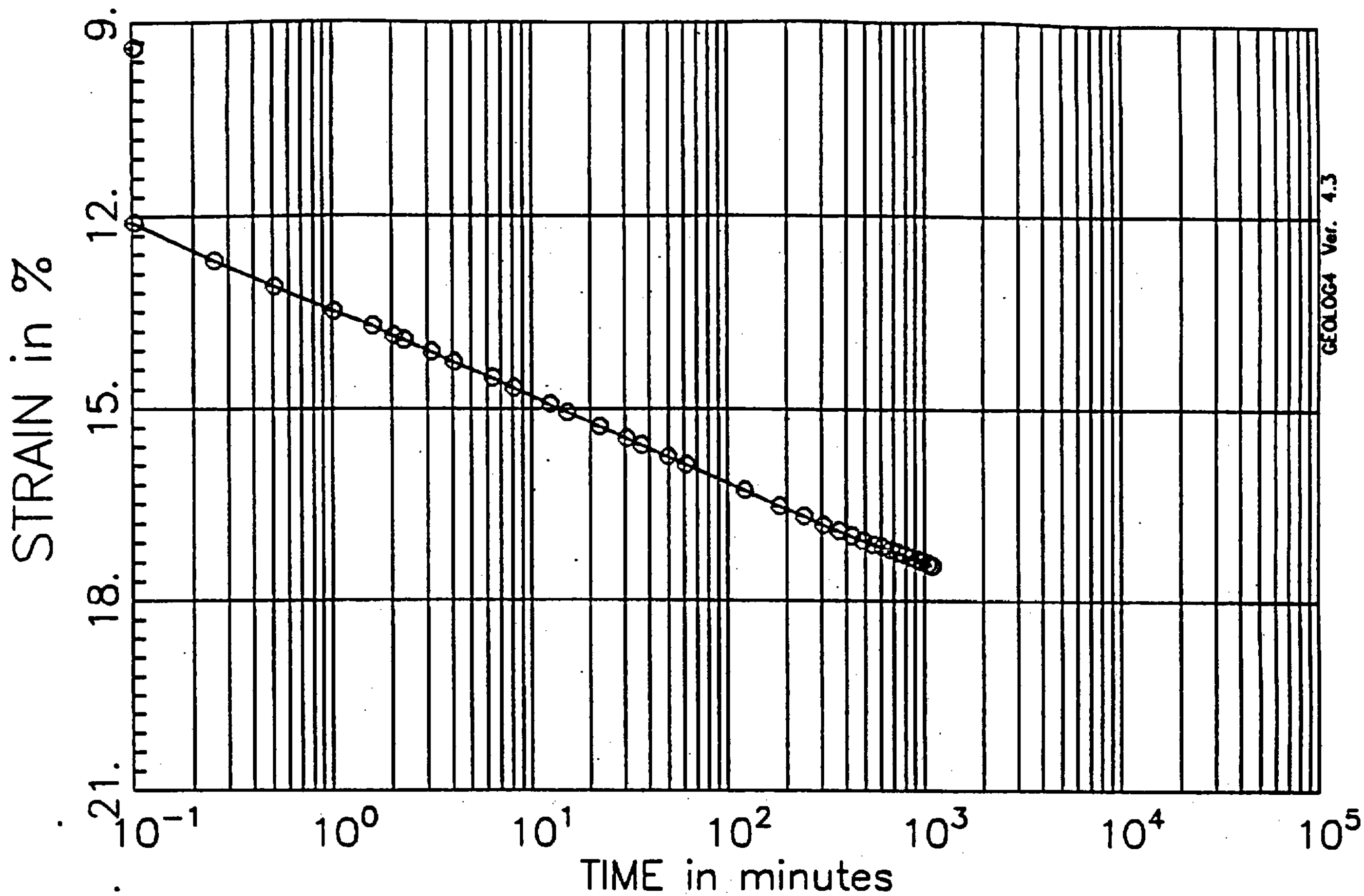
Test No: 3
Testname: C2–U2C

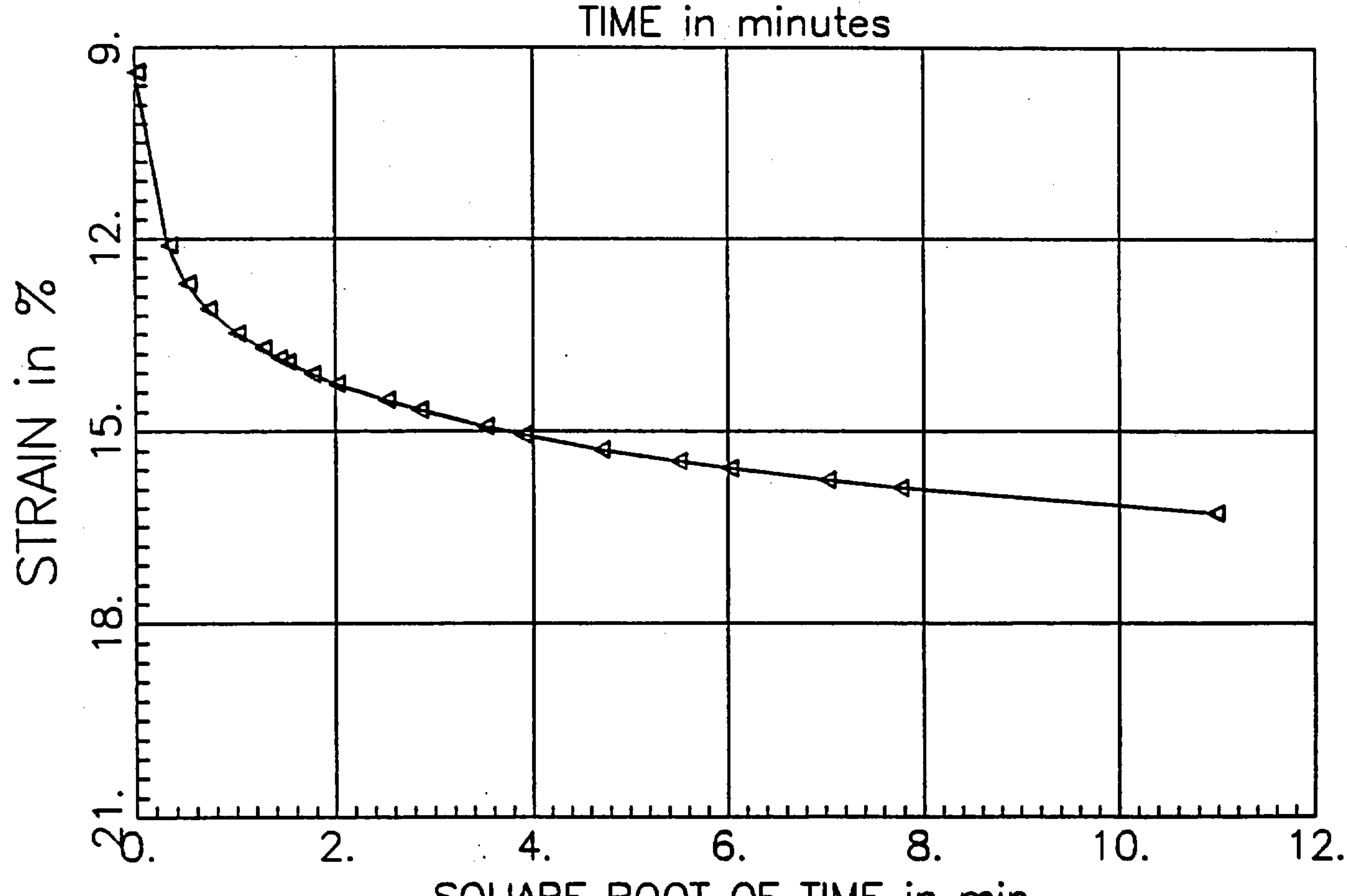




PRESSURE INCREMENT
from 2.00 tsf to 4.00 tsf

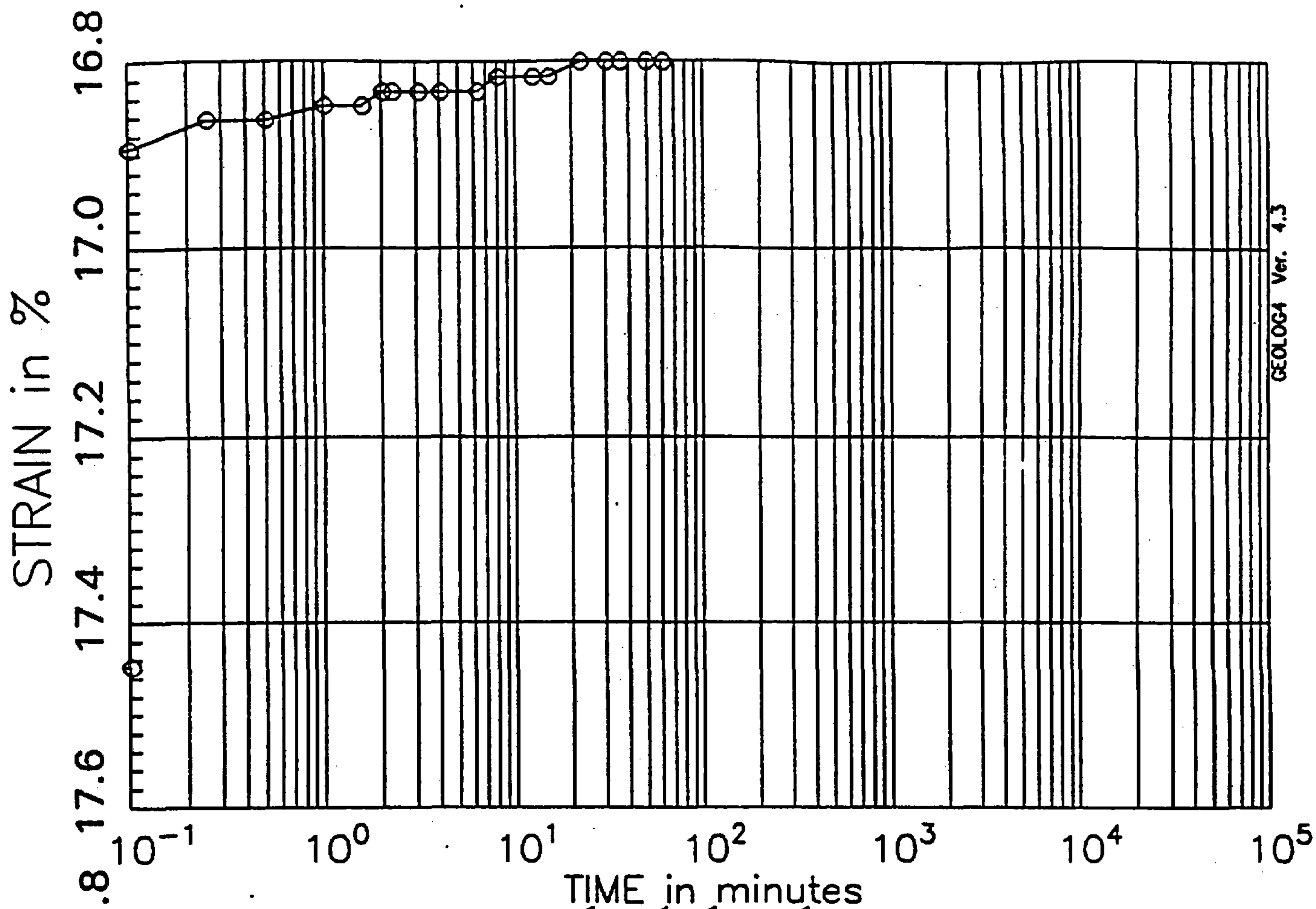
Test No: 3
Testname: C2–U2C

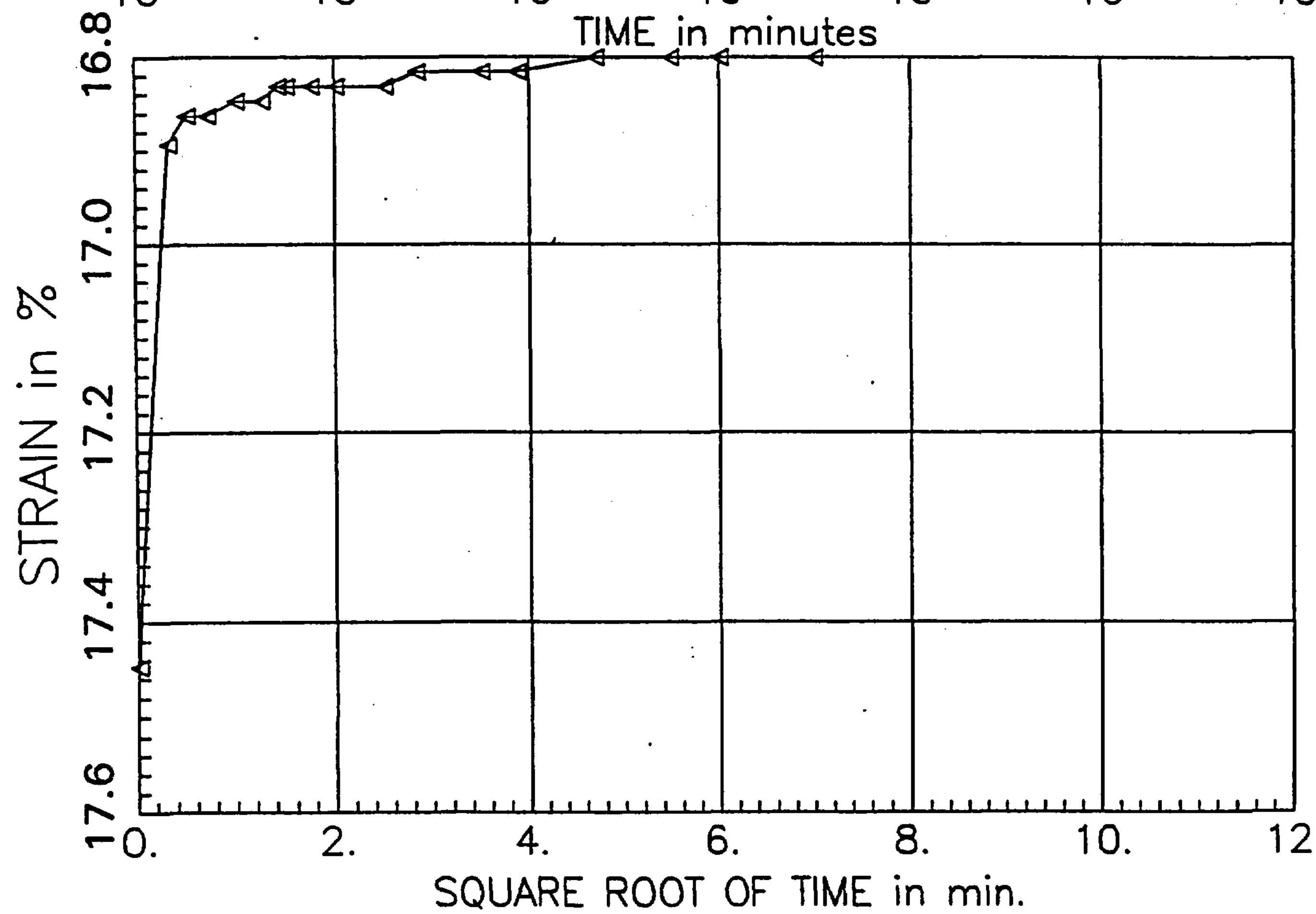




PRESSURE INCREMENT
from 4.00 tsf to 8.00 tsf

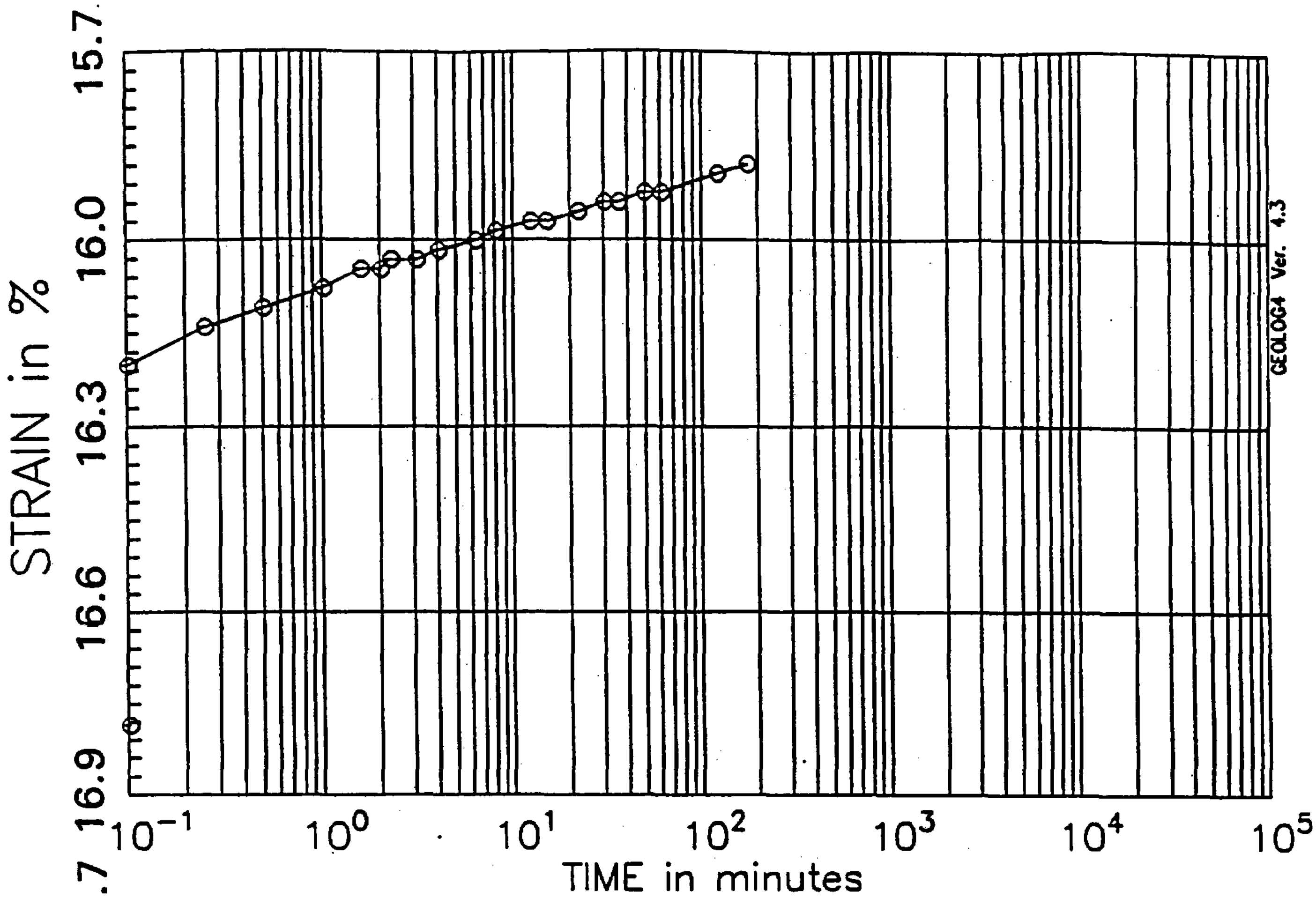
Test No: 3
Testname: C2–U2C

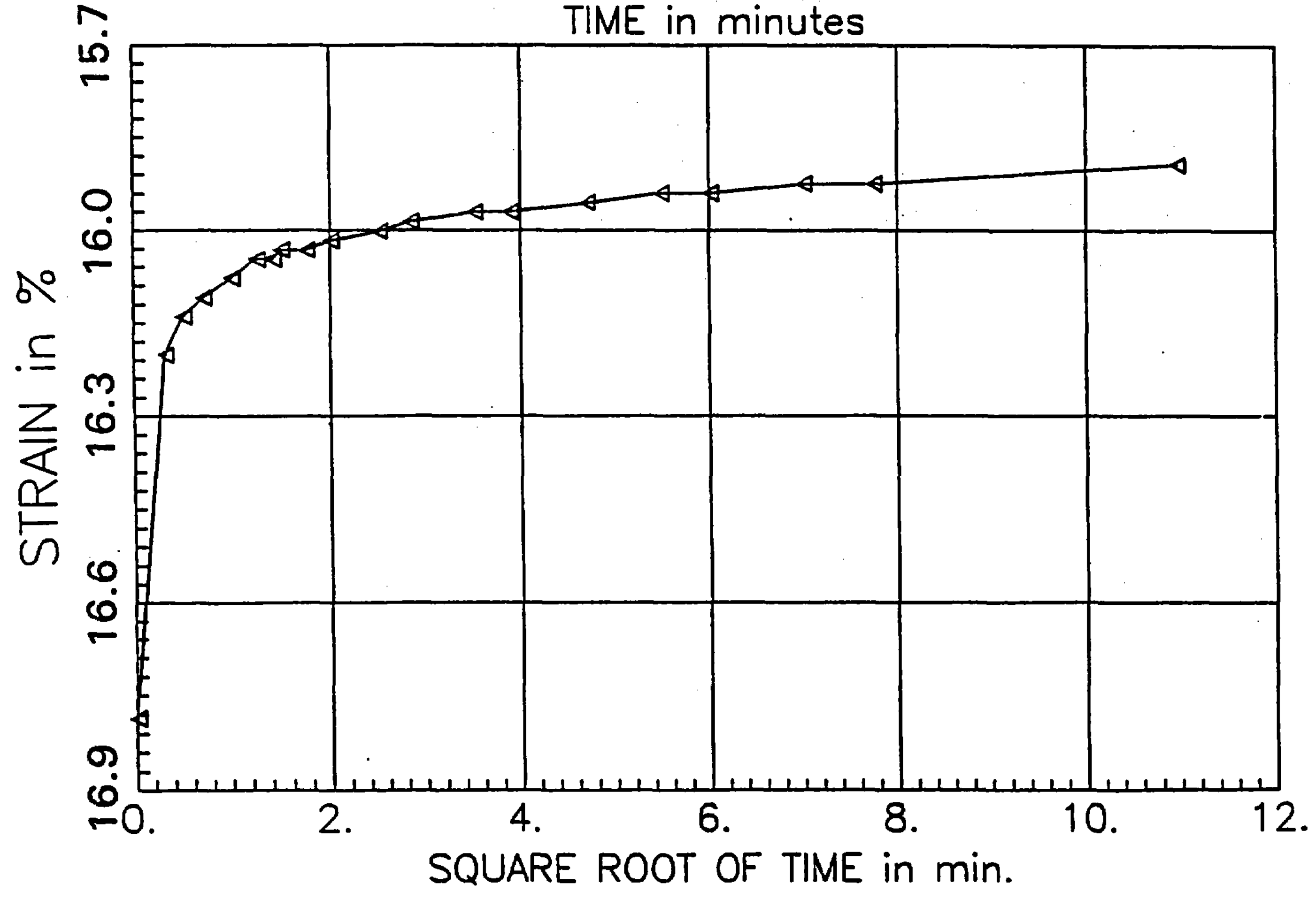




PRESSURE INCREMENT
from 8.00 tsf to 2.00 tsf

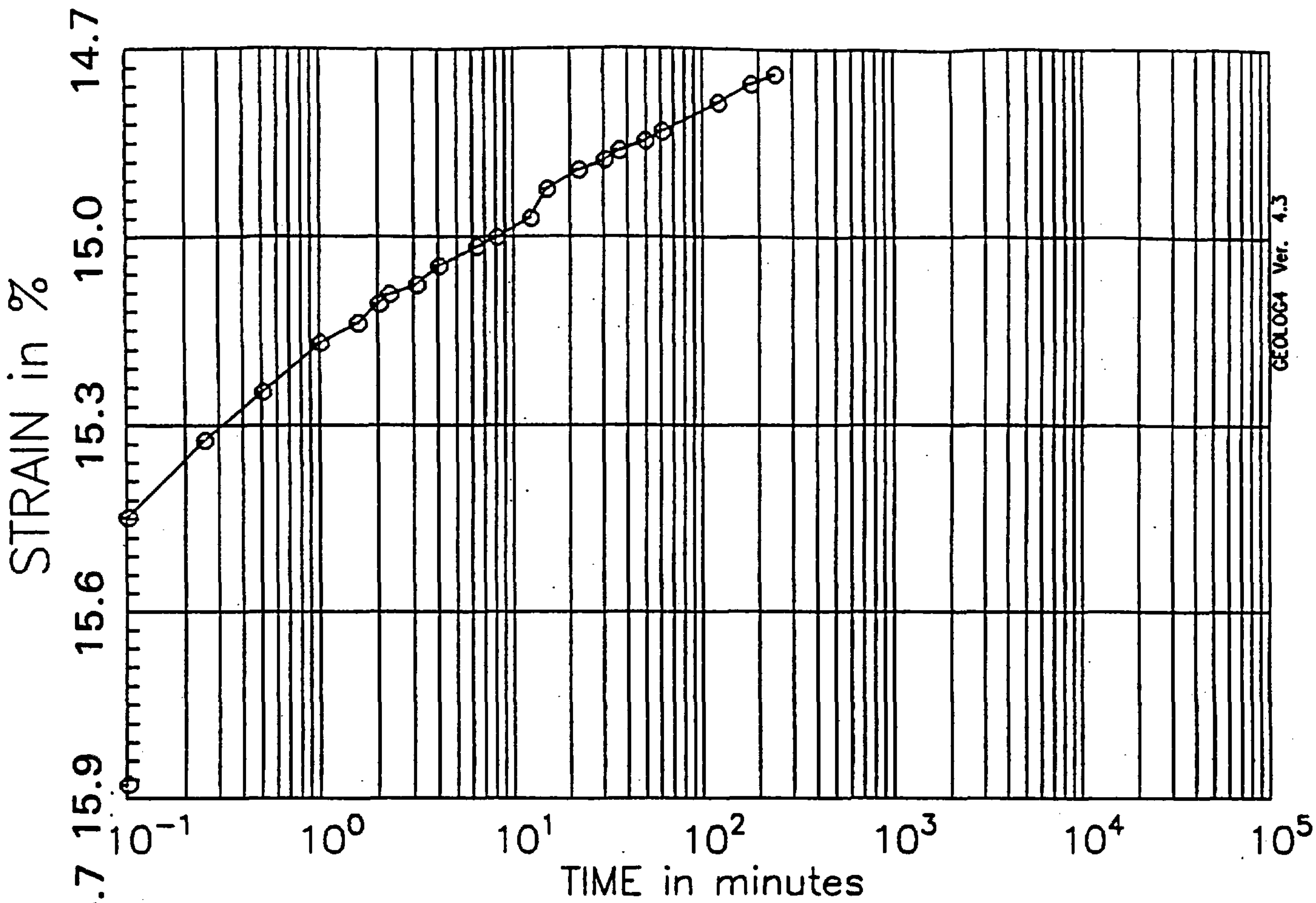
Test No: 3
Testname: C2-U2C

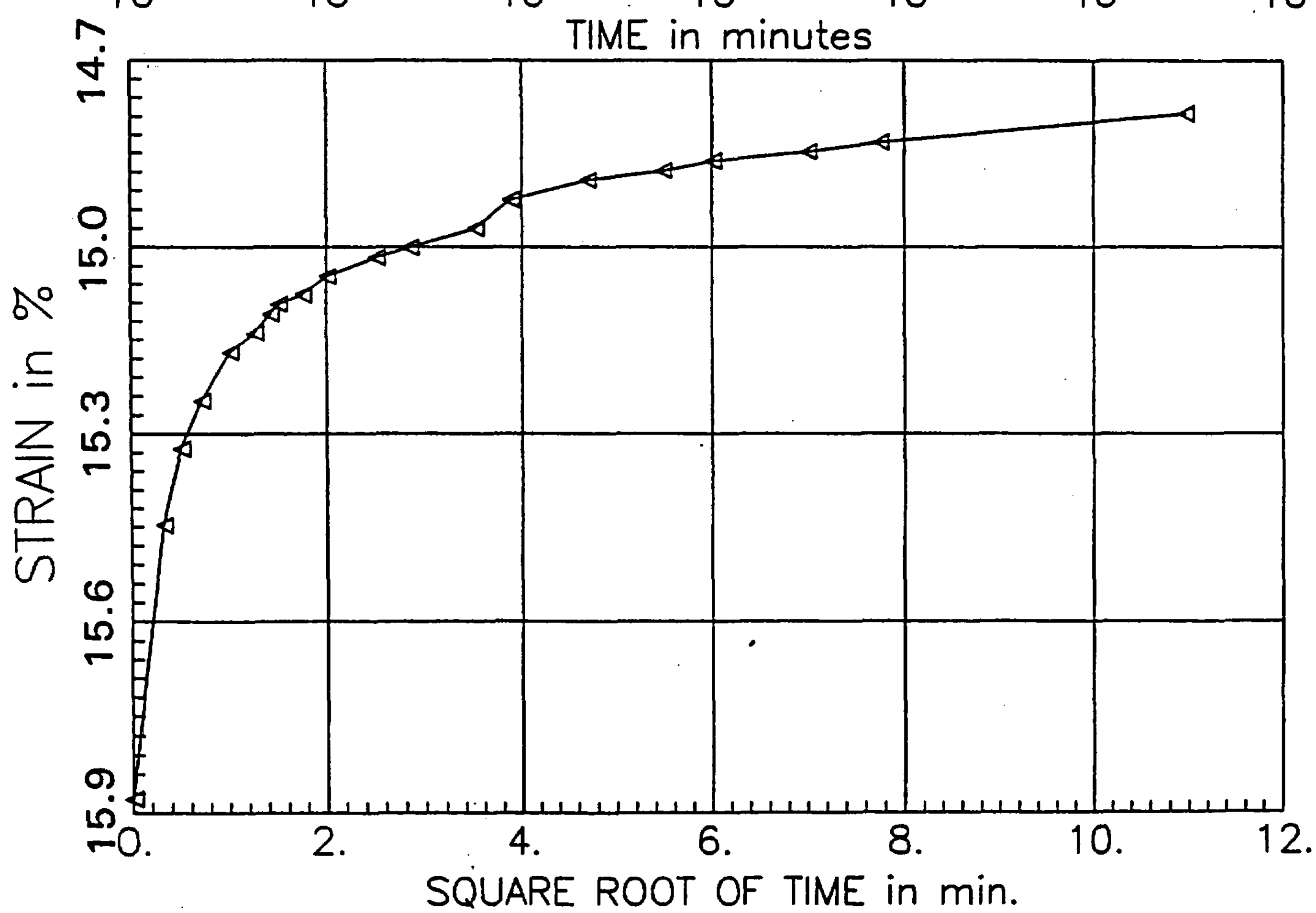




PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

Test No: 3
Testname: C2-U2C





PRESSURE INCREMENT
from 0.50 tsf to 0.10 tsf

Test No: 3
Testname: C2-U2C

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/17/96
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	10.9 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	27.6 %	44.2 %
DRY UNIT WEIGHT:	64.9 pcf	76.2 pcf
VOID RATIO:	1.615	1.230
SATURATION:	46.4 %	97.7 %
HEIGHT:	1.901 cm	1.621 cm
AREA:	31.65 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE tsf	STRAIN %
0.10	0.09
0.25	0.28
0.50	0.60
1.00	1.00
2.00	1.85
2.00	3.03
0.50	2.65
1.00	2.74
2.00	3.02
4.00	6.03
8.00	14.66
8.00	17.45
2.00	16.82
0.50	15.99
0.10	15.00

LOAD INCREMENT DATA

TEST NAME: C2-U2C

TESTED BY: ACS

PAGE NO: 1

JO: 05996.01

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-17-96	17:02:30	0.10	0.32	0.0006	1.613	0.08
		0.25	0.50	0.0006	1.613	0.08
		0.50	0.71	0.0006	1.613	0.08
		1.00	1.00	0.0007	1.613	0.09
		1.57	1.25	0.0007	1.613	0.09
		2.00	1.41	0.0007	1.613	0.09
		2.25	1.50	0.0007	1.613	0.09
		3.07	1.75	0.0007	1.613	0.09
		4.00	2.00	0.0007	1.613	0.09
		6.25	2.50	0.0007	1.613	0.09
		8.00	2.83	0.0007	1.613	0.09
		12.25	3.50	0.0008	1.612	0.11
		15.00	3.87	0.0008	1.612	0.11
		22.00	4.69	0.0008	1.612	0.11

LOAD INCREMENT DATA

TEST NAME C2-U2C

PAGE NO: 2

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-17-96	17:31:17	0.00	0.00	0.0008	1.612	0.11
		0.10	0.32	0.0017	1.609	0.23
		0.25	0.50	0.0018	1.609	0.25
		0.50	0.71	0.0018	1.609	0.25
		1.00	1.00	0.0020	1.608	0.26
		1.57	1.25	0.0020	1.608	0.26
		2.00	1.41	0.0020	1.608	0.26
		2.25	1.50	0.0020	1.608	0.26
		3.07	1.75	0.0020	1.608	0.26
		4.00	2.00	0.0020	1.608	0.26
		6.25	2.50	0.0021	1.608	0.28
		8.00	2.83	0.0021	1.608	0.28
		12.25	3.50	0.0021	1.608	0.28
		15.00	3.87	0.0021	1.608	0.28
		22.00	4.69	0.0023	1.607	0.31
		30.00	5.48	0.0024	1.607	0.32
		36.00	6.00	0.0024	1.607	0.32
		49.00	7.00	0.0024	1.607	0.32
		60.00	7.75	0.0024	1.607	0.32
		120.00	10.95	0.0025	1.606	0.34
		180.00	13.42	0.0026	1.606	0.35
		240.00	15.49	0.0028	1.605	0.37
		300.00	17.32	0.0028	1.605	0.37
		360.00	18.97	0.0029	1.605	0.38
12-18-96	00:31:17	420.00	20.49	0.0029	1.605	0.38
		480.00	21.91	0.0029	1.605	0.38
		540.00	23.24	0.0030	1.605	0.40
		600.00	24.49	0.0030	1.605	0.40
		660.00	25.69	0.0030	1.605	0.40
		720.00	26.83	0.0030	1.605	0.40
		780.00	27.93	0.0030	1.605	0.40
		840.00	28.98	0.0030	1.605	0.40
		900.00	30.00	0.0030	1.605	0.40
		960.00	30.98	0.0029	1.605	0.38
		1020.00	31.94	0.0029	1.605	0.38
		1080.00	32.86	0.0028	1.605	0.37
		1140.00	33.76	0.0029	1.605	0.38
		1200.00	34.64	0.0029	1.605	0.38
		1260.00	35.50	0.0029	1.605	0.38
		1320.00	36.33	0.0029	1.605	0.38
		1380.00	37.15	0.0029	1.605	0.38

LOAD INCREMENT DATA

TEST NAME C2-U2C PAGE NO: 3

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-18-96	16:33:21	0.00	0.00	0.0029	1.605	0.38
		0.10	0.32	0.0040	1.601	0.54
		0.25	0.50	0.0041	1.601	0.55
		0.50	0.71	0.0043	1.600	0.57
		1.00	1.00	0.0043	1.600	0.57
		1.57	1.25	0.0043	1.600	0.57
		2.00	1.41	0.0044	1.600	0.58
		2.25	1.50	0.0044	1.600	0.58
		3.07	1.75	0.0044	1.600	0.58
		4.00	2.00	0.0044	1.600	0.58
		6.25	2.50	0.0045	1.599	0.60
		8.00	2.83	0.0045	1.599	0.60
		12.25	3.50	0.0046	1.599	0.62
		15.00	3.87	0.0046	1.599	0.62
		22.00	4.69	0.0046	1.599	0.62
		30.00	5.48	0.0047	1.599	0.63
		36.00	6.00	0.0047	1.599	0.63
		49.00	7.00	0.0047	1.599	0.63
		60.00	7.75	0.0047	1.599	0.63
		120.00	10.95	0.0048	1.598	0.65
		180.00	13.42	0.0050	1.598	0.66
		240.00	15.49	0.0050	1.598	0.66
		300.00	17.32	0.0050	1.598	0.66
		360.00	18.97	0.0050	1.598	0.66
		420.00	20.49	0.0051	1.597	0.68
12-19-96	00:33:21	480.00	21.91	0.0051	1.597	0.68
		540.00	23.24	0.0051	1.597	0.68
		600.00	24.49	0.0051	1.597	0.68
		660.00	25.69	0.0051	1.597	0.68
		720.00	26.83	0.0051	1.597	0.68
		780.00	27.93	0.0051	1.597	0.68
		840.00	28.98	0.0052	1.597	0.69
		900.00	30.00	0.0051	1.597	0.68
		960.00	30.98	0.0052	1.597	0.69
		1020.00	31.94	0.0052	1.597	0.69
		1080.00	32.86	0.0052	1.597	0.69

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 4
JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0052	1.597	0.69
		1200.00	34.64	0.0052	1.597	0.69
		1260.00	35.50	0.0052	1.597	0.69
		1320.00	36.33	0.0052	1.597	0.69
		1380.00	37.15	0.0052	1.597	0.69
		1440.00	37.95	0.0052	1.597	0.69
		1500.00	38.73	0.0052	1.597	0.69
		1560.00	39.50	0.0053	1.596	0.71
		1620.00	40.25	0.0053	1.596	0.71
		1680.00	40.99	0.0053	1.596	0.71
		1740.00	41.71	0.0054	1.596	0.72
		1800.00	42.43	0.0054	1.596	0.72
		1860.00	43.13	0.0054	1.596	0.72
12-20-96	00:33:21	1920.00	43.82	0.0054	1.596	0.72
		1980.00	44.50	0.0054	1.596	0.72
		2040.00	45.17	0.0054	1.596	0.72
		2100.00	45.83	0.0054	1.596	0.72
		2160.00	46.48	0.0055	1.596	0.74
		2220.00	47.12	0.0055	1.596	0.74
		2280.00	47.75	0.0055	1.596	0.74
		2340.00	48.37	0.0055	1.596	0.74
		2400.00	48.99	0.0055	1.596	0.74
		2460.00	49.60	0.0054	1.596	0.72
		2520.00	50.20	0.0054	1.596	0.72
		2580.00	50.79	0.0053	1.596	0.71
		2640.00	51.38	0.0054	1.596	0.72
		2700.00	51.96	0.0053	1.596	0.71
		2760.00	52.54	0.0053	1.596	0.71
		2820.00	53.10	0.0053	1.596	0.71
		2880.00	53.67	0.0053	1.596	0.71
		2940.00	54.22	0.0054	1.596	0.72
		2989.77	54.68	0.0053	1.596	0.71

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 5
JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-20-96	18:23:08	0.00	0.00	0.0053	1.596	0.71
		0.10	0.32	0.0068	1.591	0.91
		0.25	0.50	0.0070	1.590	0.94
		0.50	0.71	0.0070	1.590	0.94
		1.00	1.00	0.0071	1.590	0.95
		1.57	1.25	0.0073	1.590	0.97
		2.00	1.41	0.0073	1.590	0.97
		2.25	1.50	0.0074	1.589	0.98
		3.07	1.75	0.0074	1.589	0.98
		4.00	2.00	0.0074	1.589	0.98
		6.25	2.50	0.0075	1.589	1.00
		8.00	2.83	0.0075	1.589	1.00
		12.25	3.50	0.0076	1.588	1.02
		15.00	3.87	0.0076	1.588	1.02
		22.00	4.69	0.0077	1.588	1.03
		30.00	5.48	0.0078	1.588	1.05
		36.00	6.00	0.0078	1.588	1.05
		49.00	7.00	0.0078	1.588	1.05
		60.00	7.75	0.0079	1.587	1.06
		120.00	10.95	0.0082	1.586	1.09
		180.00	13.42	0.0083	1.586	1.11
		240.00	15.49	0.0084	1.586	1.12
		300.00	17.32	0.0084	1.586	1.12
12-21-96	00:23:08	360.00	18.97	0.0085	1.585	1.14
		420.00	20.49	0.0086	1.585	1.15
		480.00	21.91	0.0086	1.585	1.15
		540.00	23.24	0.0088	1.584	1.17
		600.00	24.49	0.0088	1.584	1.17
		660.00	25.69	0.0088	1.584	1.17
		720.00	26.83	0.0088	1.584	1.17
		780.00	27.93	0.0089	1.584	1.18
		840.00	28.98	0.0089	1.584	1.18
		900.00	30.00	0.0089	1.584	1.18
		960.00	30.98	0.0089	1.584	1.18
		1020.00	31.94	0.0090	1.584	1.20
		1080.00	32.86	0.0090	1.584	1.20

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 6

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0090	1.584	1.20
		1200.00	34.64	0.0089	1.584	1.18
		1260.00	35.50	0.0090	1.584	1.20
		1320.00	36.33	0.0089	1.584	1.18
		1380.00	37.15	0.0090	1.584	1.20
		1440.00	37.95	0.0089	1.584	1.18
		1500.00	38.73	0.0089	1.584	1.18

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 7
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-21-96	19:33:48	0.00	0.00	0.0090	1.584	1.20
		0.10	0.32	0.0119	1.574	1.58
		0.25	0.50	0.0123	1.572	1.65
		0.50	0.71	0.0126	1.571	1.68
		1.00	1.00	0.0128	1.570	1.71
		1.57	1.25	0.0130	1.570	1.74
		2.00	1.41	0.0131	1.569	1.75
		2.25	1.50	0.0131	1.569	1.75
		3.07	1.75	0.0132	1.569	1.77
		4.00	2.00	0.0135	1.568	1.80
		6.25	2.50	0.0136	1.568	1.82
		8.00	2.83	0.0138	1.567	1.85
		12.25	3.50	0.0140	1.566	1.88
		15.00	3.87	0.0142	1.566	1.89
		22.00	4.69	0.0144	1.565	1.92
		30.00	5.48	0.0146	1.564	1.95
		36.00	6.00	0.0147	1.564	1.97
		49.00	7.00	0.0150	1.563	2.00
		60.00	7.75	0.0152	1.562	2.03
		120.00	10.95	0.0158	1.560	2.11
		180.00	13.42	0.0161	1.559	2.15
		240.00	15.49	0.0165	1.557	2.20
12-22-96	00:33:48	300.00	17.32	0.0167	1.557	2.23
		360.00	18.97	0.0169	1.556	2.26
		420.00	20.49	0.0170	1.555	2.28
		480.00	21.91	0.0173	1.555	2.31
		540.00	23.24	0.0174	1.554	2.32
		600.00	24.49	0.0175	1.554	2.34
		660.00	25.69	0.0176	1.553	2.35
		720.00	26.83	0.0177	1.553	2.37
		780.00	27.93	0.0178	1.553	2.38
		840.00	28.98	0.0180	1.552	2.40
		900.00	30.00	0.0180	1.552	2.40
		960.00	30.98	0.0181	1.552	2.42
		1020.00	31.94	0.0182	1.551	2.43
		1080.00	32.86	0.0182	1.551	2.43

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 8

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0183	1.551	2.45
		1200.00	34.64	0.0184	1.551	2.46
		1260.00	35.50	0.0184	1.551	2.46
		1320.00	36.33	0.0184	1.551	2.46
		1380.00	37.15	0.0184	1.551	2.46
		1440.00	37.95	0.0185	1.550	2.48
		1500.00	38.73	0.0185	1.550	2.48
		1560.00	39.50	0.0187	1.550	2.49
		1620.00	40.25	0.0188	1.549	2.51
		1680.00	40.99	0.0188	1.549	2.51
12-23-96	00:33:48	1740.00	41.71	0.0189	1.549	2.52
		1800.00	42.43	0.0189	1.549	2.52
		1860.00	43.13	0.0189	1.549	2.52
		1920.00	43.82	0.0190	1.549	2.54
		1980.00	44.50	0.0191	1.548	2.55
		2040.00	45.17	0.0190	1.549	2.54
		2100.00	45.83	0.0191	1.548	2.55
		2160.00	46.48	0.0190	1.549	2.54
		2220.00	47.12	0.0191	1.548	2.55
		2280.00	47.75	0.0191	1.548	2.55
		2340.00	48.37	0.0191	1.548	2.55
		2400.00	48.99	0.0191	1.548	2.55
		2460.00	49.60	0.0191	1.548	2.55
		2520.00	50.20	0.0191	1.548	2.55
		2580.00	50.79	0.0191	1.548	2.55
		2640.00	51.38	0.0192	1.548	2.57
		2700.00	51.96	0.0192	1.548	2.57
		2760.00	52.54	0.0192	1.548	2.57
		2820.00	53.10	0.0192	1.548	2.57
		2880.00	53.67	0.0193	1.547	2.58
		2940.00	54.22	0.0195	1.547	2.60
		3000.00	54.77	0.0195	1.547	2.60
		3060.00	55.32	0.0196	1.547	2.62
		3120.00	55.86	0.0196	1.547	2.62

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-24-96	00:33:48	3180.00	56.39	0.0197	1.546	2.63
		3240.00	56.92	0.0197	1.546	2.63
		3300.00	57.45	0.0198	1.546	2.65
		3360.00	57.97	0.0198	1.546	2.65
		3420.00	58.48	0.0198	1.546	2.65
		3480.00	58.99	0.0198	1.546	2.65
		3540.00	59.50	0.0198	1.546	2.65
		3600.00	60.00	0.0198	1.546	2.65
		3660.00	60.50	0.0198	1.546	2.65
		3720.00	60.99	0.0198	1.546	2.65
		3780.00	61.48	0.0197	1.546	2.63
		3840.00	61.97	0.0198	1.546	2.65
		3900.00	62.45	0.0198	1.546	2.65
		3960.00	62.93	0.0198	1.546	2.65
		4020.00	63.40	0.0199	1.545	2.66
		4080.00	63.87	0.0199	1.545	2.66
		4140.00	64.34	0.0199	1.545	2.66
		4200.00	64.81	0.0200	1.545	2.68
		4260.00	65.27	0.0200	1.545	2.68
		4320.00	65.73	0.0200	1.545	2.68
		4380.00	66.18	0.0202	1.545	2.69
		4440.00	66.63	0.0202	1.545	2.69
		4500.00	67.08	0.0203	1.544	2.71
		4560.00	67.53	0.0203	1.544	2.71
12-25-96	00:33:48	4620.00	67.97	0.0203	1.544	2.71
		4680.00	68.41	0.0203	1.544	2.71
		4740.00	68.85	0.0204	1.544	2.72
		4800.00	69.28	0.0204	1.544	2.72
		4860.00	69.71	0.0204	1.544	2.72
		4920.00	70.14	0.0204	1.544	2.72
		4980.00	70.57	0.0204	1.544	2.72
		5040.00	70.99	0.0204	1.544	2.72
		5100.00	71.41	0.0205	1.543	2.74
		5160.00	71.83	0.0205	1.543	2.74

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 10

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		5220.00	72.25	0.0205	1.543	2.74
		5280.00	72.66	0.0206	1.543	2.75
		5340.00	73.08	0.0205	1.543	2.74
		5400.00	73.48	0.0206	1.543	2.75
		5460.00	73.89	0.0206	1.543	2.75
		5520.00	74.30	0.0206	1.543	2.75
		5580.00	74.70	0.0206	1.543	2.75
		5640.00	75.10	0.0206	1.543	2.75
		5700.00	75.50	0.0207	1.543	2.77
		5760.00	75.89	0.0207	1.543	2.77
		5820.00	76.29	0.0207	1.543	2.77
		5880.00	76.68	0.0207	1.543	2.77
		5940.00	77.07	0.0207	1.543	2.77
		6000.00	77.46	0.0208	1.542	2.78
12-26-96	00:33:48	6060.00	77.85	0.0208	1.542	2.78
		6120.00	78.23	0.0208	1.542	2.78
		6180.00	78.61	0.0208	1.542	2.78
		6240.00	78.99	0.0208	1.542	2.78
		6300.00	79.37	0.0210	1.542	2.80
		6360.00	79.75	0.0210	1.542	2.80
		6420.00	80.12	0.0210	1.542	2.80
		6480.00	80.50	0.0208	1.542	2.78
		6540.00	80.87	0.0208	1.542	2.78
		6600.00	81.24	0.0208	1.542	2.78
		6660.00	81.61	0.0208	1.542	2.78
		6720.00	81.98	0.0208	1.542	2.78
		6780.00	82.34	0.0208	1.542	2.78
		6840.00	82.70	0.0207	1.543	2.77
		6900.00	83.07	0.0208	1.542	2.78
		6960.00	83.43	0.0208	1.542	2.78
		7020.00	83.79	0.0208	1.542	2.78
		7080.00	84.14	0.0208	1.542	2.78
		7140.00	84.50	0.0208	1.542	2.78
		7200.00	84.85	0.0208	1.542	2.78

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 11
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		7260.00	85.21	0.0210	1.542	2.80
		7320.00	85.56	0.0210	1.542	2.80
		7380.00	85.91	0.0211	1.541	2.82
		7440.00	86.26	0.0211	1.541	2.82
12-27-96	00:33:48	7500.00	86.60	0.0211	1.541	2.82
		7560.00	86.95	0.0211	1.541	2.82
		7620.00	87.29	0.0212	1.541	2.83
		7680.00	87.64	0.0211	1.541	2.82
		7740.00	87.98	0.0212	1.541	2.83
		7800.00	88.32	0.0212	1.541	2.83
		7860.00	88.66	0.0212	1.541	2.83
		7920.00	88.99	0.0211	1.541	2.82
		7980.00	89.33	0.0211	1.541	2.82
		8040.00	89.67	0.0211	1.541	2.82
		8100.00	90.00	0.0211	1.541	2.82
		8160.00	90.33	0.0211	1.541	2.82
		8220.00	90.66	0.0211	1.541	2.82
		8280.00	90.99	0.0211	1.541	2.82
		8340.00	91.32	0.0211	1.541	2.82
		8400.00	91.65	0.0211	1.541	2.82
		8460.00	91.98	0.0211	1.541	2.82
		8520.00	92.30	0.0212	1.541	2.83
		8580.00	92.63	0.0212	1.541	2.83
		8640.00	92.95	0.0212	1.541	2.83
		8700.00	93.27	0.0213	1.541	2.85
		8760.00	93.59	0.0213	1.541	2.85
		8820.00	93.91	0.0213	1.541	2.85
		8880.00	94.23	0.0213	1.541	2.85
12-28-96	00:33:48	8940.00	94.55	0.0214	1.540	2.86
		9000.00	94.87	0.0214	1.540	2.86
		9060.00	95.18	0.0214	1.540	2.86
		9120.00	95.50	0.0214	1.540	2.86
		9180.00	95.81	0.0215	1.540	2.88
		9240.00	96.12	0.0214	1.540	2.86

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 12
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		9300.00	96.44	0.0215	1.540	2.88
		9360.00	96.75	0.0215	1.540	2.88
		9420.00	97.06	0.0215	1.540	2.88
		9480.00	97.37	0.0215	1.540	2.88
		9540.00	97.67	0.0215	1.540	2.88
		9600.00	97.98	0.0215	1.540	2.88
		9660.00	98.29	0.0216	1.539	2.89
		9720.00	98.59	0.0216	1.539	2.89
		9780.00	98.89	0.0215	1.540	2.88
		9840.00	99.20	0.0216	1.539	2.89
		9900.00	99.50	0.0216	1.539	2.89
		9960.00	99.80	0.0216	1.539	2.89
		1002.00E+01	100.10	0.0216	1.539	2.89
		1008.00E+01	100.40	0.0216	1.539	2.89
		1014.00E+01	100.70	0.0216	1.539	2.89
		1020.00E+01	101.00	0.0216	1.539	2.89
		1026.00E+01	101.29	0.0216	1.539	2.89
		1032.00E+01	101.59	0.0216	1.539	2.89
12-29-96	00:33:48	1038.00E+01	101.88	0.0216	1.539	2.89
		1044.00E+01	102.18	0.0218	1.539	2.91
		1050.00E+01	102.47	0.0218	1.539	2.91
		1056.00E+01	102.76	0.0218	1.539	2.91
		1062.00E+01	103.05	0.0218	1.539	2.91
		1068.00E+01	103.34	0.0218	1.539	2.91
		1074.00E+01	103.63	0.0218	1.539	2.91
		1080.00E+01	103.92	0.0218	1.539	2.91
		1086.00E+01	104.21	0.0218	1.539	2.91
		1092.00E+01	104.50	0.0219	1.539	2.92
		1098.00E+01	104.79	0.0219	1.539	2.92
		1104.00E+01	105.07	0.0219	1.539	2.92
		1110.00E+01	105.36	0.0219	1.539	2.92
		1116.00E+01	105.64	0.0219	1.539	2.92
		1122.00E+01	105.92	0.0219	1.539	2.92
		1128.00E+01	106.21	0.0219	1.539	2.92

LOAD INCREMENT DATA

TEST NAME C2-U2C PAGE NO: 13
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1134.00E+01	106.49	0.0219	1.539	2.92
		1140.00E+01	106.77	0.0219	1.539	2.92
		1146.00E+01	107.05	0.0219	1.539	2.92
		1152.00E+01	107.33	0.0219	1.539	2.92
		1158.00E+01	107.61	0.0219	1.539	2.92
		1164.00E+01	107.89	0.0220	1.538	2.94
		1170.00E+01	108.17	0.0220	1.538	2.94
		1176.00E+01	108.44	0.0220	1.538	2.94
12-30-96	00:33:48	1182.00E+01	108.72	0.0220	1.538	2.94
		1188.00E+01	109.00	0.0220	1.538	2.94
		1194.00E+01	109.27	0.0220	1.538	2.94
		1200.00E+01	109.54	0.0220	1.538	2.94
		1206.00E+01	109.82	0.0220	1.538	2.94
		1212.00E+01	110.09	0.0220	1.538	2.94
		1218.00E+01	110.36	0.0220	1.538	2.94
		1224.00E+01	110.63	0.0219	1.539	2.92
		1230.00E+01	110.91	0.0219	1.539	2.92
		1236.00E+01	111.18	0.0219	1.539	2.92
		1242.00E+01	111.45	0.0219	1.539	2.92
		1248.00E+01	111.71	0.0219	1.539	2.92
		1254.00E+01	111.98	0.0219	1.539	2.92
		1260.00E+01	112.25	0.0219	1.539	2.92
		1266.00E+01	112.52	0.0219	1.539	2.92
		1272.00E+01	112.78	0.0219	1.539	2.92
		1278.00E+01	113.05	0.0219	1.539	2.92
		1284.00E+01	113.31	0.0219	1.539	2.92
		1290.00E+01	113.58	0.0219	1.539	2.92
		1296.00E+01	113.84	0.0219	1.539	2.92
		1302.00E+01	114.11	0.0220	1.538	2.94
		1308.00E+01	114.37	0.0220	1.538	2.94
		1314.00E+01	114.63	0.0220	1.538	2.94
		1320.00E+01	114.89	0.0221	1.538	2.95
12-31-96	00:33:48	1326.00E+01	115.15	0.0221	1.538	2.95
		1332.00E+01	115.41	0.0221	1.538	2.95

LOAD INCREMENT DATA

TEST NAME C2-U2C PAGE NO: 14
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1338.00E+01	115.67	0.0221	1.538	2.95
		1344.00E+01	115.93	0.0221	1.538	2.95
		1350.00E+01	116.19	0.0221	1.538	2.95
		1356.00E+01	116.45	0.0222	1.537	2.97
		1362.00E+01	116.70	0.0221	1.538	2.95
		1368.00E+01	116.96	0.0221	1.538	2.95
		1374.00E+01	117.22	0.0220	1.538	2.94
		1380.00E+01	117.47	0.0220	1.538	2.94
		1386.00E+01	117.73	0.0220	1.538	2.94
		1392.00E+01	117.98	0.0220	1.538	2.94
		1398.00E+01	118.24	0.0220	1.538	2.94
		1404.00E+01	118.49	0.0220	1.538	2.94
		1410.00E+01	118.74	0.0220	1.538	2.94
		1416.00E+01	119.00	0.0220	1.538	2.94
		1422.00E+01	119.25	0.0220	1.538	2.94
		1428.00E+01	119.50	0.0220	1.538	2.94
		1434.00E+01	119.75	0.0220	1.538	2.94
		1440.00E+01	120.00	0.0221	1.538	2.95
		1446.00E+01	120.25	0.0221	1.538	2.95
		1452.00E+01	120.50	0.0222	1.537	2.97
		1458.00E+01	120.75	0.0222	1.537	2.97
		1464.00E+01	121.00	0.0222	1.537	2.97
01-01-97	00:33:48	1470.00E+01	121.24	0.0222	1.537	2.97
		1476.00E+01	121.49	0.0223	1.537	2.98
		1482.00E+01	121.74	0.0223	1.537	2.98
		1488.00E+01	121.98	0.0223	1.537	2.98
		1494.00E+01	122.23	0.0225	1.537	3.00
		1500.00E+01	122.47	0.0225	1.537	3.00
		1506.00E+01	122.72	0.0225	1.537	3.00
		1512.00E+01	122.96	0.0225	1.537	3.00
		1518.00E+01	123.21	0.0225	1.537	3.00
		1524.00E+01	123.45	0.0225	1.537	3.00
		1530.00E+01	123.69	0.0225	1.537	3.00
		1536.00E+01	123.94	0.0226	1.536	3.02

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 15

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1542.00E+01	124.18	0.0226	1.536	3.02
		1548.00E+01	124.42	0.0226	1.536	3.02
		1554.00E+01	124.66	0.0226	1.536	3.02
		1560.00E+01	124.90	0.0226	1.536	3.02
		1566.00E+01	125.14	0.0226	1.536	3.02
		1572.00E+01	125.38	0.0226	1.536	3.02
		1578.00E+01	125.62	0.0226	1.536	3.02
		1584.00E+01	125.86	0.0226	1.536	3.02
		1590.00E+01	126.10	0.0226	1.536	3.02
		1596.00E+01	126.33	0.0227	1.536	3.03
		1602.00E+01	126.57	0.0226	1.536	3.02
		1608.00E+01	126.81	0.0226	1.536	3.02
01-02-97	00:33:48	1614.00E+01	127.04	0.0227	1.536	3.03
		1620.00E+01	127.28	0.0227	1.536	3.03
		1626.00E+01	127.51	0.0227	1.536	3.03
		1632.00E+01	127.75	0.0227	1.536	3.03
		1638.00E+01	127.98	0.0227	1.536	3.03
		1644.00E+01	128.22	0.0226	1.536	3.02
		1650.00E+01	128.45	0.0226	1.536	3.02
		1656.00E+01	128.69	0.0226	1.536	3.02
		1662.00E+01	128.92	0.0226	1.536	3.02
		1668.00E+01	129.15	0.0225	1.537	3.00
		1674.00E+01	129.38	0.0225	1.537	3.00
		1680.00E+01	129.61	0.0225	1.537	3.00

LOAD INCREMENT DATA

TEST NAME C2-U2C

PAGE NO: 16

TESTED BY: ACS

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	11:42:46	0.00	0.00	0.0225	1.537	3.00
		0.10	0.32	0.0202	1.545	2.69
		0.25	0.50	0.0200	1.545	2.68
		0.50	0.71	0.0200	1.545	2.68
		1.00	1.00	0.0200	1.545	2.68
		1.57	1.25	0.0199	1.545	2.66
		2.00	1.41	0.0199	1.545	2.66
		2.25	1.50	0.0199	1.545	2.66
		3.07	1.75	0.0199	1.545	2.66
		4.00	2.00	0.0199	1.545	2.66
		6.25	2.50	0.0198	1.546	2.65
		8.00	2.83	0.0198	1.546	2.65
		12.25	3.50	0.0198	1.546	2.65
		15.00	3.87	0.0198	1.546	2.65
		22.00	4.69	0.0198	1.546	2.65
		30.00	5.48	0.0197	1.546	2.63
		36.00	6.00	0.0198	1.546	2.65
		49.00	7.00	0.0197	1.546	2.63
		60.00	7.75	0.0197	1.546	2.63
		120.00	10.95	0.0196	1.547	2.62
		180.00	13.42	0.0196	1.547	2.62
		240.00	15.49	0.0196	1.547	2.62
		300.00	17.32	0.0195	1.547	2.60

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 17
JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-02-97	16:53:53	0.00	0.00	0.0196	1.547	2.62
		0.10	0.32	0.0204	1.544	2.72
		0.25	0.50	0.0204	1.544	2.72
		0.50	0.71	0.0204	1.544	2.72
		1.00	1.00	0.0204	1.544	2.72
		1.57	1.25	0.0205	1.543	2.74
		2.00	1.41	0.0204	1.544	2.72
		2.25	1.50	0.0204	1.544	2.72
		3.07	1.75	0.0205	1.543	2.74
		4.00	2.00	0.0205	1.543	2.74
		6.25	2.50	0.0205	1.543	2.74
		8.00	2.83	0.0205	1.543	2.74
		12.25	3.50	0.0205	1.543	2.74
		15.00	3.87	0.0204	1.544	2.72
		22.00	4.69	0.0205	1.543	2.74
		30.00	5.48	0.0205	1.543	2.74
		36.00	6.00	0.0205	1.543	2.74
		49.00	7.00	0.0205	1.543	2.74
		60.00	7.75	0.0205	1.543	2.74
		120.00	10.95	0.0206	1.543	2.75
		180.00	13.42	0.0206	1.543	2.75
		240.00	15.49	0.0207	1.543	2.77
		300.00	17.32	0.0207	1.543	2.77
		360.00	18.97	0.0207	1.543	2.77
		420.00	20.49	0.0207	1.543	2.77
01-03-97	00:53:53	480.00	21.91	0.0208	1.542	2.78
		540.00	23.24	0.0208	1.542	2.78
		600.00	24.49	0.0207	1.543	2.77
		660.00	25.69	0.0208	1.542	2.78
		720.00	26.83	0.0208	1.542	2.78
		780.00	27.93	0.0208	1.542	2.78
		840.00	28.98	0.0207	1.543	2.77
		900.00	30.00	0.0207	1.543	2.77
		960.00	30.98	0.0207	1.543	2.77

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 18
JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	09:00:15	0.00	0.00	0.0207	1.543	2.77
		0.10	0.32	0.0223	1.537	2.98
		0.25	0.50	0.0223	1.537	2.98
		0.50	0.71	0.0225	1.537	3.00
		1.00	1.00	0.0225	1.537	3.00
		1.57	1.25	0.0225	1.537	3.00
		2.00	1.41	0.0225	1.537	3.00
		2.25	1.50	0.0225	1.537	3.00
		3.07	1.75	0.0225	1.537	3.00
		4.00	2.00	0.0225	1.537	3.00
		6.25	2.50	0.0226	1.536	3.02
		8.00	2.83	0.0226	1.536	3.02
		12.25	3.50	0.0226	1.536	3.02
		15.00	3.87	0.0226	1.536	3.02
		22.00	4.69	0.0227	1.536	3.03
		30.00	5.48	0.0227	1.536	3.03
		36.00	6.00	0.0227	1.536	3.03
		49.00	7.00	0.0227	1.536	3.03
		60.00	7.75	0.0227	1.536	3.03
		120.00	10.95	0.0227	1.536	3.03
		180.00	13.42	0.0228	1.535	3.05

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 19
JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-03-97	12:18:35	0.00	0.00	0.0228	1.535	3.05
		0.10	0.32	0.0325	1.502	4.34
		0.25	0.50	0.0350	1.493	4.68
		0.50	0.71	0.0367	1.487	4.91
		1.00	1.00	0.0387	1.480	5.17
		1.57	1.25	0.0400	1.475	5.34
		2.00	1.41	0.0406	1.473	5.43
		2.25	1.50	0.0410	1.472	5.48
		3.07	1.75	0.0419	1.469	5.60
		4.00	2.00	0.0428	1.465	5.72
		6.25	2.50	0.0443	1.460	5.92
		8.00	2.83	0.0451	1.457	6.03
		12.25	3.50	0.0468	1.452	6.25
		15.00	3.87	0.0474	1.449	6.34
		22.00	4.69	0.0488	1.444	6.52
		30.00	5.48	0.0501	1.440	6.69
		36.00	6.00	0.0508	1.438	6.79
		49.00	7.00	0.0520	1.433	6.95
		60.00	7.75	0.0529	1.430	7.06
		120.00	10.95	0.0557	1.420	7.45
		180.00	13.42	0.0575	1.414	7.68
		240.00	15.49	0.0586	1.410	7.83
		300.00	17.32	0.0596	1.407	7.97
		360.00	18.97	0.0603	1.404	8.06
		420.00	20.49	0.0610	1.402	8.15
		480.00	21.91	0.0616	1.400	8.23
		540.00	23.24	0.0621	1.398	8.29
		600.00	24.49	0.0625	1.397	8.35
		660.00	25.69	0.0629	1.395	8.40
01-04-97	00:18:35	720.00	26.83	0.0632	1.394	8.45
		780.00	27.93	0.0636	1.393	8.49
		840.00	28.98	0.0638	1.392	8.52
		900.00	30.00	0.0641	1.391	8.57
		960.00	30.98	0.0644	1.390	8.60
		1020.00	31.94	0.0646	1.389	8.63
		1080.00	32.86	0.0648	1.388	8.66

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 20
JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0649	1.388	8.68
		1200.00	34.64	0.0652	1.387	8.71
		1260.00	35.50	0.0654	1.386	8.74
		1320.00	36.33	0.0655	1.386	8.75
		1380.00	37.15	0.0658	1.385	8.79
		1440.00	37.95	0.0659	1.385	8.80
		1500.00	38.73	0.0661	1.384	8.83
		1560.00	39.50	0.0662	1.384	8.85
		1620.00	40.25	0.0663	1.383	8.86
		1680.00	40.99	0.0664	1.383	8.88
		1740.00	41.71	0.0667	1.382	8.91
		1800.00	42.43	0.0667	1.382	8.91
		1860.00	43.13	0.0669	1.381	8.94
		1920.00	43.82	0.0670	1.381	8.95
		1980.00	44.50	0.0671	1.380	8.97
		2040.00	45.17	0.0672	1.380	8.99
		2100.00	45.83	0.0674	1.380	9.00
01-05-97	00:18:35	2160.00	46.48	0.0674	1.380	9.00
		2220.00	47.12	0.0675	1.379	9.02
		2280.00	47.75	0.0676	1.379	9.03
		2340.00	48.37	0.0677	1.378	9.05
		2400.00	48.99	0.0678	1.378	9.06
		2460.00	49.60	0.0679	1.378	9.08
		2520.00	50.20	0.0681	1.377	9.09
		2580.00	50.79	0.0681	1.377	9.09
		2640.00	51.38	0.0682	1.377	9.11
		2700.00	51.96	0.0683	1.376	9.12
		2760.00	52.54	0.0683	1.376	9.12
		2820.00	53.10	0.0684	1.376	9.14
		2880.00	53.67	0.0685	1.376	9.15
		2940.00	54.22	0.0686	1.375	9.17
		3000.00	54.77	0.0686	1.375	9.17
		3060.00	55.32	0.0687	1.375	9.19
		3120.00	55.86	0.0689	1.374	9.20

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 21

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0689	1.374	9.20
		3240.00	56.92	0.0690	1.374	9.22
		3300.00	57.45	0.0690	1.374	9.22
		3360.00	57.97	0.0691	1.374	9.23
		3420.00	58.48	0.0692	1.373	9.25
		3480.00	58.99	0.0692	1.373	9.25
		3540.00	59.50	0.0693	1.373	9.26
01-06-97	00:18:35	3600.00	60.00	0.0693	1.373	9.26
		3660.00	60.50	0.0694	1.372	9.28
		3720.00	60.99	0.0694	1.372	9.28
		3780.00	61.48	0.0696	1.372	9.29
		3840.00	61.97	0.0696	1.372	9.29
		3900.00	62.45	0.0697	1.372	9.31
		3960.00	62.93	0.0697	1.372	9.31
		4020.00	63.40	0.0697	1.372	9.31
		4080.00	63.87	0.0698	1.371	9.32
		4319.92	65.73	0.0698	1.371	9.32

LOAD INCREMENT DATA

TEST NAME C2-U2C
TESTED BY: ACS

PAGE NO: 22
JO: 05996.01

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-06-97	13:43:29	0.00	0.00	0.0702	1.370	9.39
		0.10	0.32	0.0905	1.299	12.09
		0.25	0.50	0.0950	1.283	12.69
		0.50	0.71	0.0980	1.273	13.09
		1.00	1.00	0.1009	1.263	13.48
		1.57	1.25	0.1026	1.257	13.71
		2.00	1.41	0.1038	1.253	13.86
		2.25	1.50	0.1042	1.251	13.92
		3.07	1.75	0.1056	1.246	14.11
		4.00	2.00	0.1067	1.242	14.26
		6.25	2.50	0.1086	1.236	14.51
		8.00	2.83	0.1097	1.232	14.66
		12.25	3.50	0.1117	1.225	14.92
		15.00	3.87	0.1126	1.222	15.05
		22.00	4.69	0.1143	1.215	15.28
		30.00	5.48	0.1157	1.211	15.46
		36.00	6.00	0.1165	1.208	15.57
		49.00	7.00	0.1179	1.203	15.75
		60.00	7.75	0.1188	1.200	15.88
		120.00	10.95	0.1218	1.189	16.28
		180.00	13.42	0.1237	1.183	16.52
		240.00	15.49	0.1248	1.179	16.68
		300.00	17.32	0.1259	1.175	16.82
		360.00	18.97	0.1265	1.173	16.91
		420.00	20.49	0.1271	1.171	16.99
		480.00	21.91	0.1277	1.169	17.06
		540.00	23.24	0.1282	1.167	17.12
		600.00	24.49	0.1284	1.166	17.15
01-07-97	00:43:29	660.00	25.69	0.1287	1.165	17.20
		720.00	26.83	0.1291	1.164	17.25
		780.00	27.93	0.1293	1.163	17.28
		840.00	28.98	0.1297	1.162	17.32
		900.00	30.00	0.1299	1.161	17.35
		960.00	30.98	0.1301	1.160	17.39
		1020.00	31.94	0.1303	1.160	17.42
		1080.00	32.86	0.1306	1.159	17.45

LOAD INCREMENT DATA

TEST NAME C2-U2C PAGE NO: 23

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 8.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	08:01:01	0.00	0.00	0.1306	1.159	17.45
		0.10	0.32	0.1264	1.173	16.89
		0.25	0.50	0.1262	1.174	16.86
		0.50	0.71	0.1262	1.174	16.86
		1.00	1.00	0.1261	1.174	16.85
		1.57	1.25	0.1261	1.174	16.85
		2.00	1.41	0.1260	1.175	16.83
		2.25	1.50	0.1260	1.175	16.83
		3.07	1.75	0.1260	1.175	16.83
		4.00	2.00	0.1260	1.175	16.83
		6.25	2.50	0.1260	1.175	16.83
		8.00	2.83	0.1259	1.175	16.82
		12.25	3.50	0.1259	1.175	16.82
		15.00	3.87	0.1259	1.175	16.82
		22.00	4.69	0.1257	1.176	16.80
		30.00	5.48	0.1257	1.176	16.80
		36.00	6.00	0.1257	1.176	16.80
		49.00	7.00	0.1257	1.176	16.80
		60.00	7.75	0.1257	1.176	16.80

LOAD INCREMENT DATA

TEST NAME C2-U2C

TESTED BY: ACS

PAGE NO: 24

JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	09:15:12	0.00	0.00	0.1256	1.176	16.79
		0.10	0.32	0.1213	1.191	16.20
		0.25	0.50	0.1208	1.193	16.14
		0.50	0.71	0.1206	1.194	16.11
		1.00	1.00	0.1203	1.195	16.08
		1.57	1.25	0.1201	1.195	16.05
		2.00	1.41	0.1201	1.195	16.05
		2.25	1.50	0.1200	1.196	16.03
		3.07	1.75	0.1200	1.196	16.03
		4.00	2.00	0.1199	1.196	16.02
		6.25	2.50	0.1198	1.197	16.00
		8.00	2.83	0.1196	1.197	15.99
		12.25	3.50	0.1195	1.197	15.97
		15.00	3.87	0.1195	1.197	15.97
		22.00	4.69	0.1194	1.198	15.95
		30.00	5.48	0.1193	1.198	15.94
		36.00	6.00	0.1193	1.198	15.94
		49.00	7.00	0.1192	1.199	15.92
		60.00	7.75	0.1192	1.199	15.92
		120.00	10.95	0.1189	1.199	15.89
		174.00	13.19	0.1188	1.200	15.88

LOAD INCREMENT DATA

TEST NAME C2-U2C

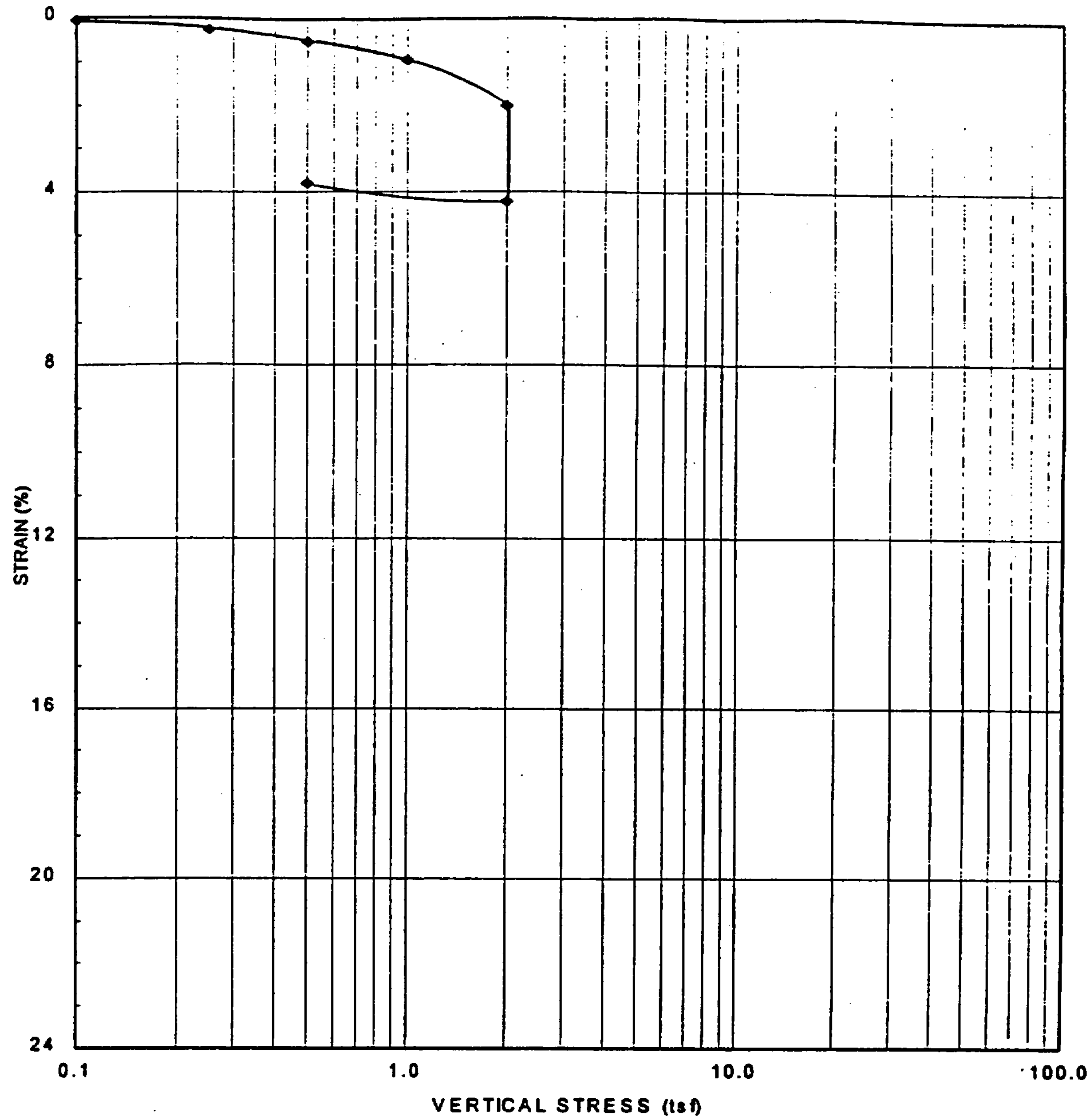
TESTED BY: ACS

PAGE NO: 25

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
01-07-97	12:09:39	0.00	0.00	0.1188	1.200	15.88
		0.10	0.32	0.1156	1.211	15.45
		0.25	0.50	0.1147	1.214	15.32
		0.50	0.71	0.1141	1.216	15.25
		1.00	1.00	0.1135	1.218	15.17
		1.57	1.25	0.1133	1.219	15.14
		2.00	1.41	0.1131	1.220	15.11
		2.25	1.50	0.1130	1.220	15.09
		3.07	1.75	0.1128	1.221	15.08
		4.00	2.00	0.1126	1.222	15.05
		6.25	2.50	0.1124	1.222	15.02
		8.00	2.83	0.1123	1.223	15.00
		12.25	3.50	0.1120	1.224	14.97
		15.00	3.87	0.1117	1.225	14.92
		22.00	4.69	0.1115	1.226	14.89
		30.00	5.48	0.1113	1.226	14.88
		36.00	6.00	0.1112	1.226	14.86
		49.00	7.00	0.1111	1.227	14.85
		60.00	7.75	0.1110	1.227	14.83
		120.00	10.95	0.1107	1.228	14.79
		180.00	13.42	0.1104	1.229	14.75
		240.00	15.49	0.1103	1.230	14.74



SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/10/96
SAMPLE:	U-2E	TESTED BY:	ACS
DEPTH:	11.7 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	39.7 %	65.0 %
DRY UNIT WEIGHT:	57.5 pcf	59.8 pcf
VOID RATIO:	1.952	1.840
SATURATION:	55.3 %	96.0 %

SPECIFIC GRAVITY: 2.72 (est)

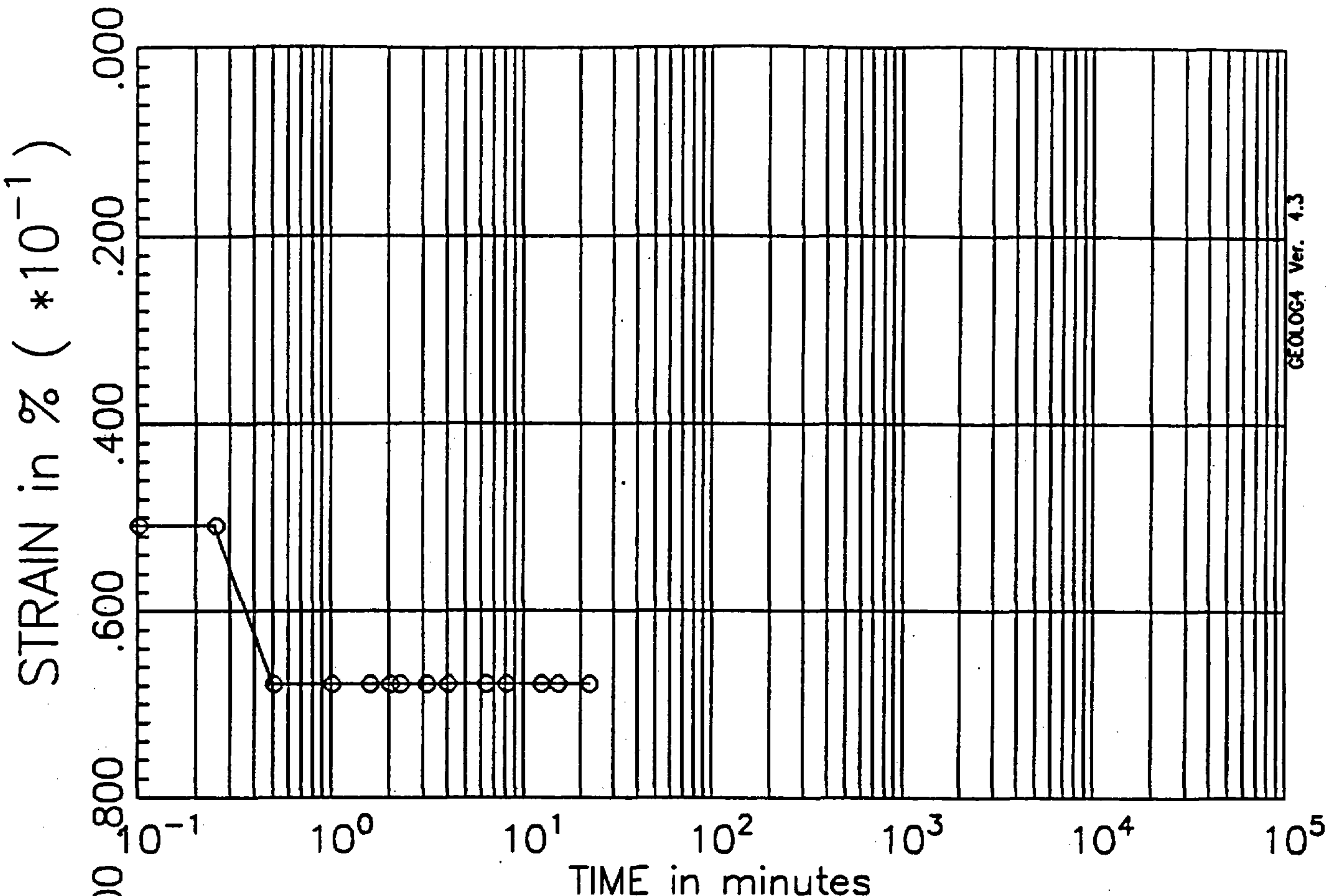
NOTE: Sample was inundated when the applied pressure was 0.5 tsf.

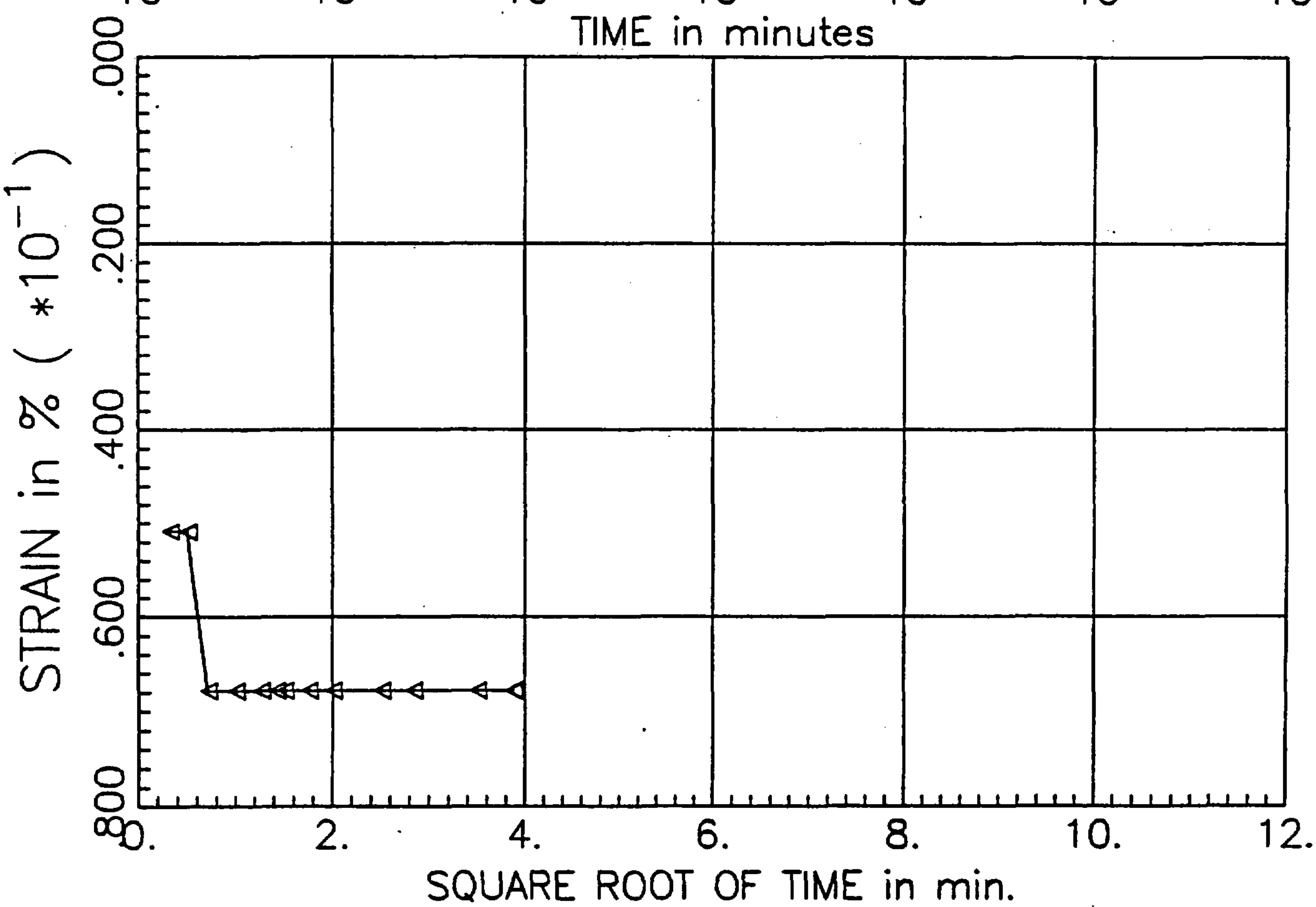
PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

CONSOLIDATION TEST RESULTS
BORING C-2, SAMPLE U-2E

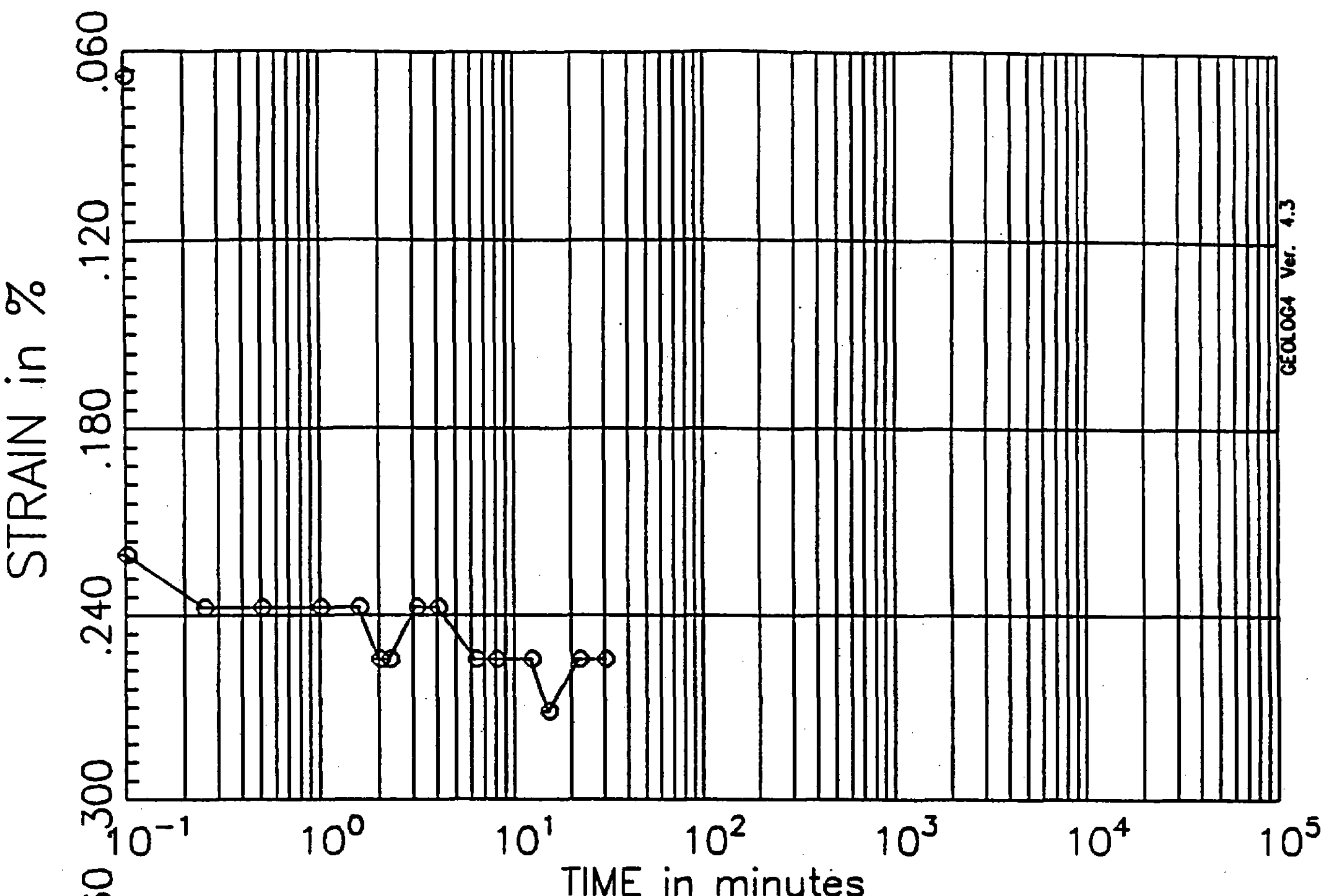
JO 05996.01
January 1997

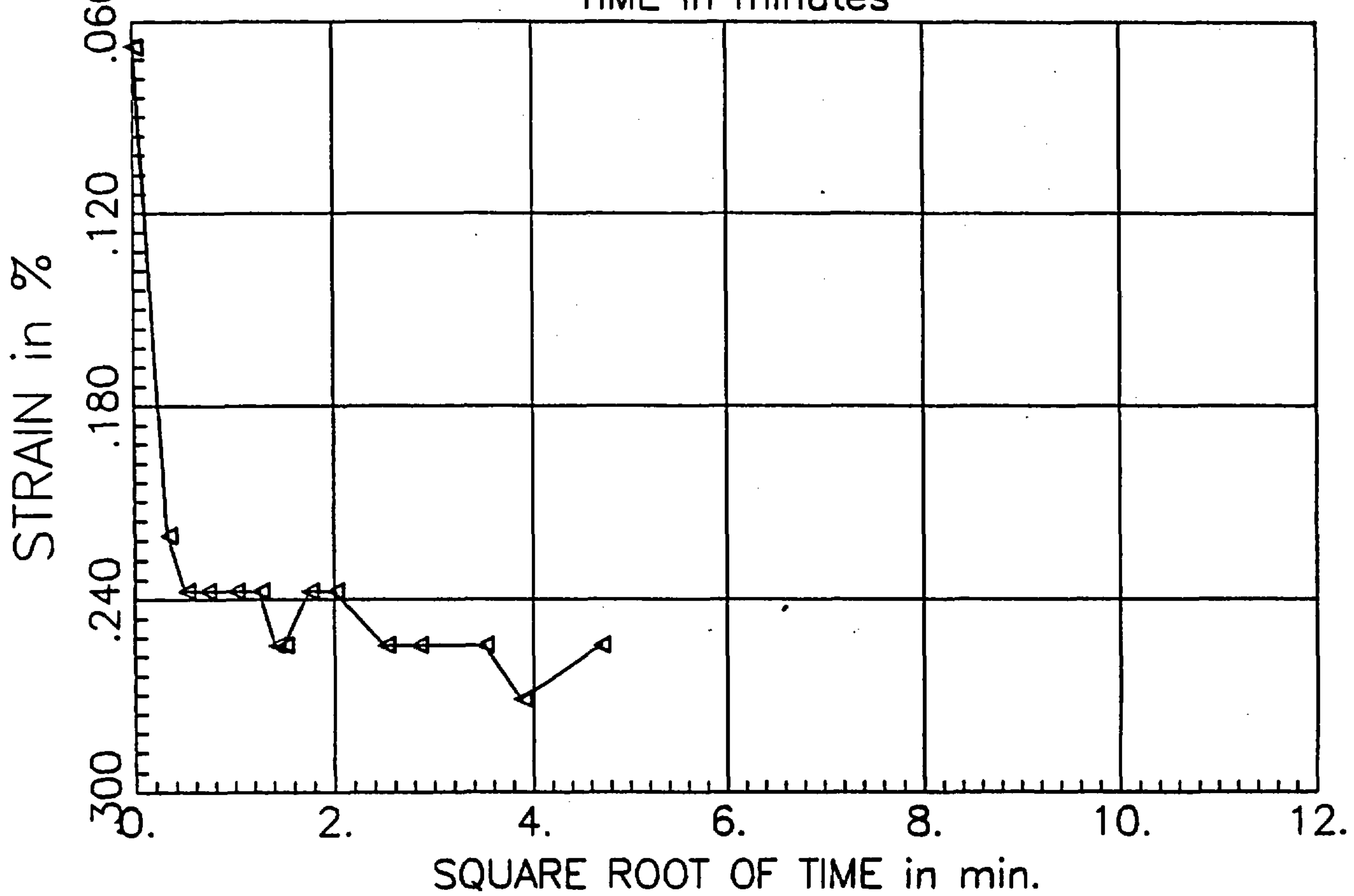




PRESSURE INCREMENT
from 0.00 tsf to 0.10 tsf

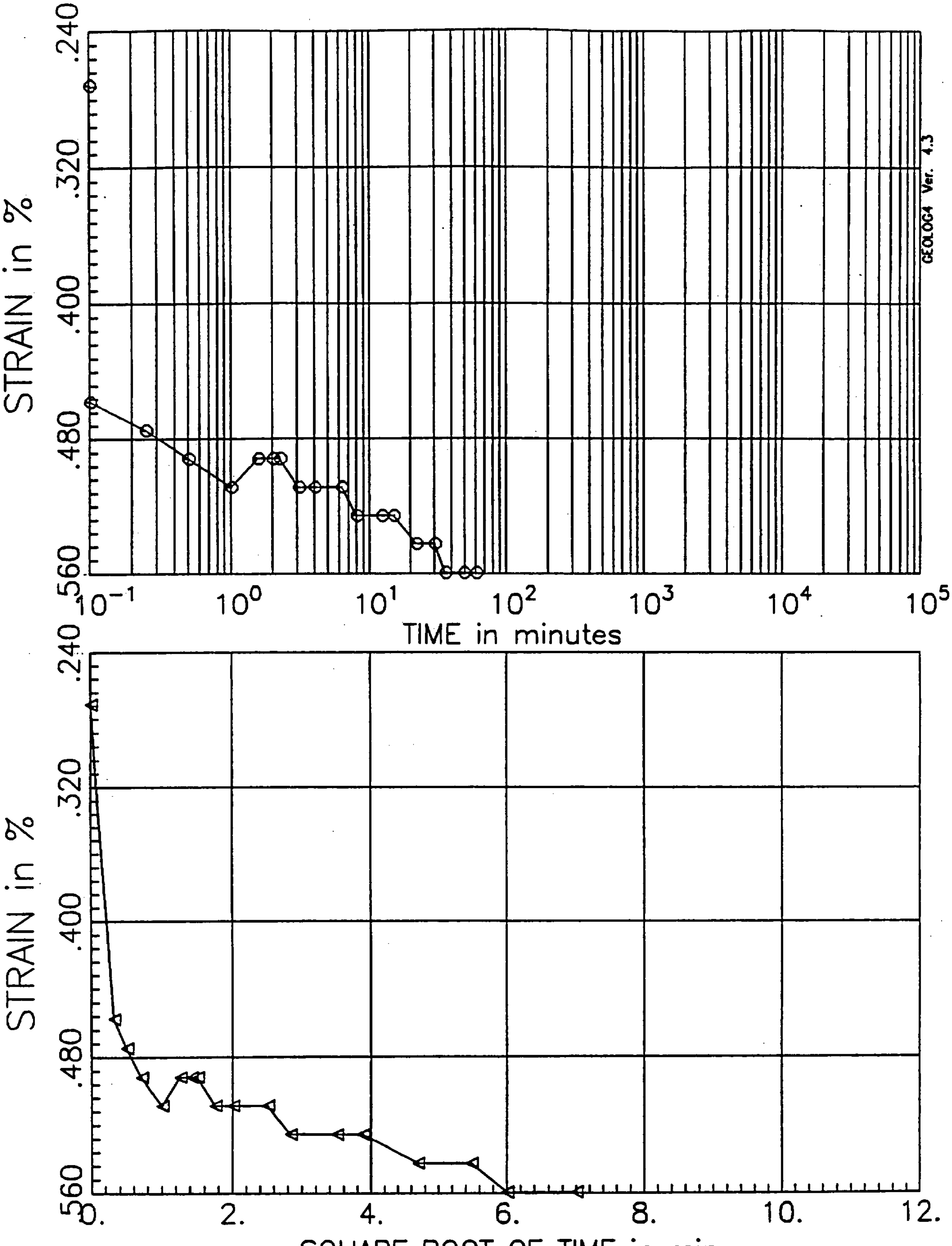
Test No: 1
Testname: C2-U2E





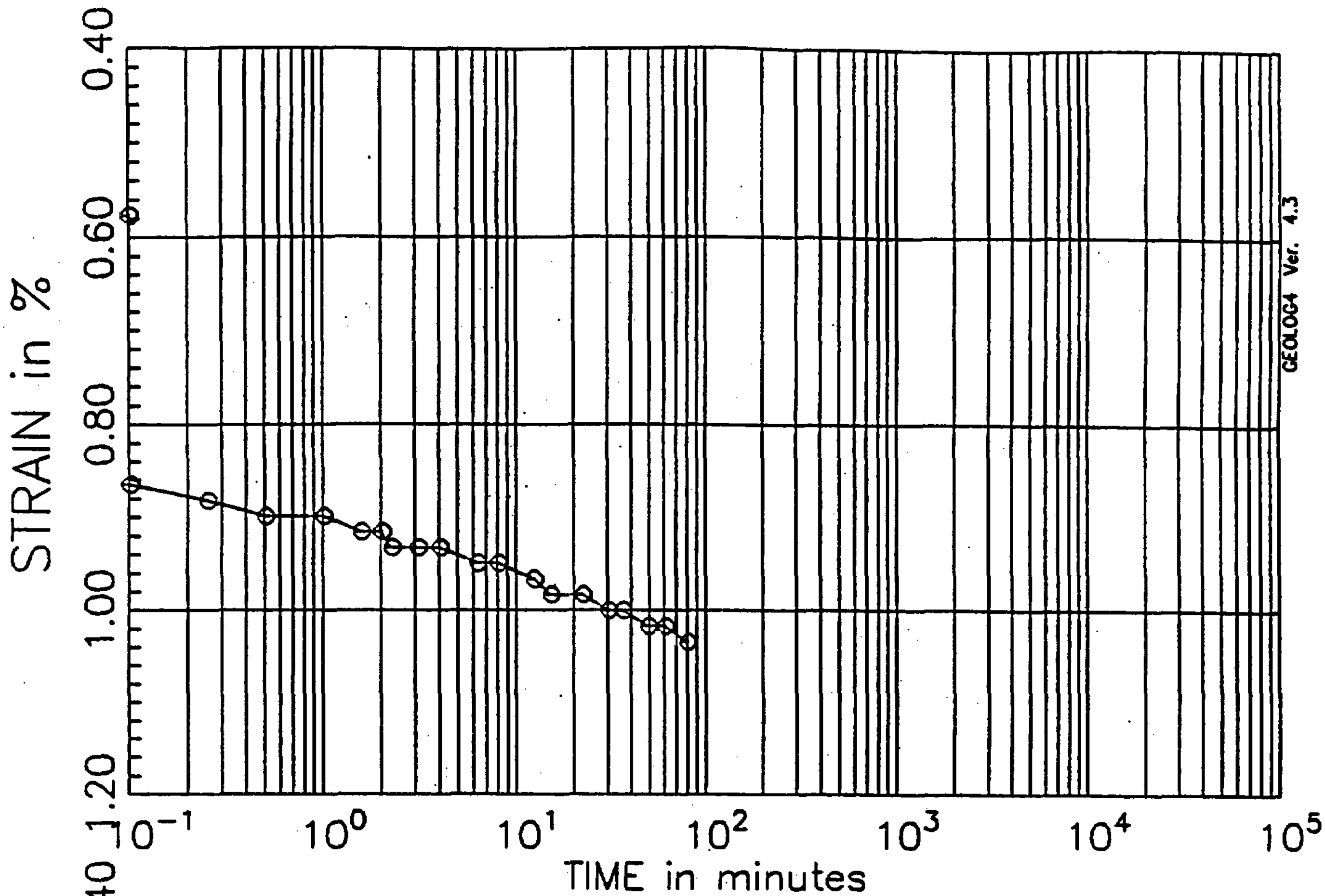
PRESSURE INCREMENT
from 0.10 tsf to 0.25 tsf

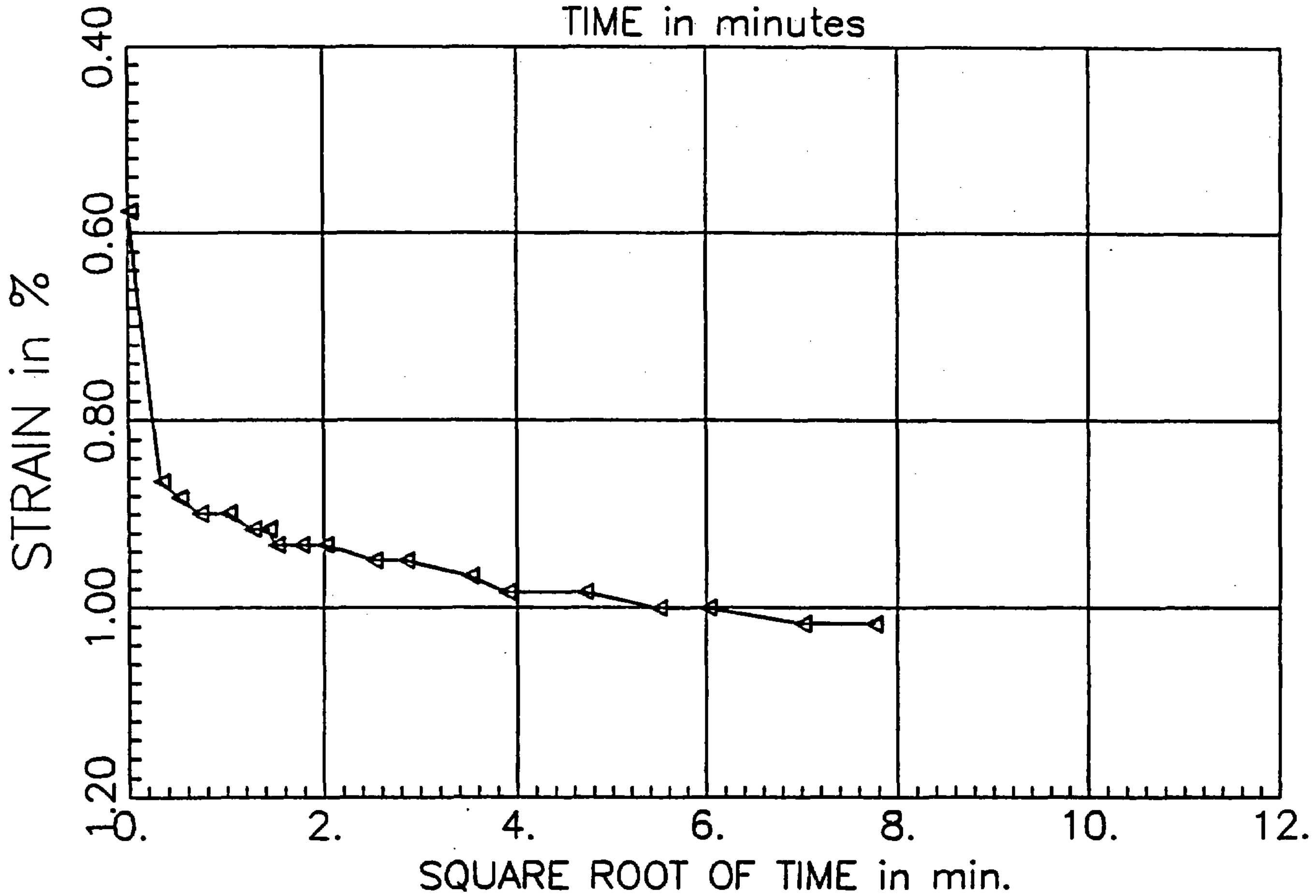
Test No: 1
Testname: C2-U2E



PRESSURE INCREMENT
from 0.25 tsf to 0.50 tsf

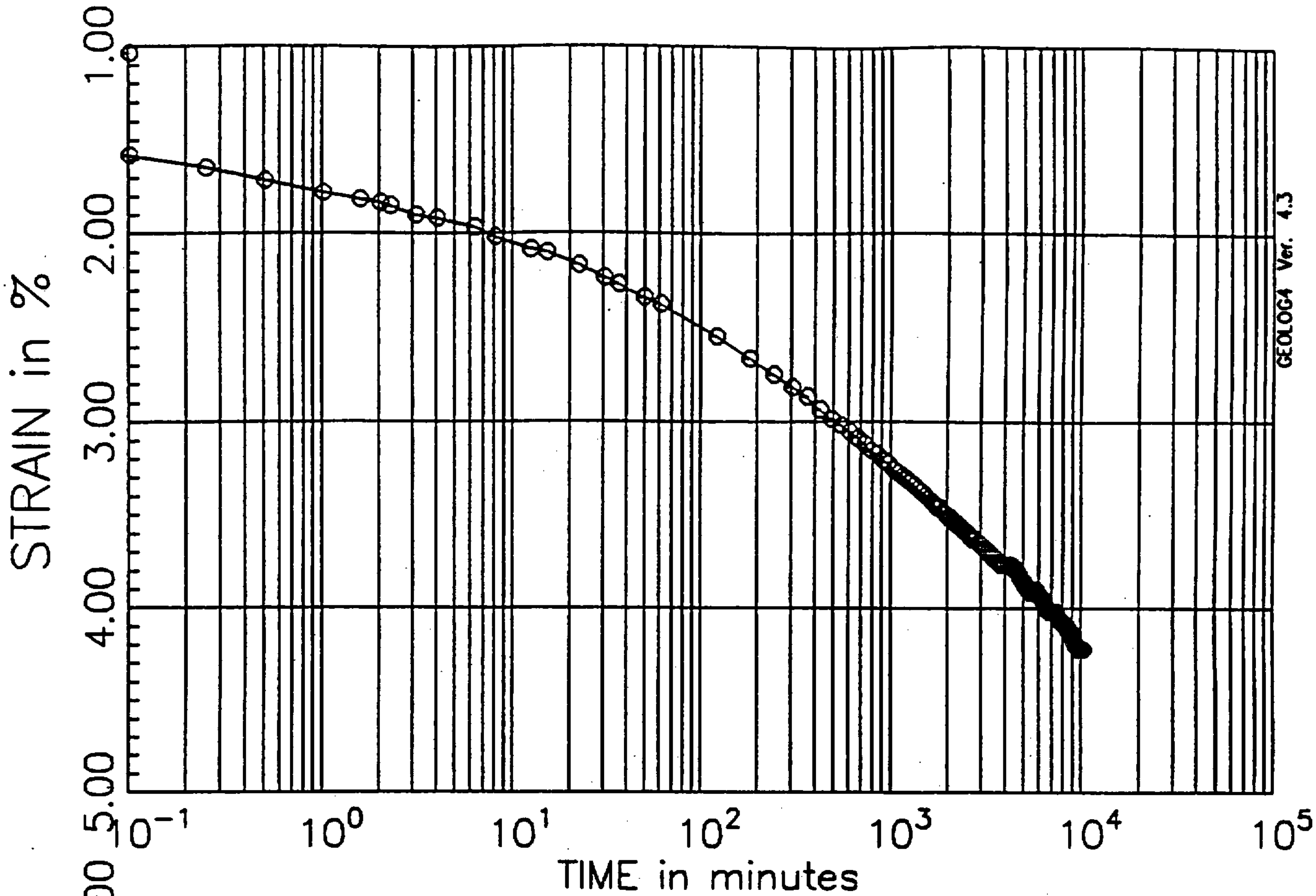
Test No: 1
Testname: C2–U2E

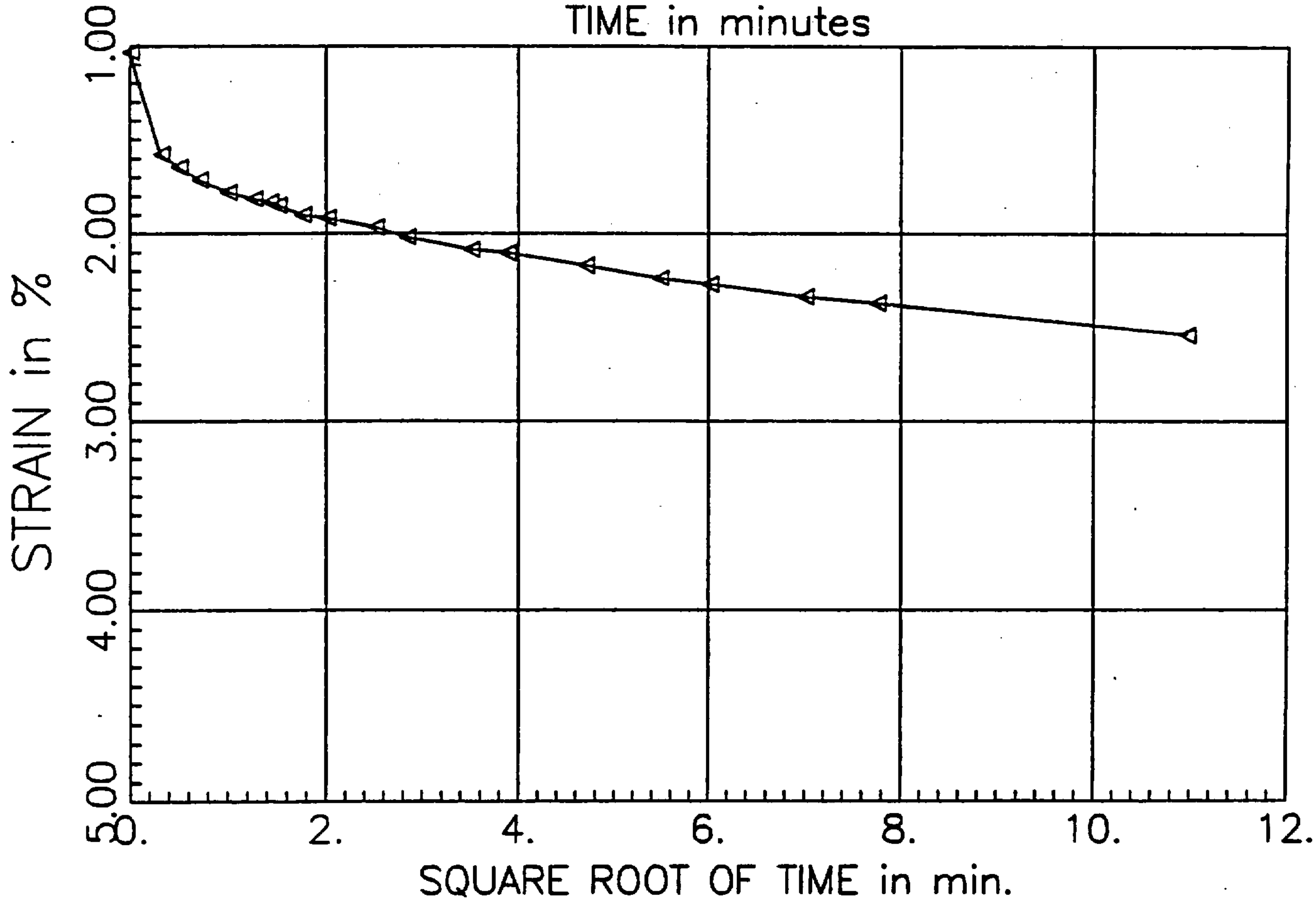




PRESSURE INCREMENT
from 0.50 tsf to 1.00 tsf

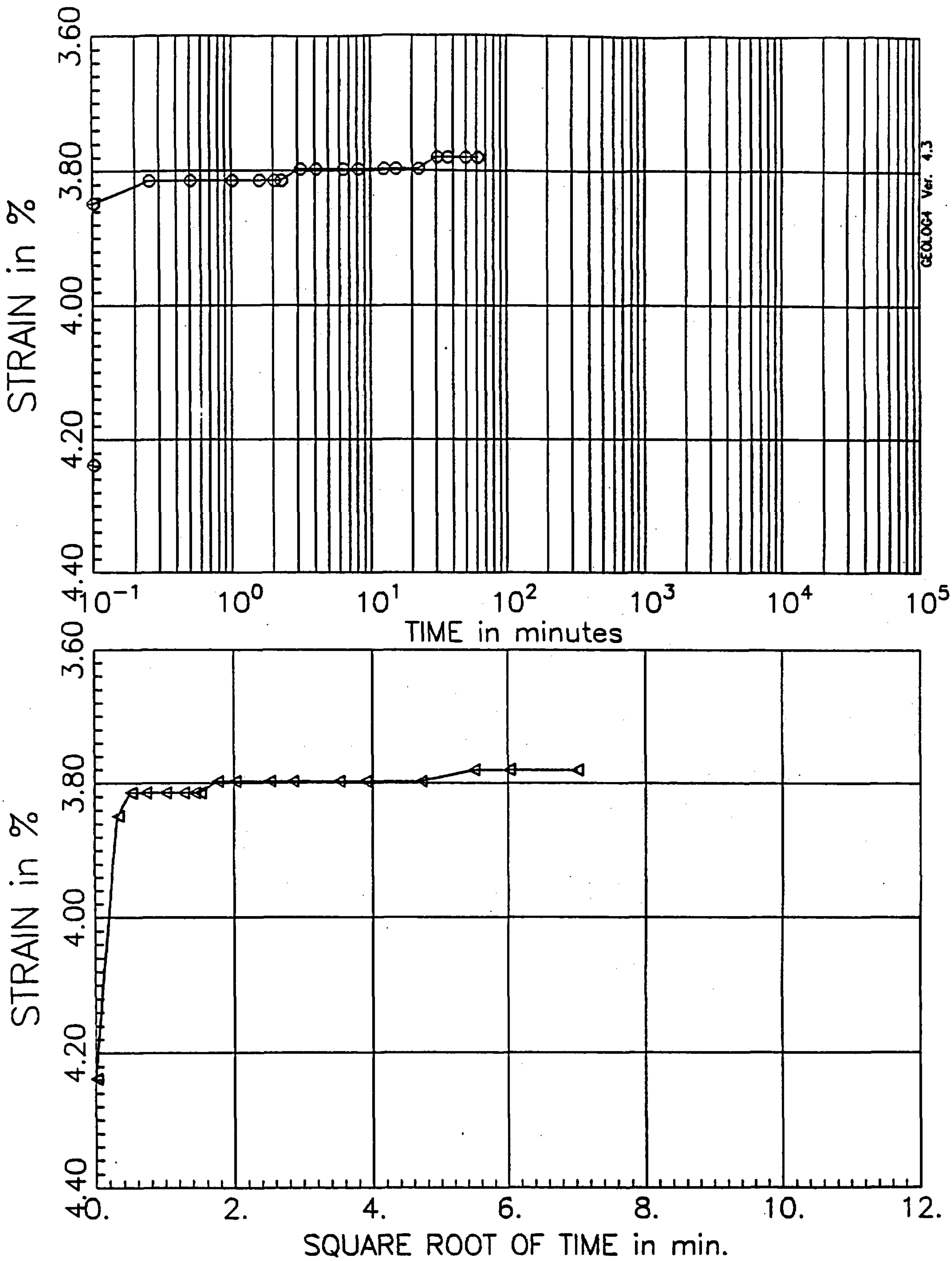
Test No: 1
Testname: C2-U2E





PRESSURE INCREMENT
from 1.00 tsf to 2.00 tsf

Test No: 1
Testname: C2–U2E



PRESSURE INCREMENT
from 2.00 tsf to 0.50 tsf

Test No: 1
Testname: C2-U2E

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/10/96
SAMPLE:	U-2E	TESTED BY:	ACS
DEPTH:	11.7 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	39.7 %	65.0 %
DRY UNIT WEIGHT:	57.5 pcf	59.8 pcf
VOID RATIO:	1.952	1.640
SATURATION:	55.3 %	96.0 %
HEIGHT:	1.901 cm	1.829 cm
AREA:	31.61 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE tsf	STRAIN %
0.10	0.07
0.25	0.25
0.50	0.53
1.00	0.95
2.00	2.02
2.00	4.22
0.50	3.80

LOAD INCREMENT DATA

TEST NAME: C2-U2E PAGE NO: 1

TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	09:17:19	0.10	0.32	0.0004	1.950	0.05
		0.25	0.50	0.0004	1.950	0.05
		0.50	0.71	0.0005	1.950	0.07
		1.00	1.00	0.0005	1.950	0.07
		1.57	1.25	0.0005	1.950	0.07
		2.00	1.41	0.0005	1.950	0.07
		2.25	1.50	0.0005	1.950	0.07
		3.07	1.75	0.0005	1.950	0.07
		4.00	2.00	0.0005	1.950	0.07
		6.25	2.50	0.0005	1.950	0.07
		8.00	2.83	0.0005	1.950	0.07
		12.25	3.50	0.0005	1.950	0.07
		15.00	3.87	0.0005	1.950	0.07
		22.00	4.69	0.0005	1.950	0.07

LOAD INCREMENT DATA

TEST NAME C2-U2E
TESTED BY: ACS

PAGE NO: 2
JO: 05996.01

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	09:43:58	0.00	0.00	0.0005	1.950	0.07
		0.10	0.32	0.0016	1.945	0.22
		0.25	0.50	0.0018	1.945	0.24
		0.50	0.71	0.0018	1.945	0.24
		1.00	1.00	0.0018	1.945	0.24
		1.57	1.25	0.0018	1.945	0.24
		2.00	1.41	0.0019	1.944	0.25
		2.25	1.50	0.0019	1.944	0.25
		3.07	1.75	0.0018	1.945	0.24
		4.00	2.00	0.0018	1.945	0.24
		6.25	2.50	0.0019	1.944	0.25
		8.00	2.83	0.0019	1.944	0.25
		12.25	3.50	0.0019	1.944	0.25
		15.00	3.87	0.0020	1.944	0.27
		22.00	4.69	0.0019	1.944	0.25
		30.00	5.48	0.0019	1.944	0.25

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 3

JO: 05996.01

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	10:19:46	0.00	0.00	0.0020	1.944	0.27
		0.10	0.32	0.0034	1.938	0.46
		0.25	0.50	0.0036	1.938	0.47
		0.50	0.71	0.0037	1.937	0.49
		1.00	1.00	0.0038	1.937	0.51
		1.57	1.25	0.0037	1.937	0.49
		2.00	1.41	0.0037	1.937	0.49
		2.25	1.50	0.0037	1.937	0.49
		3.07	1.75	0.0038	1.937	0.51
		4.00	2.00	0.0038	1.937	0.51
		6.25	2.50	0.0038	1.937	0.51
		8.00	2.83	0.0039	1.936	0.53
		12.25	3.50	0.0039	1.936	0.53
		15.00	3.87	0.0039	1.936	0.53
		22.00	4.69	0.0041	1.936	0.54
		30.00	5.48	0.0041	1.936	0.54
		36.00	6.00	0.0042	1.935	0.56
		49.00	7.00	0.0042	1.935	0.56
		60.00	7.75	0.0042	1.935	0.56

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 4

JO: 05996.01

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	11:23:52	0.00	0.00	0.0043	1.935	0.58
		0.10	0.32	0.0065	1.926	0.86
		0.25	0.50	0.0066	1.926	0.88
		0.50	0.71	0.0067	1.925	0.90
		1.00	1.00	0.0067	1.925	0.90
		1.57	1.25	0.0069	1.925	0.92
		2.00	1.41	0.0069	1.925	0.92
		2.25	1.50	0.0070	1.924	0.93
		3.07	1.75	0.0070	1.924	0.93
		4.00	2.00	0.0070	1.924	0.93
		6.25	2.50	0.0071	1.924	0.95
		8.00	2.83	0.0071	1.924	0.95
		12.25	3.50	0.0072	1.923	0.97
		15.00	3.87	0.0074	1.923	0.98
		22.00	4.69	0.0074	1.923	0.98
		30.00	5.48	0.0075	1.922	1.00
		36.00	6.00	0.0075	1.922	1.00
		49.00	7.00	0.0076	1.922	1.02
		60.00	7.75	0.0076	1.922	1.02
		79.52	8.92	0.0077	1.921	1.03

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 5

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-13-96	12:43:24	0.00	0.00	0.0077	1.921	1.03
		0.10	0.32	0.0118	1.905	1.58
		0.25	0.50	0.0123	1.903	1.64
		0.50	0.71	0.0128	1.901	1.71
		1.00	1.00	0.0133	1.899	1.78
		1.57	1.25	0.0136	1.898	1.81
		2.00	1.41	0.0137	1.898	1.83
		2.25	1.50	0.0138	1.897	1.85
		3.07	1.75	0.0142	1.896	1.90
		4.00	2.00	0.0143	1.895	1.92
		6.25	2.50	0.0147	1.894	1.97
		8.00	2.83	0.0151	1.892	2.02
		12.25	3.50	0.0156	1.890	2.09
		15.00	3.87	0.0157	1.890	2.10
		22.00	4.69	0.0162	1.888	2.17
		30.00	5.48	0.0167	1.886	2.24
		36.00	6.00	0.0170	1.885	2.27
		49.00	7.00	0.0175	1.883	2.34
		60.00	7.75	0.0178	1.882	2.37
		120.00	10.95	0.0190	1.877	2.54
		180.00	13.42	0.0199	1.873	2.66
		240.00	15.49	0.0206	1.871	2.75
		300.00	17.32	0.0211	1.869	2.81
		360.00	18.97	0.0214	1.867	2.86
		420.00	20.49	0.0219	1.865	2.93
		480.00	21.91	0.0223	1.864	2.98
		540.00	23.24	0.0226	1.863	3.02
		600.00	24.49	0.0228	1.862	3.05
		660.00	25.69	0.0231	1.861	3.09
12-14-96	00:43:24	720.00	26.83	0.0233	1.860	3.12
		780.00	27.93	0.0236	1.859	3.15
		840.00	28.98	0.0237	1.858	3.17
		900.00	30.00	0.0240	1.857	3.20
		960.00	30.98	0.0241	1.857	3.22
		1020.00	31.94	0.0244	1.856	3.25
		1080.00	32.86	0.0245	1.855	3.27

LOAD INCREMENT DATA

TEST NAME C2-U2E PAGE NO: 6
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		1140.00	33.76	0.0246	1.855	3.29
		1200.00	34.64	0.0247	1.854	3.31
		1260.00	35.50	0.0249	1.854	3.32
		1320.00	36.33	0.0250	1.853	3.34
		1380.00	37.15	0.0251	1.853	3.36
		1440.00	37.95	0.0252	1.852	3.37
		1500.00	38.73	0.0254	1.852	3.39
		1560.00	39.50	0.0255	1.851	3.41
		1620.00	40.25	0.0256	1.851	3.42
		1680.00	40.99	0.0258	1.850	3.44
		1740.00	41.71	0.0259	1.850	3.46
		1800.00	42.43	0.0259	1.850	3.46
		1860.00	43.13	0.0260	1.849	3.48
		1920.00	43.82	0.0261	1.849	3.49
		1980.00	44.50	0.0263	1.848	3.51
		2040.00	45.17	0.0263	1.848	3.51
		2100.00	45.83	0.0264	1.848	3.53
12-15-96	00:43:24	2160.00	46.48	0.0265	1.847	3.54
		2220.00	47.12	0.0265	1.847	3.54
		2280.00	47.75	0.0266	1.847	3.56
		2340.00	48.37	0.0268	1.846	3.58
		2400.00	48.99	0.0268	1.846	3.58
		2460.00	49.60	0.0269	1.846	3.59
		2520.00	50.20	0.0269	1.846	3.59
		2580.00	50.79	0.0270	1.845	3.61
		2640.00	51.38	0.0272	1.845	3.63
		2700.00	51.96	0.0272	1.845	3.63
		2760.00	52.54	0.0272	1.845	3.63
		2820.00	53.10	0.0273	1.844	3.64
		2880.00	53.67	0.0273	1.844	3.64
		2940.00	54.22	0.0274	1.844	3.66
		3000.00	54.77	0.0274	1.844	3.66
		3060.00	55.32	0.0275	1.843	3.68
		3120.00	55.86	0.0275	1.843	3.68

LOAD INCREMENT DATA

TEST NAME C2-U2E PAGE NO: 7
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		3180.00	56.39	0.0277	1.843	3.70
		3240.00	56.92	0.0277	1.843	3.70
		3300.00	57.45	0.0278	1.842	3.71
		3360.00	57.97	0.0278	1.842	3.71
		3420.00	58.48	0.0279	1.842	3.73
		3480.00	58.99	0.0279	1.842	3.73
		3540.00	59.50	0.0279	1.842	3.73
12-16-96	00:43:24	3600.00	60.00	0.0280	1.841	3.75
		3660.00	60.50	0.0280	1.841	3.75
		3720.00	60.99	0.0282	1.841	3.76
		3780.00	61.48	0.0282	1.841	3.76
		3840.00	61.97	0.0282	1.841	3.76
		3900.00	62.45	0.0282	1.841	3.76
		3960.00	62.93	0.0282	1.841	3.76
		4020.00	63.40	0.0282	1.841	3.76
		4080.00	63.87	0.0282	1.841	3.76
		4140.00	64.34	0.0282	1.841	3.76
		4200.00	64.81	0.0282	1.841	3.76
		4260.00	65.27	0.0283	1.840	3.78
		4320.00	65.73	0.0283	1.840	3.78
		4380.00	66.18	0.0283	1.840	3.78
		4440.00	66.63	0.0283	1.840	3.78
		4500.00	67.08	0.0284	1.840	3.80
		4560.00	67.53	0.0284	1.840	3.80
		4620.00	67.97	0.0285	1.839	3.81
		4680.00	68.41	0.0285	1.839	3.81
		4740.00	68.85	0.0287	1.839	3.83
		4800.00	69.28	0.0288	1.838	3.85
		4860.00	69.71	0.0288	1.838	3.85
		4920.00	70.14	0.0289	1.838	3.87
		4980.00	70.57	0.0289	1.838	3.87
12-17-96	00:43:24	5040.00	70.99	0.0291	1.837	3.88
		5100.00	71.41	0.0291	1.837	3.88
		5160.00	71.83	0.0292	1.837	3.90

LOAD INCREMENT DATA

TEST NAME C2-U2E PAGE NO: 8
TESTED BY: ACS JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		5220.00	72.25	0.0292	1.837	3.90
		5280.00	72.66	0.0292	1.837	3.90
		5340.00	73.08	0.0293	1.836	3.92
		5400.00	73.48	0.0292	1.837	3.90
		5460.00	73.89	0.0292	1.837	3.90
		5520.00	74.30	0.0292	1.837	3.90
		5580.00	74.70	0.0292	1.837	3.90
		5640.00	75.10	0.0292	1.837	3.90
		5700.00	75.50	0.0292	1.837	3.90
		5760.00	75.89	0.0293	1.836	3.92
		5820.00	76.29	0.0293	1.836	3.92
		5880.00	76.68	0.0293	1.836	3.92
		5940.00	77.07	0.0294	1.836	3.93
		6000.00	77.46	0.0294	1.836	3.93
		6060.00	77.85	0.0294	1.836	3.93
		6120.00	78.23	0.0296	1.835	3.95
		6180.00	78.61	0.0296	1.835	3.95
		6240.00	78.99	0.0297	1.835	3.97
		6300.00	79.37	0.0297	1.835	3.97
		6360.00	79.75	0.0298	1.834	3.98
		6420.00	80.12	0.0298	1.834	3.98
12-18-96	00:43:24	6480.00	80.50	0.0299	1.834	4.00
		6540.00	80.87	0.0299	1.834	4.00
		6600.00	81.24	0.0299	1.834	4.00
		6660.00	81.61	0.0301	1.833	4.02
		6720.00	81.98	0.0301	1.833	4.02
		6780.00	82.34	0.0301	1.833	4.02
		6840.00	82.70	0.0301	1.833	4.02
		6900.00	83.07	0.0301	1.833	4.02
		6960.00	83.43	0.0301	1.833	4.02
		7020.00	83.79	0.0301	1.833	4.02
		7080.00	84.14	0.0301	1.833	4.02
		7140.00	84.50	0.0301	1.833	4.02
		7200.00	84.85	0.0301	1.833	4.02

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 9

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		7260.00	85.21	0.0301	1.833	4.02
		7320.00	85.56	0.0302	1.833	4.03
		7380.00	85.91	0.0302	1.833	4.03
		7440.00	86.26	0.0303	1.832	4.05
		7500.00	86.60	0.0303	1.832	4.05
		7560.00	86.95	0.0303	1.832	4.05
		7620.00	87.29	0.0303	1.832	4.05
		7680.00	87.64	0.0305	1.832	4.07
		7740.00	87.98	0.0305	1.832	4.07
		7800.00	88.32	0.0305	1.832	4.07
		7860.00	88.66	0.0305	1.832	4.07
12-19-96	00:43:24	7920.00	88.99	0.0306	1.831	4.09
		7980.00	89.33	0.0306	1.831	4.09
		8040.00	89.67	0.0306	1.831	4.09
		8100.00	90.00	0.0306	1.831	4.09
		8160.00	90.33	0.0306	1.831	4.09
		8220.00	90.66	0.0307	1.831	4.10
		8280.00	90.99	0.0307	1.831	4.10
		8340.00	91.32	0.0307	1.831	4.10
		8400.00	91.65	0.0308	1.830	4.12
		8460.00	91.98	0.0308	1.830	4.12
		8520.00	92.30	0.0308	1.830	4.12
		8580.00	92.63	0.0310	1.830	4.14
		8640.00	92.95	0.0308	1.830	4.12
		8700.00	93.27	0.0310	1.830	4.14
		8760.00	93.59	0.0310	1.830	4.14
		8820.00	93.91	0.0311	1.829	4.15
		8880.00	94.23	0.0311	1.829	4.15
		8940.00	94.55	0.0311	1.829	4.15
		9000.00	94.87	0.0311	1.829	4.15
		9060.00	95.18	0.0312	1.829	4.17
		9120.00	95.50	0.0313	1.828	4.19
		9180.00	95.81	0.0313	1.828	4.19
		9240.00	96.12	0.0313	1.828	4.19

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 10

JO: 05996.01

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
		9300.00	96.44	0.0315	1.828	4.20
12-20-96	00:43:24	9360.00	96.75	0.0315	1.828	4.20
		9420.00	97.06	0.0315	1.828	4.20
		9480.00	97.37	0.0315	1.828	4.20
		9540.00	97.67	0.0316	1.827	4.22
		9600.00	97.98	0.0316	1.827	4.22
		9660.00	98.29	0.0316	1.827	4.22
		9720.00	98.59	0.0316	1.827	4.22
		9780.00	98.89	0.0316	1.827	4.22
		9840.00	99.20	0.0316	1.827	4.22
		9900.00	99.50	0.0316	1.827	4.22
		9960.00	99.80	0.0316	1.827	4.22
		1002.00E+01	100.10	0.0316	1.827	4.22
		1008.00E+01	100.40	0.0316	1.827	4.22
		1014.00E+01	100.70	0.0316	1.827	4.22

LOAD INCREMENT DATA

TEST NAME C2-U2E

TESTED BY: ACS

PAGE NO: 11

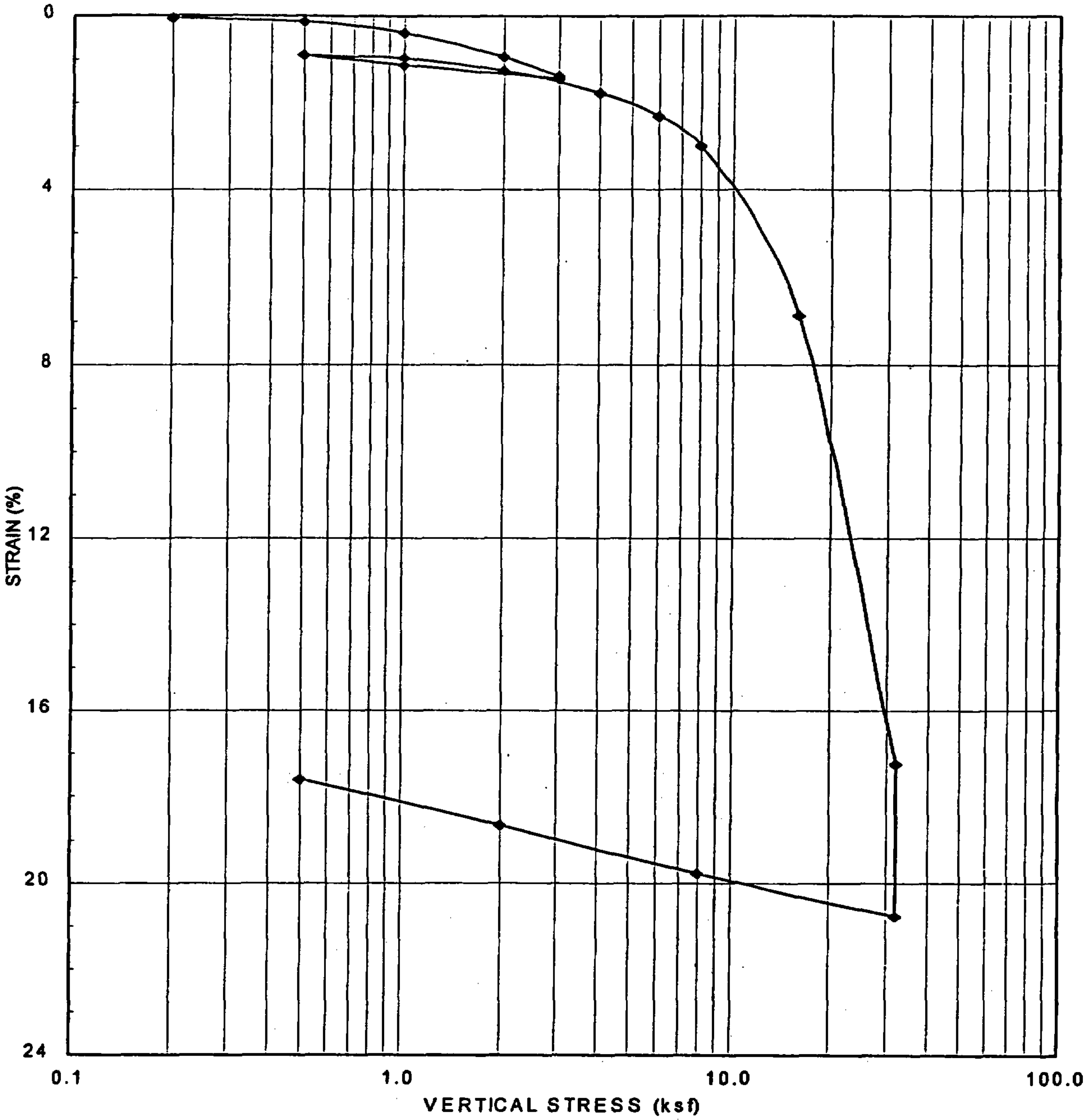
JO: 05996.01

PRESSURE INCREMENT FROM 2.00 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT(in)	VOID RATIO	STRAIN (%)
12-20-96	13:55:53	0.00	0.00	0.0317	1.827	4.24
		0.10	0.32	0.0288	1.838	3.85
		0.25	0.50	0.0285	1.839	3.81
		0.50	0.71	0.0285	1.839	3.81
		1.00	1.00	0.0285	1.839	3.81
		1.57	1.25	0.0285	1.839	3.81
		2.00	1.41	0.0285	1.839	3.81
		2.25	1.50	0.0285	1.839	3.81
		3.07	1.75	0.0284	1.840	3.80
		4.00	2.00	0.0284	1.840	3.80
		6.25	2.50	0.0284	1.840	3.80
		8.00	2.83	0.0284	1.840	3.80
		12.25	3.50	0.0284	1.840	3.80
		15.00	3.87	0.0284	1.840	3.80
		22.00	4.69	0.0284	1.840	3.80
		30.00	5.48	0.0283	1.840	3.78
		36.00	6.00	0.0283	1.840	3.78
		49.00	7.00	0.0283	1.840	3.78
		60.00	7.75	0.0283	1.840	3.78

APPENDIX A

Consolidation Test Plots and Data



SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	4/21/99
SAMPLE:	U-2E	TESTED BY:	ACS
DEPTH:	9.8 ft	CHECKED:	TYC
DESCRIPTION:	CLAY (CH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	48.9 %	42.1 %	
DRY UNIT WEIGHT:	63.2 pcf	75.8 pcf	2.72 (est)
VOID RATIO:	1.687	1.240	
SATURATION:	78.8 %	92.3 %	

NOTE: Sample was not inundated

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING CTB-4, SAMPLE U-2E	JO 05996.02 April 1999

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	4/21/99
SAMPLE:	U-2E	TESTED BY:	ACS
DEPTH:	9.8 ft	CHECKED:	TYC
DESCRIPTION:	CLAY (CH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	48.9 %	42.1 %
DRY UNIT WEIGHT:	63.2 pcf	75.8 pcf
VOID RATIO:	1.687	1.240
SATURATION:	78.8 %	92.3 %
HEIGHT:	1.901 cm	1.585 cm
AREA:	31.60 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE ksf	STRAIN %
0.20	0.07
0.50	0.13
1.00	0.38
2.00	0.92
3.00	1.37
3.00	1.46
1.00	1.14
0.50	0.91
1.00	0.98
2.00	1.24
4.00	1.76
6.00	2.33
8.00	3.00
16.00	6.87
32.00	17.23
32.00	20.79
8.00	19.78
2.00	18.67
0.50	17.61

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 1
TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	14:42:42	0.10	0.32	0.0005	1.685	0.07
		0.25	0.50	0.0004	1.686	0.05
		0.50	0.71	0.0005	1.685	0.07
		1.00	1.00	0.0005	1.685	0.07
		1.57	1.25	0.0005	1.685	0.07
		2.00	1.41	0.0005	1.685	0.07
		2.25	1.50	0.0004	1.686	0.05
		3.07	1.75	0.0004	1.686	0.05
		4.00	2.00	0.0002	1.686	0.03
		6.25	2.50	0.0001	1.687	0.02
		8.00	2.83	0.0001	1.687	0.02

LOAD INCREMENT DATA

TEST NAME CTB4-U2E

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	14:51:20	0.00	0.00	0.0004	1.686	0.05
		0.10	0.32	0.0009	1.684	0.12
		0.25	0.50	0.0010	1.683	0.13
		0.50	0.71	0.0009	1.684	0.12
		1.00	1.00	0.0009	1.684	0.12
		1.57	1.25	0.0010	1.683	0.13
		2.00	1.41	0.0010	1.683	0.13
		2.25	1.50	0.0010	1.683	0.13
		3.07	1.75	0.0010	1.683	0.13
		4.00	2.00	0.0010	1.683	0.13
		6.25	2.50	0.0010	1.683	0.13
		8.00	2.83	0.0010	1.683	0.13
		12.25	3.50	0.0011	1.683	0.15
		15.00	3.87	0.0010	1.683	0.13
		22.00	4.69	0.0010	1.683	0.13
		30.00	5.48	0.0011	1.683	0.15
		36.00	6.00	0.0011	1.683	0.15

LOAD INCREMENT DATA

TEST NAME CTB4-U2E

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	15:28:05	0.00	0.00	0.0011	1.683	0.15
		0.10	0.32	0.0027	1.677	0.36
		0.25	0.50	0.0027	1.677	0.36
		0.50	0.71	0.0029	1.677	0.38
		1.00	1.00	0.0029	1.677	0.38
		1.57	1.25	0.0029	1.677	0.38
		2.00	1.41	0.0030	1.676	0.40
		2.25	1.50	0.0030	1.676	0.40
		3.07	1.75	0.0030	1.676	0.40
		4.00	2.00	0.0030	1.676	0.40
		6.25	2.50	0.0030	1.676	0.40
		8.00	2.83	0.0031	1.676	0.41
		12.25	3.50	0.0031	1.676	0.41
		15.00	3.87	0.0031	1.676	0.41
		22.00	4.69	0.0032	1.675	0.43
		30.00	5.48	0.0032	1.675	0.43
		36.00	6.00	0.0032	1.675	0.43
		49.00	7.00	0.0032	1.675	0.43

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 4

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	16:20:51	0.00	0.00	0.0032	1.675	0.43
		0.10	0.32	0.0063	1.664	0.84
		0.25	0.50	0.0064	1.664	0.86
		0.50	0.71	0.0067	1.663	0.89
		1.00	1.00	0.0067	1.663	0.89
		1.57	1.25	0.0068	1.663	0.91
		2.00	1.41	0.0068	1.663	0.91
		2.25	1.50	0.0068	1.663	0.91
		3.07	1.75	0.0068	1.663	0.91
		4.00	2.00	0.0069	1.662	0.93
		6.25	2.50	0.0071	1.662	0.94
		8.00	2.83	0.0071	1.662	0.94
		12.25	3.50	0.0071	1.662	0.94
		15.00	3.87	0.0072	1.661	0.96
		22.00	4.69	0.0072	1.661	0.96
		30.00	5.48	0.0073	1.661	0.98
		36.00	6.00	0.0073	1.661	0.98
		49.00	7.00	0.0073	1.661	0.98
		60.00	7.75	0.0074	1.660	0.99
		120.00	10.95	0.0076	1.660	1.01
		180.00	13.42	0.0077	1.659	1.03
		240.00	15.49	0.0078	1.659	1.04
		360.00	18.97	0.0079	1.659	1.06
04-23-99	00:20:51	480.00	21.91	0.0081	1.658	1.08
		720.00	26.83	0.0082	1.658	1.09
		900.00	30.00	0.0082	1.658	1.09

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 5

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 1.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	07:56:13	0.00	0.00	0.0083	1.657	1.11
		0.10	0.32	0.0099	1.651	1.33
		0.25	0.50	0.0100	1.651	1.34
		0.50	0.71	0.0102	1.650	1.36
		1.00	1.00	0.0102	1.650	1.36
		1.57	1.25	0.0103	1.650	1.38
		2.00	1.41	0.0103	1.650	1.38
		2.25	1.50	0.0103	1.650	1.38
		3.07	1.75	0.0103	1.650	1.38
		4.00	2.00	0.0104	1.650	1.39
		6.25	2.50	0.0105	1.649	1.41
		8.00	2.83	0.0105	1.649	1.41
		12.25	3.50	0.0107	1.649	1.42
		15.00	3.87	0.0107	1.649	1.42
		22.00	4.69	0.0108	1.648	1.44
		30.00	5.48	0.0108	1.648	1.44
		36.00	6.00	0.0108	1.648	1.44
		49.00	7.00	0.0109	1.648	1.46

LOAD INCREMENT DATA

TEST NAME CTB4-U2E

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.50 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	08:50:05	0.00	0.00	0.0109	1.648	1.46
		0.10	0.32	0.0088	1.655	1.18
		0.25	0.50	0.0087	1.656	1.16
		0.50	0.71	0.0086	1.656	1.14
		1.00	1.00	0.0086	1.656	1.14
		1.57	1.25	0.0084	1.657	1.13
		2.00	1.41	0.0084	1.657	1.13
		2.25	1.50	0.0084	1.657	1.13
		3.07	1.75	0.0084	1.657	1.13
		4.00	2.00	0.0084	1.657	1.13
		6.25	2.50	0.0084	1.657	1.13
		8.00	2.83	0.0084	1.657	1.13
		12.25	3.50	0.0083	1.657	1.11
		15.00	3.87	0.0084	1.657	1.13
		22.00	4.69	0.0083	1.657	1.11

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 7

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	09:19:06	0.00	0.00	0.0084	1.657	1.13
		0.10	0.32	0.0071	1.662	0.94
		0.25	0.50	0.0069	1.662	0.93
		0.50	0.71	0.0068	1.663	0.91
		1.00	1.00	0.0068	1.663	0.91
		1.57	1.25	0.0068	1.663	0.91
		2.00	1.41	0.0067	1.663	0.89
		2.25	1.50	0.0067	1.663	0.89
		3.07	1.75	0.0067	1.663	0.89
		4.00	2.00	0.0067	1.663	0.89
		6.25	2.50	0.0067	1.663	0.89
		8.00	2.83	0.0066	1.663	0.88
		12.25	3.50	0.0066	1.663	0.88
		15.00	3.87	0.0066	1.663	0.88
		22.00	4.69	0.0066	1.663	0.88

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 8

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	09:45:16	0.00	0.00	0.0066	1.663	0.88
		0.10	0.32	0.0073	1.661	0.98
		0.25	0.50	0.0073	1.661	0.98
		0.50	0.71	0.0073	1.661	0.98
		1.00	1.00	0.0073	1.661	0.98
		1.57	1.25	0.0073	1.661	0.98
		2.00	1.41	0.0073	1.661	0.98
		2.25	1.50	0.0073	1.661	0.98
		3.07	1.75	0.0073	1.661	0.98
		4.00	2.00	0.0073	1.661	0.98
		6.25	2.50	0.0074	1.660	0.99
		8.00	2.83	0.0074	1.660	0.99
		12.25	3.50	0.0073	1.661	0.98
		15.00	3.87	0.0074	1.660	0.99
		22.00	4.69	0.0074	1.660	0.99

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 9

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	10:08:02	0.00	0.00	0.0073	1.661	0.98
		0.10	0.32	0.0092	1.654	1.23
		0.25	0.50	0.0092	1.654	1.23
		0.50	0.71	0.0093	1.654	1.24
		1.00	1.00	0.0093	1.654	1.24
		1.57	1.25	0.0093	1.654	1.24
		2.00	1.41	0.0093	1.654	1.24
		2.25	1.50	0.0093	1.654	1.24
		3.07	1.75	0.0093	1.654	1.24
		4.00	2.00	0.0093	1.654	1.24
		6.25	2.50	0.0093	1.654	1.24
		8.00	2.83	0.0093	1.654	1.24
		12.25	3.50	0.0093	1.654	1.24
		15.00	3.87	0.0093	1.654	1.24
		22.00	4.69	0.0093	1.654	1.24

LOAD INCREMENT DATA

TEST NAME CTB4-U2E

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	10:38:09	0.00	0.00	0.0094	1.653	1.26
		0.10	0.32	0.0126	1.642	1.69
		0.25	0.50	0.0128	1.641	1.71
		0.50	0.71	0.0129	1.641	1.72
		1.00	1.00	0.0130	1.640	1.74
		1.57	1.25	0.0131	1.640	1.76
		2.00	1.41	0.0131	1.640	1.76
		2.25	1.50	0.0131	1.640	1.76
		3.07	1.75	0.0133	1.639	1.77
		4.00	2.00	0.0133	1.639	1.77
		6.25	2.50	0.0134	1.639	1.79
		8.00	2.83	0.0134	1.639	1.79
		12.25	3.50	0.0135	1.638	1.81
		15.00	3.87	0.0136	1.638	1.82
		22.00	4.69	0.0136	1.638	1.82
		30.00	5.48	0.0138	1.638	1.84
		36.00	6.00	0.0138	1.638	1.84
		49.00	7.00	0.0139	1.637	1.86
		60.00	7.75	0.0139	1.637	1.86

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 11
TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 3.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	11:49:00	0.00	0.00	0.0140	1.637	1.87
		0.10	0.32	0.0166	1.627	2.22
		0.25	0.50	0.0170	1.626	2.27
		0.50	0.71	0.0174	1.625	2.32
		1.00	1.00	0.0176	1.624	2.35
		1.57	1.25	0.0177	1.623	2.37
		2.00	1.41	0.0179	1.623	2.39
		2.25	1.50	0.0179	1.623	2.39
		3.07	1.75	0.0180	1.622	2.40
		4.00	2.00	0.0181	1.622	2.42
		6.25	2.50	0.0182	1.622	2.44
		8.00	2.83	0.0184	1.621	2.45
		12.25	3.50	0.0186	1.620	2.49
		15.00	3.87	0.0187	1.620	2.50
		22.00	4.69	0.0188	1.619	2.52
		30.00	5.48	0.0190	1.619	2.53
		36.00	6.00	0.0192	1.618	2.57
		49.00	7.00	0.0193	1.618	2.58
		60.00	7.75	0.0195	1.617	2.60

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 12

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 3.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	12:57:01	0.00	0.00	0.0195	1.617	2.60
		0.10	0.32	0.0217	1.609	2.90
		0.25	0.50	0.0221	1.608	2.95
		0.50	0.71	0.0223	1.607	2.98
		1.00	1.00	0.0227	1.606	3.03
		1.57	1.25	0.0229	1.605	3.06
		2.00	1.41	0.0231	1.604	3.08
		2.25	1.50	0.0231	1.604	3.08
		3.07	1.75	0.0233	1.603	3.11
		4.00	2.00	0.0234	1.603	3.13
		6.25	2.50	0.0238	1.602	3.18
		8.00	2.83	0.0239	1.601	3.20
		12.25	3.50	0.0243	1.600	3.25
		15.00	3.87	0.0246	1.599	3.28
		22.00	4.69	0.0249	1.598	3.33
		30.00	5.48	0.0253	1.596	3.38
		36.00	6.00	0.0254	1.596	3.40
		49.00	7.00	0.0258	1.594	3.45
		60.00	7.75	0.0260	1.594	3.48
		92.00	9.59	0.0267	1.591	3.56

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 13

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	14:29:07	0.00	0.00	0.0267	1.591	3.56
		0.10	0.32	0.0454	1.524	6.06
		0.25	0.50	0.0493	1.510	6.59
		0.50	0.71	0.0523	1.499	6.99
		1.00	1.00	0.0554	1.488	7.41
		1.57	1.25	0.0575	1.480	7.69
		2.00	1.41	0.0588	1.476	7.85
		2.25	1.50	0.0594	1.474	7.94
		3.07	1.75	0.0609	1.468	8.13
		4.00	2.00	0.0622	1.464	8.32
		6.25	2.50	0.0646	1.455	8.63
		8.00	2.83	0.0660	1.450	8.81
		12.25	3.50	0.0684	1.441	9.15
		15.00	3.87	0.0696	1.437	9.29
		22.00	4.69	0.0718	1.429	9.59
		30.00	5.48	0.0737	1.423	9.84
		36.00	6.00	0.0748	1.419	9.99
		49.00	7.00	0.0766	1.412	10.24
		60.00	7.75	0.0779	1.407	10.40

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 14
TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 8.00 tsf to 16.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	15:48:02	0.00	0.00	0.0797	1.401	10.65
		0.10	0.32	0.1166	1.269	15.57
		0.25	0.50	0.1231	1.245	16.45
		0.50	0.71	0.1280	1.228	17.10
		1.00	1.00	0.1324	1.212	17.69
		1.57	1.25	0.1352	1.202	18.06
		2.00	1.41	0.1365	1.197	18.24
		2.25	1.50	0.1373	1.194	18.34
		3.07	1.75	0.1390	1.188	18.57
		4.00	2.00	0.1404	1.183	18.75
		6.25	2.50	0.1428	1.174	19.09
		8.00	2.83	0.1441	1.170	19.25
		12.25	3.50	0.1464	1.161	19.57
		15.00	3.87	0.1474	1.158	19.70
		22.00	4.69	0.1494	1.151	19.96
		30.00	5.48	0.1510	1.145	20.18
		36.00	6.00	0.1520	1.141	20.31
		49.00	7.00	0.1536	1.135	20.53
		60.00	7.75	0.1546	1.132	20.66
		75.00	8.66	0.1556	1.128	20.79

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 15
TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 16.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	17:03:08	0.00	0.00	0.1556	1.128	20.79
		0.10	0.32	0.1485	1.154	19.85
		0.25	0.50	0.1484	1.154	19.83
		0.50	0.71	0.1482	1.155	19.80
		1.00	1.00	0.1480	1.155	19.78
		1.57	1.25	0.1480	1.155	19.78
		2.00	1.41	0.1479	1.156	19.76
		2.25	1.50	0.1479	1.156	19.76
		3.07	1.75	0.1478	1.156	19.75
		4.00	2.00	0.1478	1.156	19.75
		6.25	2.50	0.1478	1.156	19.75
		8.00	2.83	0.1477	1.157	19.73
		12.25	3.50	0.1477	1.157	19.73
		15.00	3.87	0.1477	1.157	19.73
		22.00	4.69	0.1476	1.157	19.71
		30.00	5.48	0.1476	1.157	19.71

LOAD INCREMENT DATA

TEST NAME CTB4-U2E PAGE NO: 16
TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	17:36:02	0.00	0.00	0.1476	1.157	19.71
		0.10	0.32	0.1411	1.180	18.85
		0.25	0.50	0.1405	1.183	18.77
		0.50	0.71	0.1399	1.185	18.69
		1.00	1.00	0.1392	1.187	18.60
		1.57	1.25	0.1389	1.188	18.56
		2.00	1.41	0.1386	1.189	18.52
		2.25	1.50	0.1386	1.189	18.52
		3.07	1.75	0.1384	1.190	18.49
		4.00	2.00	0.1383	1.191	18.47
		6.25	2.50	0.1380	1.192	18.44
		8.00	2.83	0.1379	1.192	18.42
		12.25	3.50	0.1376	1.193	18.39
		15.00	3.87	0.1375	1.193	18.37
		22.00	4.69	0.1374	1.194	18.36
		30.00	5.48	0.1373	1.194	18.34

LOAD INCREMENT DATA

TEST NAME CTB4-U2E

PAGE NO: 17

TEST NO: 5 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

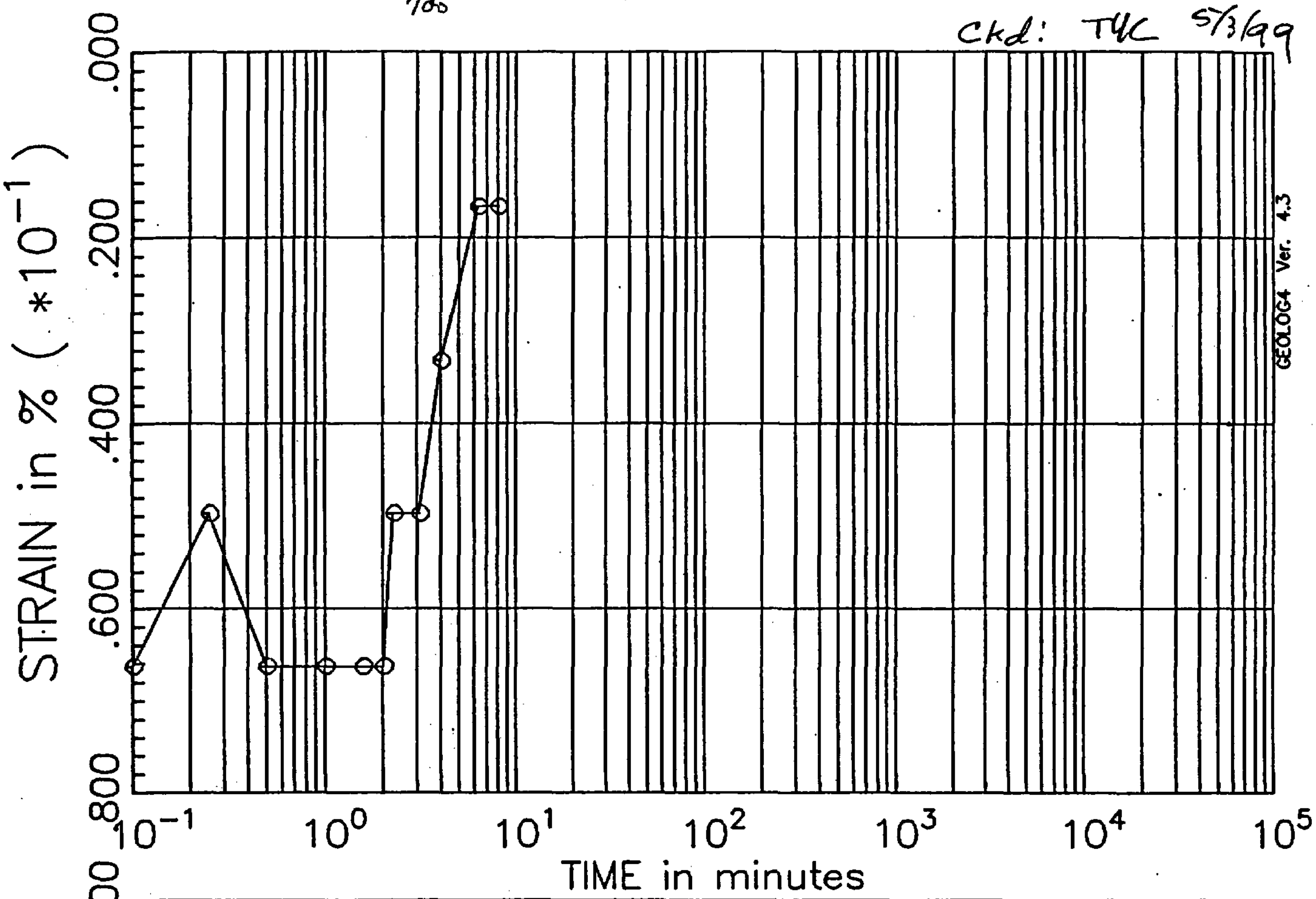
DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	18:11:12	0.00	0.00	0.1373	1.194	18.34
		0.10	0.32	0.1333	1.208	17.81
		0.25	0.50	0.1324	1.212	17.69
		0.50	0.71	0.1314	1.215	17.56
		1.00	1.00	0.1303	1.219	17.41
		1.57	1.25	0.1296	1.222	17.31
		2.00	1.41	0.1291	1.224	17.25
		2.25	1.50	0.1288	1.224	17.21
		3.07	1.75	0.1283	1.226	17.15
		4.00	2.00	0.1278	1.228	17.08
		6.25	2.50	0.1272	1.230	17.00
		8.00	2.83	0.1270	1.231	16.96
		12.25	3.50	0.1265	1.233	16.90
		15.00	3.87	0.1261	1.234	16.85
		22.00	4.69	0.1257	1.236	16.80
		30.00	5.48	0.1254	1.237	16.75
		36.00	6.00	0.1251	1.238	16.72
		49.00	7.00	0.1249	1.239	16.68
		60.00	7.75	0.1246	1.240	16.65

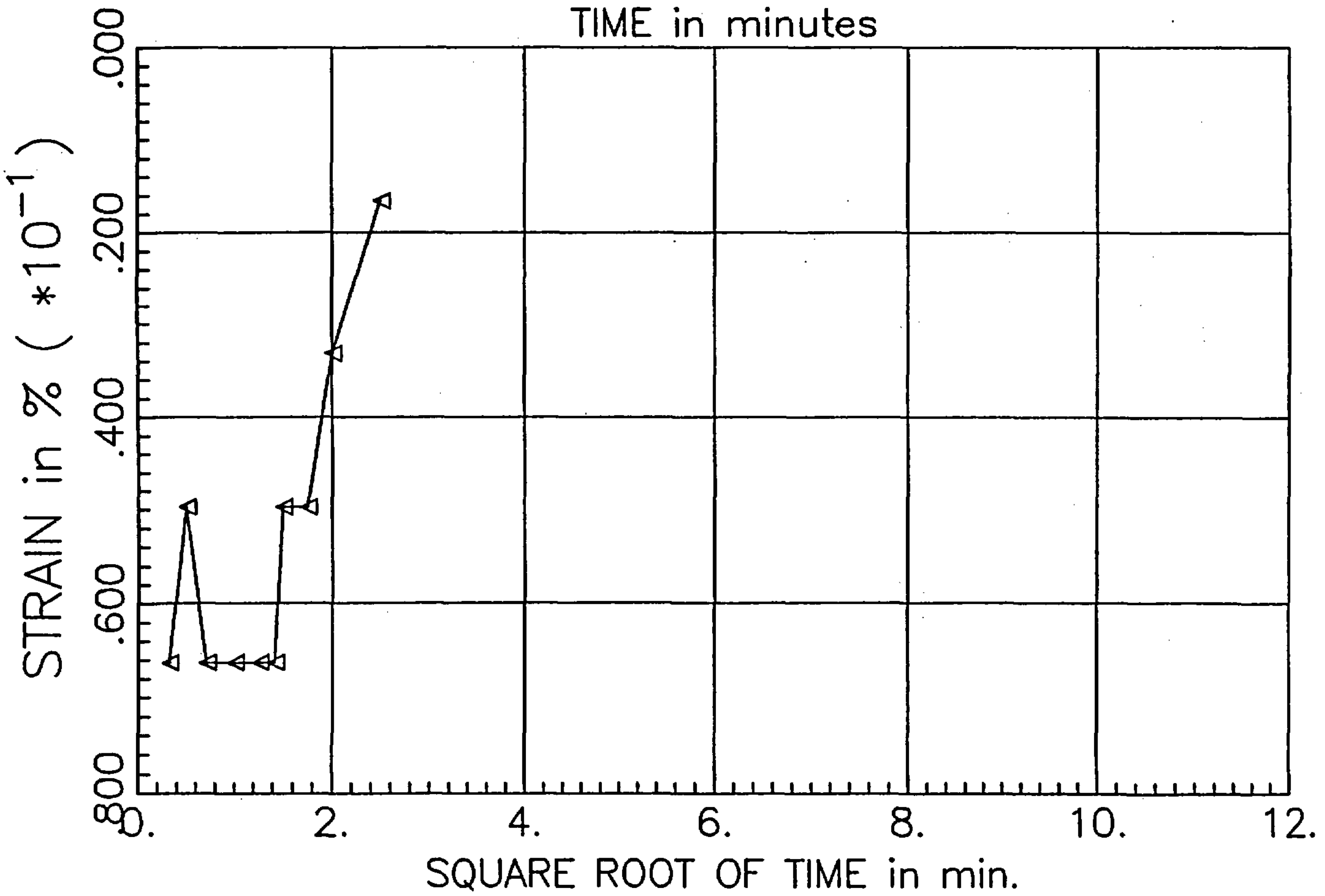
JO 05996.02

$d_{100} = 0.07\%$

Calc: ACS 4-26-99

Ckd: TWC 5/3/99





PRESSURE INCREMENT #1
from 0.00 tsf to 0.10 tsf

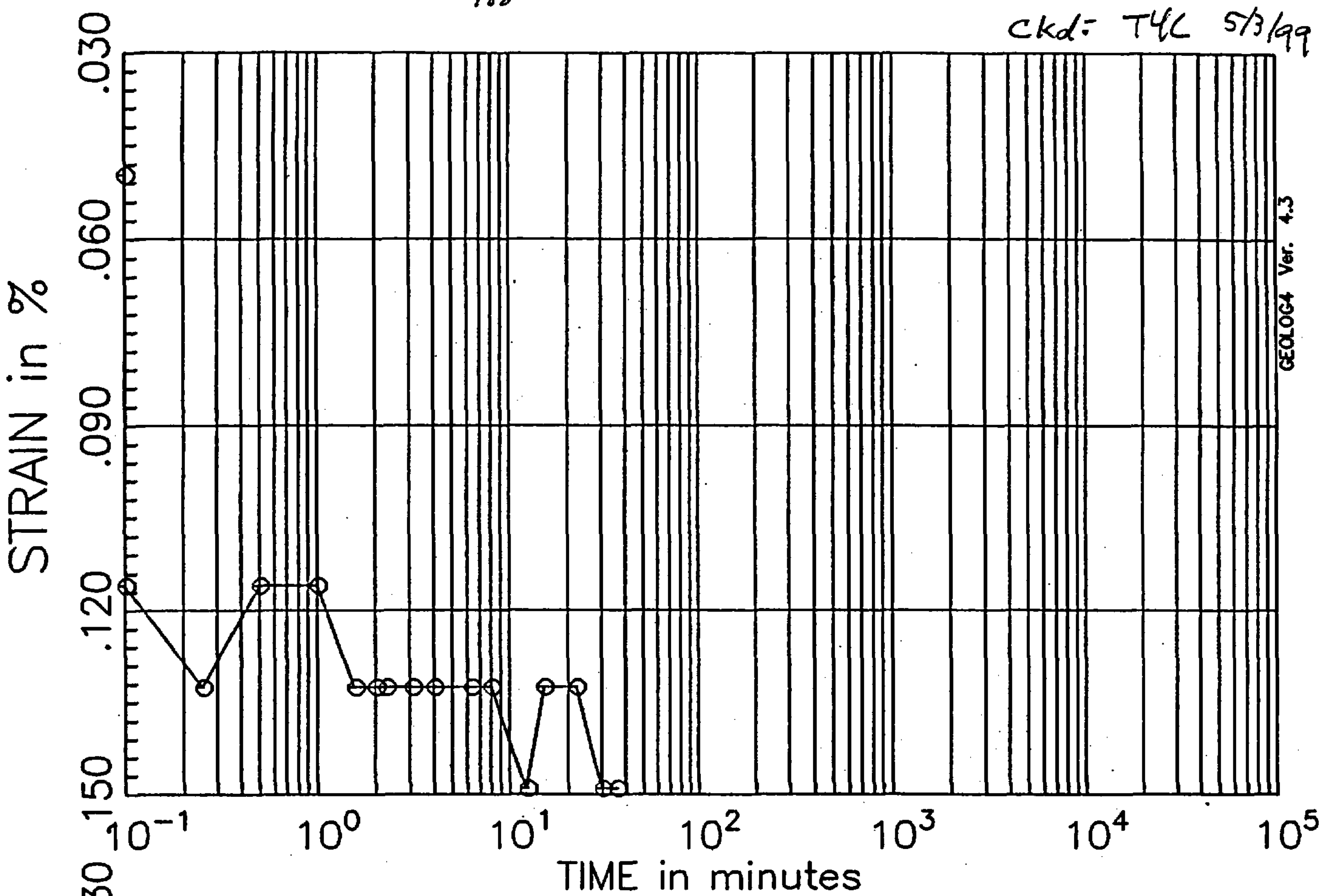
Test No: 5
Testname: CTB4-U2E

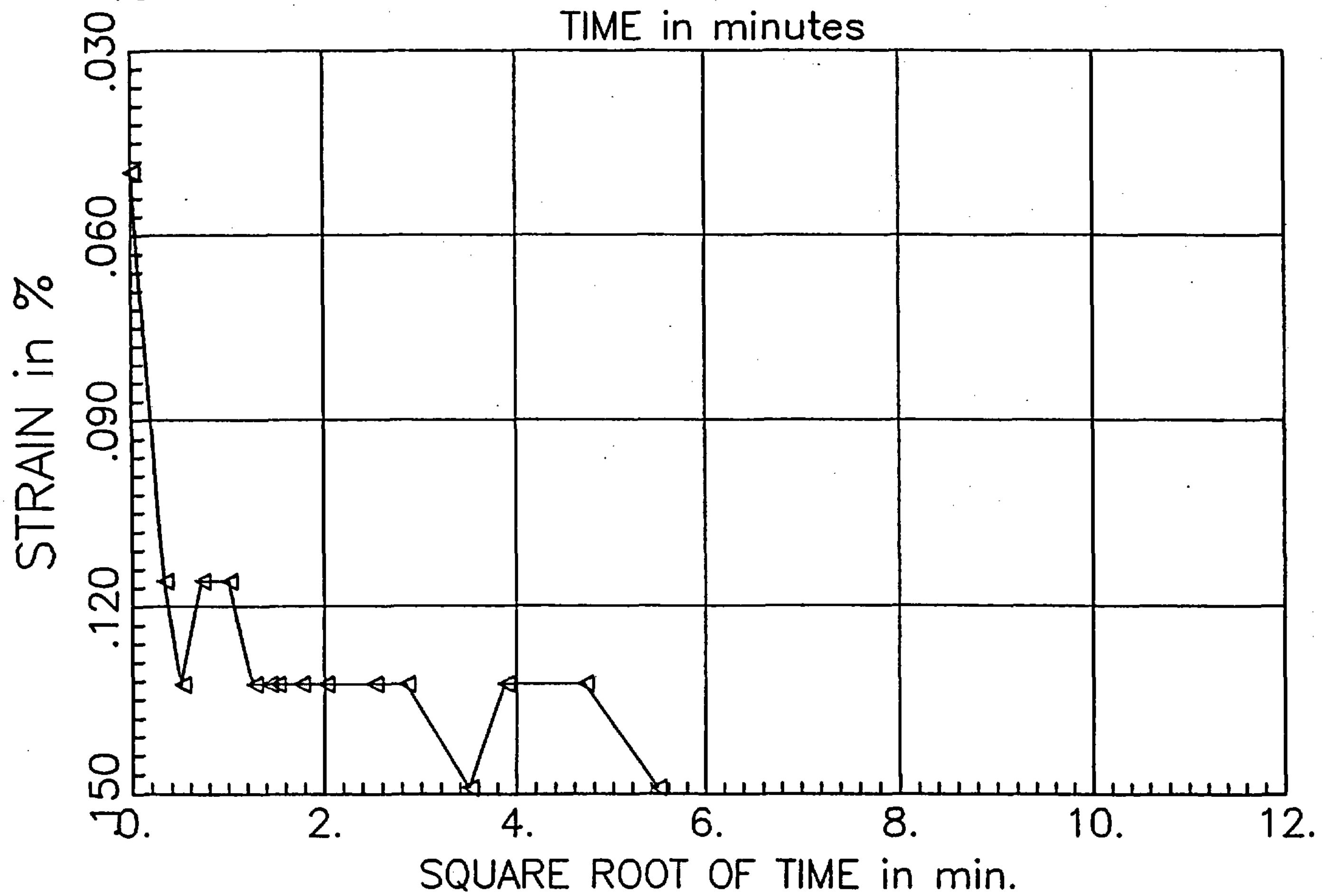
JO 05746.02.

$d_{100} = 0.13\%$

Calc: ACS 4-26-99

Ckd: TYL 5/3/99





PRESSURE INCREMENT #2
from 0.10 tsf to 0.25 tsf

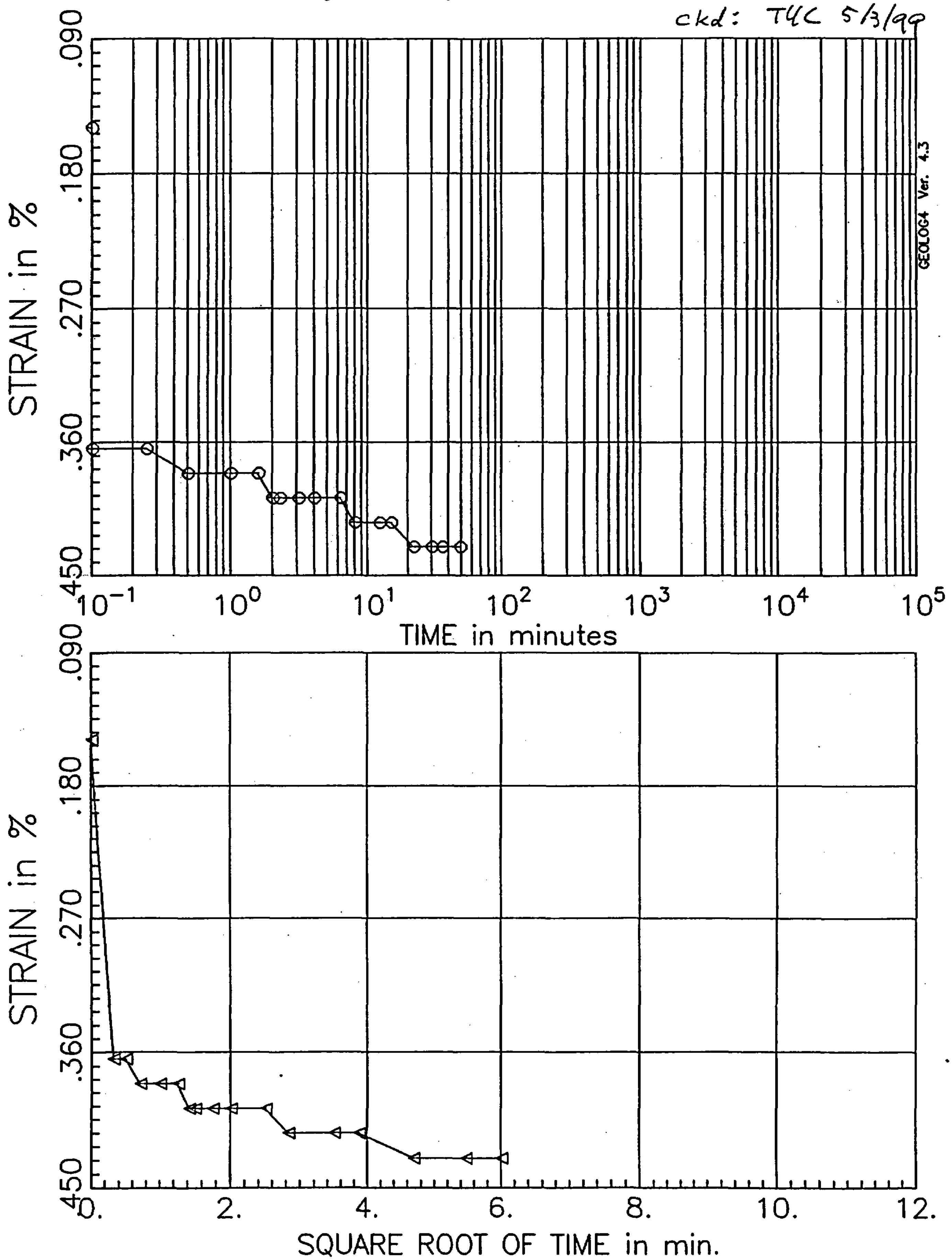
Test No: 5
Testname: CTB4-U2E

JO 05996.02

Calc: ACS 4-26-99

ckd: TYC 5/3/99

$d_{100} = 0.38\%$

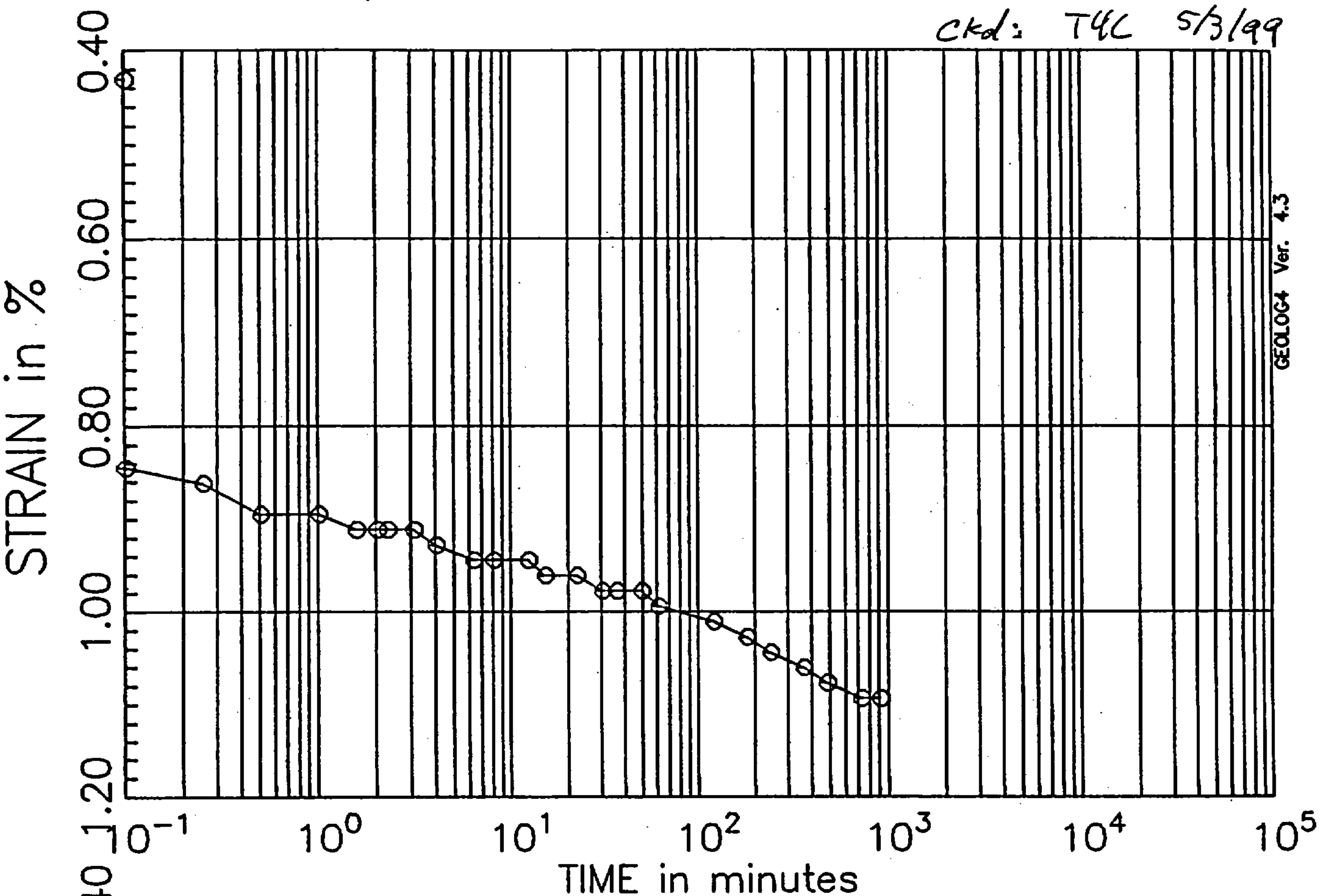


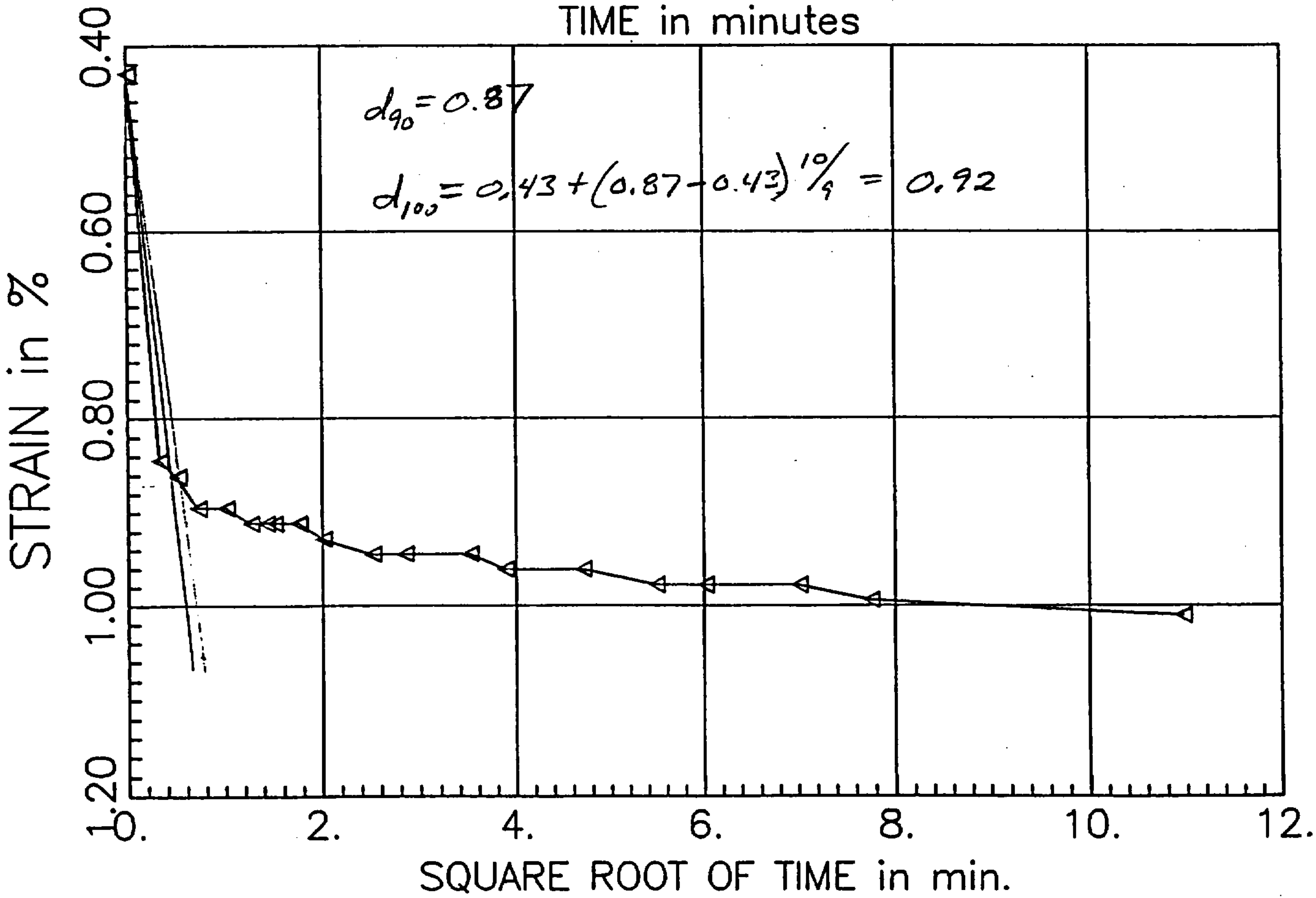
PRESSURE INCREMENT #3
from 0.25 tsf to 0.50 tsf

Test No: 5
Testname: CTB4–U2E

JO 05776.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 0.92\%$





PRESSURE INCREMENT #4
from 0.50 tsf to 1.00 tsf

Test No: 5
Testname: CTB4—U2E

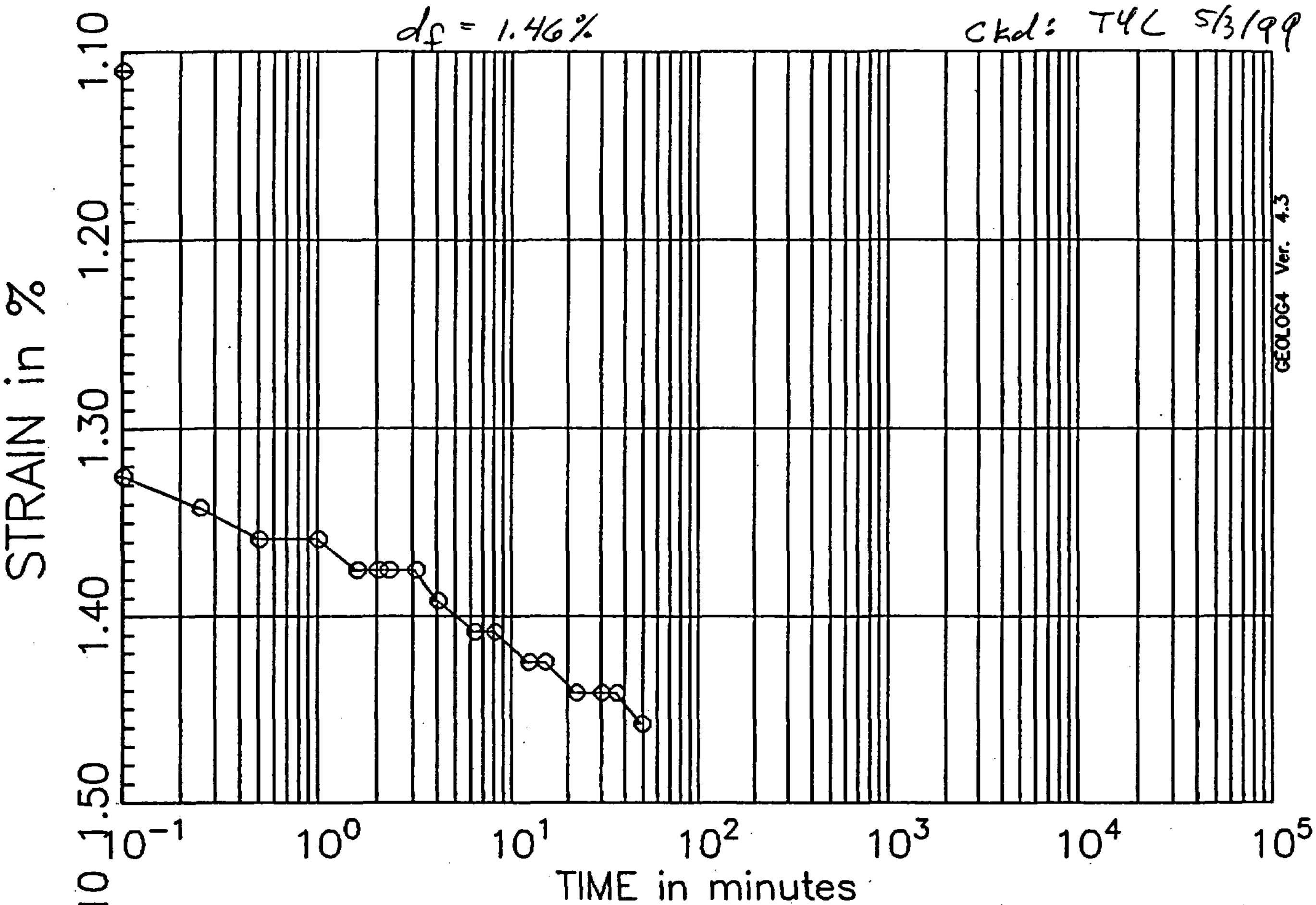
JO 05996.02

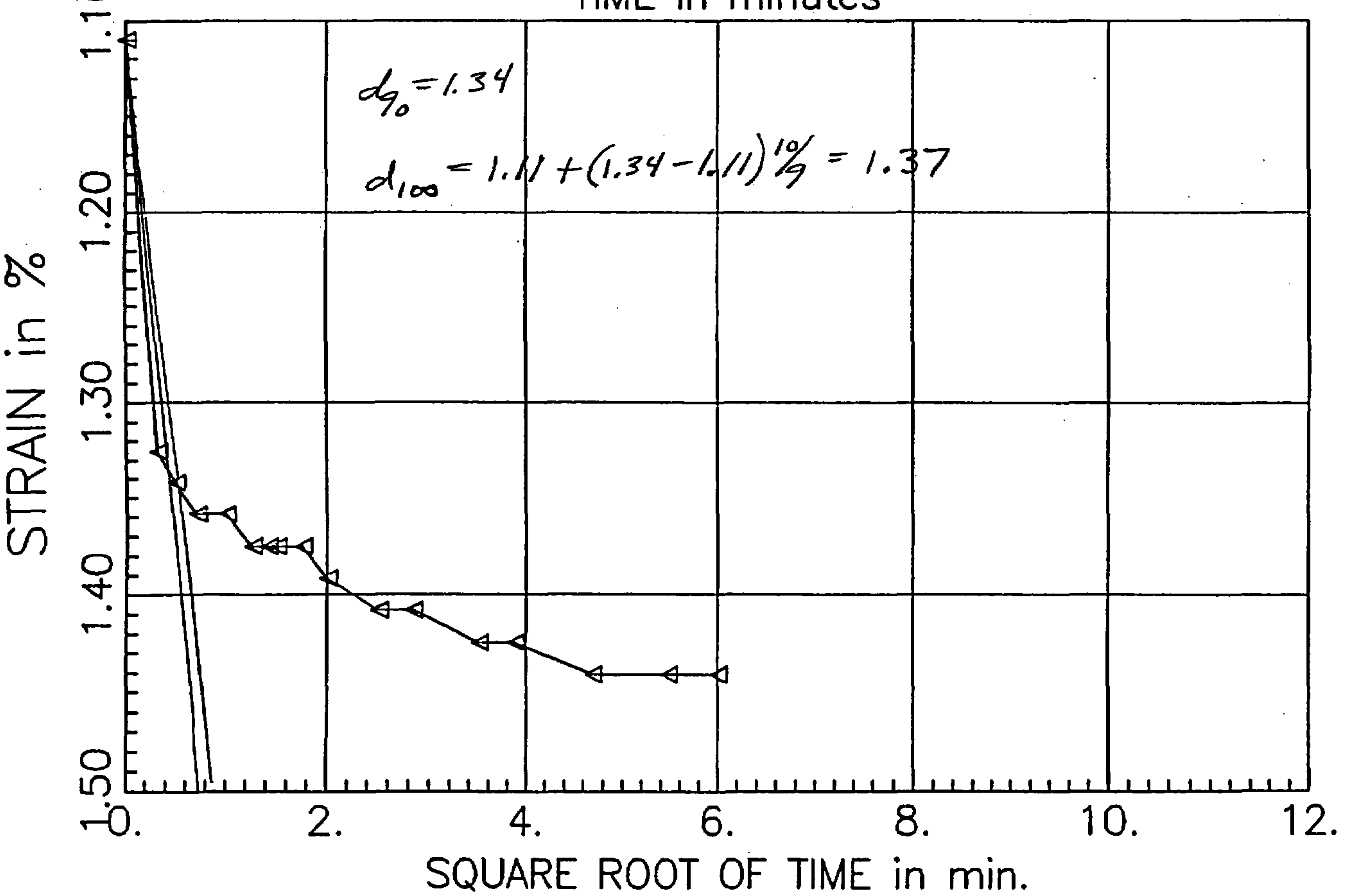
Calc: ACS 4-26-99

Ckd: TYL 5/3/99

d_{100} = 1.37 %

d_f = 1.46 %



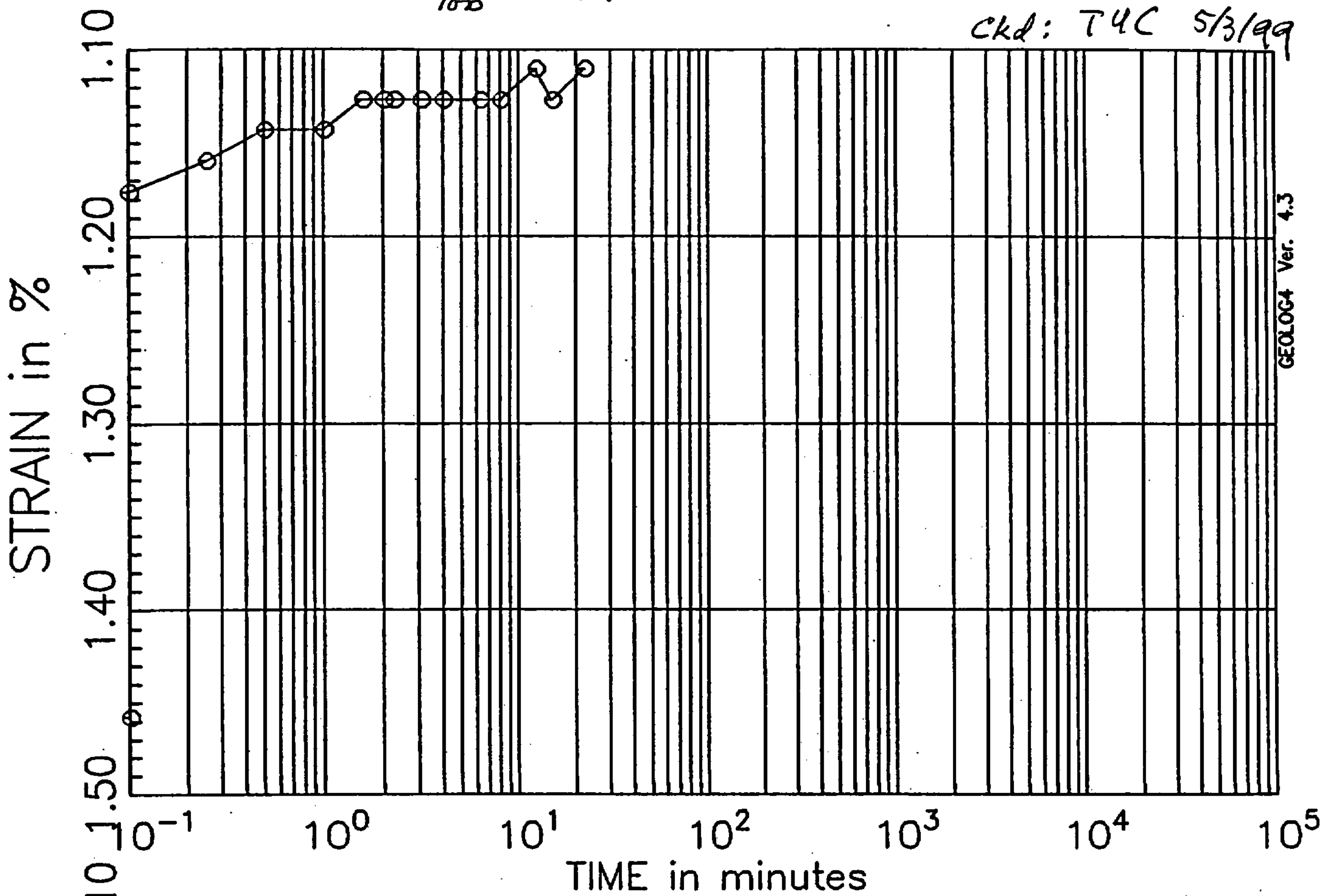


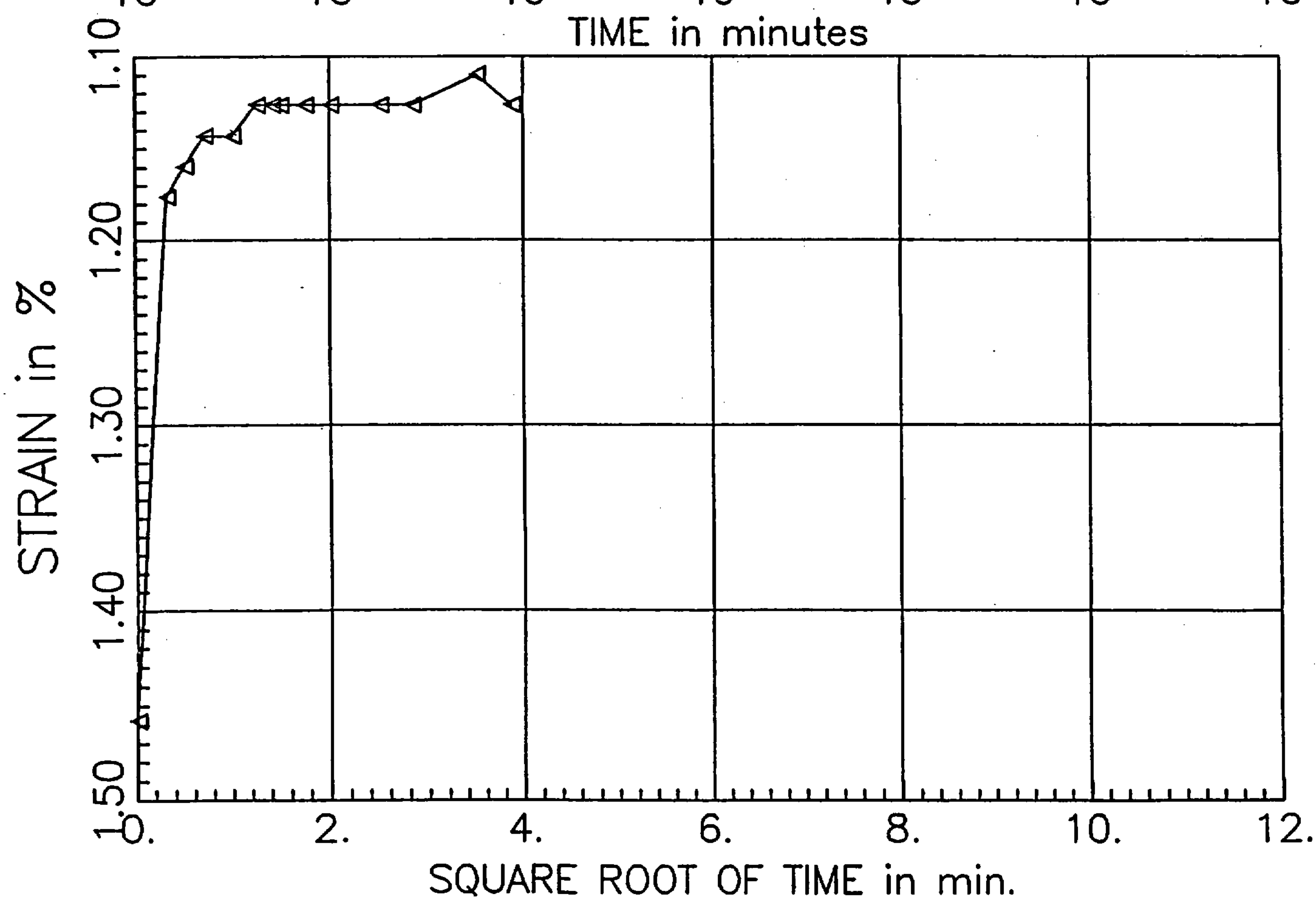
PRESSURE INCREMENT #5
from 1.00 tsf to 1.50 tsf

Test No: 5
Testname: CTB4-U2E

JO 05996.02
Calc: ACS 4-26-99
Ckd: TUC 5/3/99

$d_{100} = 1.14\%$



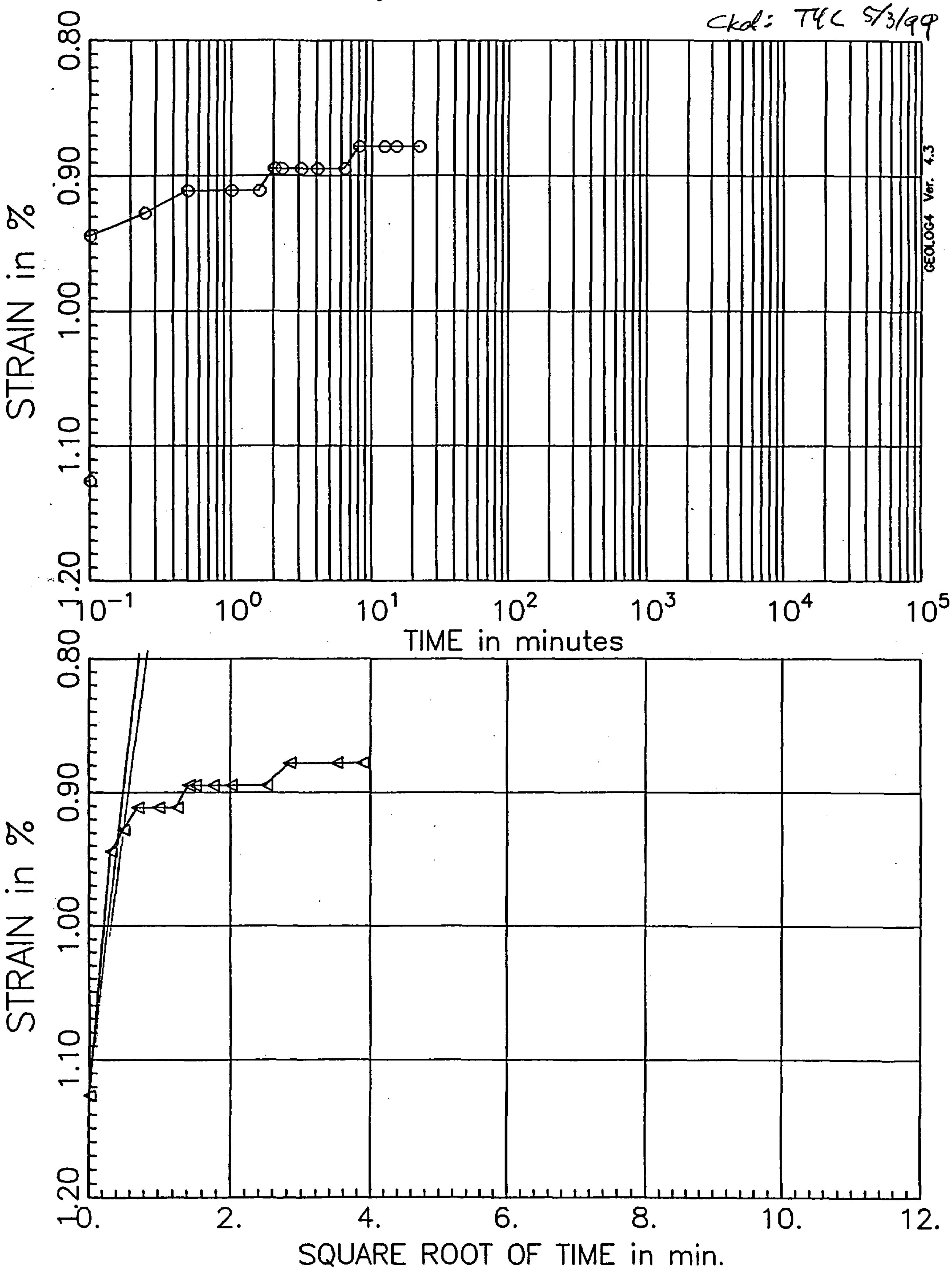


PRESSURE INCREMENT #6
from 1.50 tsf to 0.50 tsf

Test No: 5
Testname: CTB4–U2E

Jo 05996.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 0.91\%$



PRESSURE INCREMENT #7
from 0.50 tsf to 0.25 tsf

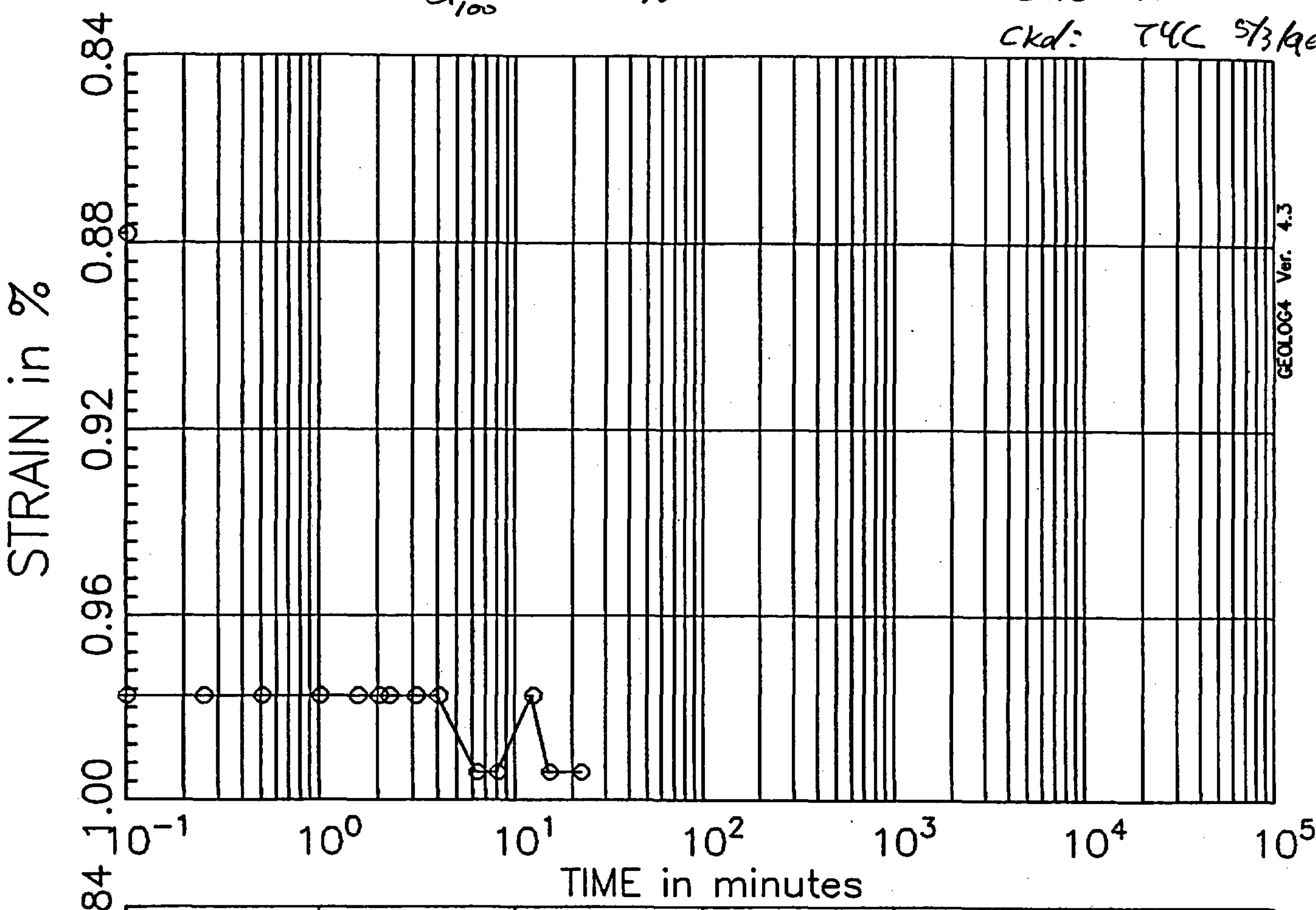
Test No: 5
Testname: CTB4-U2E

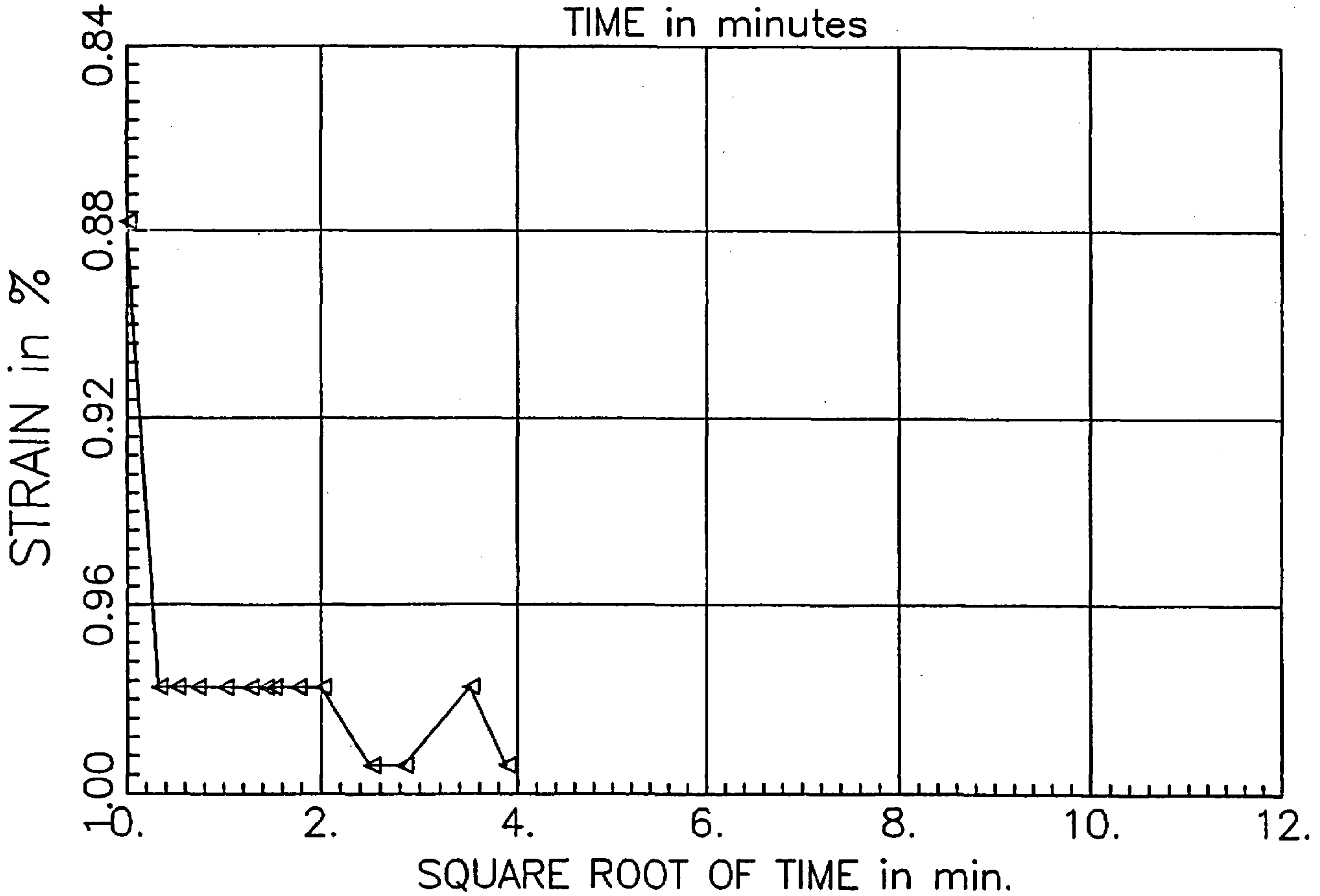
JO 05776.02

Calc: ACS 4-26-99

Ckd: TYC 5/3/99

$d_{100} = 0.98\%$





PRESSURE INCREMENT #8
from 0.25 tsf to 0.50 tsf

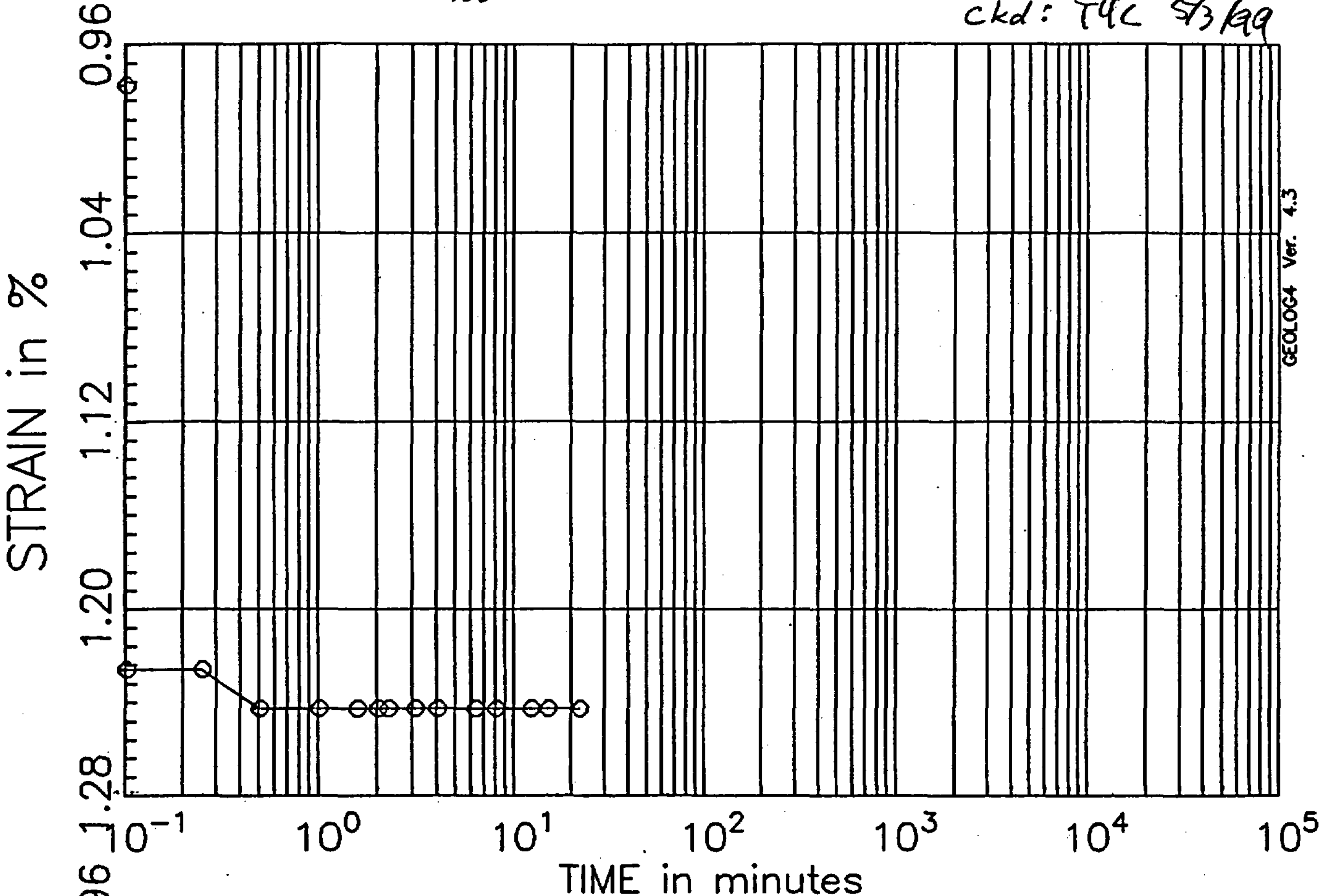
Test No: 5
Testname: CTB4-U2E

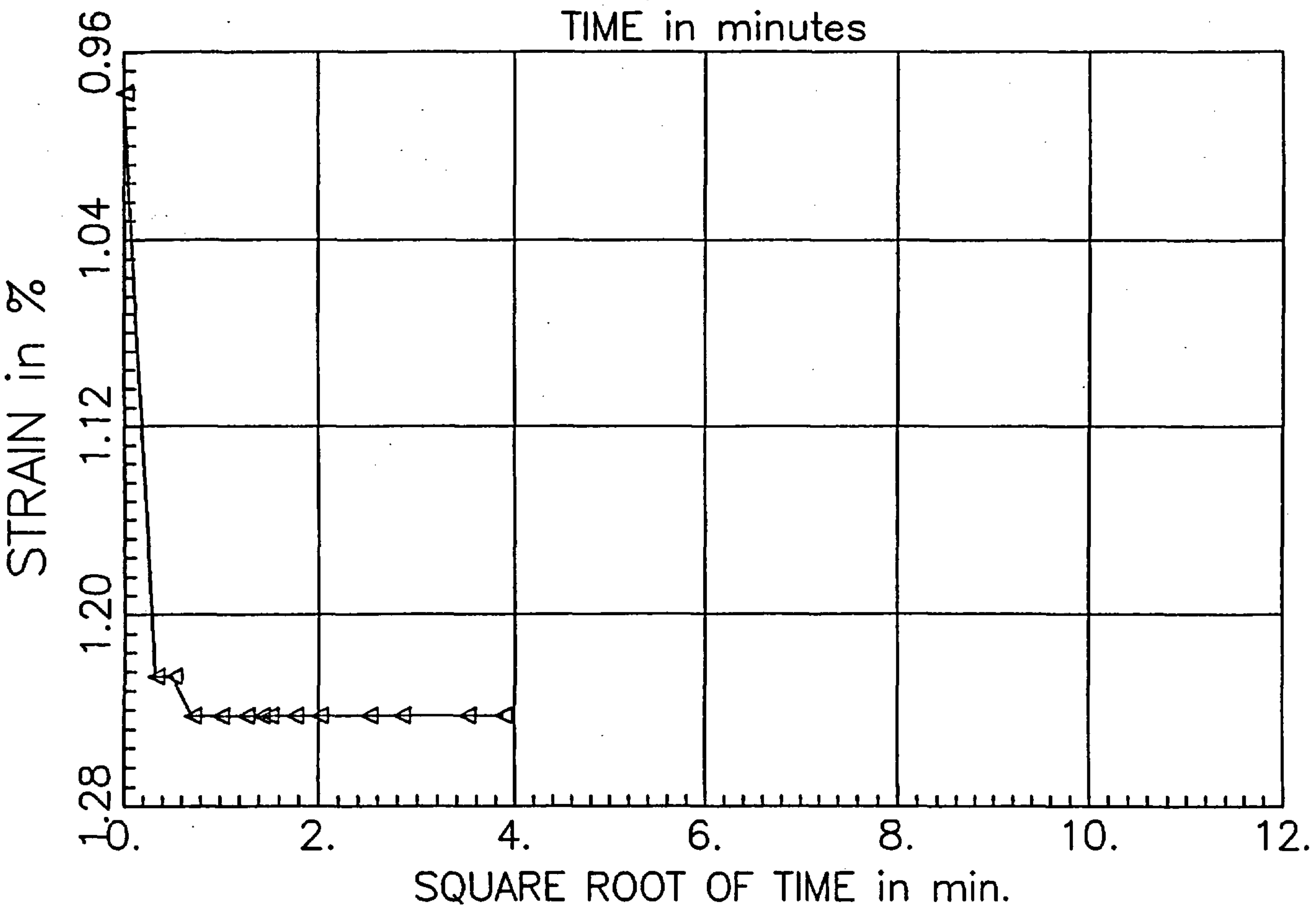
Jo 05996.02

Calc: ACS 4-26-99

Ckd: TYC 5/3/99

$d_{100} = 1.24\%$



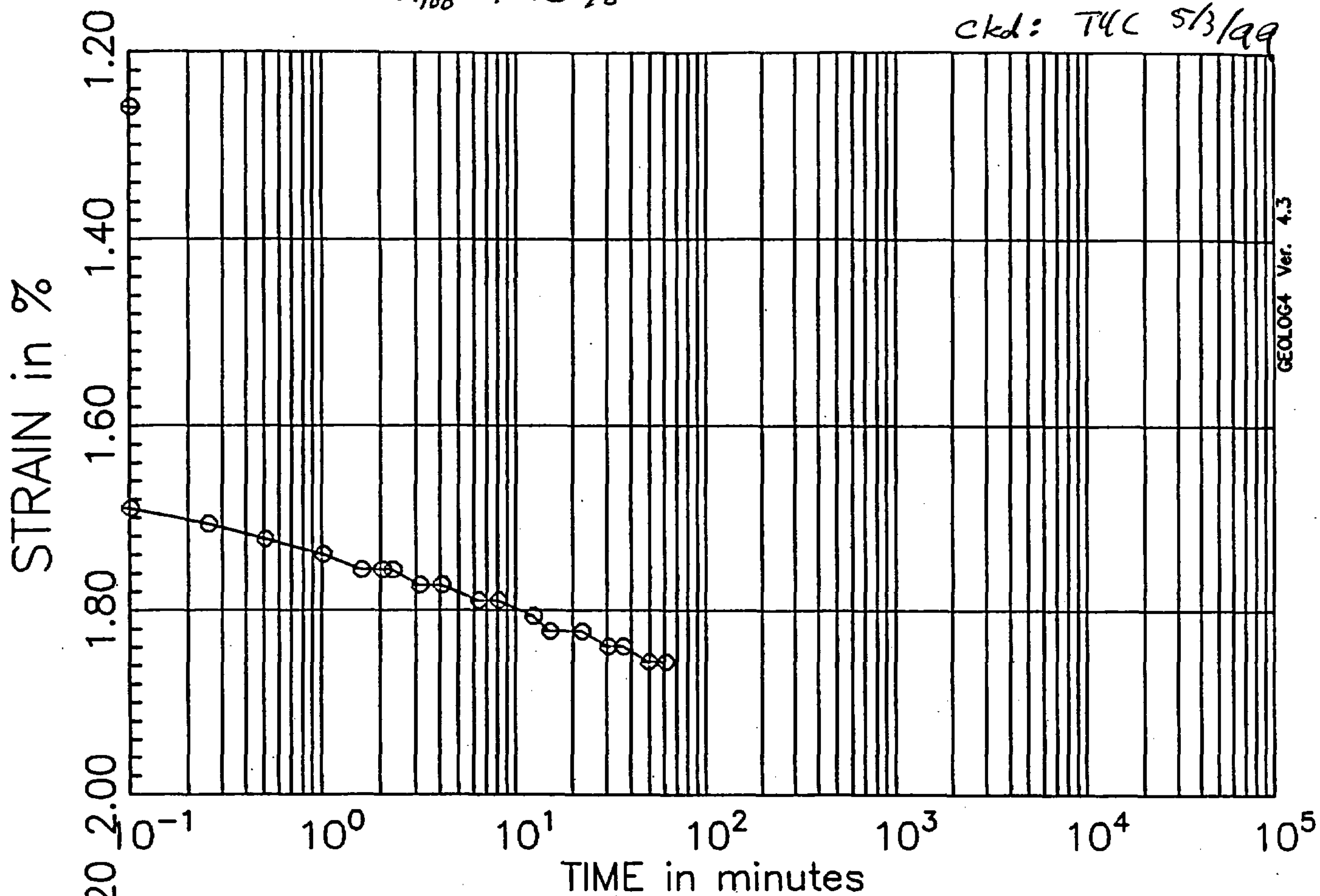


PRESSURE INCREMENT #9
from 0.50 tsf to 1.00 tsf

Test No: 5
Testname: CTB4–U2E

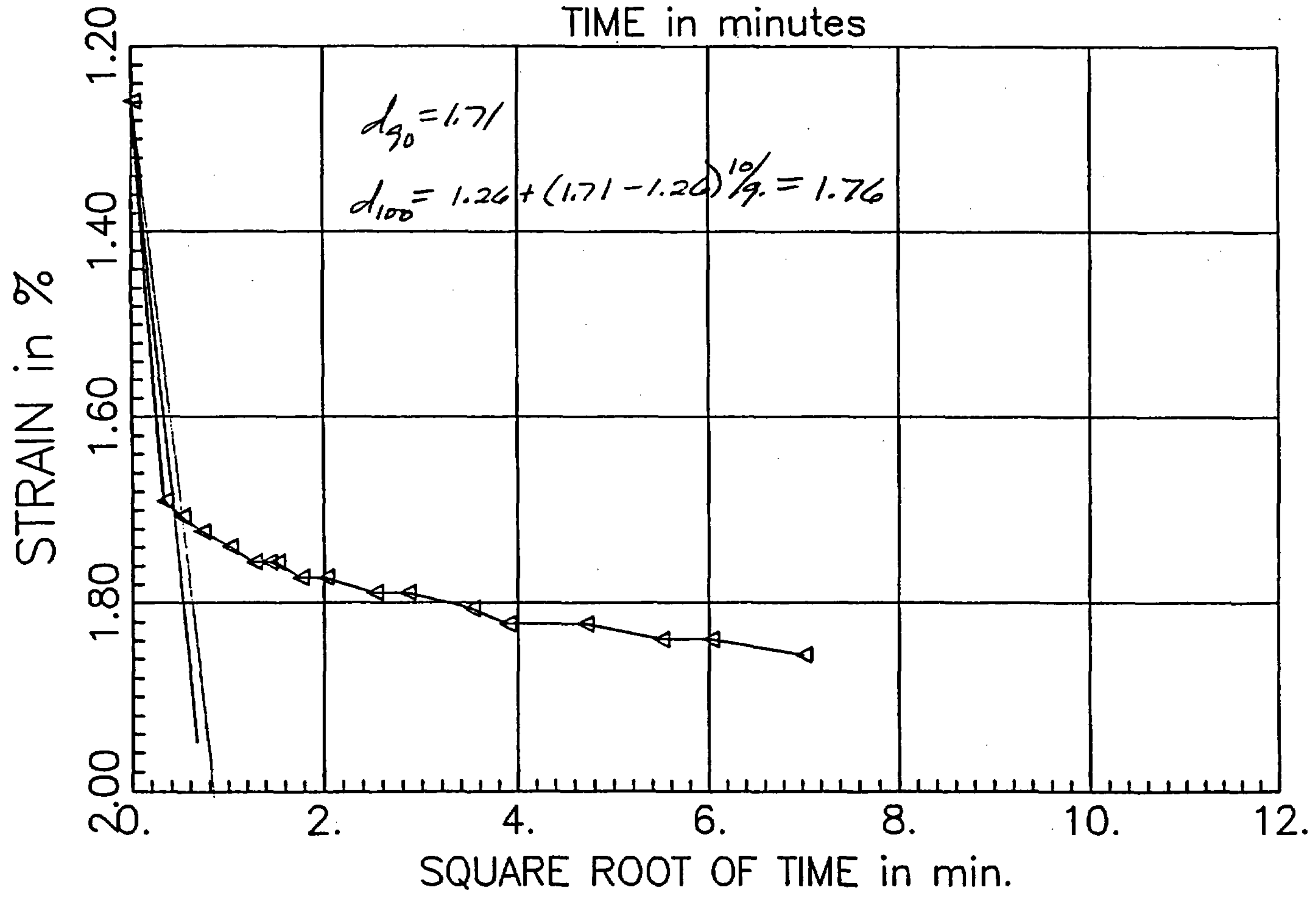
JO 05446.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 1.76\%$



$d_{90} = 1.71$

$d_{100} = 1.26 + (1.71 - 1.26)\frac{10}{9} = 1.76$

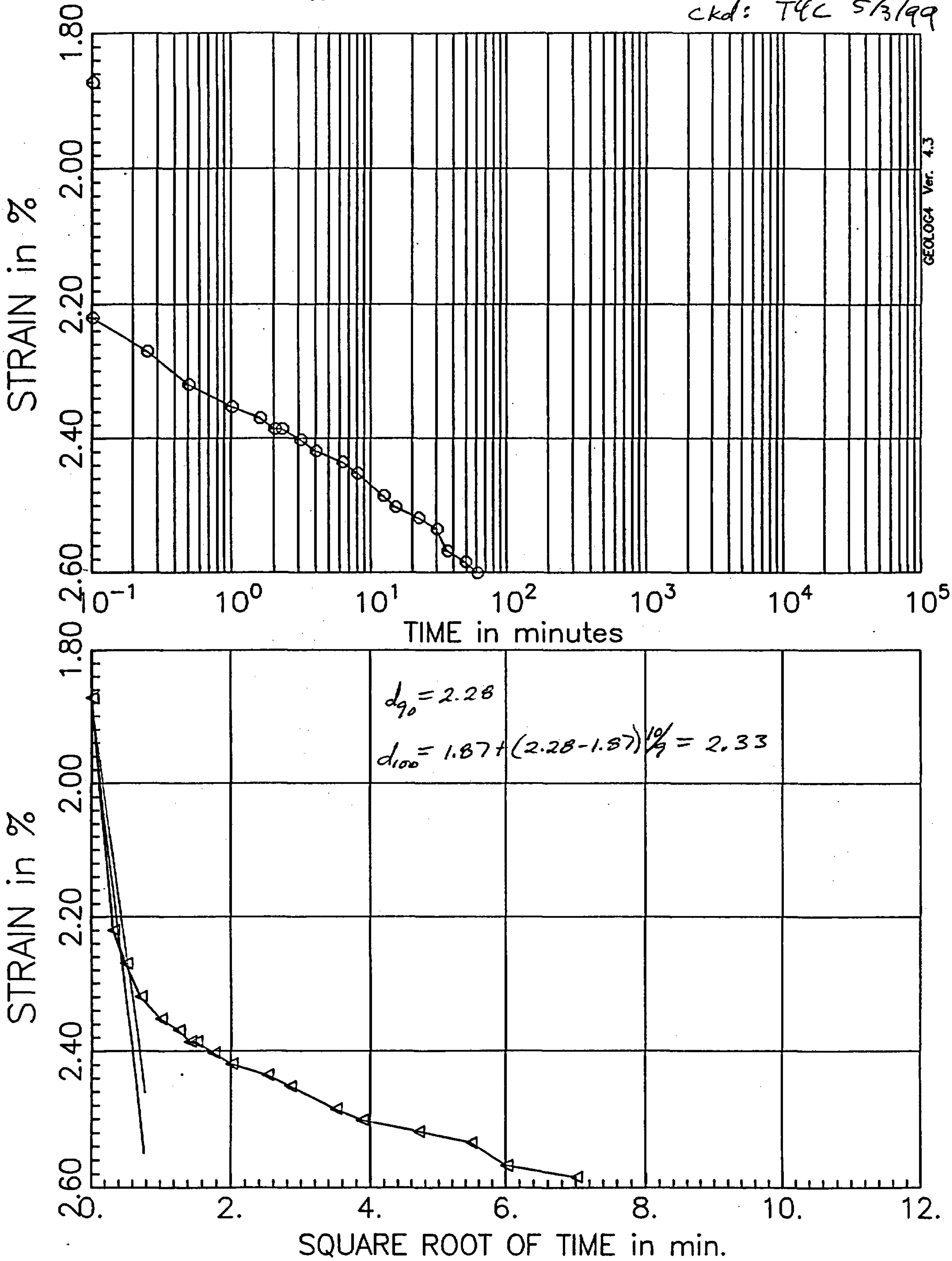


PRESSURE INCREMENT #10
from 1.00 tsf to 2.00 tsf

Test No: 5
Testname: CTB4–U2E

JO 05996.02
Calc: ACS 4-26-99
ckd: TYC 5/3/99

$d_{100} = 2.33\%$

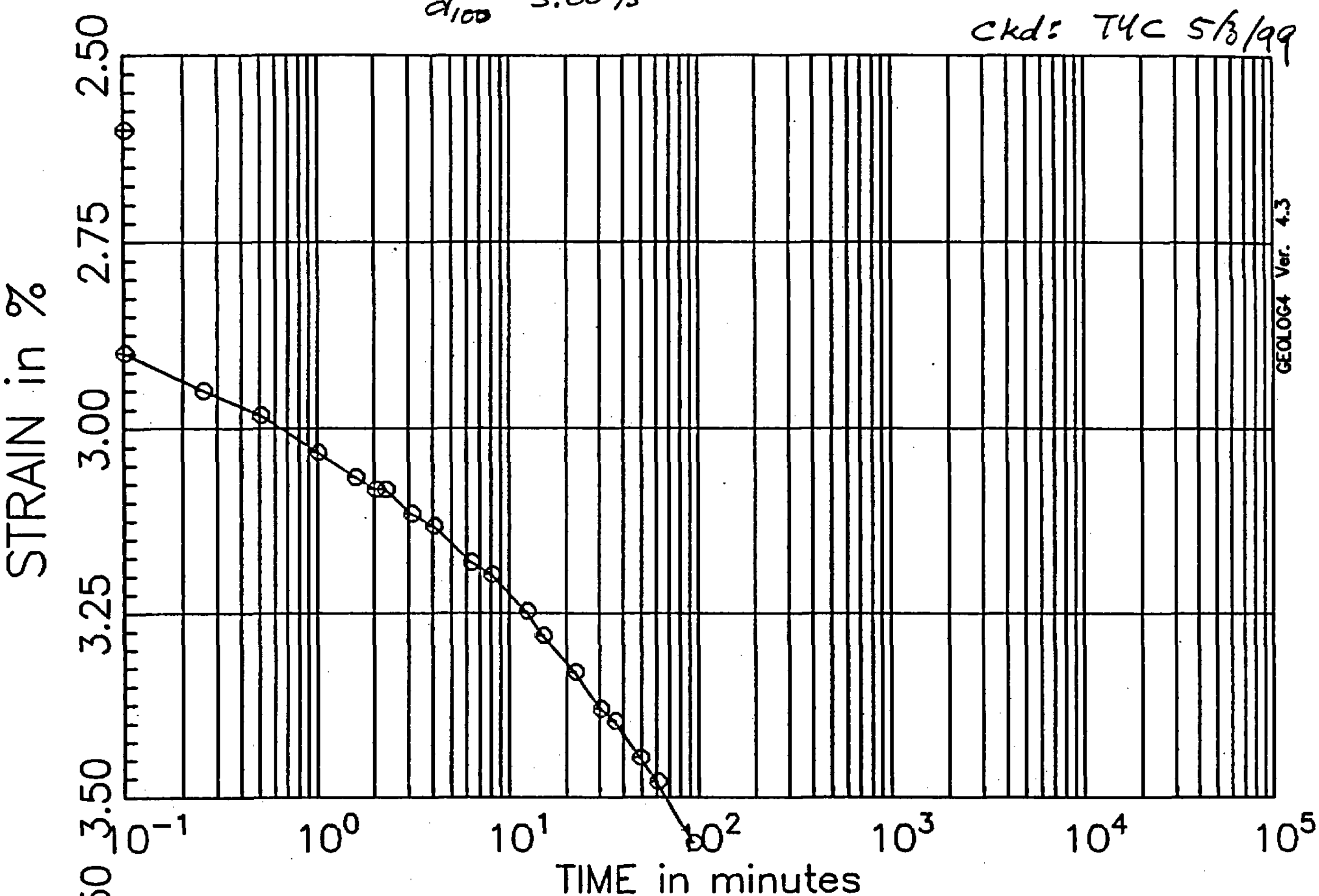


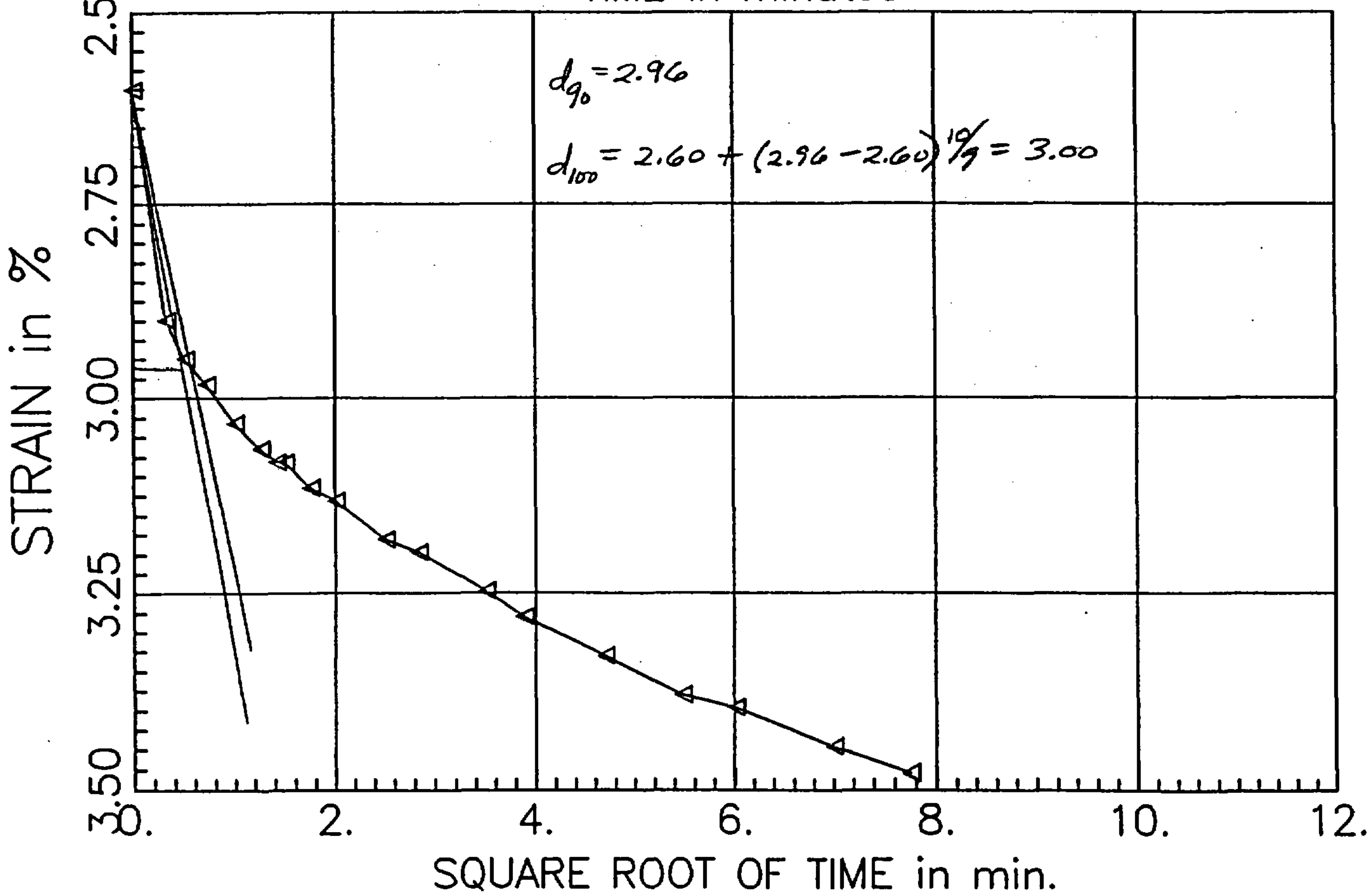
PRESSURE INCREMENT #11
from 2.00 tsf to 3.00 tsf

Test No: 5
Testname: CTB4-U2E

JO 05996.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 3.00\%$



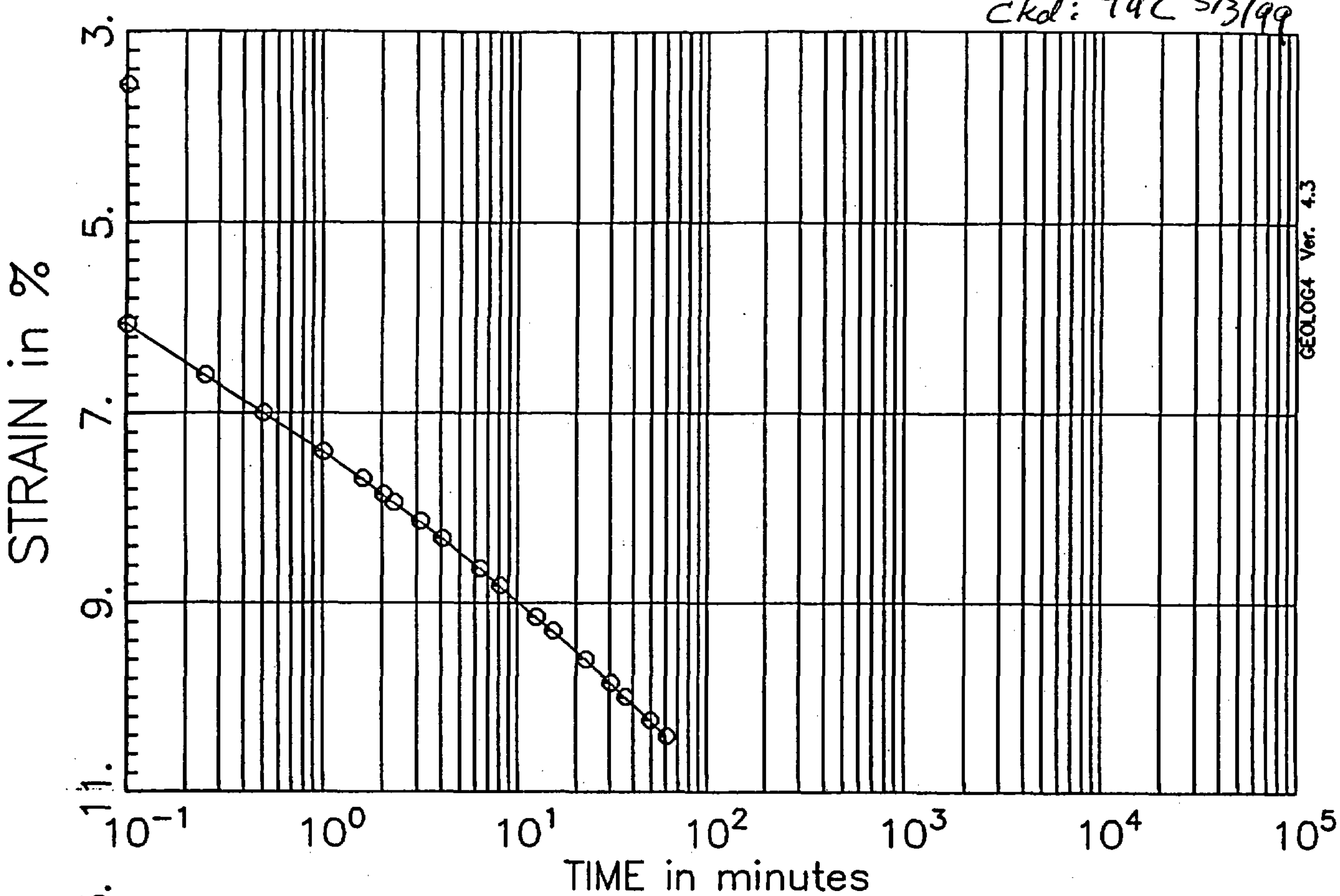


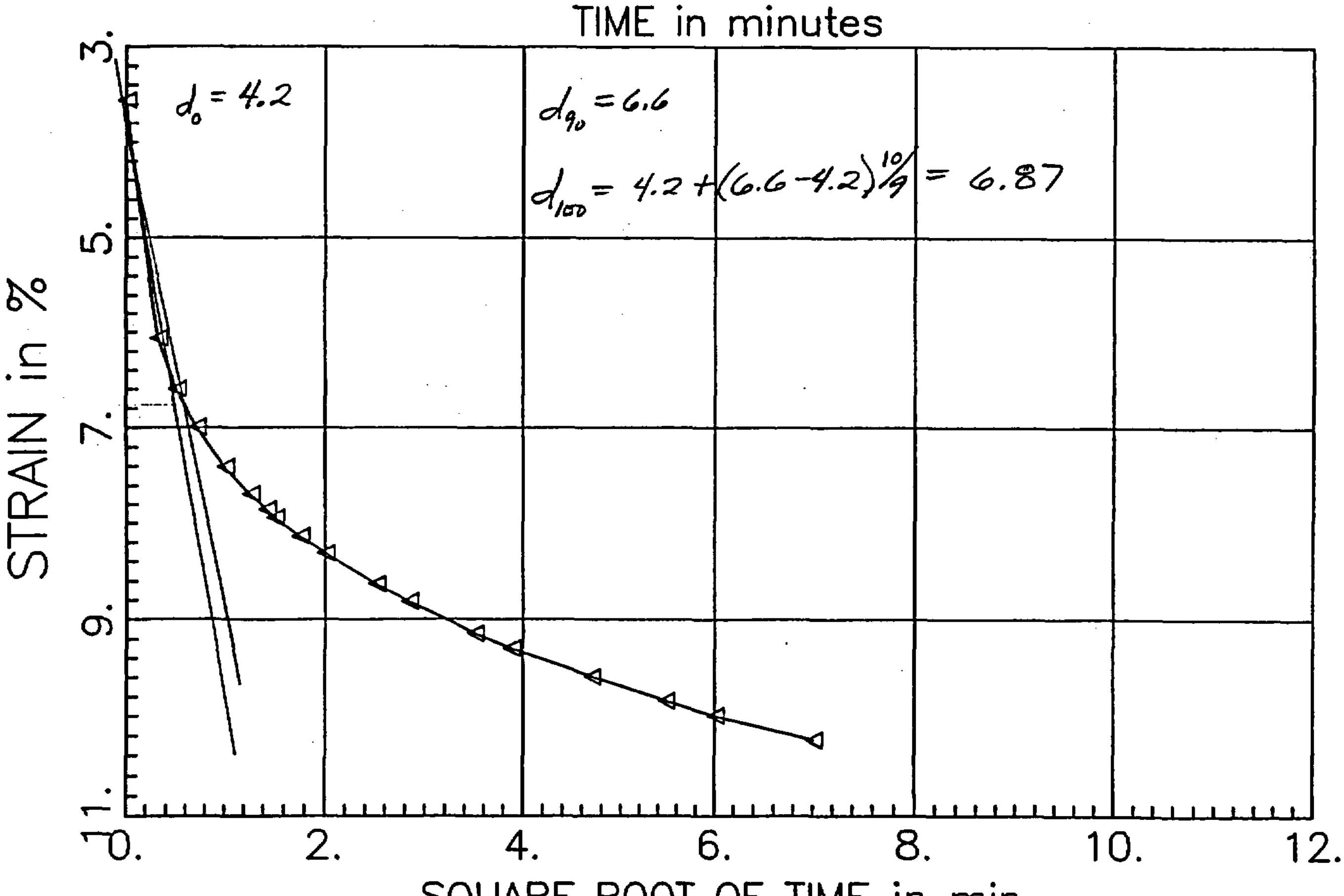
PRESSURE INCREMENT #12
from 3.00 tsf to 4.00 tsf

Test No: 5
Testname: CTB4-U2E

Jo 05996.02
Calc: ACS 4-26-99
Ckd: TQC 5/3/99

$d_{100} = 6.87\%$



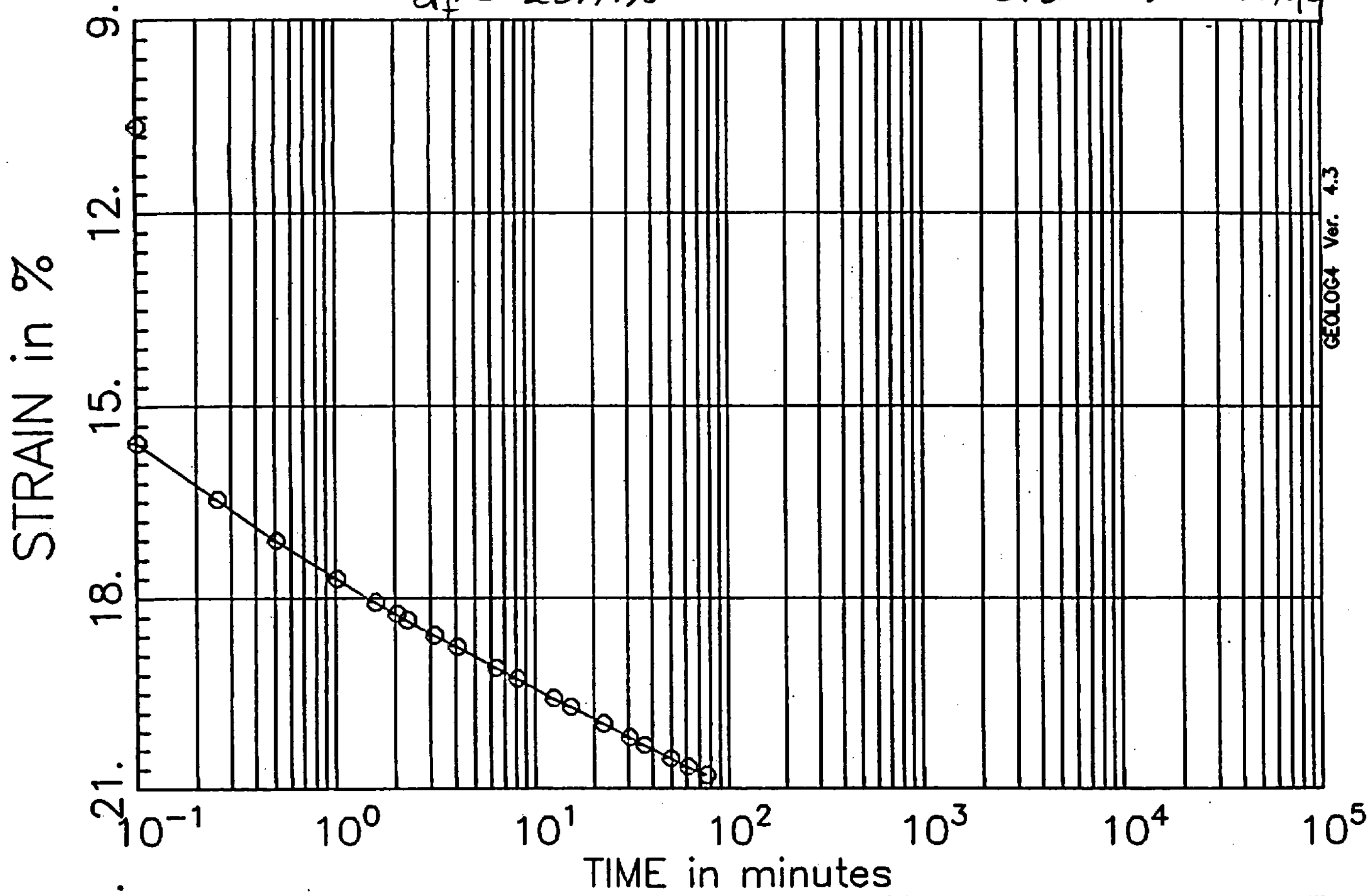


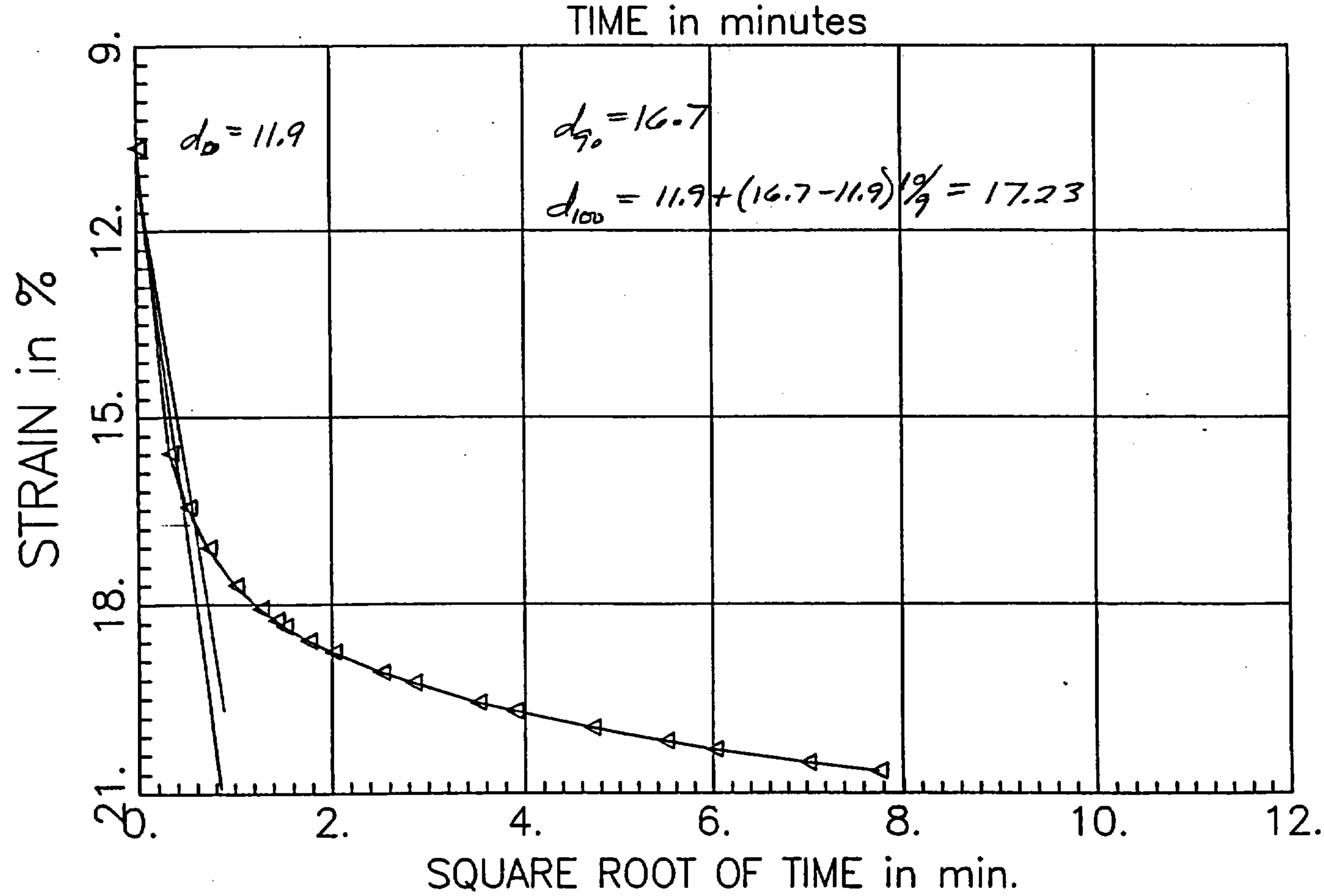
PRESSURE INCREMENT #13
from 4.00 tsf to 8.00 tsf

Test No: 5
Testname: CTB4-U2E

Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 17.23\%$
$d_f = 20.79\%$





PRESSURE INCREMENT #14
from 8.00 tsf to 16.00 tsf

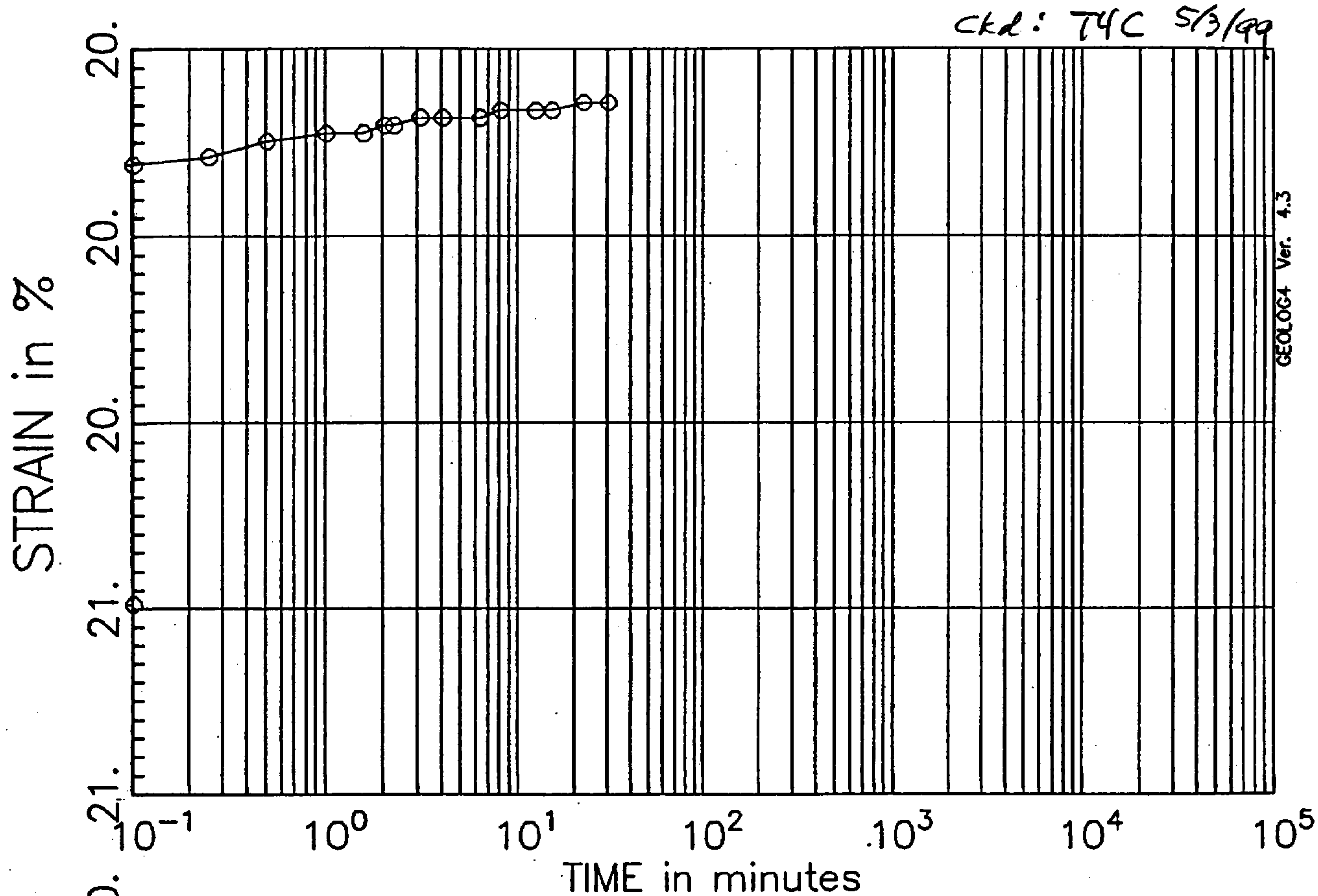
Test No: 5
Testname: CTB4-U2E

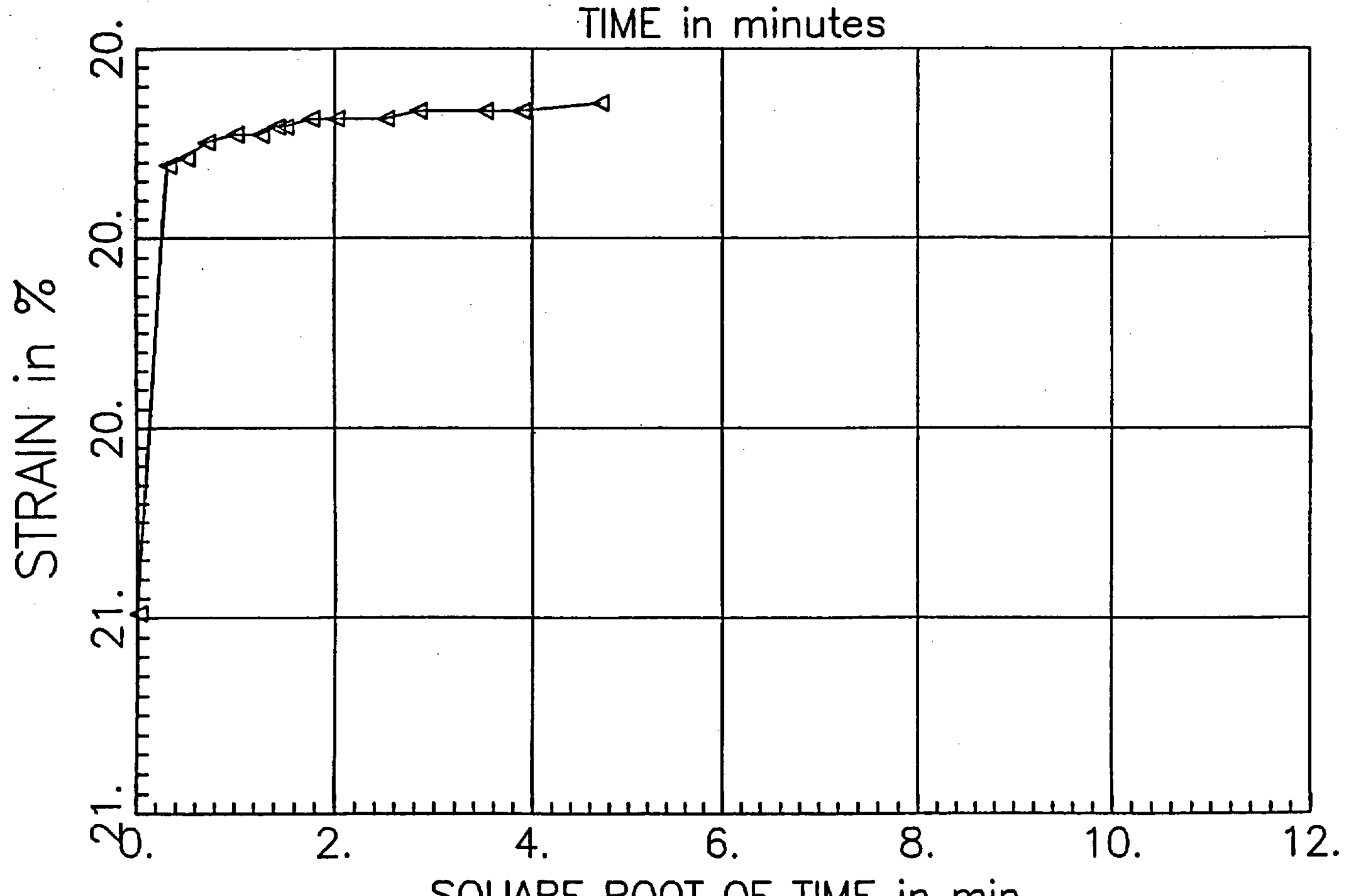
Jo 05996.02

$d_{100} = 19.78\%$

Calc: ACS 4-26-99

Ckd: TYC 5/3/99



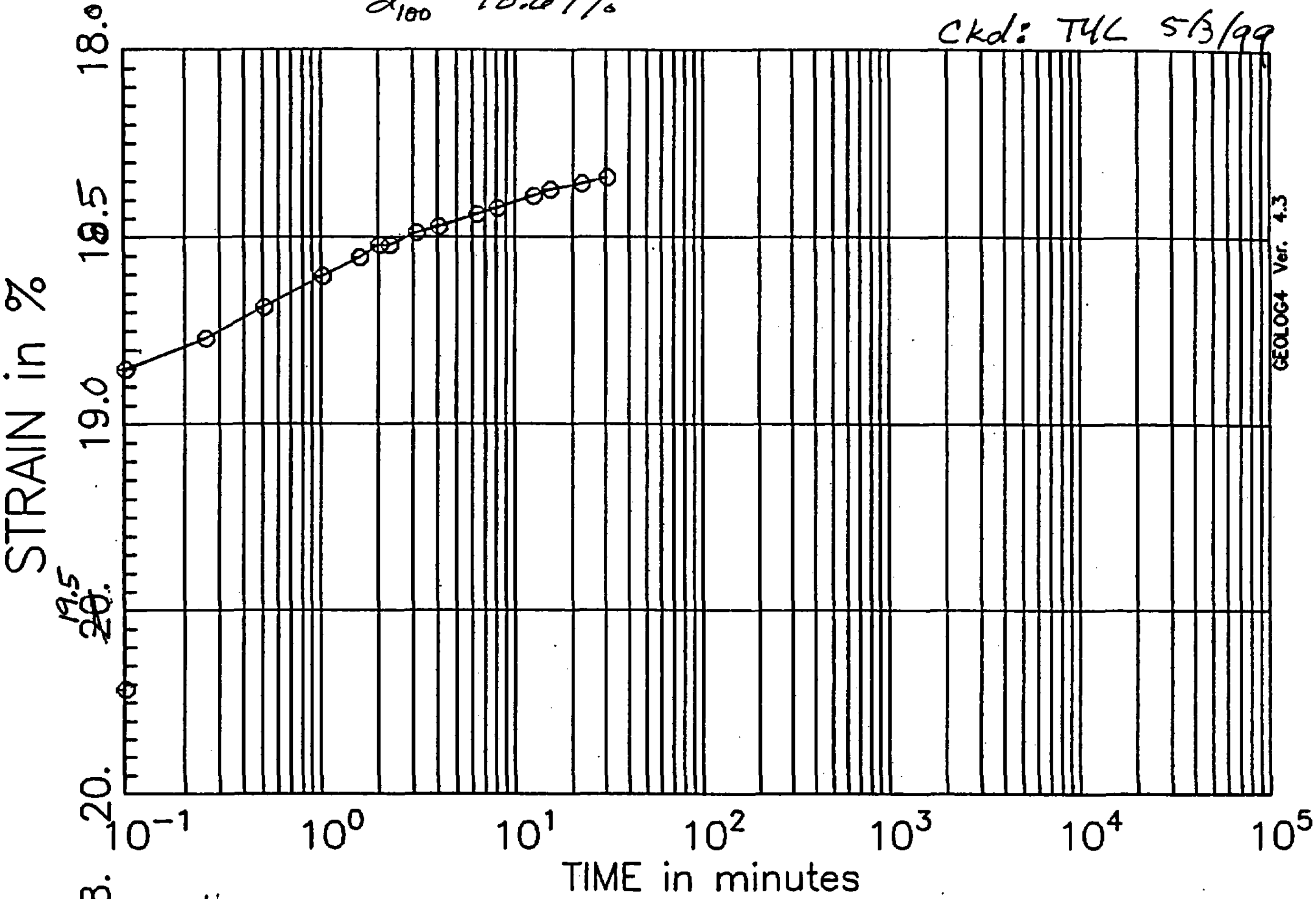


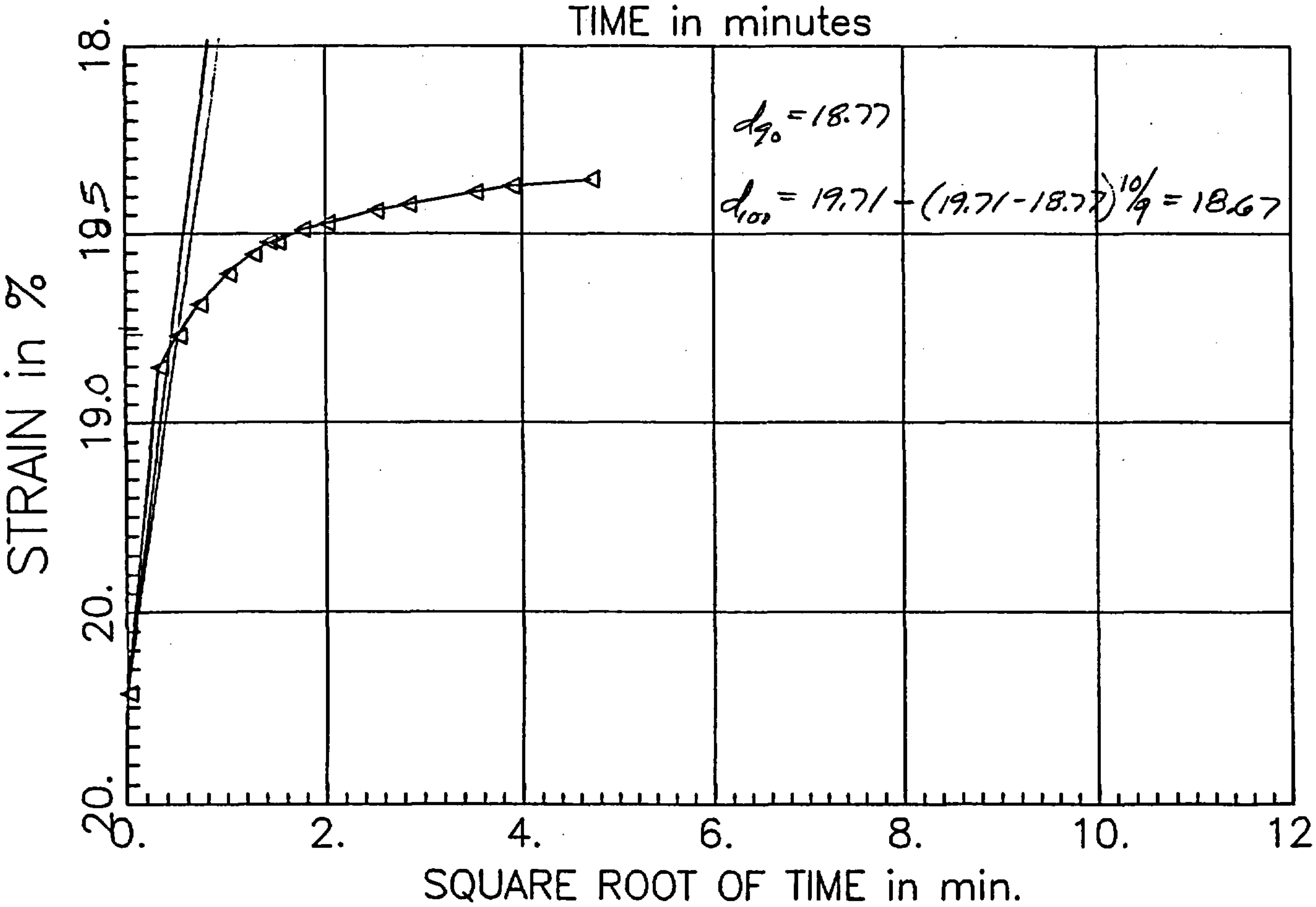
PRESSURE INCREMENT #15
from 16.00 tsf to 4.00 tsf

Test No: 5
Testname: CTB4–U2E

JO 05996.02
Calc: ACS 4-26-99
Ckd: TYL 5/3/99

$d_{100} = 18.67\%$

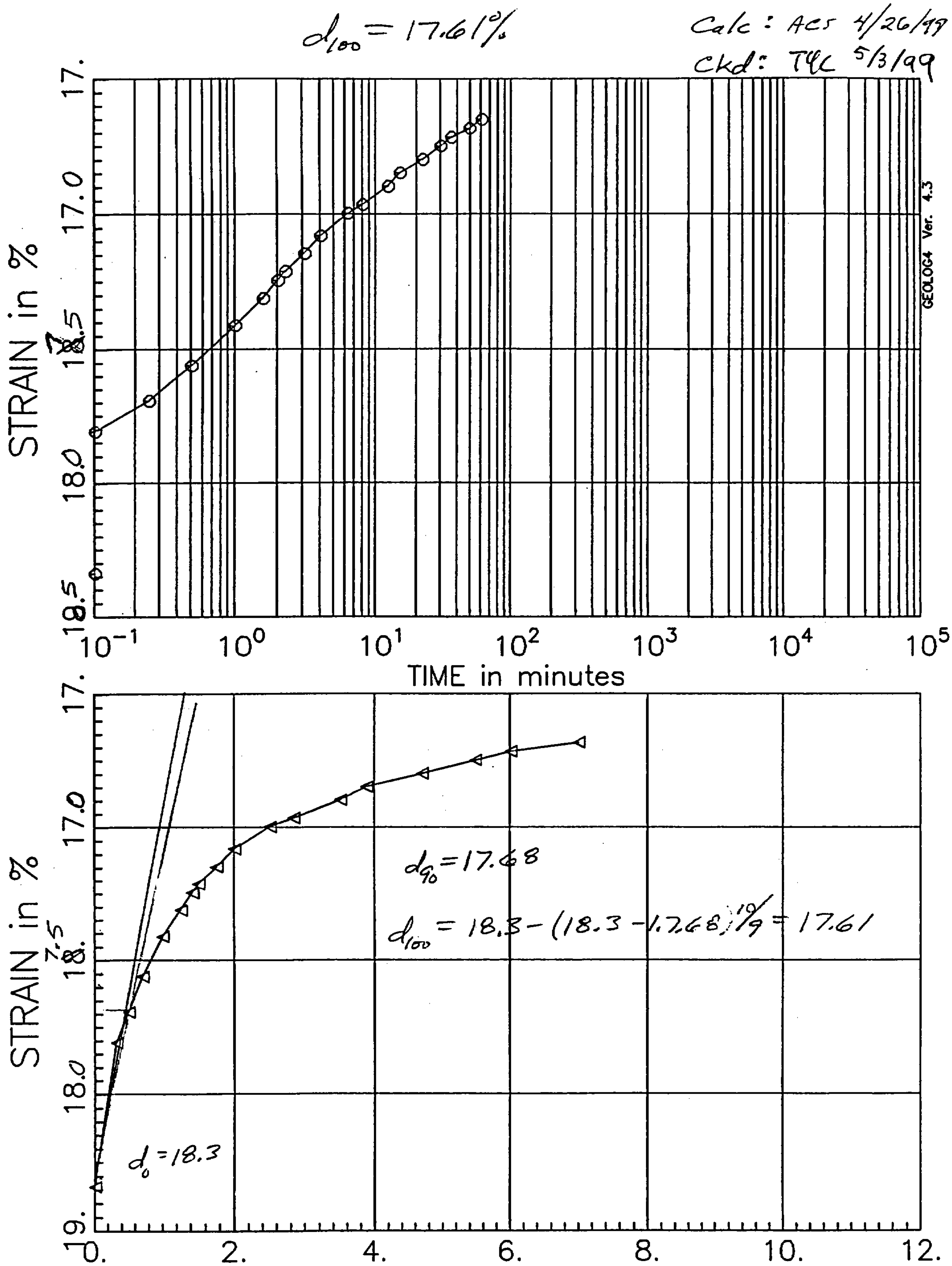




PRESSURE INCREMENT #16
from 4.00 tsf to 1.00 tsf

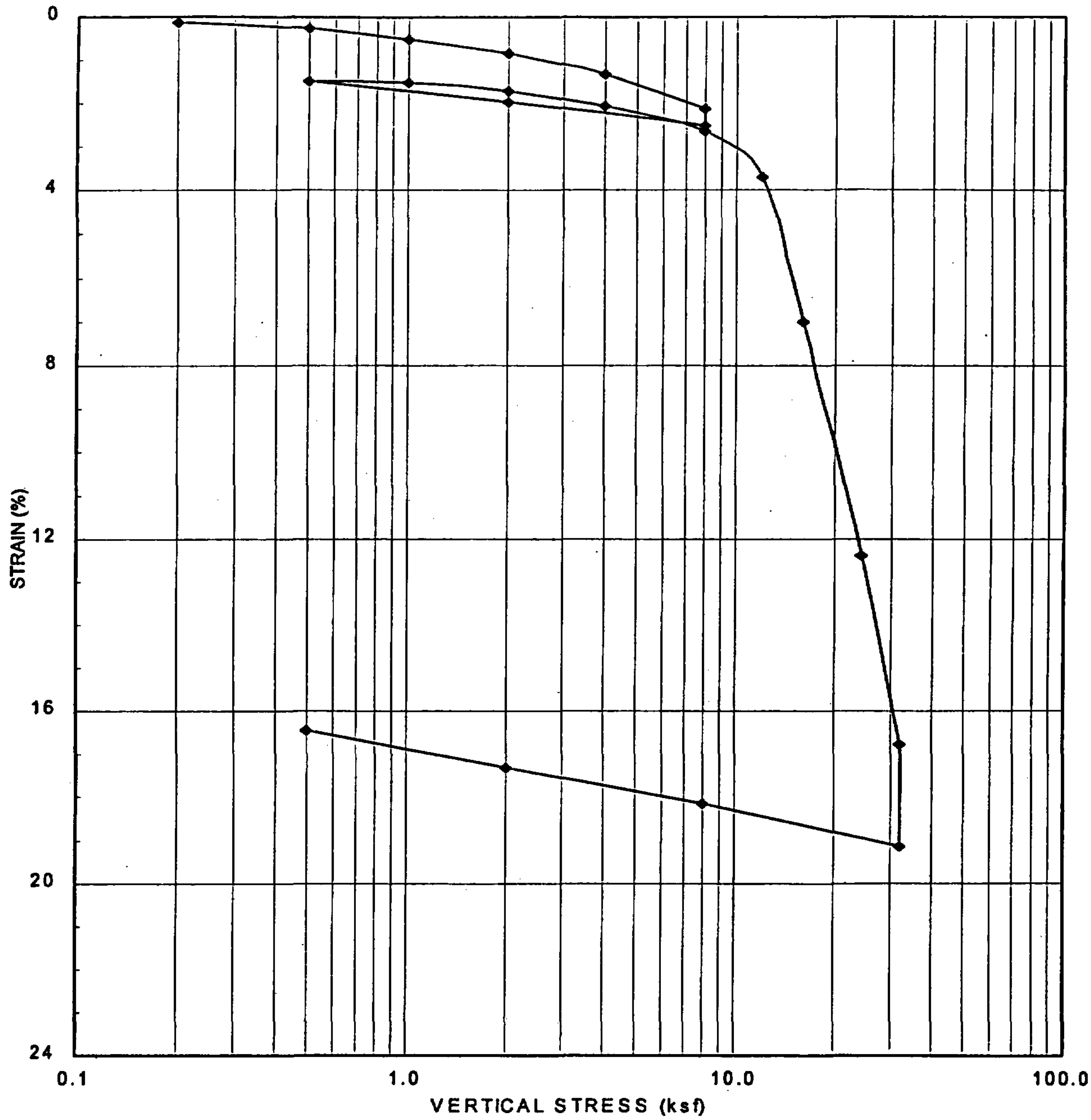
Test No: 5
Testname: CTB4–U2E

Jo 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99



PRESSURE INCREMENT #17
from 1.00 tsf to 0.25 tsf

Test No: 5
Testname: CTB4-U2E



SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	4/14/99
SAMPLE:	U-12C	TESTED BY:	ACS
DEPTH:	23.5 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	52.4 %	43.6 %	2.72 (est)
DRY UNIT WEIGHT:	63.3 pcf	75.0 pcf	
VOID RATIO:	1.683	1.265	
SATURATION:	84.6 %	93.8 %	

NOTE: Sample was not inundated

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING CTB-5, SAMPLE U-12C	JO 05996.02 April 1999

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	4/14/99
SAMPLE:	U-12C	TESTED BY:	ACS
DEPTH:	23.5 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	52.4 %	43.6 %
DRY UNIT WEIGHT:	63.3 pcf	75.0 pcf
VOID RATIO:	1.683	1.265
SATURATION:	84.6 %	93.8 %
HEIGHT:	1.901 cm	1.604 cm
AREA:	31.60 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE ksf	STRAIN %
0.20	0.12
0.50	0.27
1.00	0.50
2.00	0.84
4.00	1.32
8.00	2.13
8.00	2.50
2.00	1.97
0.50	1.47
1.00	1.52
2.00	1.72
4.00	2.07
8.00	2.64
12.00	3.69
16.00	7.02
24.00	12.37
32.00	16.76
32.00	19.13
8.00	18.16
2.00	17.32
0.50	16.44

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 1
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	15:41:09	0.10	0.32	0.0009	1.680	0.12
		0.25	0.50	0.0009	1.680	0.12
		0.50	0.71	0.0010	1.679	0.13
		1.00	1.00	0.0009	1.680	0.12
		1.57	1.25	0.0010	1.679	0.13
		2.00	1.41	0.0010	1.679	0.13
		2.25	1.50	0.0010	1.679	0.13
		3.07	1.75	0.0010	1.679	0.13
		4.00	2.00	0.0010	1.679	0.13
		6.25	2.50	0.0009	1.680	0.12
		8.00	2.83	0.0009	1.680	0.12
		12.25	3.50	0.0009	1.680	0.12
		15.00	3.87	0.0009	1.680	0.12
		22.00	4.69	0.0009	1.680	0.12

LOAD INCREMENT DATA

TEST NAME CTB5-12C
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:04:58	0.00	0.00	0.0009	1.680	0.12
		0.10	0.32	0.0017	1.677	0.23
		0.25	0.50	0.0019	1.676	0.25
		0.50	0.71	0.0019	1.676	0.25
		1.00	1.00	0.0019	1.676	0.25
		1.57	1.25	0.0019	1.676	0.25
		2.00	1.41	0.0019	1.676	0.25
		2.25	1.50	0.0019	1.676	0.25
		3.07	1.75	0.0019	1.676	0.25
		4.00	2.00	0.0019	1.676	0.25
		6.25	2.50	0.0020	1.676	0.27
		8.00	2.83	0.0020	1.676	0.27
		12.25	3.50	0.0020	1.676	0.27
		15.00	3.87	0.0020	1.676	0.27
		22.00	4.69	0.0020	1.676	0.27

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 3

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:34:04	0.00	0.00	0.0020	1.676	0.27
		0.10	0.32	0.0035	1.671	0.46
		0.25	0.50	0.0036	1.670	0.48
		0.50	0.71	0.0036	1.670	0.48
		1.00	1.00	0.0036	1.670	0.48
		1.57	1.25	0.0036	1.670	0.48
		2.00	1.41	0.0037	1.670	0.50
		2.25	1.50	0.0037	1.670	0.50
		3.07	1.75	0.0037	1.670	0.50
		4.00	2.00	0.0037	1.670	0.50
		6.25	2.50	0.0037	1.670	0.50
		8.00	2.83	0.0037	1.670	0.50
		12.25	3.50	0.0038	1.669	0.51
		15.00	3.87	0.0038	1.669	0.51
		22.00	4.69	0.0038	1.669	0.51
		30.00	5.48	0.0038	1.669	0.51

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 4
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	17:09:33	0.00	0.00	0.0040	1.669	0.53
		0.10	0.32	0.0060	1.662	0.80
		0.25	0.50	0.0061	1.661	0.81
		0.50	0.71	0.0062	1.661	0.83
		1.00	1.00	0.0062	1.661	0.83
		1.57	1.25	0.0062	1.661	0.83
		2.00	1.41	0.0062	1.661	0.83
		2.25	1.50	0.0062	1.661	0.83
		3.07	1.75	0.0063	1.660	0.84
		4.00	2.00	0.0063	1.660	0.84
		6.25	2.50	0.0063	1.660	0.84
		8.00	2.83	0.0064	1.660	0.86
		12.25	3.50	0.0064	1.660	0.86
		15.00	3.87	0.0064	1.660	0.86
		22.00	4.69	0.0064	1.660	0.86
		30.00	5.48	0.0064	1.660	0.86
		36.00	6.00	0.0064	1.660	0.86
		49.00	7.00	0.0066	1.659	0.88
		60.00	7.75	0.0066	1.659	0.88
		120.00	10.95	0.0067	1.659	0.89
		180.00	13.42	0.0068	1.659	0.91
		240.00	15.49	0.0068	1.659	0.91
		360.00	18.97	0.0069	1.658	0.93
04-15-99	01:09:33	480.00	21.91	0.0069	1.658	0.93
		720.00	26.83	0.0071	1.658	0.94
		840.00	28.98	0.0069	1.658	0.93

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 5
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	07:58:57	0.00	0.00	0.0069	1.658	0.93
		0.10	0.32	0.0094	1.649	1.26
		0.25	0.50	0.0095	1.649	1.28
		0.50	0.71	0.0097	1.648	1.29
		1.00	1.00	0.0098	1.648	1.31
		1.57	1.25	0.0098	1.648	1.31
		2.00	1.41	0.0098	1.648	1.31
		2.25	1.50	0.0098	1.648	1.31
		3.07	1.75	0.0099	1.647	1.33
		4.00	2.00	0.0099	1.647	1.33
		6.25	2.50	0.0099	1.647	1.33
		8.00	2.83	0.0099	1.647	1.33
		12.25	3.50	0.0100	1.647	1.34
		15.00	3.87	0.0102	1.647	1.36
		22.00	4.69	0.0102	1.647	1.36
		30.00	5.48	0.0102	1.647	1.36
		36.00	6.00	0.0102	1.647	1.36
		49.00	7.00	0.0103	1.646	1.38
		60.00	7.75	0.0103	1.646	1.38

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 6
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	09:04:38	0.00	0.00	0.0104	1.646	1.39
		0.10	0.32	0.0150	1.629	2.00
		0.25	0.50	0.0154	1.628	2.05
		0.50	0.71	0.0156	1.627	2.09
		1.00	1.00	0.0160	1.626	2.14
		1.57	1.25	0.0161	1.625	2.15
		2.00	1.41	0.0164	1.624	2.19
		2.25	1.50	0.0164	1.624	2.19
		3.07	1.75	0.0165	1.624	2.20
		4.00	2.00	0.0166	1.623	2.22
		6.25	2.50	0.0169	1.623	2.25
		8.00	2.83	0.0170	1.622	2.27
		12.25	3.50	0.0172	1.621	2.30
		15.00	3.87	0.0175	1.620	2.34
		22.00	4.69	0.0177	1.619	2.37
		30.00	5.48	0.0180	1.619	2.40
		36.00	6.00	0.0181	1.618	2.42
		49.00	7.00	0.0185	1.617	2.47
		60.00	7.75	0.0186	1.616	2.49

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 7

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	10:18:33	0.00	0.00	0.0187	1.616	2.50
		0.10	0.32	0.0151	1.629	2.02
		0.25	0.50	0.0150	1.629	2.00
		0.50	0.71	0.0149	1.630	1.99
		1.00	1.00	0.0149	1.630	1.99
		1.57	1.25	0.0149	1.630	1.99
		2.00	1.41	0.0148	1.630	1.97
		2.25	1.50	0.0148	1.630	1.97
		3.07	1.75	0.0148	1.630	1.97
		4.00	2.00	0.0148	1.630	1.97
		6.25	2.50	0.0148	1.630	1.97
		8.00	2.83	0.0148	1.630	1.97
		12.25	3.50	0.0146	1.631	1.95
		15.00	3.87	0.0148	1.630	1.97
		22.00	4.69	0.0146	1.631	1.95
		30.00	5.48	0.0146	1.631	1.95
		36.00	6.00	0.0146	1.631	1.95

LOAD INCREMENT DATA

TEST NAME CTB5-12C   PAGE NO: 8

TEST NO: 2   TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	11:07:02	0.00	0.00	0.0146	1.631	1.95
		0.10	0.32	0.0117	1.641	1.56
		0.25	0.50	0.0114	1.642	1.52
		0.50	0.71	0.0112	1.643	1.49
		1.00	1.00	0.0110	1.643	1.47
		1.57	1.25	0.0110	1.643	1.47
		2.00	1.41	0.0110	1.643	1.47
		2.25	1.50	0.0110	1.643	1.47
		3.07	1.75	0.0109	1.644	1.46
		4.00	2.00	0.0109	1.644	1.46
		6.25	2.50	0.0109	1.644	1.46
		8.00	2.83	0.0108	1.644	1.44
		12.25	3.50	0.0108	1.644	1.44
		15.00	3.87	0.0108	1.644	1.44
		22.00	4.69	0.0107	1.645	1.42
		30.00	5.48	0.0107	1.645	1.42
		36.00	6.00	0.0107	1.645	1.42
		49.00	7.00	0.0107	1.645	1.42
		60.00	7.75	0.0107	1.645	1.42

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 9

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:12:35	0.00	0.00	0.0107	1.645	1.42
		0.10	0.32	0.0114	1.642	1.52
		0.25	0.50	0.0113	1.643	1.51
		0.50	0.71	0.0113	1.643	1.51
		1.00	1.00	0.0114	1.642	1.52
		1.57	1.25	0.0114	1.642	1.52
		2.00	1.41	0.0114	1.642	1.52
		2.25	1.50	0.0114	1.642	1.52
		3.07	1.75	0.0114	1.642	1.52
		4.00	2.00	0.0114	1.642	1.52
		6.25	2.50	0.0113	1.643	1.51
		8.00	2.83	0.0114	1.642	1.52
		12.25	3.50	0.0113	1.643	1.51
		15.00	3.87	0.0114	1.642	1.52
		22.00	4.69	0.0113	1.643	1.51
		30.00	5.48	0.0113	1.643	1.51
		36.00	6.00	0.0114	1.642	1.52

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 10
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:49:49	0.00	0.00	0.0114	1.642	1.52
		0.10	0.32	0.0128	1.637	1.71
		0.25	0.50	0.0128	1.637	1.71
		0.50	0.71	0.0129	1.637	1.72
		1.00	1.00	0.0128	1.637	1.71
		1.57	1.25	0.0129	1.637	1.72
		2.00	1.41	0.0129	1.637	1.72
		2.25	1.50	0.0129	1.637	1.72
		3.07	1.75	0.0129	1.637	1.72
		4.00	2.00	0.0129	1.637	1.72
		6.25	2.50	0.0129	1.637	1.72
		8.00	2.83	0.0129	1.637	1.72
		12.25	3.50	0.0130	1.636	1.74
		15.00	3.87	0.0130	1.636	1.74
		22.00	4.69	0.0130	1.636	1.74
		30.00	5.48	0.0130	1.636	1.74

LOAD INCREMENT DATA

TEST NAME CTB5-12C

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	13:25:46	0.00	0.00	0.0130	1.636	1.74
		0.10	0.32	0.0153	1.628	2.04
		0.25	0.50	0.0154	1.628	2.05
		0.50	0.71	0.0154	1.628	2.05
		1.00	1.00	0.0155	1.627	2.07
		1.57	1.25	0.0155	1.627	2.07
		2.00	1.41	0.0155	1.627	2.07
		2.25	1.50	0.0155	1.627	2.07
		3.07	1.75	0.0155	1.627	2.07
		4.00	2.00	0.0155	1.627	2.07
		6.25	2.50	0.0156	1.627	2.09
		8.00	2.83	0.0155	1.627	2.07
		12.25	3.50	0.0156	1.627	2.09
		15.00	3.87	0.0156	1.627	2.09
		22.00	4.69	0.0156	1.627	2.09
		30.00	5.48	0.0156	1.627	2.09
		36.00	6.00	0.0156	1.627	2.09

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 12
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	14:10:58	0.00	0.00	0.0157	1.627	2.10
		0.10	0.32	0.0191	1.615	2.55
		0.25	0.50	0.0193	1.614	2.58
		0.50	0.71	0.0195	1.613	2.60
		1.00	1.00	0.0196	1.613	2.62
		1.57	1.25	0.0197	1.612	2.63
		2.00	1.41	0.0197	1.612	2.63
		2.25	1.50	0.0197	1.612	2.63
		3.07	1.75	0.0198	1.612	2.65
		4.00	2.00	0.0198	1.612	2.65
		6.25	2.50	0.0200	1.611	2.67
		8.00	2.83	0.0201	1.611	2.68
		12.25	3.50	0.0202	1.611	2.70
		15.00	3.87	0.0202	1.611	2.70
		22.00	4.69	0.0203	1.610	2.72
		30.00	5.48	0.0206	1.609	2.75
		36.00	6.00	0.0206	1.609	2.75

LOAD INCREMENT DATA

TEST NAME CTB5-12C

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 6.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	14:57:22	0.00	0.00	0.0207	1.609	2.77
		0.10	0.32	0.0255	1.591	3.41
		0.25	0.50	0.0268	1.587	3.58
		0.50	0.71	0.0278	1.583	3.71
		1.00	1.00	0.0290	1.579	3.88
		1.57	1.25	0.0299	1.576	3.99
		2.00	1.41	0.0304	1.574	4.06
		2.25	1.50	0.0306	1.573	4.09
		3.07	1.75	0.0314	1.571	4.19
		4.00	2.00	0.0321	1.568	4.29
		6.25	2.50	0.0332	1.564	4.44
		8.00	2.83	0.0340	1.561	4.54
		12.25	3.50	0.0352	1.557	4.71
		15.00	3.87	0.0358	1.555	4.79
		22.00	4.69	0.0371	1.550	4.95
		30.00	5.48	0.0382	1.546	5.10
		36.00	6.00	0.0388	1.544	5.19
		42.00	6.48	0.0394	1.542	5.27

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 14

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 6.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	15:39:33	0.00	0.00	0.0394	1.542	5.27
		0.10	0.32	0.0433	1.528	5.78
		0.25	0.50	0.0464	1.517	6.20
		0.50	0.71	0.0486	1.509	6.49
		1.00	1.00	0.0511	1.500	6.83
		1.57	1.25	0.0528	1.494	7.06
		2.00	1.41	0.0538	1.490	7.19
		2.25	1.50	0.0543	1.488	7.26
		3.07	1.75	0.0557	1.483	7.44
		4.00	2.00	0.0569	1.479	7.60
		6.25	2.50	0.0589	1.472	7.87
		8.00	2.83	0.0601	1.467	8.04
		12.25	3.50	0.0622	1.460	8.32
		15.00	3.87	0.0632	1.456	8.45
		22.00	4.69	0.0651	1.450	8.70
		30.00	5.48	0.0667	1.444	8.91
		36.00	6.00	0.0677	1.440	9.05
		45.50	6.75	0.0689	1.436	9.21

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 15
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 8.00 tsf to 12.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	16:25:09	0.00	0.00	0.0689	1.436	9.21
		0.10	0.32	0.0826	1.387	11.03
		0.25	0.50	0.0869	1.371	11.61
		0.50	0.71	0.0905	1.359	12.09
		1.00	1.00	0.0944	1.345	12.61
		1.57	1.25	0.0968	1.336	12.94
		2.00	1.41	0.0981	1.331	13.10
		2.25	1.50	0.0988	1.329	13.20
		3.07	1.75	0.1004	1.323	13.42
		4.00	2.00	0.1019	1.318	13.62
		6.25	2.50	0.1043	1.309	13.93
		8.00	2.83	0.1055	1.305	14.10
		12.25	3.50	0.1079	1.296	14.41
		15.00	3.87	0.1089	1.293	14.55
		22.00	4.69	0.1108	1.286	14.81
		30.00	5.48	0.1125	1.280	15.03

LOAD INCREMENT DATA

TEST NAME CTB5-12C
TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 12.00 tsf to 16.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	16:56:20	0.00	0.00	0.1127	1.279	15.06
		0.10	0.32	0.1199	1.253	16.02
		0.25	0.50	0.1224	1.244	16.35
		0.50	0.71	0.1245	1.237	16.63
		1.00	1.00	0.1272	1.227	17.00
		1.57	1.25	0.1291	1.220	17.25
		2.00	1.41	0.1301	1.217	17.38
		2.25	1.50	0.1307	1.215	17.46
		3.07	1.75	0.1321	1.210	17.64
		4.00	2.00	0.1332	1.206	17.79
		6.25	2.50	0.1353	1.198	18.07
		8.00	2.83	0.1364	1.194	18.22
		12.25	3.50	0.1384	1.187	18.49
		15.00	3.87	0.1394	1.183	18.62
		22.00	4.69	0.1411	1.177	18.85
		30.00	5.48	0.1425	1.172	19.04
		35.20	5.93	0.1432	1.170	19.13

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 17

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 16.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	17:31:38	0.00	0.00	0.1432	1.170	19.13
		0.10	0.32	0.1364	1.194	18.22
		0.25	0.50	0.1363	1.195	18.21
		0.50	0.71	0.1361	1.195	18.19
		1.00	1.00	0.1359	1.196	18.16
		1.57	1.25	0.1359	1.196	18.16
		2.00	1.41	0.1359	1.196	18.16
		2.25	1.50	0.1359	1.196	18.16
		3.07	1.75	0.1358	1.196	18.14
		4.00	2.00	0.1358	1.196	18.14
		6.25	2.50	0.1358	1.196	18.14
		8.00	2.83	0.1356	1.197	18.12
		12.25	3.50	0.1356	1.197	18.12
		15.00	3.87	0.1356	1.197	18.12
		22.00	4.69	0.1356	1.197	18.12
		30.00	5.48	0.1355	1.197	18.11
		36.00	6.00	0.1355	1.197	18.11
		49.00	7.00	0.1355	1.197	18.11
		60.00	7.75	0.1355	1.197	18.11
		120.00	10.95	0.1355	1.197	18.11
		180.00	13.42	0.1355	1.197	18.11
		240.00	15.49	0.1355	1.197	18.11
		360.00	18.97	0.1355	1.197	18.11
04-16-99	01:31:38	480.00	21.91	0.1355	1.197	18.11
		720.00	26.83	0.1355	1.197	18.11
		840.00	28.98	0.1355	1.197	18.11

LOAD INCREMENT DATA

TEST NAME CTB5-12C PAGE NO: 18

TEST NO: 2 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	07:55:07	0.00	0.00	0.1355	1.197	18.11
		0.10	0.32	0.1308	1.214	17.48
		0.25	0.50	0.1303	1.216	17.41
		0.50	0.71	0.1299	1.217	17.36
		1.00	1.00	0.1294	1.219	17.30
		1.57	1.25	0.1292	1.220	17.26
		2.00	1.41	0.1291	1.220	17.25
		2.25	1.50	0.1291	1.220	17.25
		3.07	1.75	0.1290	1.221	17.23
		4.00	2.00	0.1288	1.221	17.21
		6.25	2.50	0.1287	1.222	17.20
		8.00	2.83	0.1286	1.222	17.18
		12.25	3.50	0.1285	1.223	17.16
		15.00	3.87	0.1283	1.223	17.15
		22.00	4.69	0.1282	1.223	17.13
		30.00	5.48	0.1282	1.223	17.13
		36.00	6.00	0.1281	1.224	17.11
		49.00	7.00	0.1280	1.224	17.10

LOAD INCREMENT DATA

TEST NAME CTB5-12C

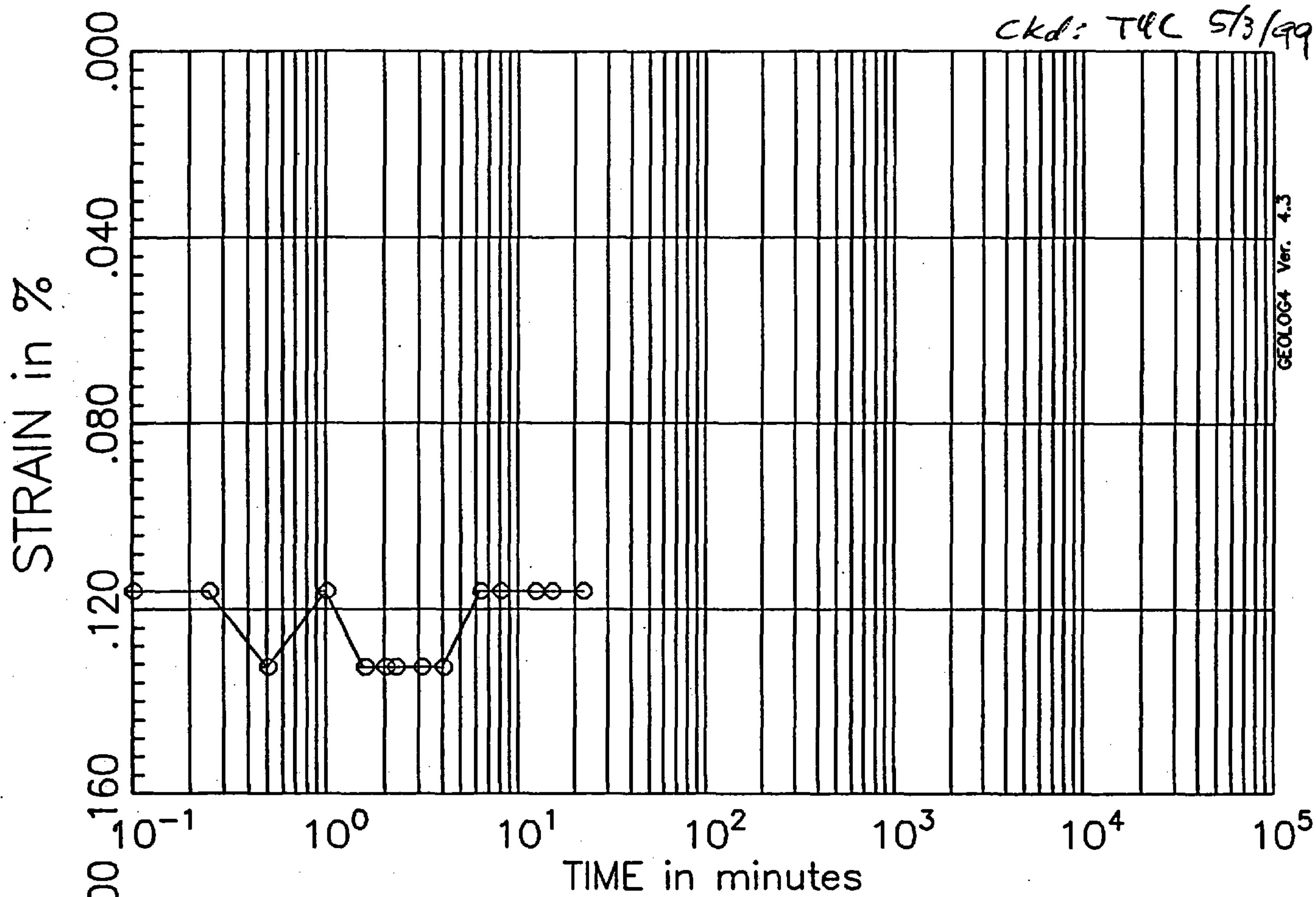
TEST NO: 2 TESTED BY: ACS

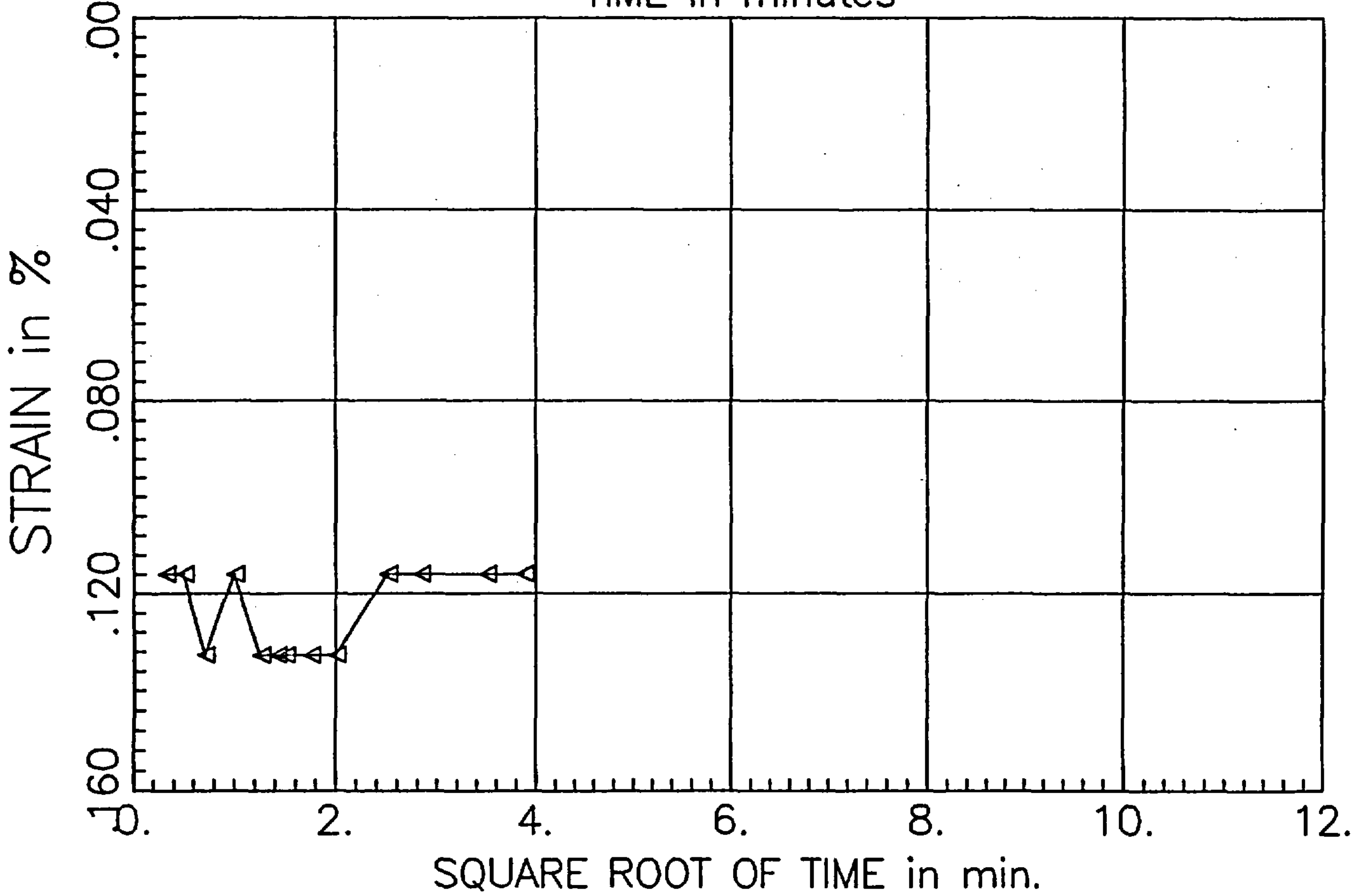
PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	08:50:35	0.00	0.00	0.1278	1.225	17.08
		0.10	0.32	0.1242	1.238	16.60
		0.25	0.50	0.1235	1.240	16.50
		0.50	0.71	0.1226	1.243	16.38
		1.00	1.00	0.1218	1.247	16.27
		1.57	1.25	0.1210	1.249	16.17
		2.00	1.41	0.1206	1.251	16.12
		2.25	1.50	0.1205	1.251	16.10
		3.07	1.75	0.1200	1.253	16.04
		4.00	2.00	0.1198	1.254	16.00
		6.25	2.50	0.1193	1.255	15.94
		8.00	2.83	0.1190	1.256	15.90
		12.25	3.50	0.1188	1.257	15.87
		15.00	3.87	0.1185	1.258	15.84
		22.00	4.69	0.1183	1.259	15.80
		30.00	5.48	0.1180	1.260	15.77
		36.00	6.00	0.1179	1.260	15.76
		49.00	7.00	0.1177	1.261	15.72
		60.00	7.75	0.1175	1.262	15.71
		120.00	10.95	0.1170	1.263	15.64
		149.00	12.21	0.1168	1.264	15.61

$d_{100} = 0.12\%$

JO 05996.02
Calc: ACS 4/26/99
Ckd: TWC 5/3/99



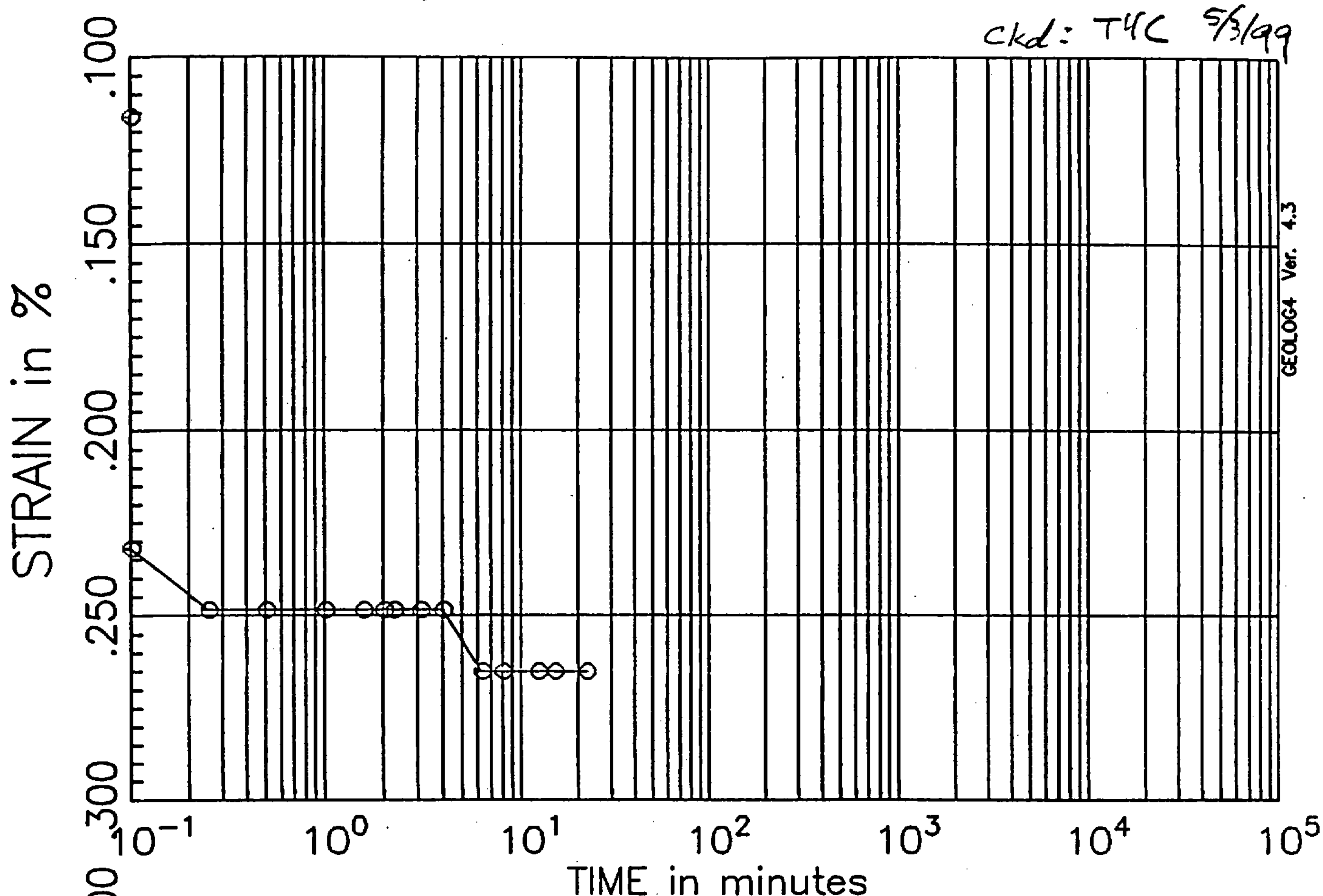


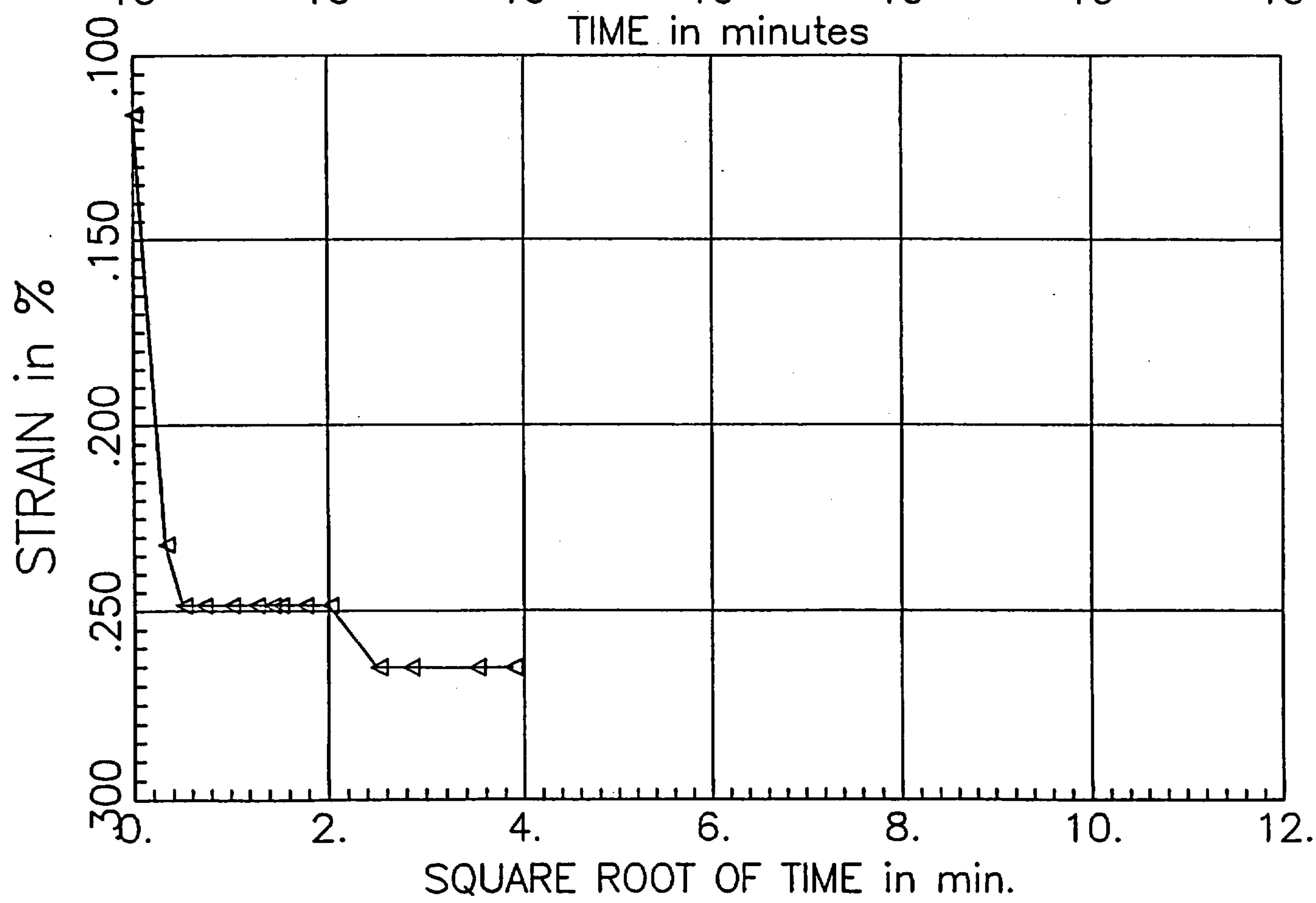
PRESSURE INCREMENT #1
from 0.00 tsf to 0.10 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 0.27\%$

Calc: ACS 4/26/99
Ckd: TYC 5/3/99



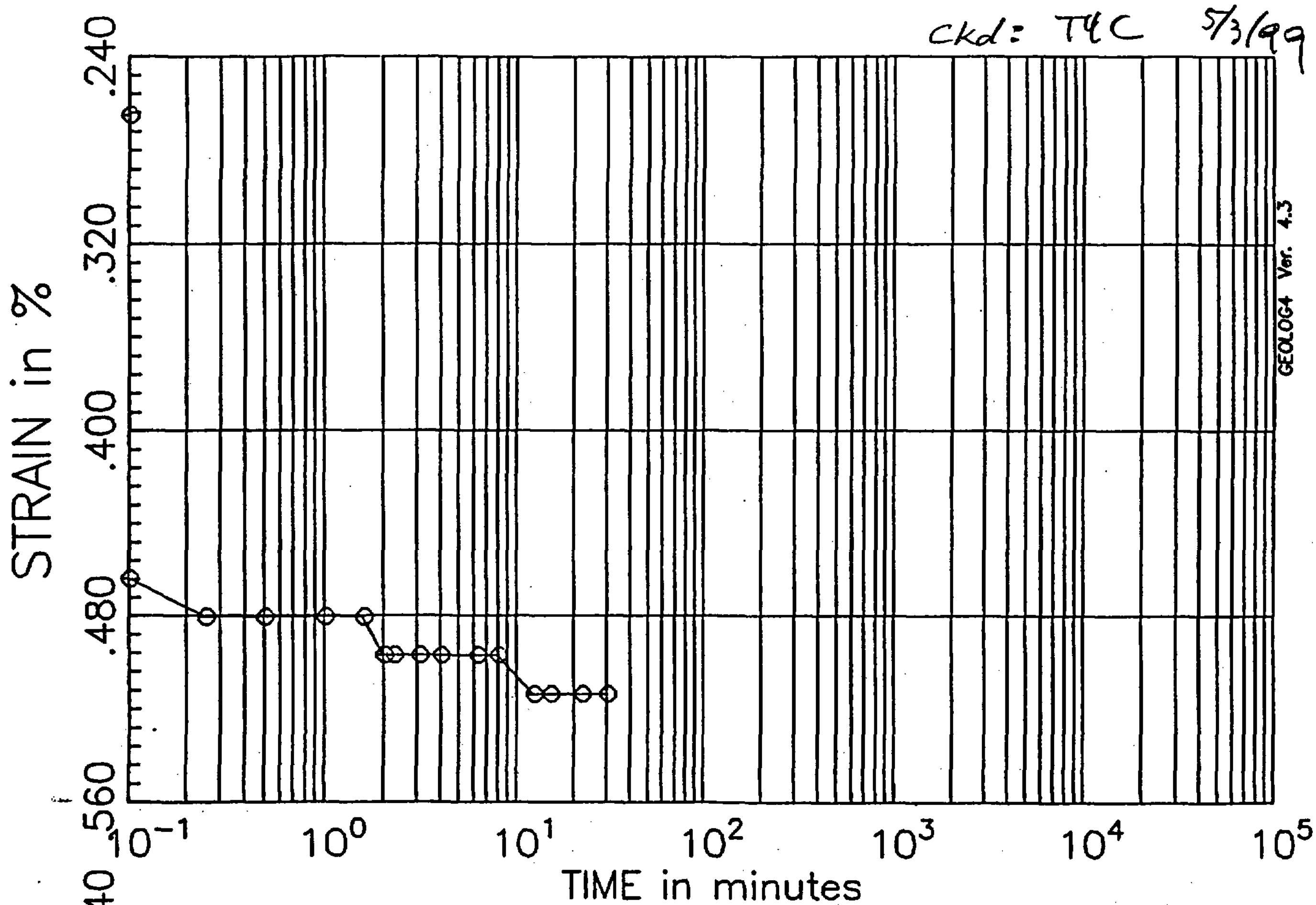


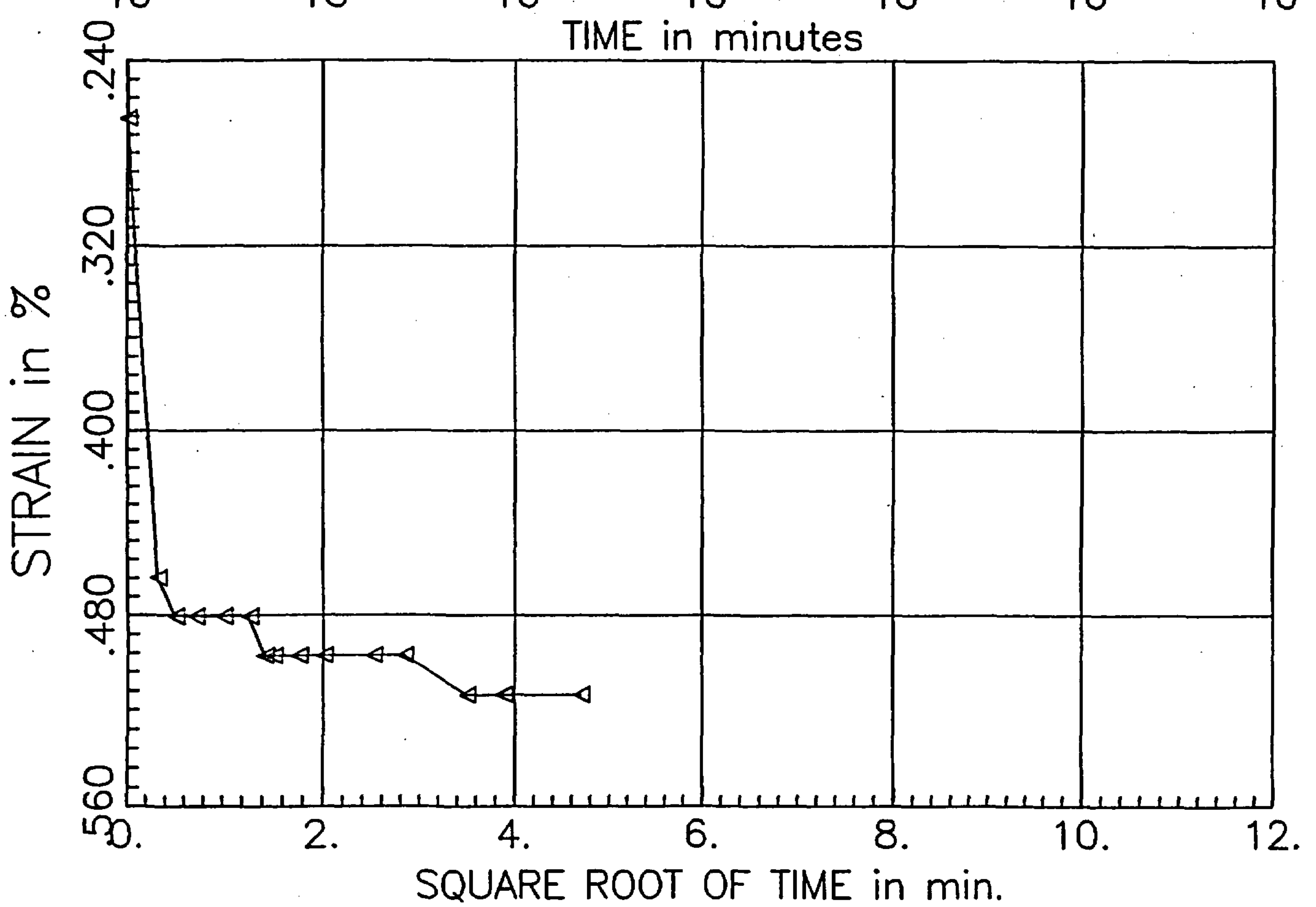
PRESSURE INCREMENT #2
from 0.10 tsf to 0.25 tsf

Test No: 2
Testname: CTB5-12C

JO 05996.02
Calc: ACS 4/26/99
Ckd: T4C 5/3/99

$d_{100} = 0.50\%$



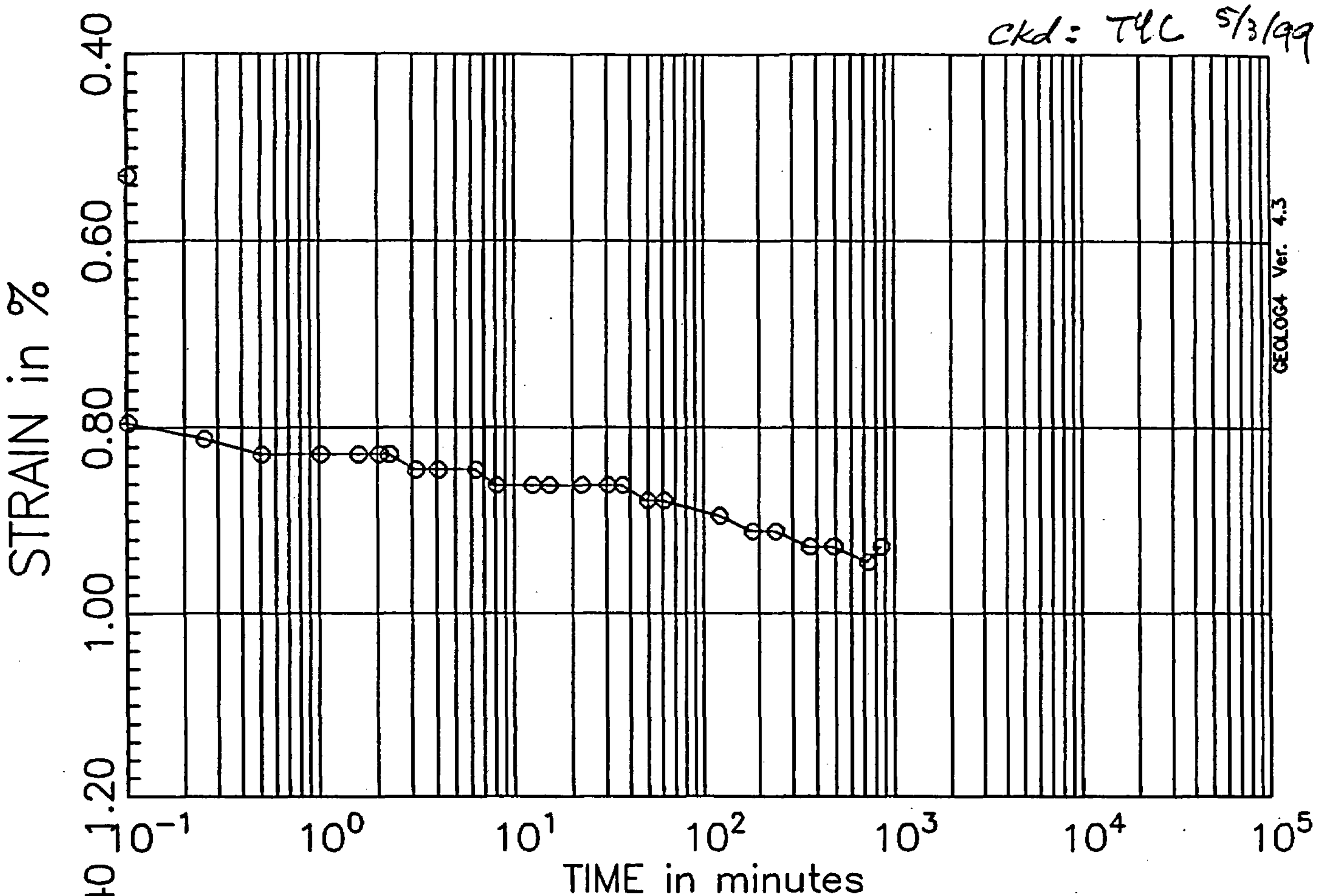


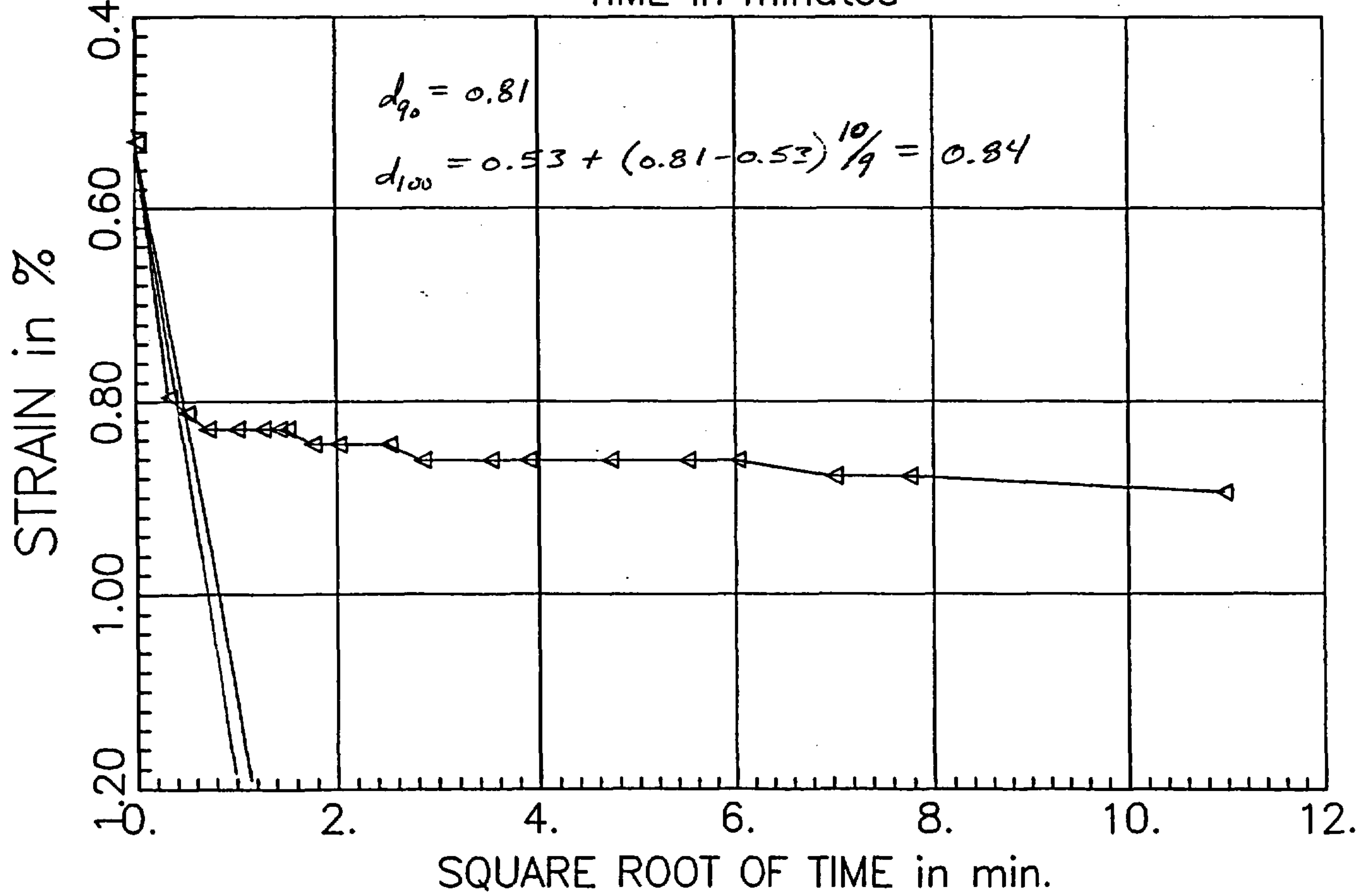
PRESSURE INCREMENT #3
from 0.25 tsf to 0.50 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 0.84\%$

JO 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99



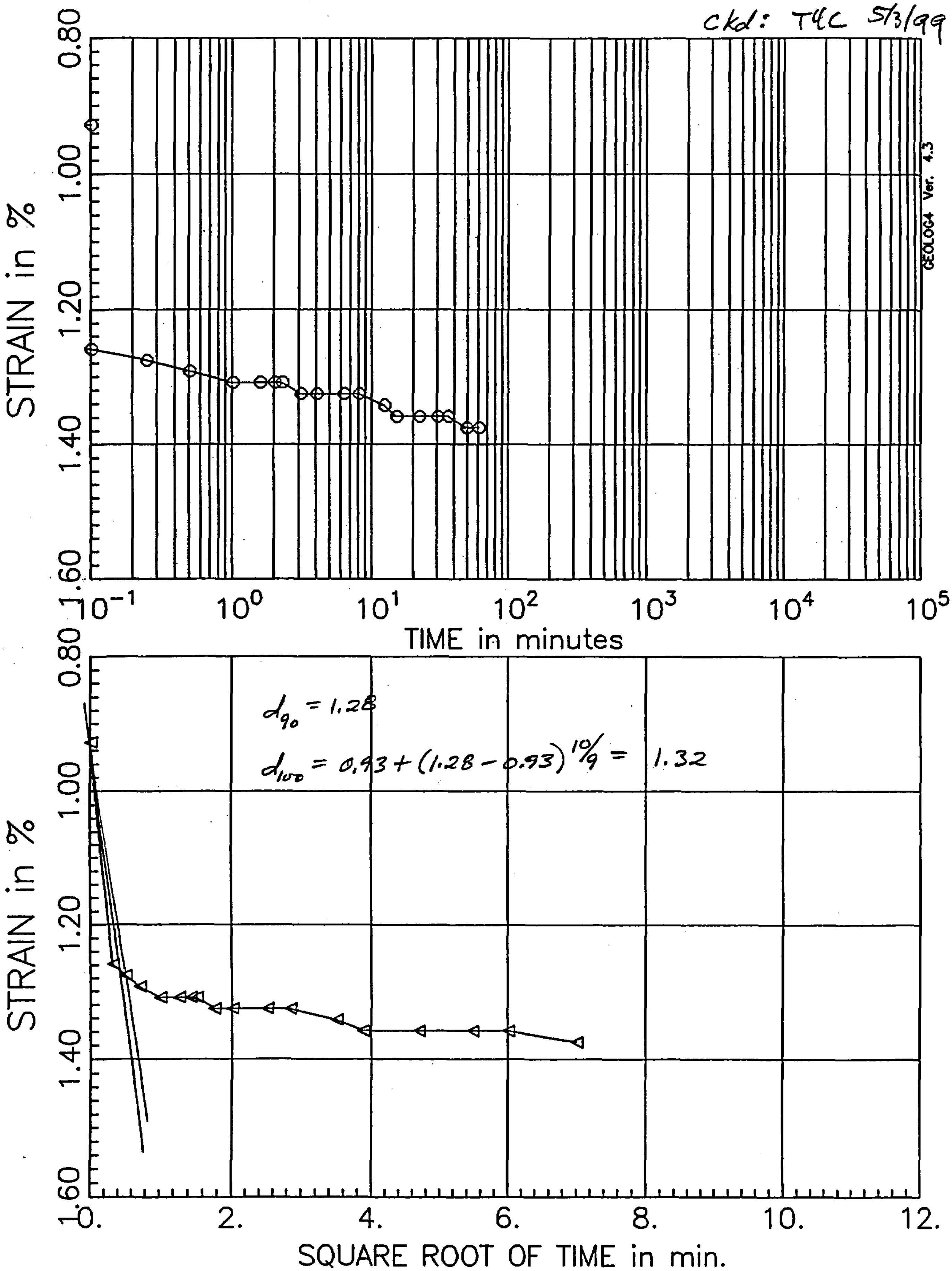


PRESSURE INCREMENT #4
from 0.50 tsf to 1.00 tsf

Test No: 2
Testname: CTB5–12C

JO 05996.02
Calc: ACS 4/26/99
Ckd: T&C 5/3/99

$d_{100} = 1.32\%$



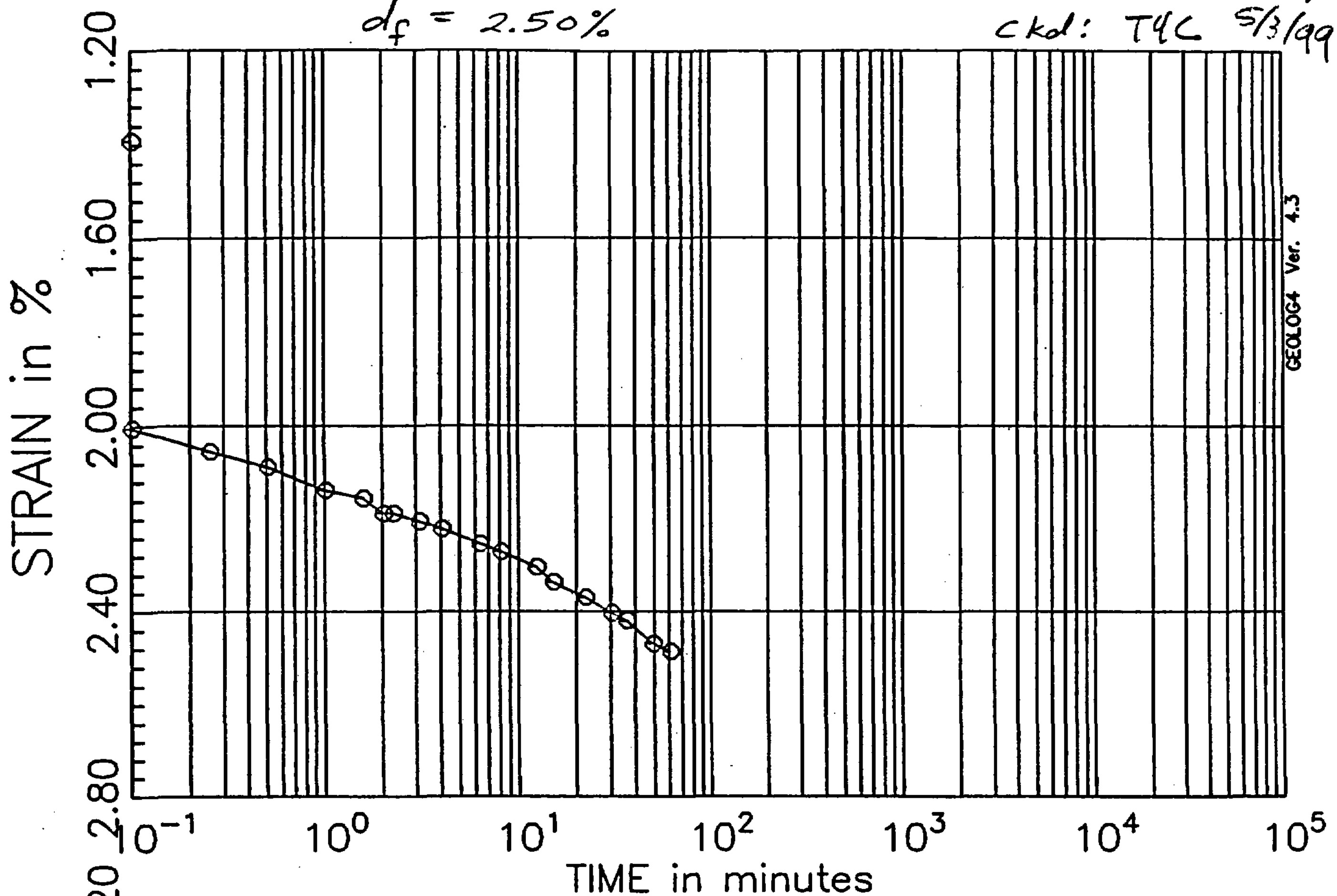
PRESSURE INCREMENT #5
from 1.00 tsf to 2.00 tsf

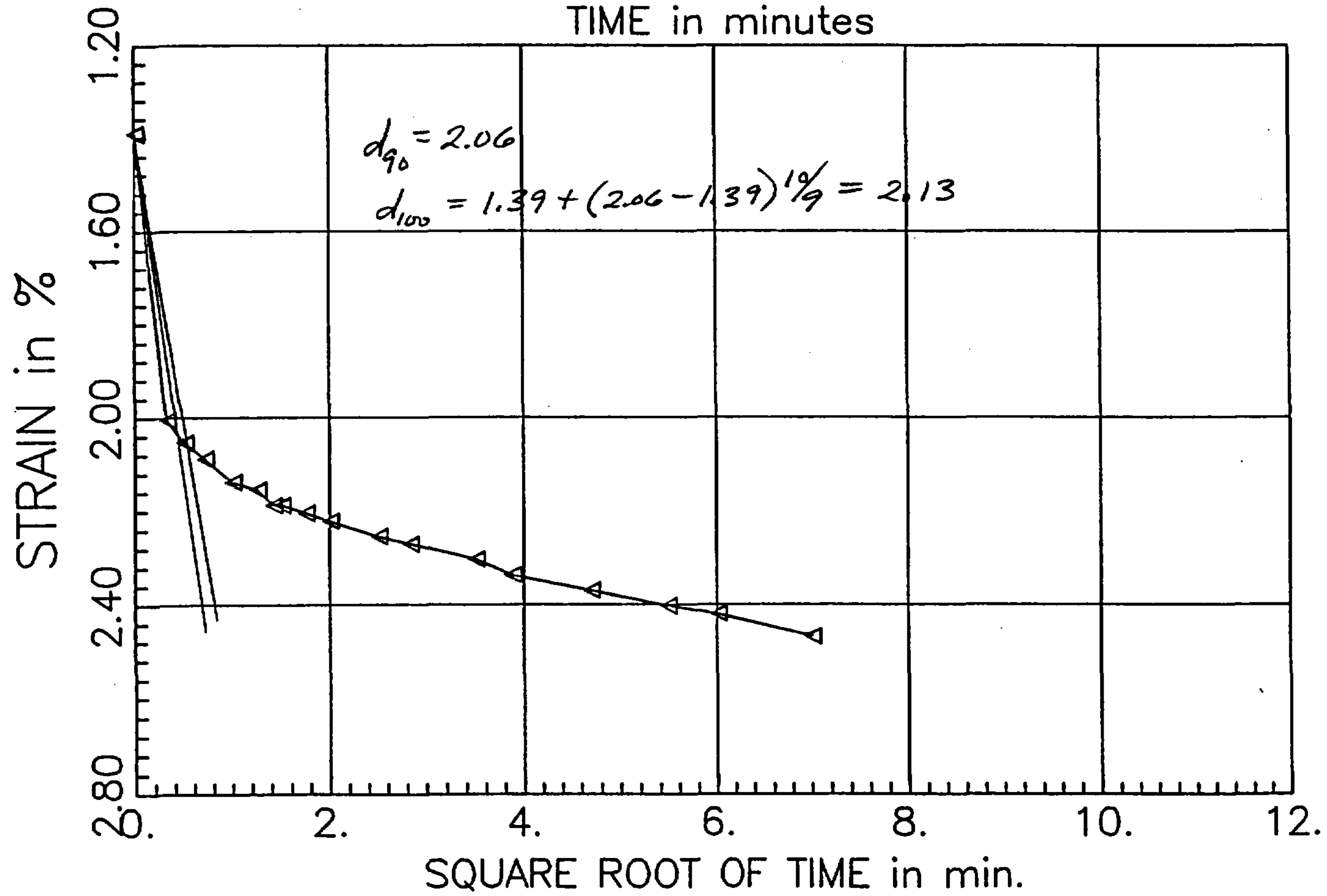
Test No: 2
Testname: CTB5-12C

$d_{100} = 2.13\%$

$d_f = 2.50\%$

JO 02776.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99



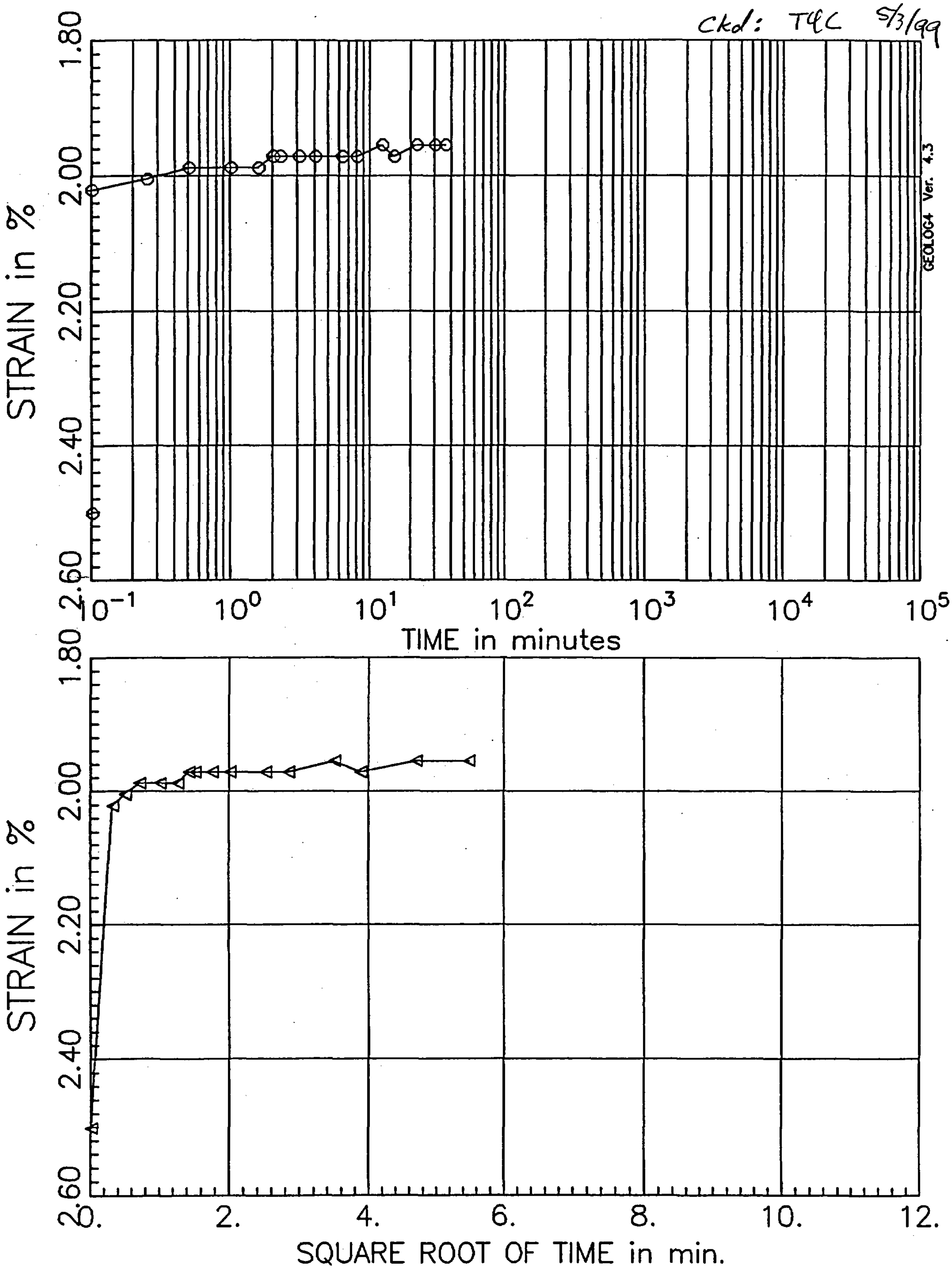


PRESSURE INCREMENT #6
from 2.00 tsf to 4.00 tsf

Test No: 2
Testname: CTB5-12C

Jo 05996.02
Calc: ACS 4/26/99
Ckd: TQC 5/3/99

$d_{100} = 1.97\%$

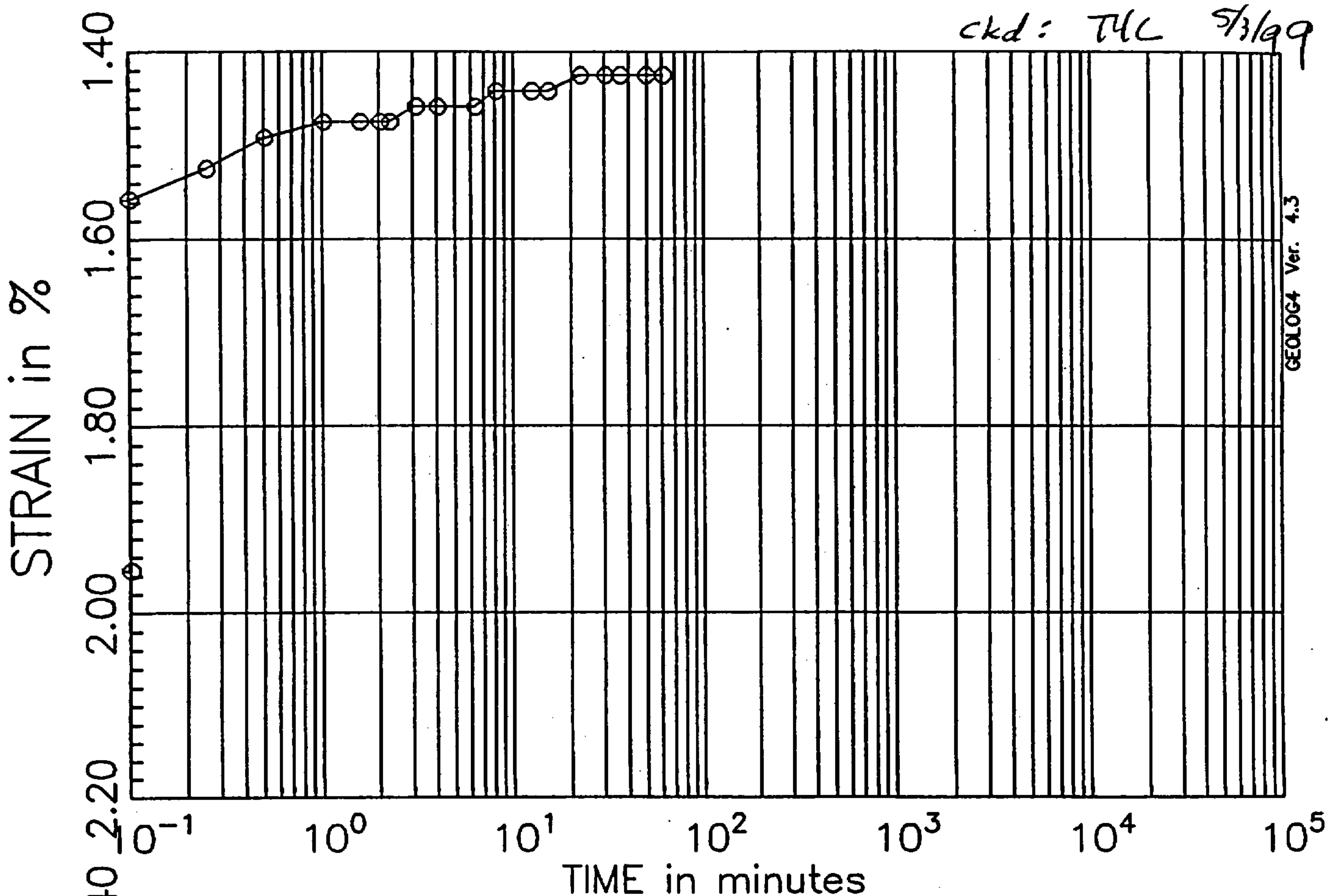


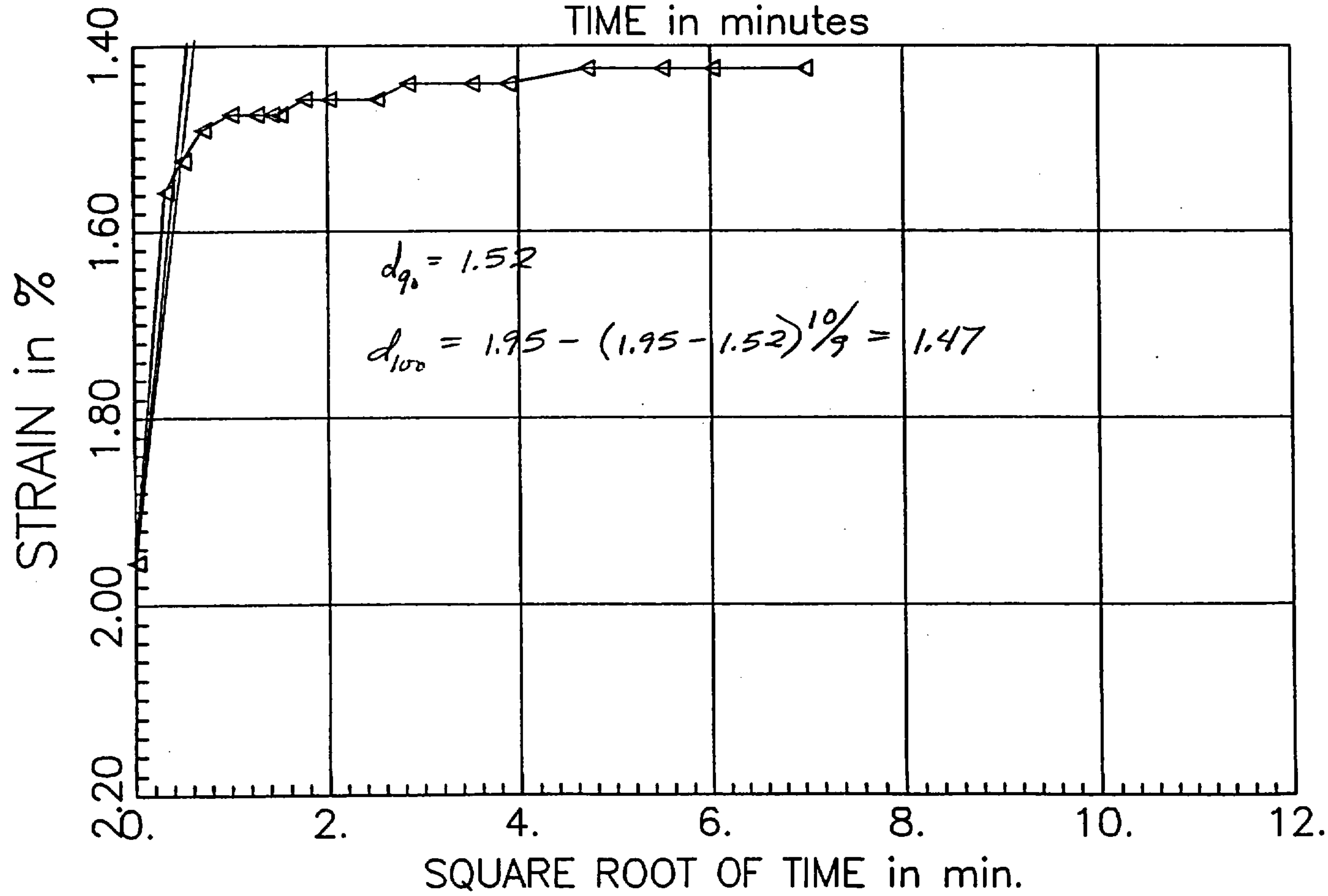
PRESSURE INCREMENT #7
from 4.00 tsf to 1.00 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 1.47\%$

Calc: ACS 4/26/99
Ckd: THC 5/3/99



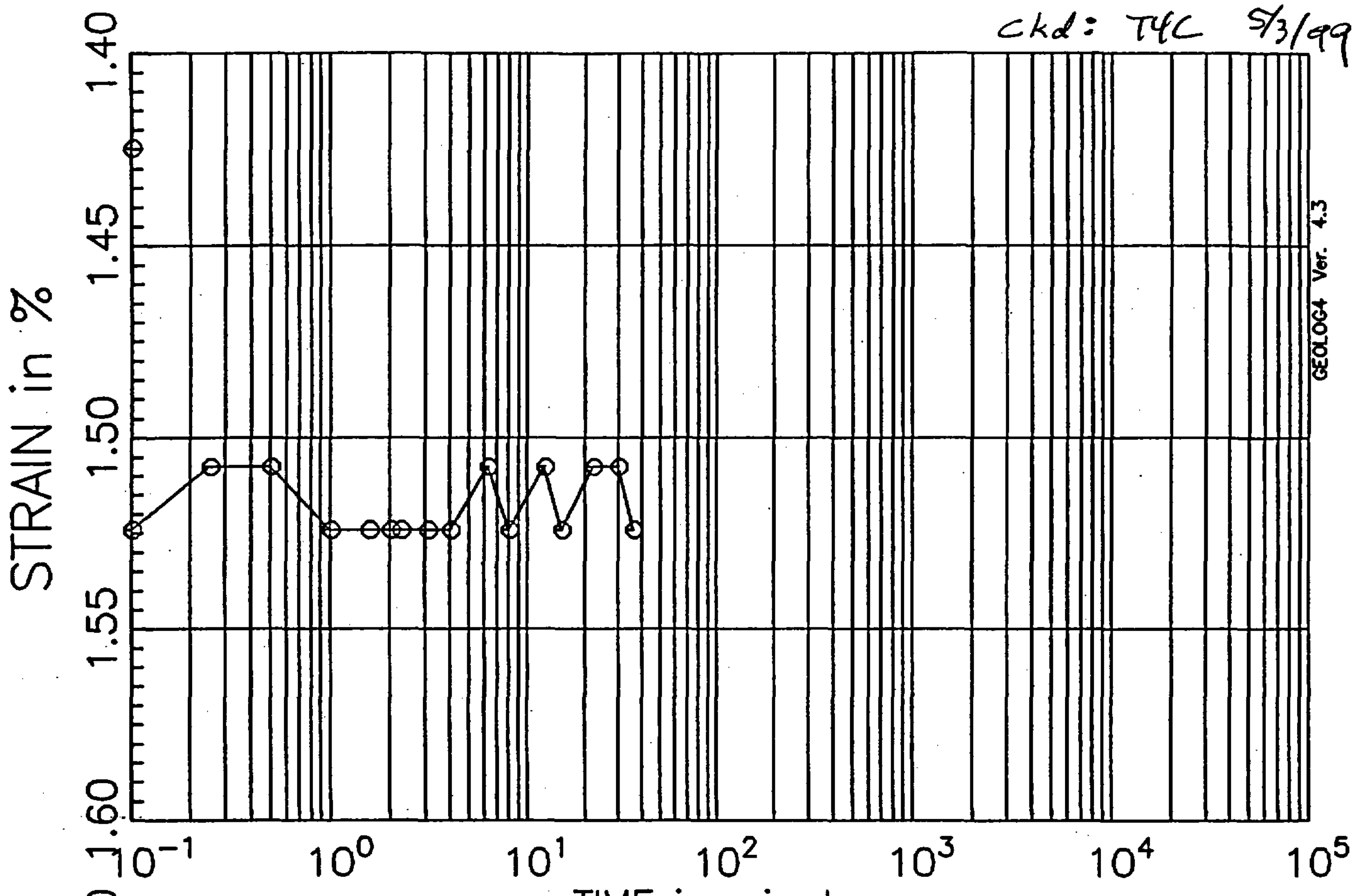


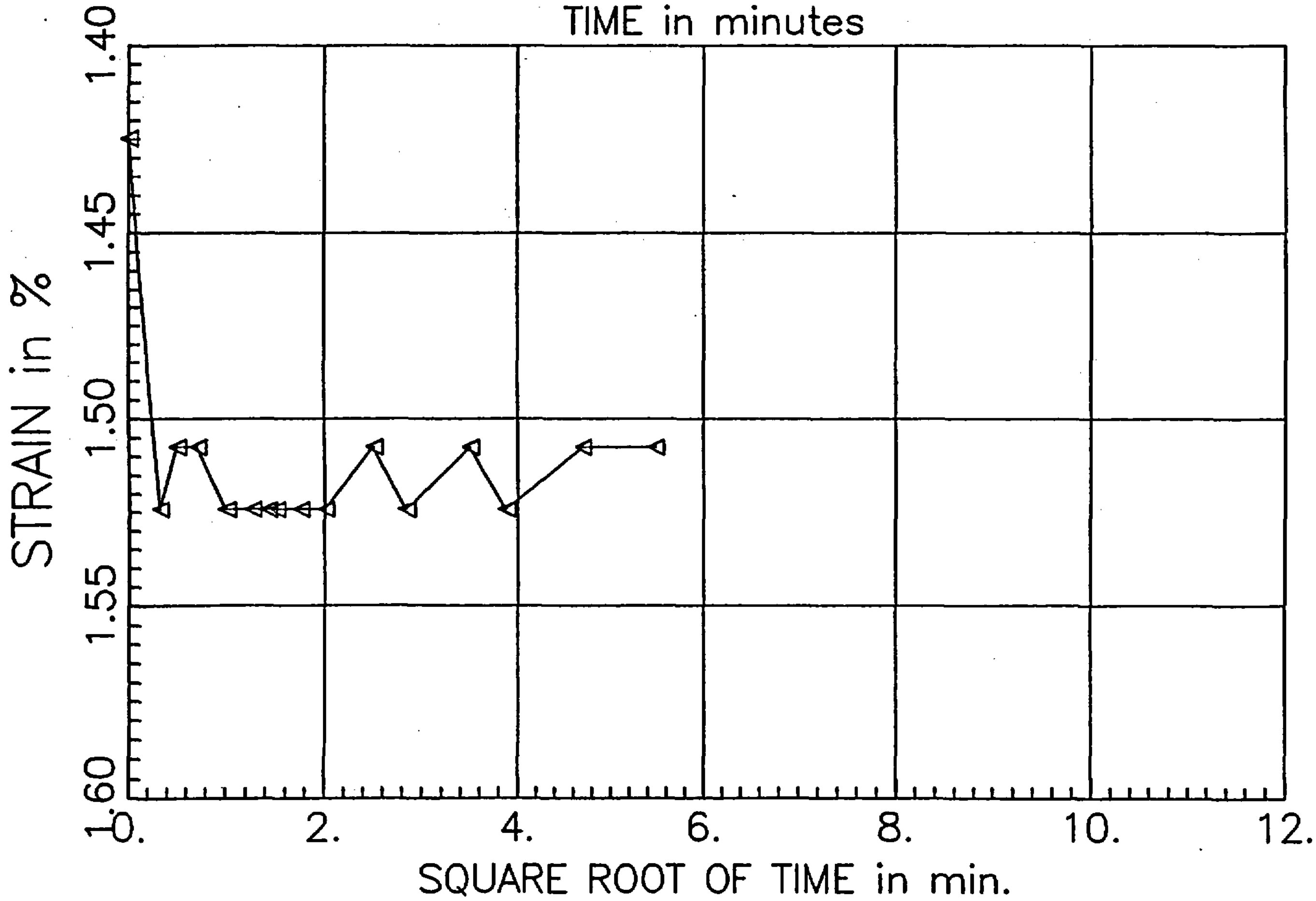
PRESSURE INCREMENT #8
from 1.00 tsf to 0.25 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 1.52\%$

JO 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99





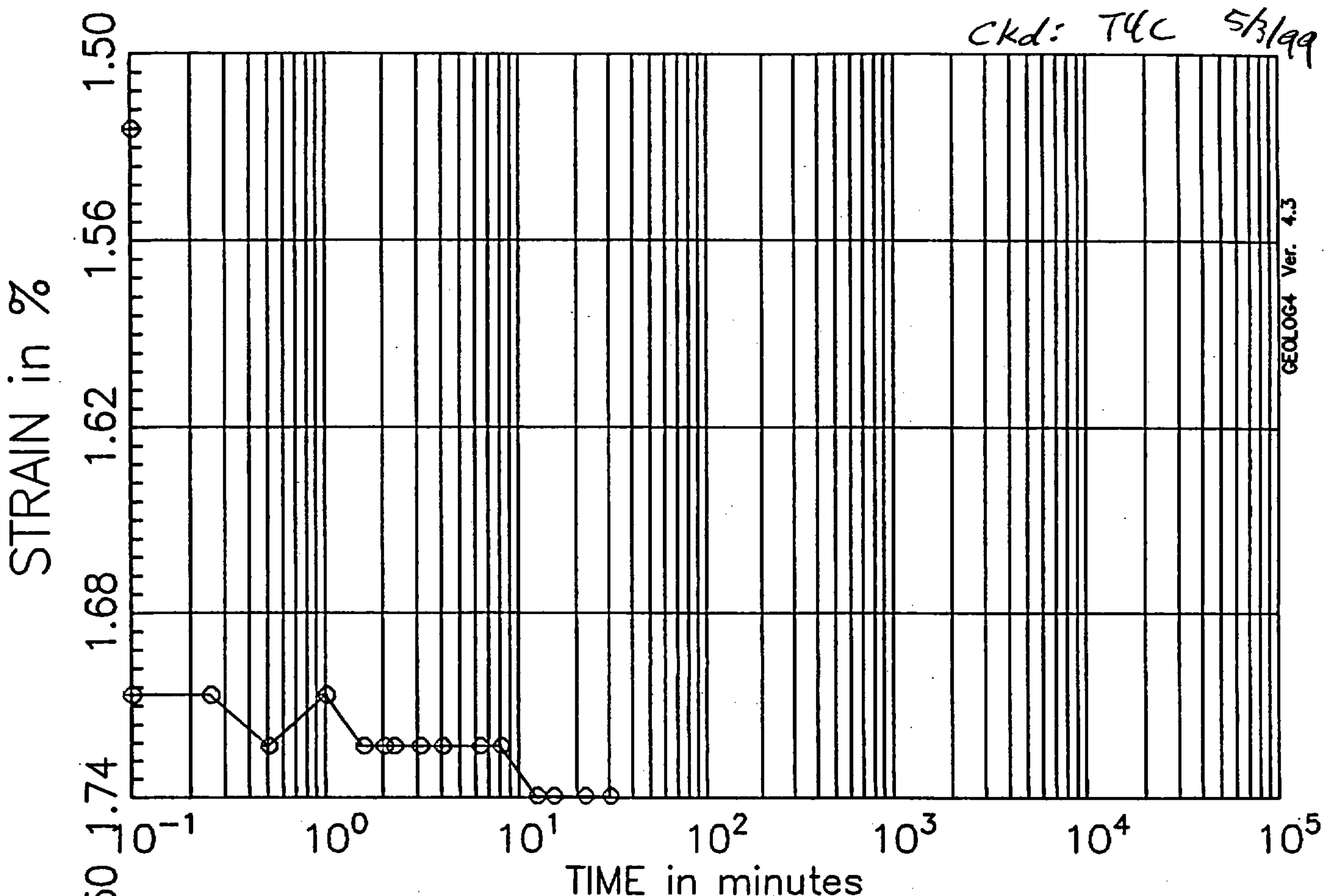
PRESSURE INCREMENT # 9
from 0.25 tsf to 0.50 tsf

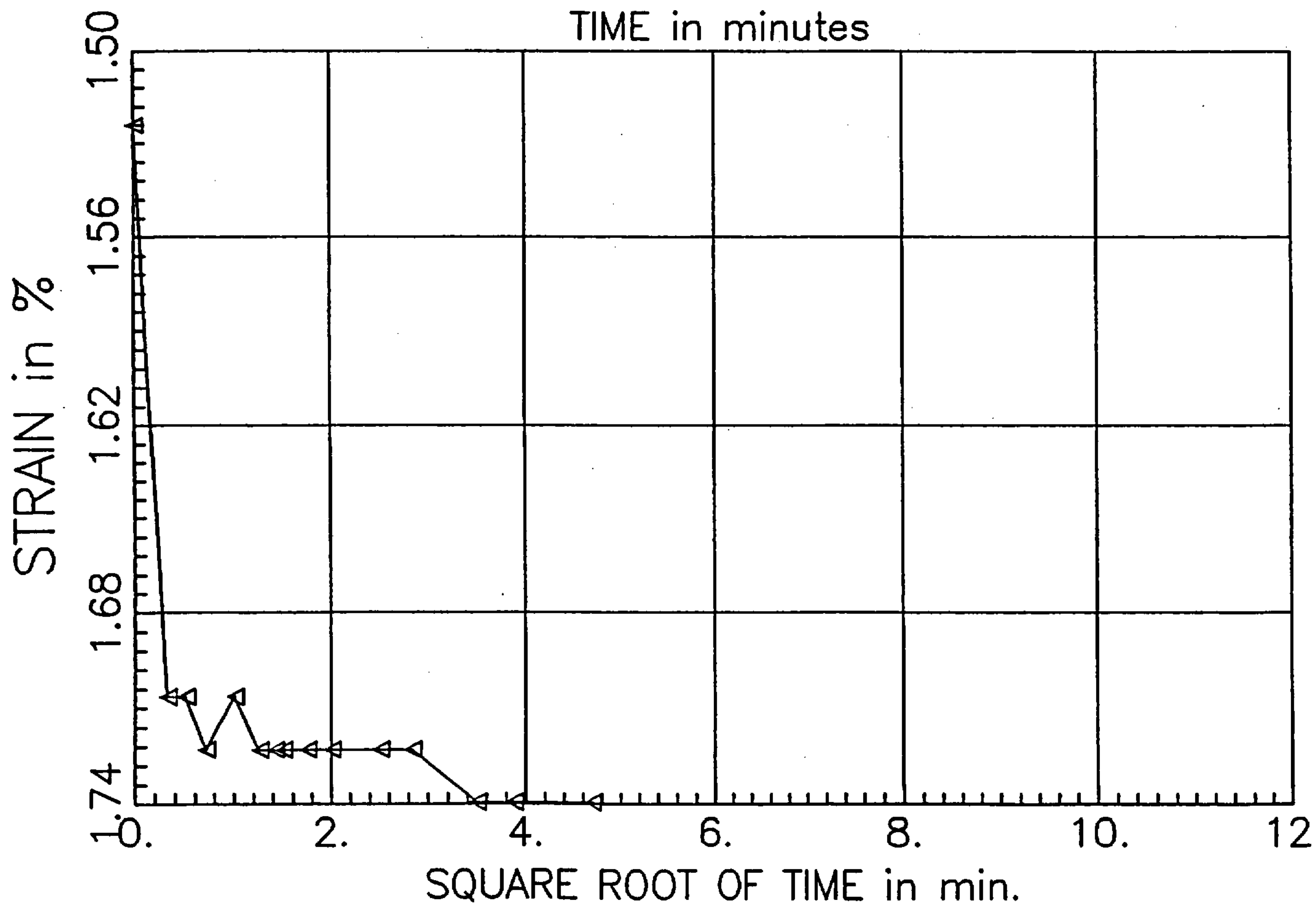
Test No: 2
Testname: CTB5-12C

$d_{100} = 1.72\%$

Calc: ACS 4/26/99

Ckd: T4C 5/3/99



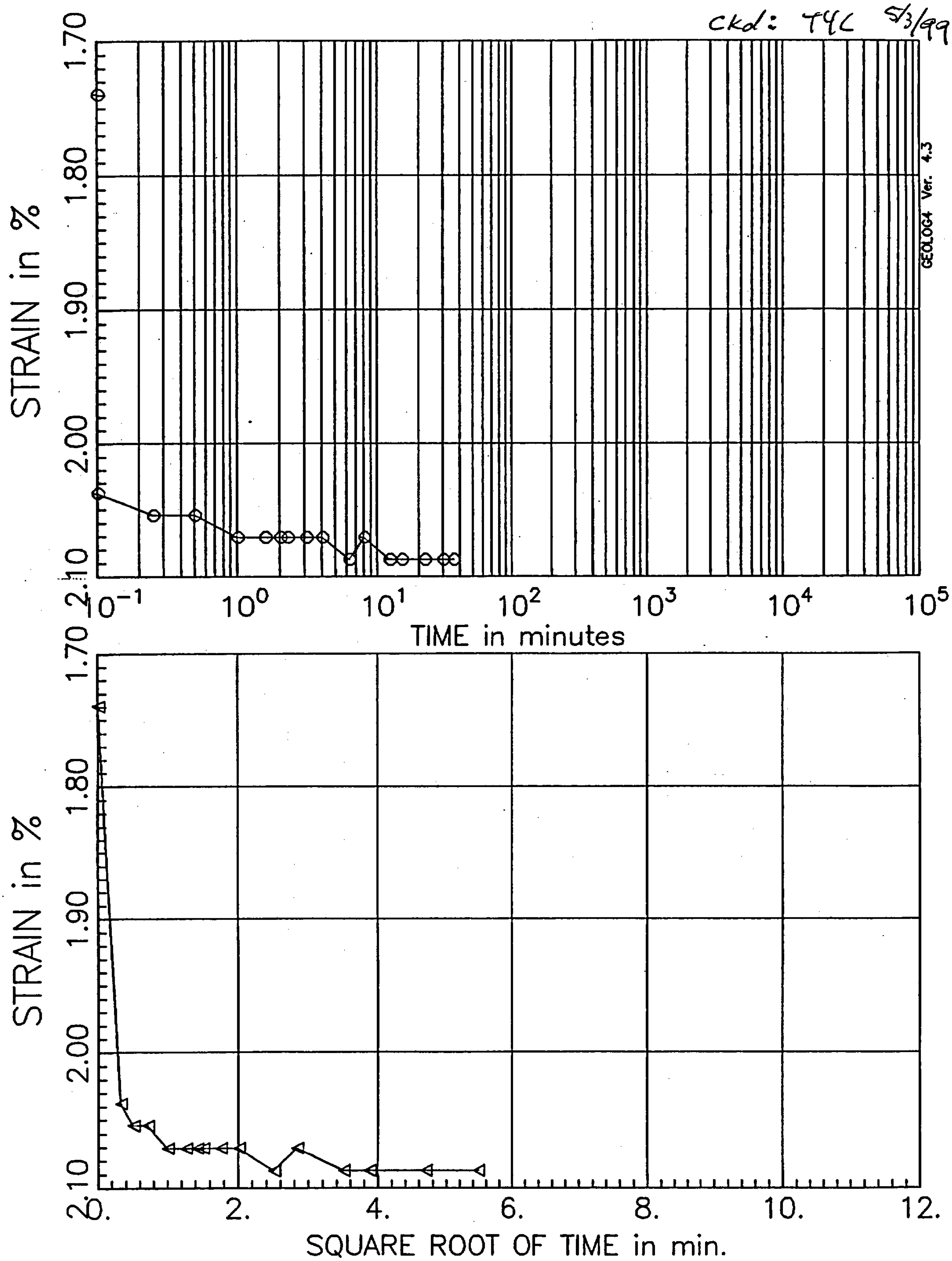


PRESSURE INCREMENT #10
from 0.50 tsf to 1.00 tsf

Test No: 2
Testname: CTB5-12C

JO 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 2.07\%$

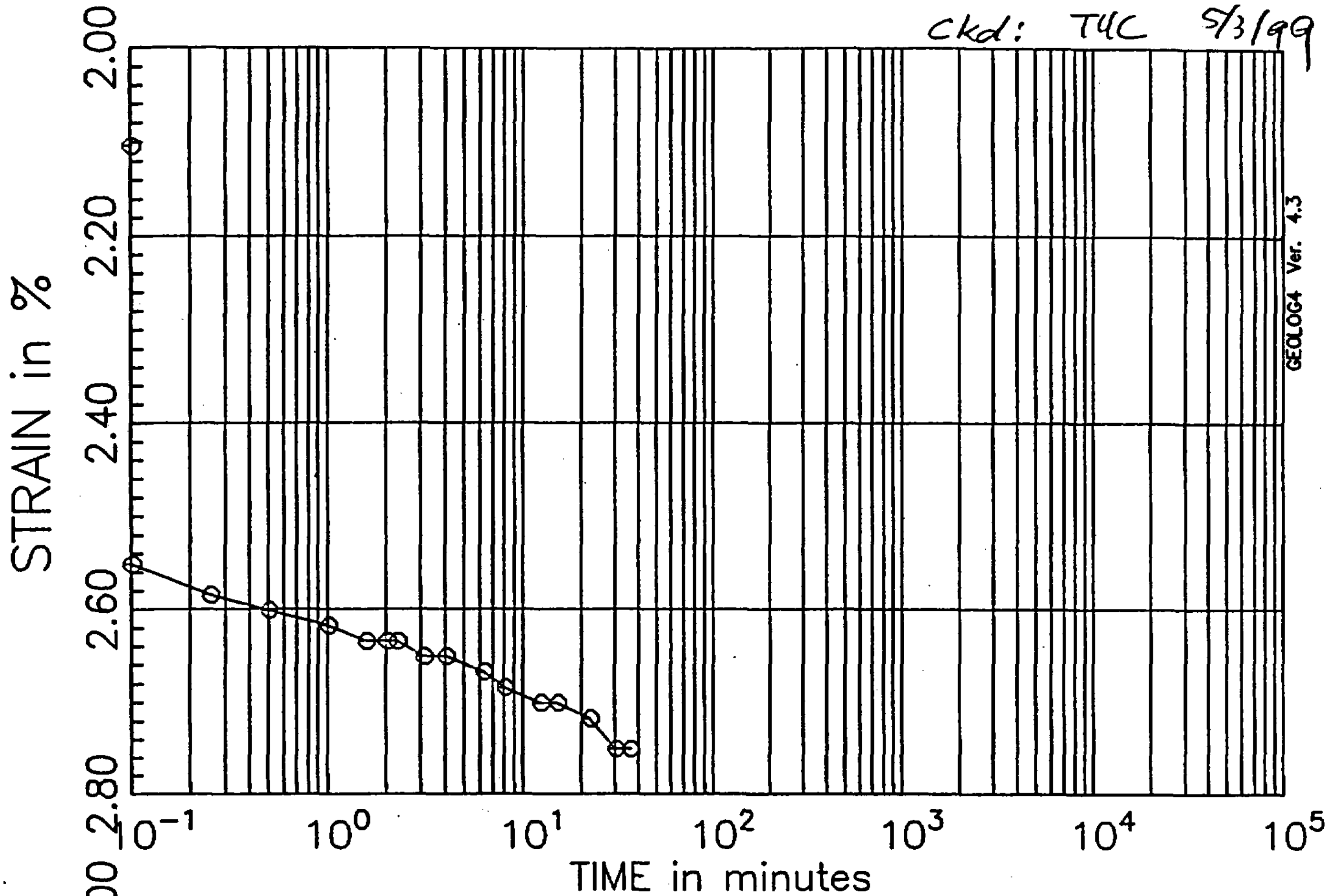


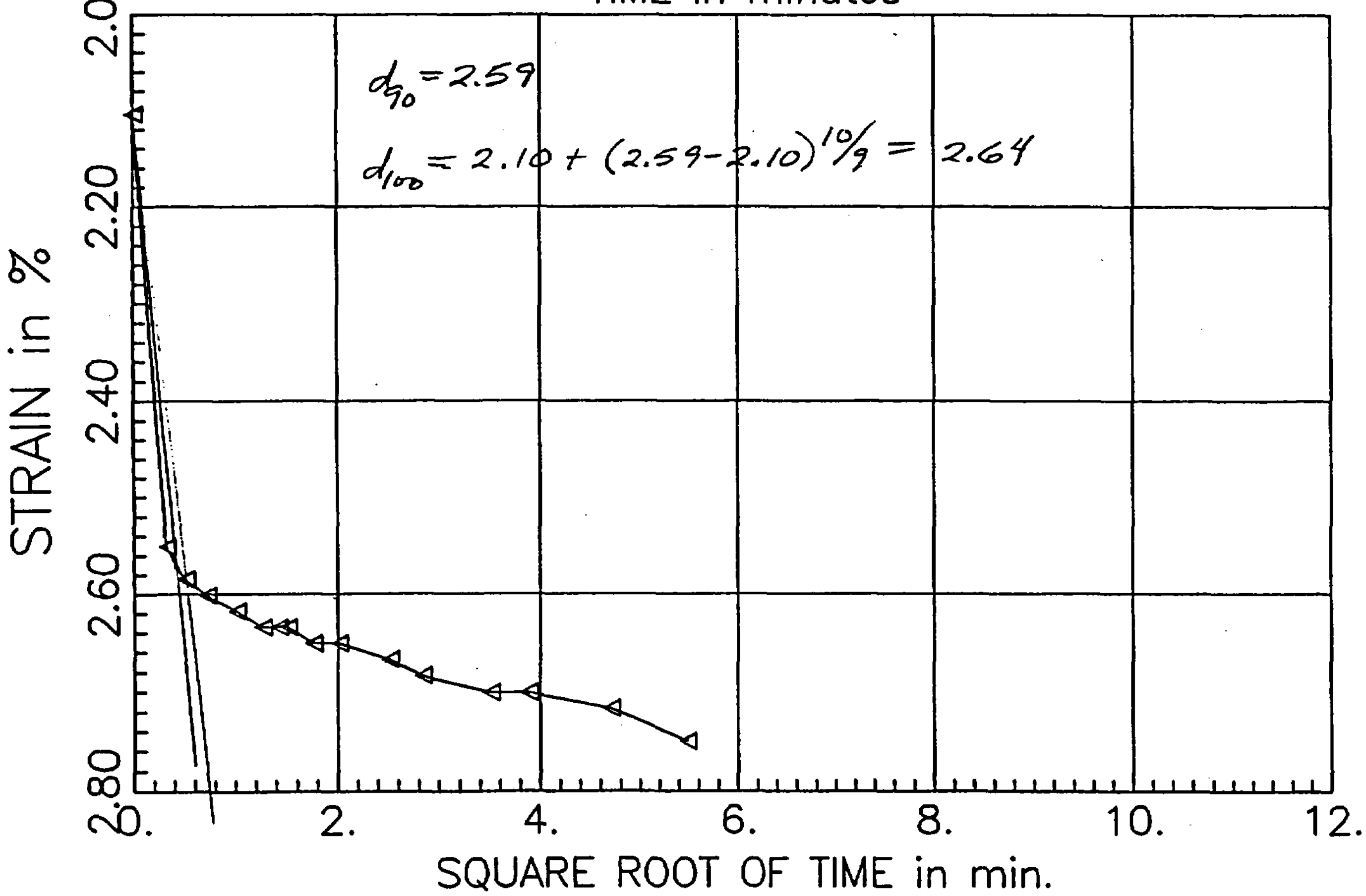
PRESSURE INCREMENT #11
from 1.00 tsf to 2.00 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 2.64\%$

Calc: ACS 4/26/99
Ckd: TWC 5/3/99



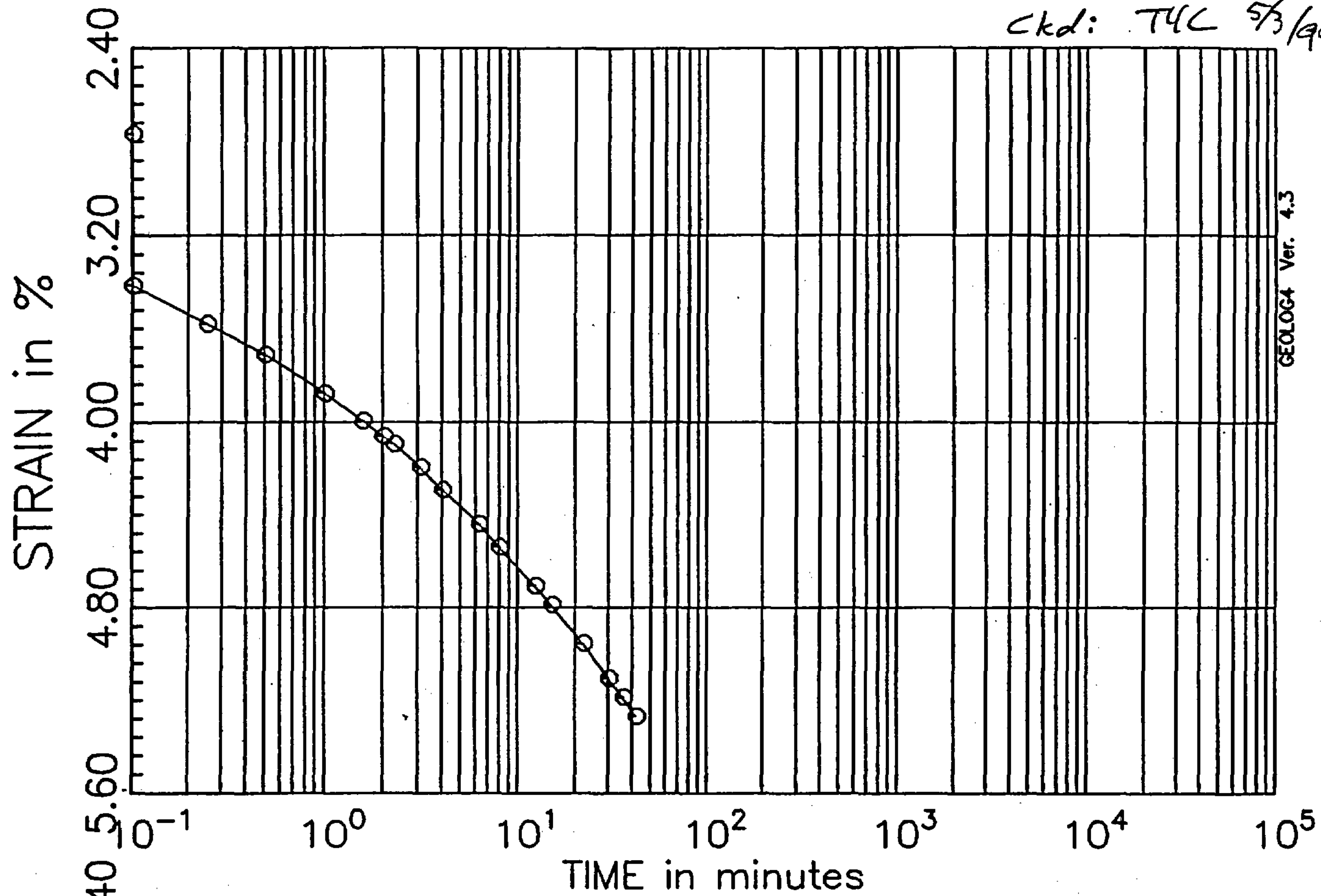


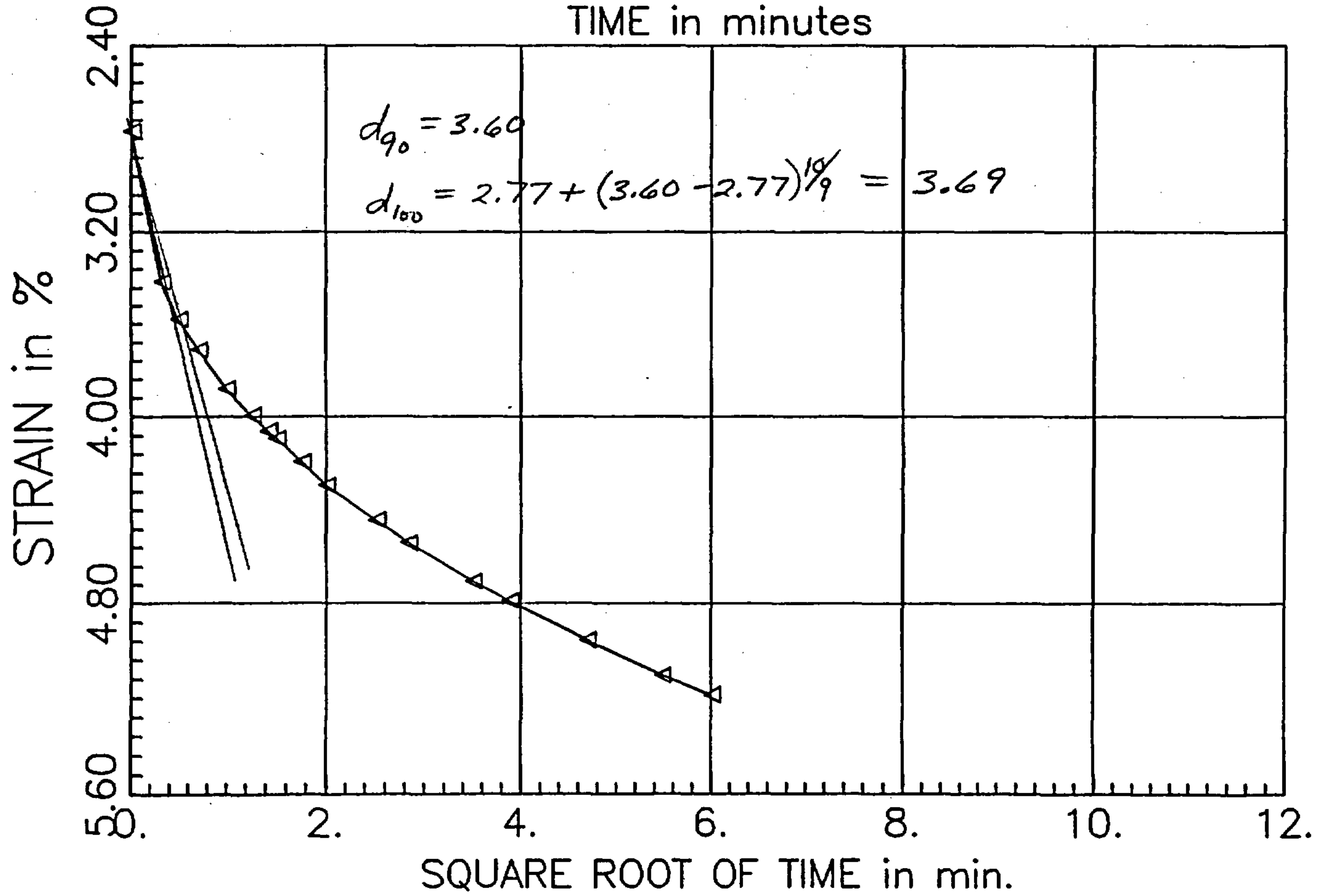
PRESSURE INCREMENT #12
from 2.00 tsf to 4.00 tsf

Test No: 2
Testname: CTB5-12C

Jo 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 3.69\%$



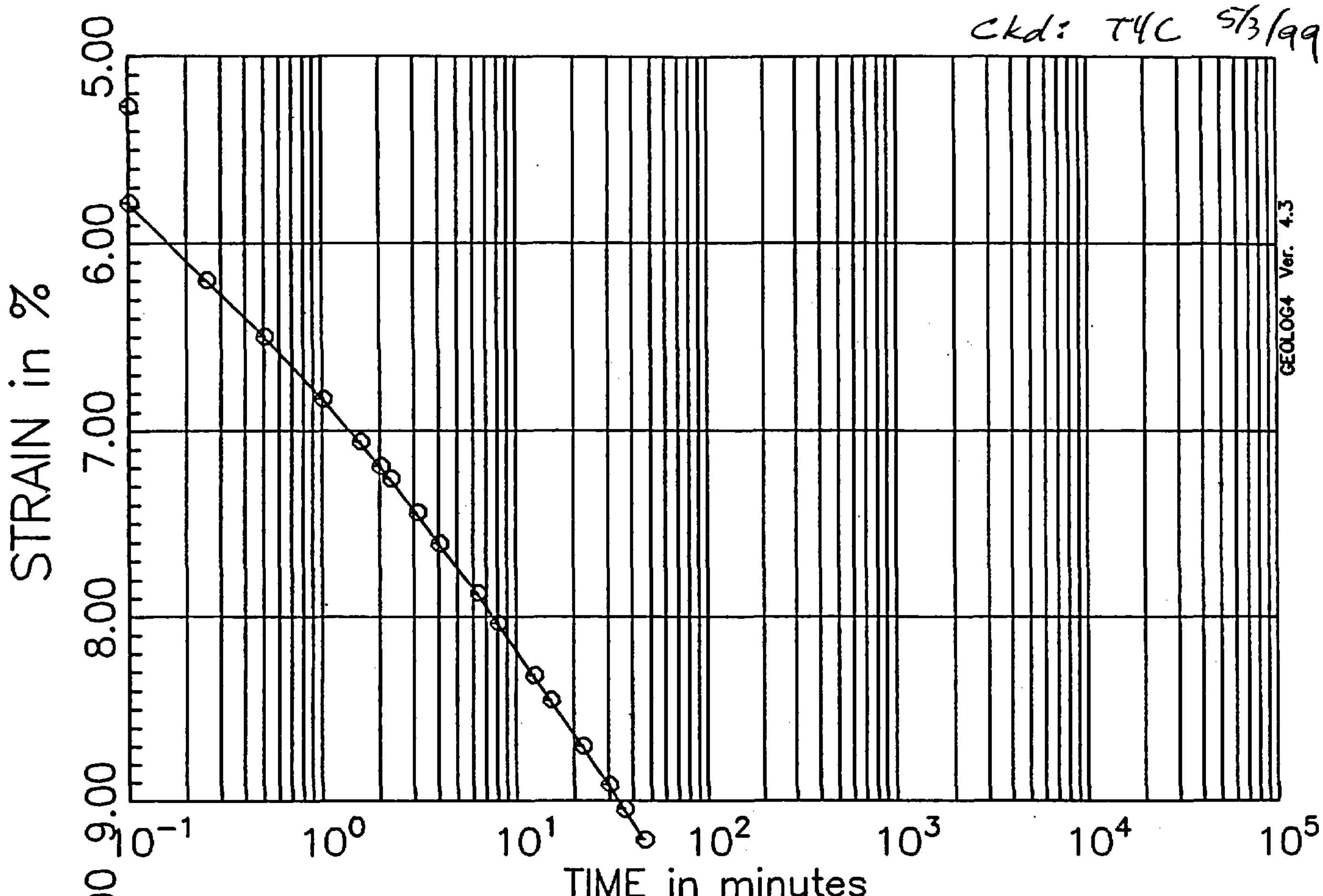


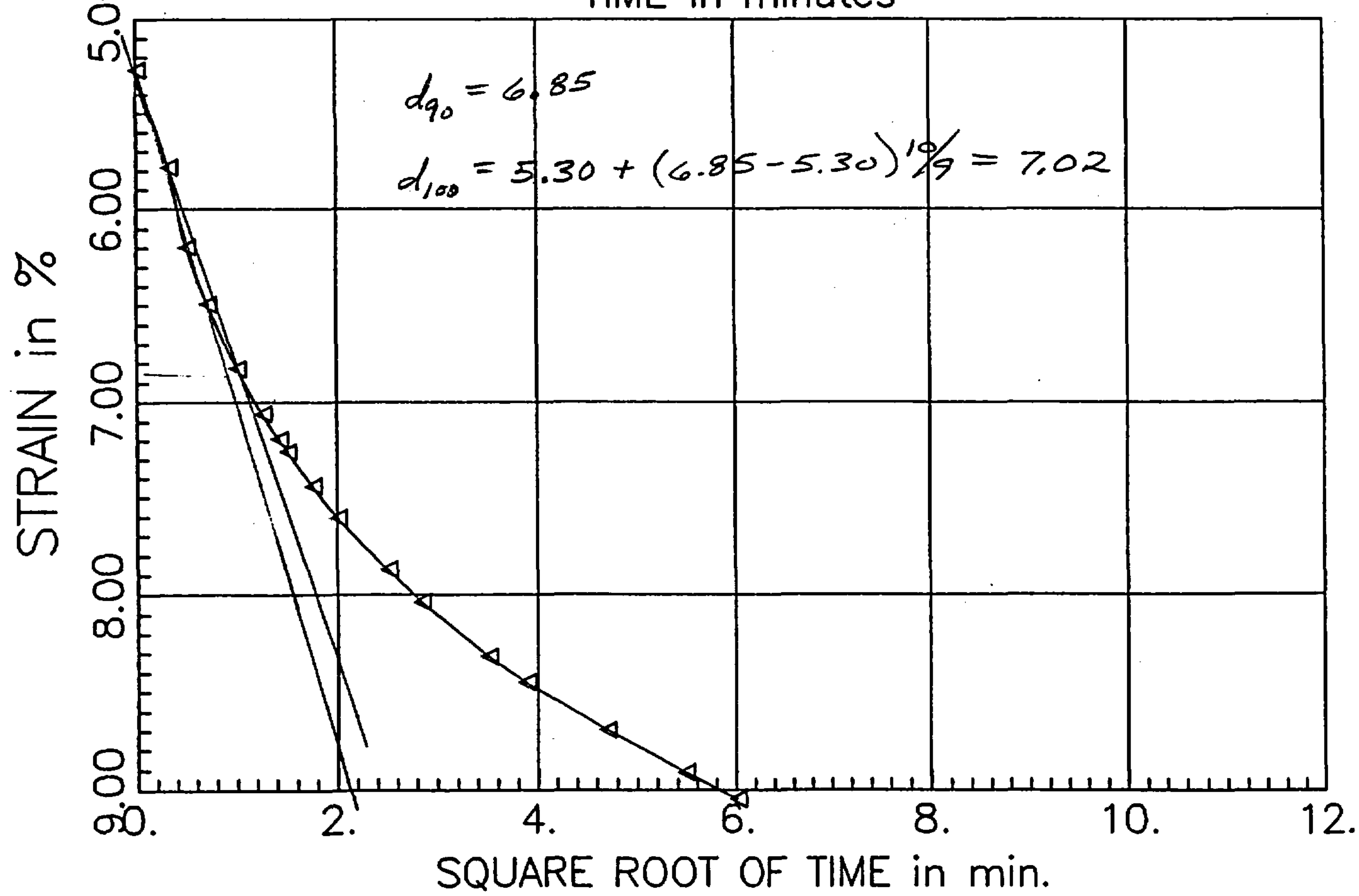
PRESSURE INCREMENT #13
from 4.00 tsf to 6.00 tsf

Test No: 2
Testname: CTB5-12C

$d_{100} = 7.02\%$

Calc: ACS 4/26/99
Ckd: TYC 5/3/99



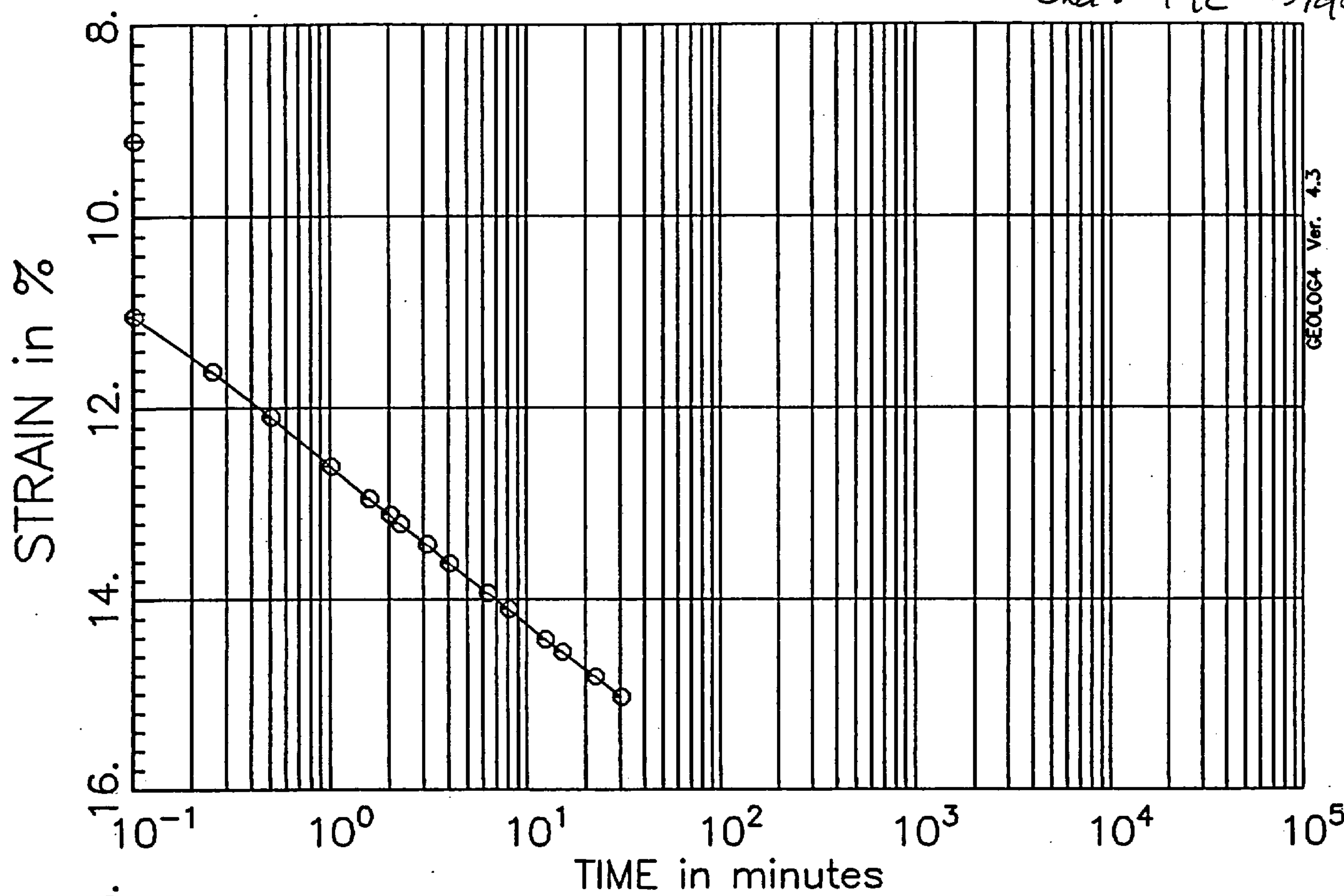


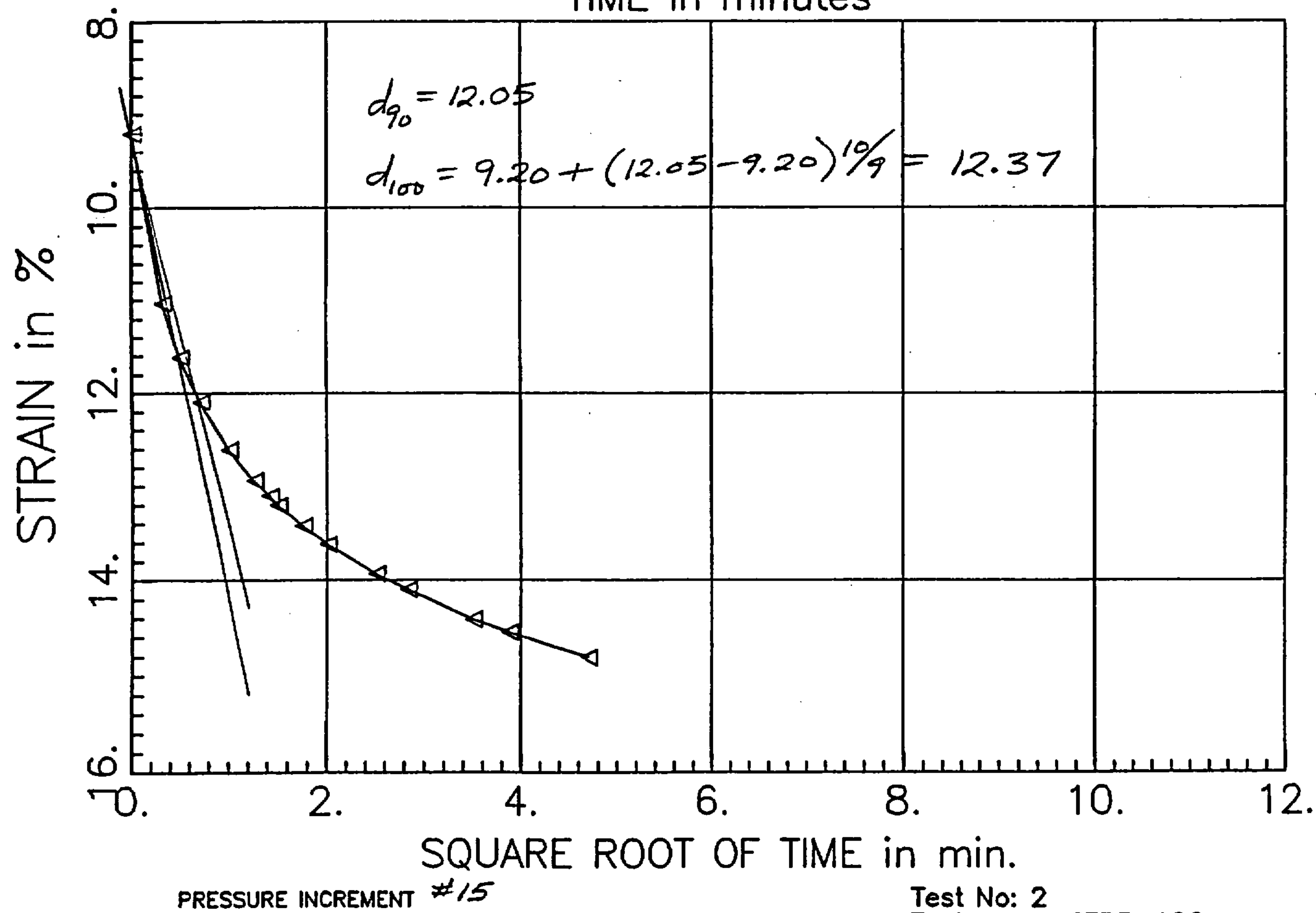
PRESSURE INCREMENT #14
from 6.00 tsf to 8.00 tsf

Test No: 2
Testname: CTB5-12C

JO 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 12.37\%$





$d_{90} = 12.05$

$d_{100} = 9.20 + (12.05 - 9.20)\,10/9 = 12.37$

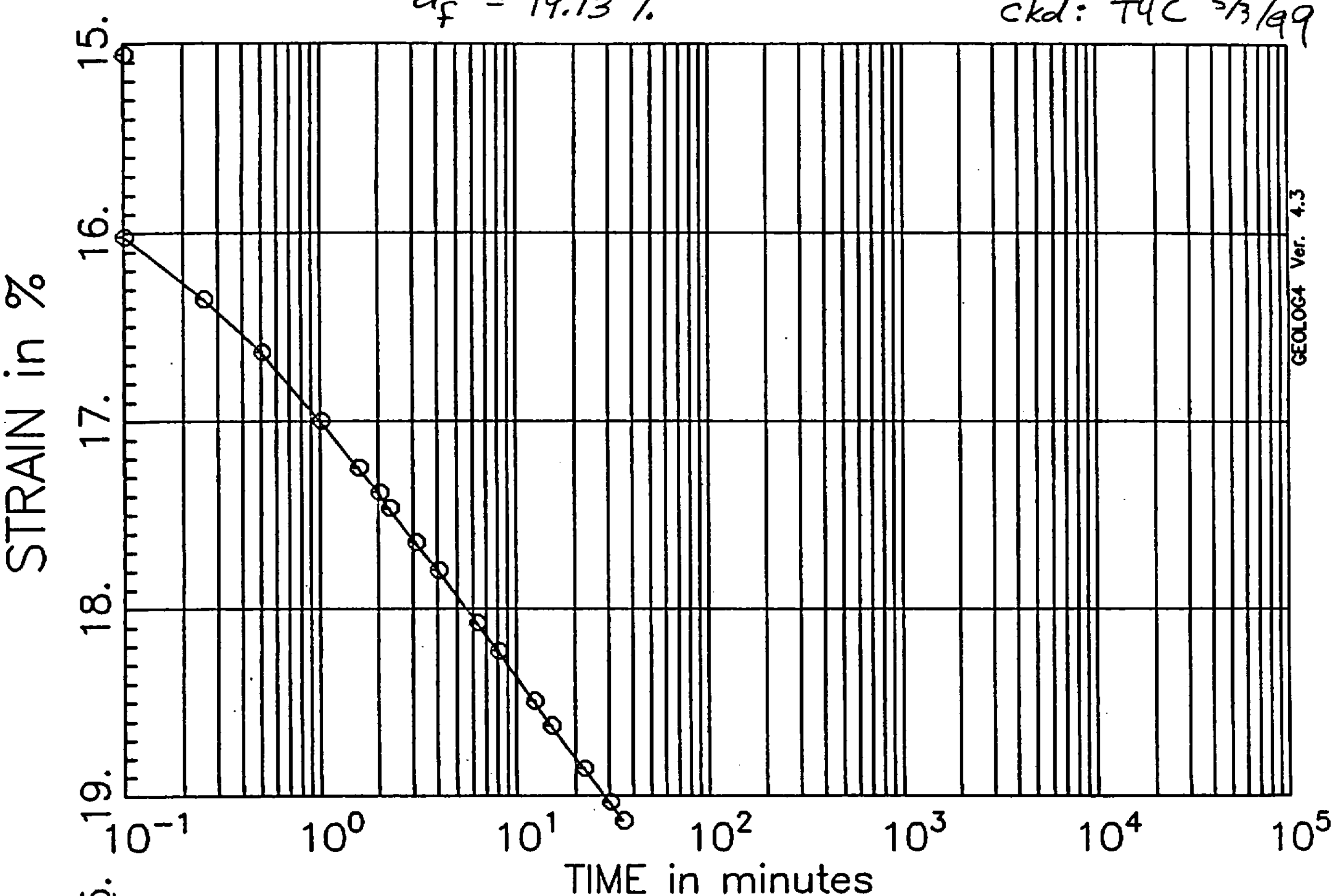
PRESSURE INCREMENT #15
from 8.00 tsf to 12.00 tsf

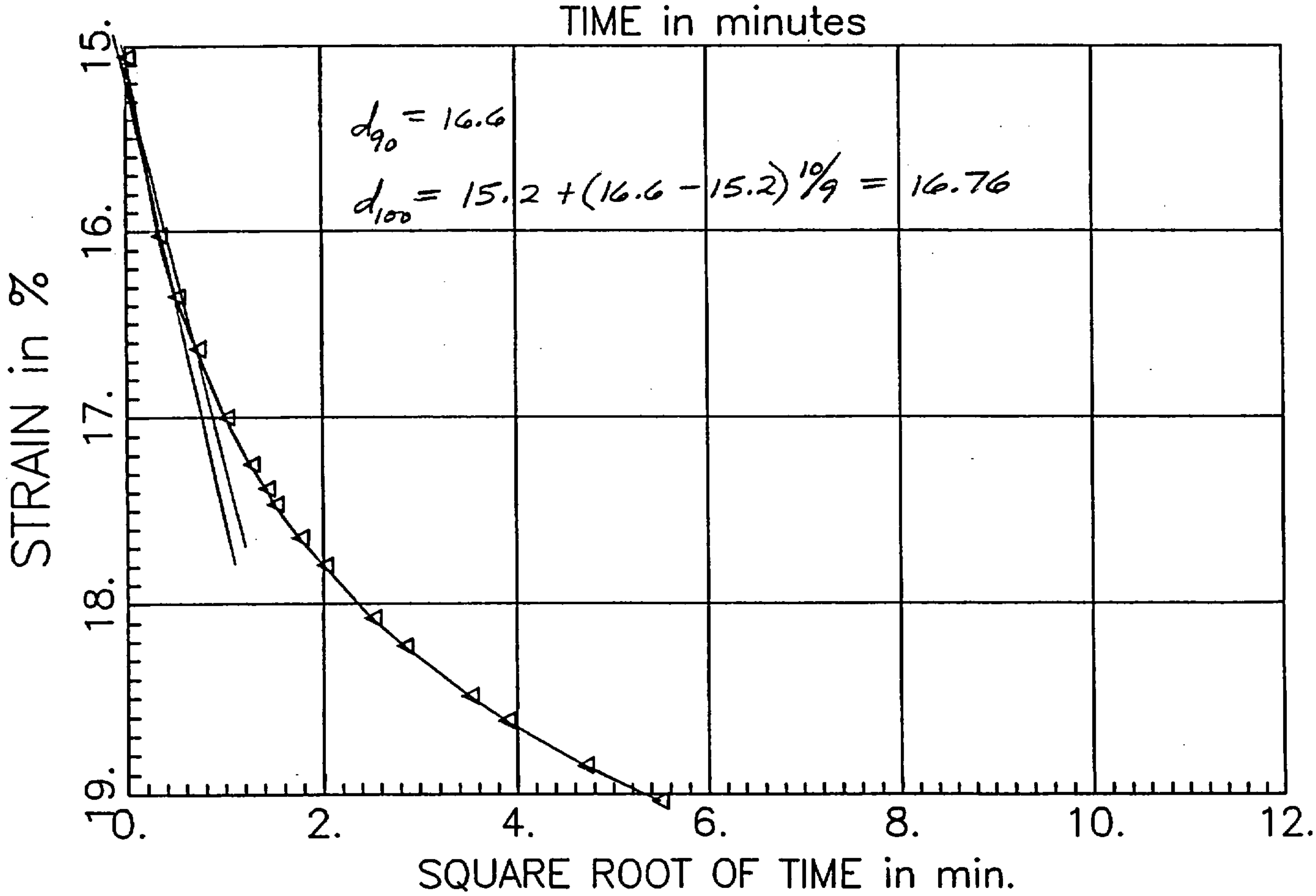
Test No: 2
Testname: CTB5-12C

$d_{100} = 16.76$ %

$d_f = 19.13$ %

Calc: ACS 4/26/99
Ckd: TYC 5/3/99



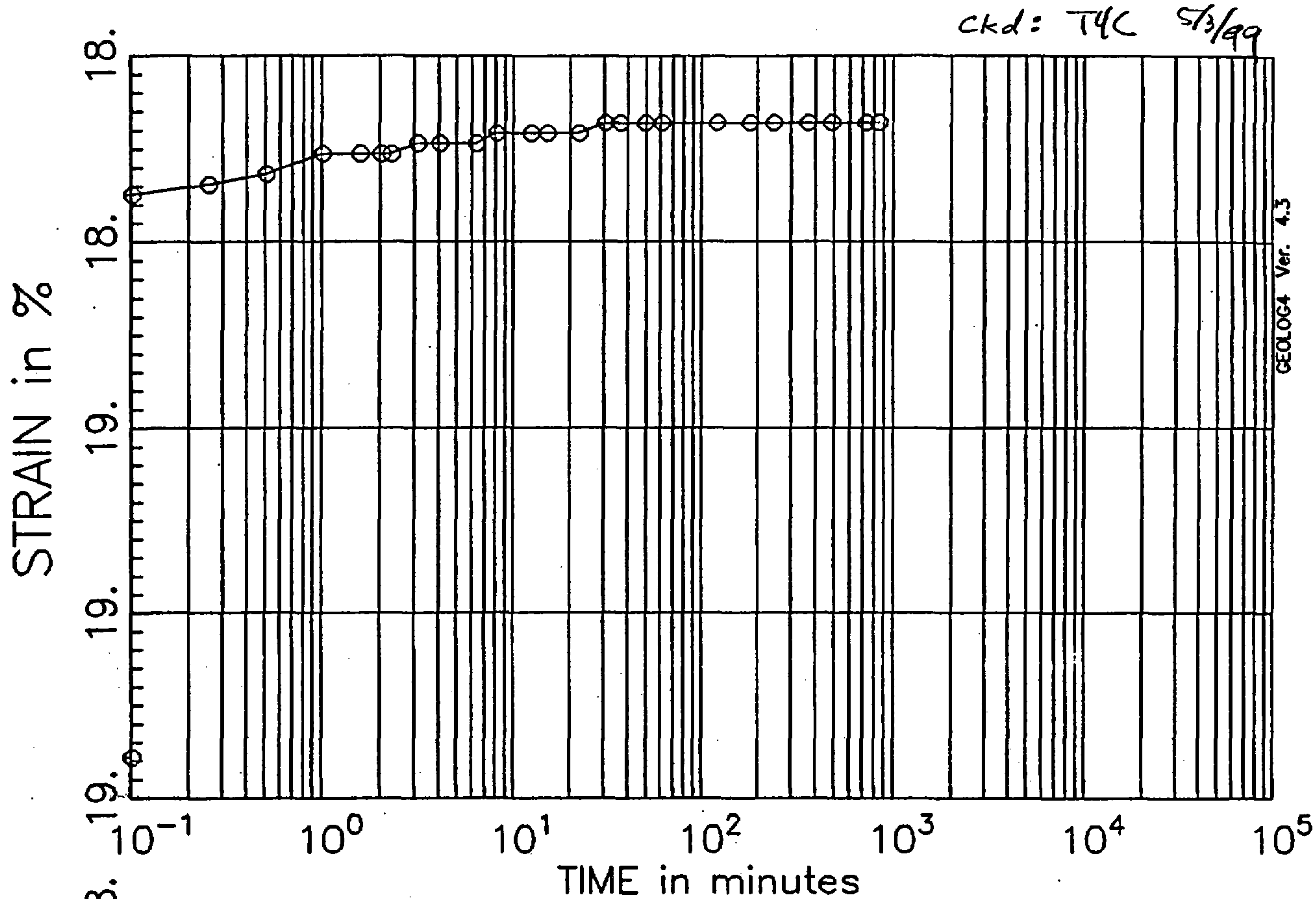


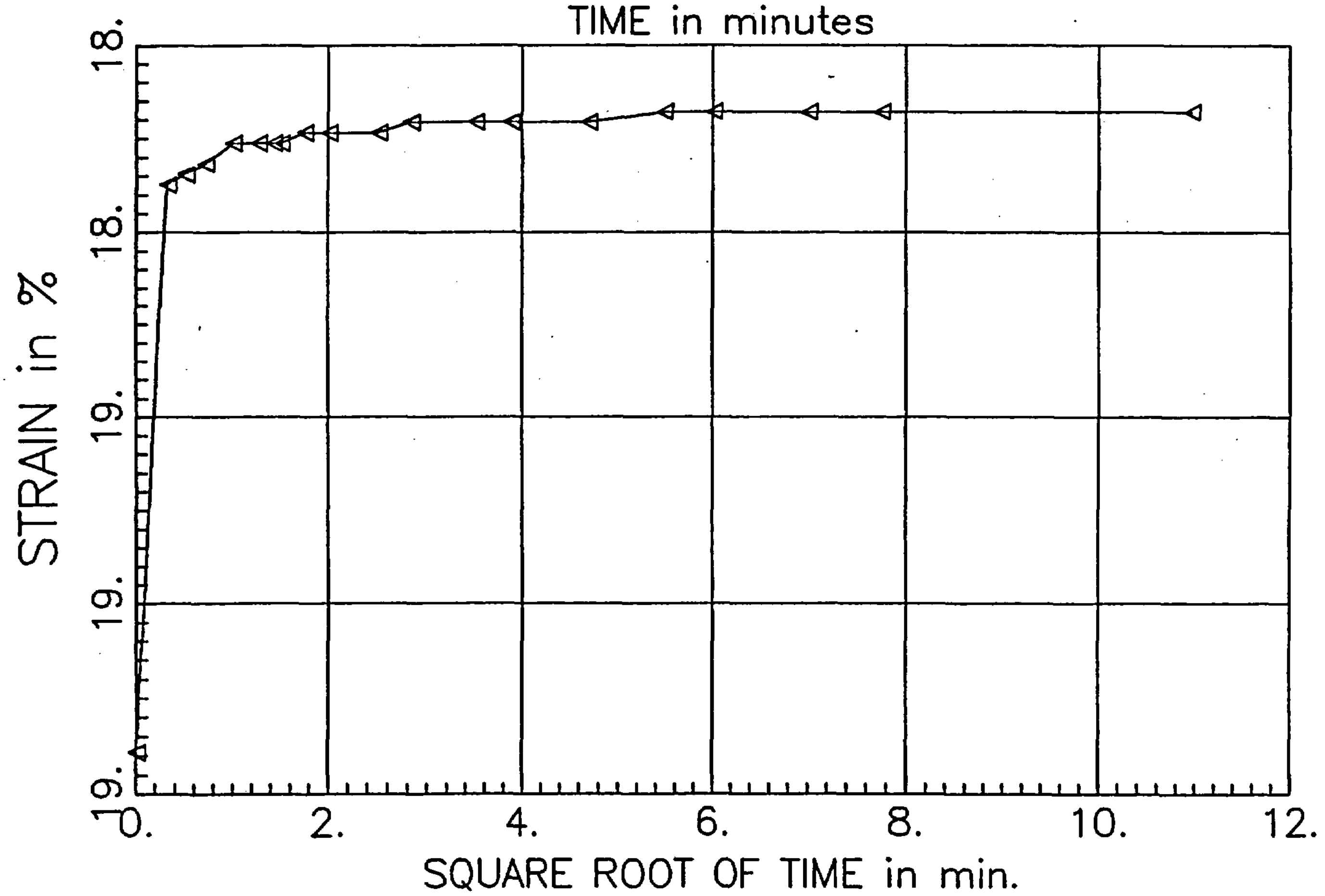
PRESSURE INCREMENT #16
from 12.00 tsf to 16.00 tsf

Test No: 2
Testname: CTB5-12C

Jo 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 18.16\%$





PRESSURE INCREMENT #17
from 16.00 tsf to 4.00 tsf

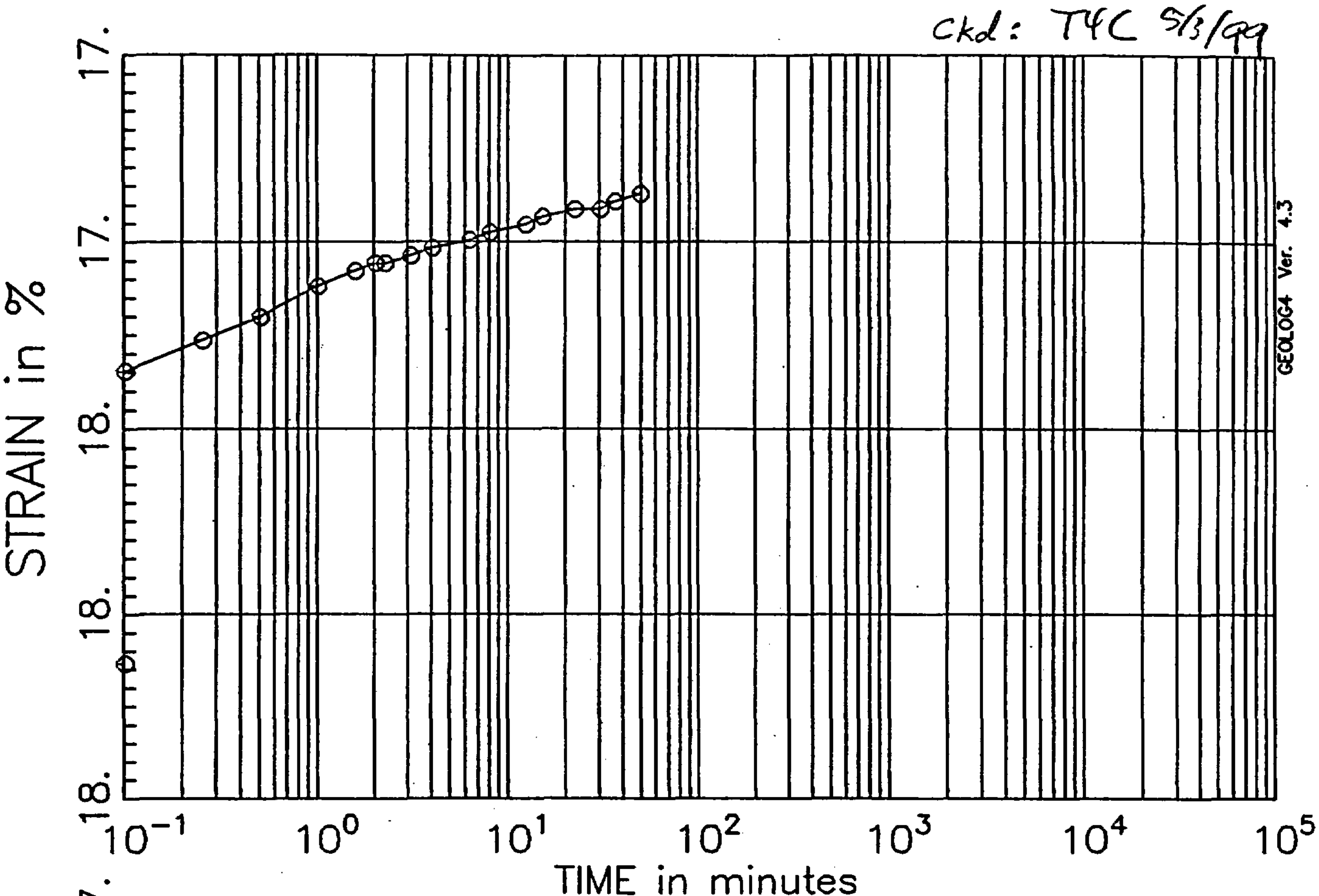
Test No: 2
Testname: CTB5-12C

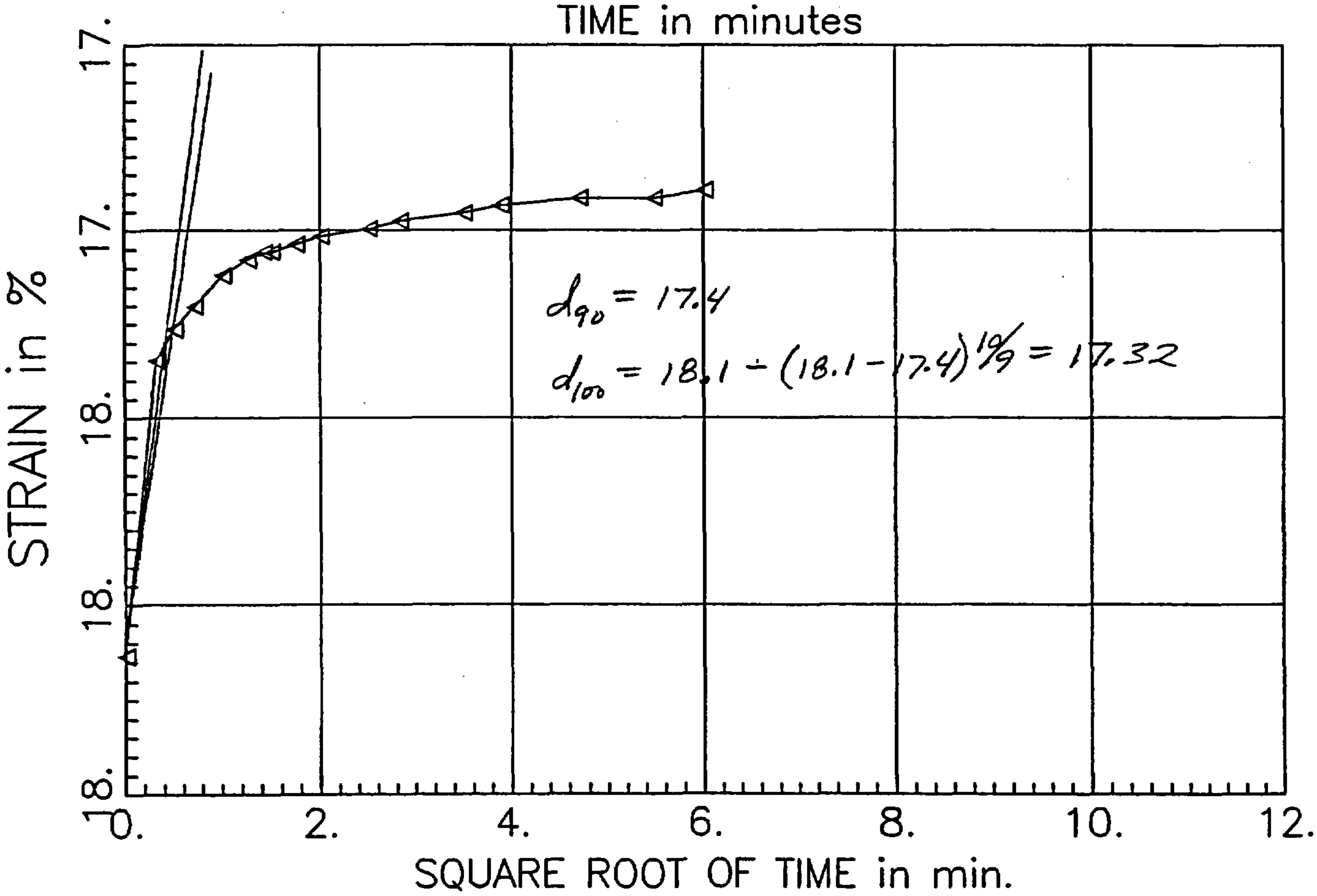
JU 05776.02

$d_{100} = 17.32\%$

Calc: ACS 4/26/99

Ckd: TYC 5/3/99



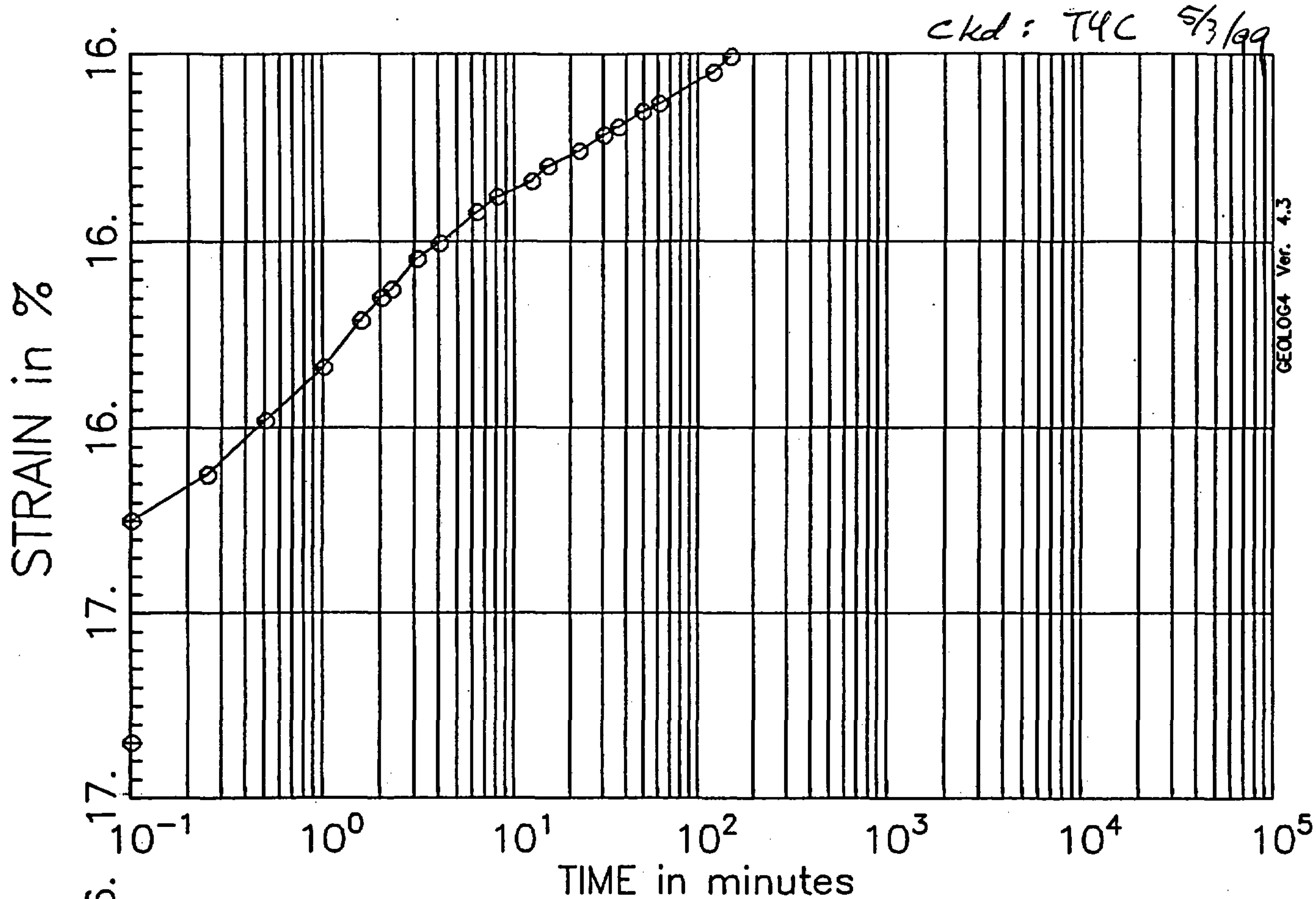


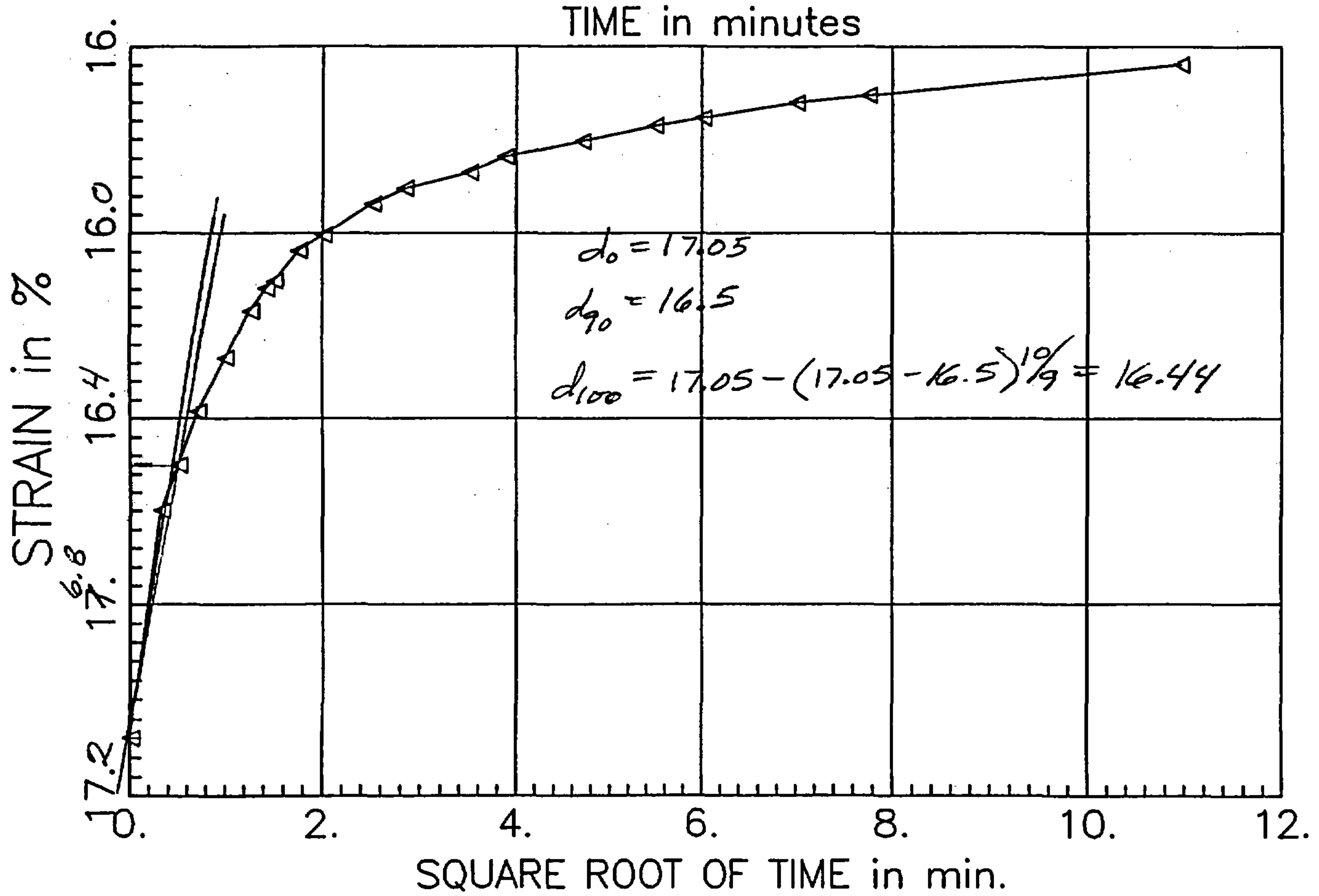
PRESSURE INCREMENT #18
from 4.00 tsf to 1.00 tsf

Test No: 2
Testname: CTB5-12C

JO 05996.02
Calc: ACS 4/26/99
Ckd: T4C 5/3/99

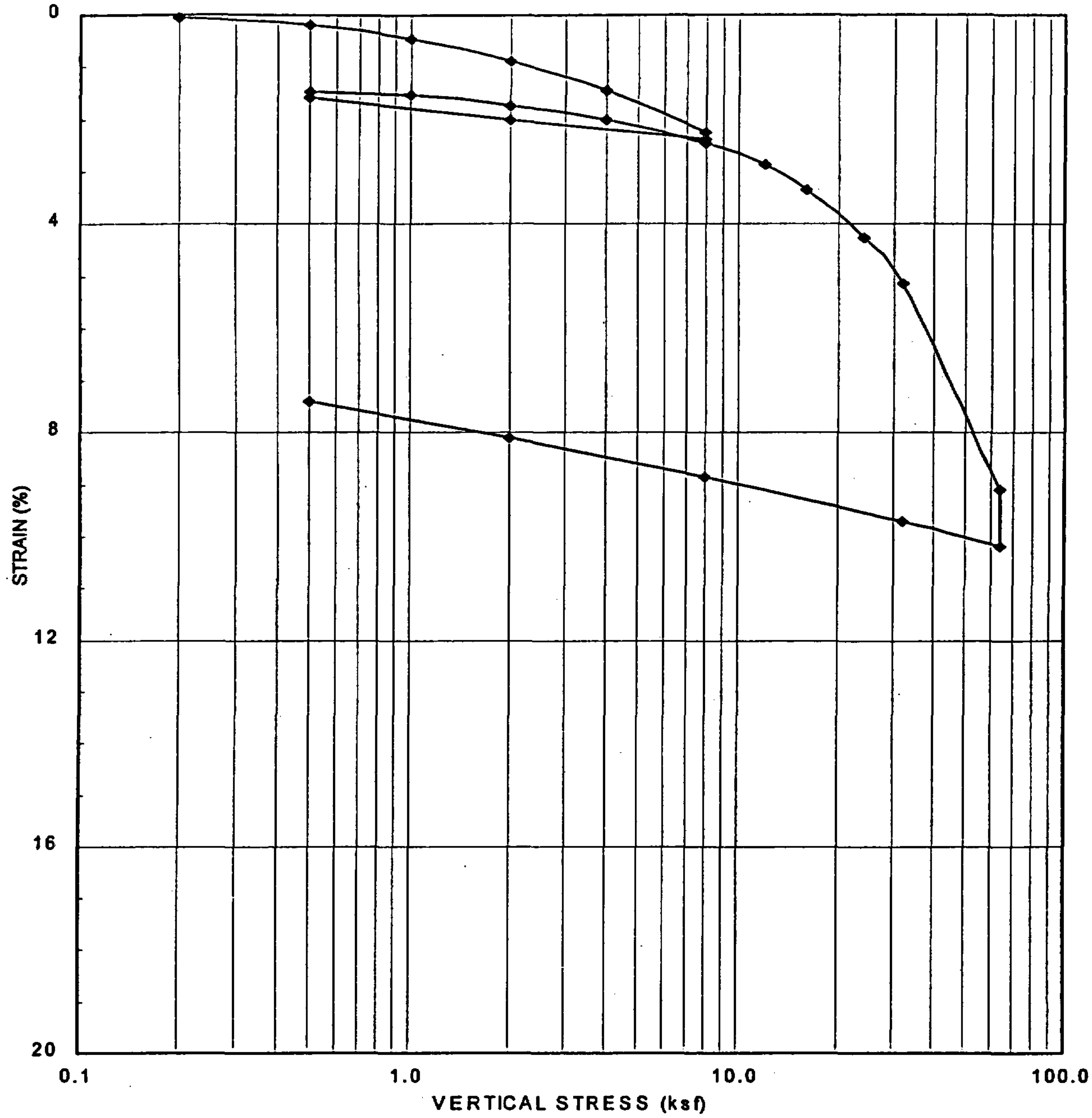
$d_{100} = 16.44\%$





PRESSURE INCREMENT #19
from 1.00 tsf to 0.25 tsf

Test No: 2
Testname: CTB5-12C



SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	4/14/99
SAMPLE:	U-14E	TESTED BY:	ACS
DEPTH:	27.3 ft	CHECKED:	TYC
DESCRIPTION:	CLAY (CL)		

SPECIMEN INFORMATION:

	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	26.2 %	24.9 %	
DRY UNIT WEIGHT:	90.9 pcf	97.9 pcf	2.72 (est)
VOID RATIO:	0.868	0.735	
SATURATION:	82.1 %	92.2 %	

<u>NOTE:</u> Sample was not inundated

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING CTB-5, SAMPLE U-14E	JO 05996.02 April 1999

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	4/14/99
SAMPLE:	U-14E	TESTED BY:	ACS
DEPTH:	27.3 ft	CHECKED:	TYC
DESCRIPTION:	CLAY (CL)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	26.2 %	24.9 %
DRY UNIT WEIGHT:	90.9 pcf	97.9 pcf
VOID RATIO:	0.868	0.735
SATURATION:	82.1 %	92.2 %
HEIGHT:	1.902 cm	1.767 cm
AREA:	31.63 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE ksf	STRAIN %
0.20	0.06
0.50	0.20
1.00	0.46
2.00	0.89
4.00	1.44
8.00	2.25
8.00	2.39
2.00	2.00
0.50	1.58
0.50	1.47
1.00	1.54
2.00	1.73
4.00	2.02
8.00	2.46
12.00	2.87
16.00	3.35
24.00	4.27
32.00	5.15
64.00	9.10
64.00	10.20
32.00	9.70
8.00	8.85
2.00	8.10
0.50	7.40

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 1
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	15:54:35	0.10	0.32	0.0003	0.867	0.05
		0.25	0.50	0.0003	0.867	0.05
		0.50	0.71	0.0003	0.867	0.05
		1.00	1.00	0.0005	0.867	0.06
		1.57	1.25	0.0005	0.867	0.06
		2.00	1.41	0.0005	0.867	0.06
		2.25	1.50	0.0005	0.867	0.06
		3.07	1.75	0.0005	0.867	0.06
		4.00	2.00	0.0005	0.867	0.06
		6.25	2.50	0.0005	0.867	0.06
		8.00	2.83	0.0005	0.867	0.06
		12.25	3.50	0.0005	0.867	0.06

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 2

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:08:10	0.00	0.00	0.0005	0.867	0.06
		0.10	0.32	0.0014	0.865	0.19
		0.25	0.50	0.0014	0.865	0.19
		0.50	0.71	0.0015	0.864	0.20
		1.00	1.00	0.0015	0.864	0.20
		1.57	1.25	0.0015	0.864	0.20
		2.00	1.41	0.0015	0.864	0.20
		2.25	1.50	0.0015	0.864	0.20
		3.07	1.75	0.0015	0.864	0.20
		4.00	2.00	0.0016	0.864	0.22
		6.25	2.50	0.0016	0.864	0.22
		8.00	2.83	0.0016	0.864	0.22
		12.25	3.50	0.0016	0.864	0.22
		15.00	3.87	0.0016	0.864	0.22
		22.00	4.69	0.0017	0.864	0.23

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 3
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:35:37	0.00	0.00	0.0017	0.864	0.23
		0.10	0.32	0.0031	0.860	0.42
		0.25	0.50	0.0034	0.860	0.45
		0.50	0.71	0.0034	0.860	0.45
		1.00	1.00	0.0035	0.859	0.46
		1.57	1.25	0.0035	0.859	0.46
		2.00	1.41	0.0036	0.859	0.48
		2.25	1.50	0.0036	0.859	0.48
		3.07	1.75	0.0036	0.859	0.48
		4.00	2.00	0.0036	0.859	0.48
		6.25	2.50	0.0037	0.859	0.49
		8.00	2.83	0.0037	0.859	0.49
		12.25	3.50	0.0037	0.859	0.49
		15.00	3.87	0.0037	0.859	0.49
		22.00	4.69	0.0038	0.858	0.51
		30.00	5.48	0.0038	0.858	0.51
		36.00	6.00	0.0039	0.858	0.52

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 4

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	17:12:10	0.00	0.00	0.0039	0.858	0.52
		0.10	0.32	0.0061	0.853	0.82
		0.25	0.50	0.0064	0.852	0.85
		0.50	0.71	0.0065	0.852	0.86
		1.00	1.00	0.0067	0.851	0.89
		1.57	1.25	0.0067	0.851	0.89
		2.00	1.41	0.0068	0.851	0.91
		2.25	1.50	0.0068	0.851	0.91
		3.07	1.75	0.0068	0.851	0.91
		4.00	2.00	0.0068	0.851	0.91
		6.25	2.50	0.0069	0.851	0.93
		8.00	2.83	0.0069	0.851	0.93
		12.25	3.50	0.0070	0.850	0.94
		15.00	3.87	0.0070	0.850	0.94
		22.00	4.69	0.0070	0.850	0.94
		30.00	5.48	0.0072	0.850	0.96
		36.00	6.00	0.0072	0.850	0.96
		49.00	7.00	0.0073	0.850	0.97
		60.00	7.75	0.0073	0.850	0.97
		120.00	10.95	0.0074	0.850	0.99
		180.00	13.42	0.0075	0.849	1.00
		240.00	15.49	0.0076	0.849	1.02
		360.00	18.97	0.0076	0.849	1.02
04-15-99	01:12:10	480.00	21.91	0.0077	0.849	1.03
		720.00	26.83	0.0079	0.848	1.05
		840.00	28.98	0.0077	0.849	1.03

LOAD INCREMENT DATA

TEST NAME CTB5-14E　　　　PAGE NO: 5
TEST NO: 3　　TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	08:00:35	0.00	0.00	0.0079	0.848	1.05
		0.10	0.32	0.0104	0.842	1.39
		0.25	0.50	0.0105	0.842	1.40
		0.50	0.71	0.0107	0.841	1.43
		1.00	1.00	0.0109	0.841	1.45
		1.57	1.25	0.0110	0.841	1.47
		2.00	1.41	0.0110	0.841	1.47
		2.25	1.50	0.0111	0.840	1.48
		3.07	1.75	0.0111	0.840	1.48
		4.00	2.00	0.0111	0.840	1.48
		6.25	2.50	0.0112	0.840	1.50
		8.00	2.83	0.0112	0.840	1.50
		12.25	3.50	0.0113	0.840	1.51
		15.00	3.87	0.0114	0.839	1.53
		22.00	4.69	0.0114	0.839	1.53
		30.00	5.48	0.0116	0.839	1.54
		36.00	6.00	0.0116	0.839	1.54
		49.00	7.00	0.0116	0.839	1.54
		60.00	7.75	0.0117	0.839	1.56

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 6

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	09:06:28	0.00	0.00	0.0117	0.839	1.56
		0.10	0.32	0.0158	0.829	2.11
		0.25	0.50	0.0163	0.827	2.18
		0.50	0.71	0.0164	0.827	2.19
		1.00	1.00	0.0166	0.826	2.22
		1.57	1.25	0.0168	0.826	2.24
		2.00	1.41	0.0169	0.826	2.25
		2.25	1.50	0.0169	0.826	2.25
		3.07	1.75	0.0170	0.826	2.27
		4.00	2.00	0.0170	0.826	2.27
		6.25	2.50	0.0171	0.825	2.28
		8.00	2.83	0.0172	0.825	2.30
		12.25	3.50	0.0173	0.825	2.31
		15.00	3.87	0.0174	0.824	2.33
		22.00	4.69	0.0176	0.824	2.35
		30.00	5.48	0.0177	0.824	2.36
		36.00	6.00	0.0177	0.824	2.36
		49.00	7.00	0.0178	0.824	2.38
		60.00	7.75	0.0178	0.824	2.38

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 7
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	10:20:36	0.00	0.00	0.0179	0.823	2.39
		0.10	0.32	0.0153	0.830	2.04
		0.25	0.50	0.0151	0.830	2.02
		0.50	0.71	0.0151	0.830	2.02
		1.00	1.00	0.0150	0.831	2.01
		1.57	1.25	0.0149	0.831	1.99
		2.00	1.41	0.0149	0.831	1.99
		2.25	1.50	0.0149	0.831	1.99
		3.07	1.75	0.0149	0.831	1.99
		4.00	2.00	0.0149	0.831	1.99
		6.25	2.50	0.0148	0.831	1.97
		8.00	2.83	0.0148	0.831	1.97
		12.25	3.50	0.0148	0.831	1.97
		15.00	3.87	0.0148	0.831	1.97
		22.00	4.69	0.0148	0.831	1.97
		30.00	5.48	0.0148	0.831	1.97
		36.00	6.00	0.0147	0.831	1.96

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 8
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	11:08:40	0.00	0.00	0.0148	0.831	1.97
		0.10	0.32	0.0124	0.837	1.65
		0.25	0.50	0.0121	0.838	1.62
		0.50	0.71	0.0119	0.838	1.59
		1.00	1.00	0.0117	0.839	1.56
		1.57	1.25	0.0116	0.839	1.54
		2.00	1.41	0.0116	0.839	1.54
		2.25	1.50	0.0116	0.839	1.54
		3.07	1.75	0.0114	0.839	1.53
		4.00	2.00	0.0114	0.839	1.53
		6.25	2.50	0.0113	0.840	1.51
		8.00	2.83	0.0113	0.840	1.51
		12.25	3.50	0.0112	0.840	1.50
		15.00	3.87	0.0112	0.840	1.50
		22.00	4.69	0.0111	0.840	1.48
		30.00	5.48	0.0111	0.840	1.48
		36.00	6.00	0.0111	0.840	1.48
		49.00	7.00	0.0110	0.841	1.47
		60.00	7.75	0.0110	0.841	1.47

LOAD INCREMENT DATA

TEST NAME CTB5-14E

PAGE NO: 9

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:14:01	0.00	0.00	0.0110	0.841	1.47
		0.10	0.32	0.0116	0.839	1.54
		0.25	0.50	0.0116	0.839	1.54
		0.50	0.71	0.0116	0.839	1.54
		1.00	1.00	0.0116	0.839	1.54
		1.57	1.25	0.0116	0.839	1.54
		2.00	1.41	0.0116	0.839	1.54
		2.25	1.50	0.0116	0.839	1.54
		3.07	1.75	0.0116	0.839	1.54
		4.00	2.00	0.0116	0.839	1.54
		6.25	2.50	0.0116	0.839	1.54
		8.00	2.83	0.0116	0.839	1.54
		12.25	3.50	0.0116	0.839	1.54
		15.00	3.87	0.0116	0.839	1.54
		22.00	4.69	0.0116	0.839	1.54
		30.00	5.48	0.0116	0.839	1.54
		36.00	6.00	0.0117	0.839	1.56

LOAD INCREMENT DATA

TEST NAME CTB5-14E

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:51:07	0.00	0.00	0.0117	0.839	1.56
		0.10	0.32	0.0128	0.836	1.71
		0.25	0.50	0.0128	0.836	1.71
		0.50	0.71	0.0129	0.836	1.73
		1.00	1.00	0.0129	0.836	1.73
		1.57	1.25	0.0129	0.836	1.73
		2.00	1.41	0.0129	0.836	1.73
		2.25	1.50	0.0129	0.836	1.73
		3.07	1.75	0.0129	0.836	1.73
		4.00	2.00	0.0129	0.836	1.73
		6.25	2.50	0.0129	0.836	1.73
		8.00	2.83	0.0129	0.836	1.73
		12.25	3.50	0.0131	0.835	1.74
		15.00	3.87	0.0129	0.836	1.73
		22.00	4.69	0.0131	0.835	1.74
		30.00	5.48	0.0131	0.835	1.74

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 11
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	13:27:13	0.00	0.00	0.0131	0.835	1.74
		0.10	0.32	0.0149	0.831	1.99
		0.25	0.50	0.0151	0.830	2.02
		0.50	0.71	0.0150	0.831	2.01
		1.00	1.00	0.0151	0.830	2.02
		1.57	1.25	0.0153	0.830	2.04
		2.00	1.41	0.0151	0.830	2.02
		2.25	1.50	0.0151	0.830	2.02
		3.07	1.75	0.0153	0.830	2.04
		4.00	2.00	0.0151	0.830	2.02
		6.25	2.50	0.0153	0.830	2.04
		8.00	2.83	0.0153	0.830	2.04
		12.25	3.50	0.0153	0.830	2.04
		15.00	3.87	0.0153	0.830	2.04
		22.00	4.69	0.0154	0.830	2.05
		30.00	5.48	0.0154	0.830	2.05
		36.00	6.00	0.0154	0.830	2.05

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 12

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	14:12:06	0.00	0.00	0.0155	0.829	2.07
		0.10	0.32	0.0180	0.823	2.41
		0.25	0.50	0.0181	0.823	2.42
		0.50	0.71	0.0181	0.823	2.42
		1.00	1.00	0.0183	0.822	2.44
		1.57	1.25	0.0183	0.822	2.44
		2.00	1.41	0.0184	0.822	2.45
		2.25	1.50	0.0184	0.822	2.45
		3.07	1.75	0.0184	0.822	2.45
		4.00	2.00	0.0185	0.822	2.47
		6.25	2.50	0.0185	0.822	2.47
		8.00	2.83	0.0185	0.822	2.47
		12.25	3.50	0.0186	0.822	2.48
		15.00	3.87	0.0186	0.822	2.48
		22.00	4.69	0.0187	0.821	2.50
		30.00	5.48	0.0187	0.821	2.50
		36.00	6.00	0.0188	0.821	2.51
		49.00	7.00	0.0188	0.821	2.51
		60.00	7.75	0.0188	0.821	2.51
		89.00	9.43	0.0189	0.821	2.53

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 13

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 6.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	15:41:18	0.00	0.00	0.0189	0.821	2.53
		0.10	0.32	0.0210	0.816	2.81
		0.25	0.50	0.0213	0.815	2.84
		0.50	0.71	0.0214	0.815	2.85
		1.00	1.00	0.0215	0.814	2.87
		1.57	1.25	0.0216	0.814	2.89
		2.00	1.41	0.0217	0.814	2.90
		2.25	1.50	0.0217	0.814	2.90
		3.07	1.75	0.0218	0.814	2.92
		4.00	2.00	0.0220	0.813	2.93
		6.25	2.50	0.0221	0.813	2.95
		8.00	2.83	0.0221	0.813	2.95
		12.25	3.50	0.0223	0.812	2.98
		15.00	3.87	0.0223	0.812	2.98
		22.00	4.69	0.0224	0.812	2.99
		30.00	5.48	0.0225	0.812	3.01
		36.00	6.00	0.0226	0.812	3.02

LOAD INCREMENT DATA

TEST NAME CTB5-14E

PAGE NO: 14

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 6.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	16:22:24	0.00	0.00	0.0228	0.811	3.04
		0.10	0.32	0.0238	0.809	3.18
		0.25	0.50	0.0246	0.807	3.29
		0.50	0.71	0.0248	0.806	3.32
		1.00	1.00	0.0251	0.805	3.35
		1.57	1.25	0.0253	0.805	3.38
		2.00	1.41	0.0253	0.805	3.38
		2.25	1.50	0.0254	0.805	3.39
		3.07	1.75	0.0254	0.805	3.39
		4.00	2.00	0.0256	0.804	3.43
		6.25	2.50	0.0258	0.804	3.44
		8.00	2.83	0.0259	0.803	3.46
		12.25	3.50	0.0261	0.803	3.49
		15.00	3.87	0.0262	0.803	3.50
		22.00	4.69	0.0263	0.802	3.52
		30.00	5.48	0.0266	0.802	3.55
		36.00	6.00	0.0267	0.801	3.56
		49.00	7.00	0.0268	0.801	3.58
		60.00	7.75	0.0269	0.801	3.59
		120.00	10.95	0.0273	0.800	3.64
		180.00	13.42	0.0276	0.799	3.69
		240.00	15.49	0.0277	0.799	3.70
		360.00	18.97	0.0281	0.798	3.75
04-16-99	00:22:24	480.00	21.91	0.0283	0.797	3.78
		720.00	26.83	0.0285	0.797	3.81
		900.00	30.00	0.0287	0.797	3.83

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 15
TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 8.00 tsf to 12.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	07:56:30	0.00	0.00	0.0287	0.797	3.83
		0.10	0.32	0.0313	0.790	4.18
		0.25	0.50	0.0317	0.789	4.23
		0.50	0.71	0.0319	0.788	4.26
		1.00	1.00	0.0321	0.788	4.29
		1.57	1.25	0.0323	0.787	4.32
		2.00	1.41	0.0325	0.787	4.34
		2.25	1.50	0.0326	0.787	4.35
		3.07	1.75	0.0327	0.786	4.37
		4.00	2.00	0.0328	0.786	4.38
		6.25	2.50	0.0330	0.786	4.41
		8.00	2.83	0.0332	0.785	4.43
		12.25	3.50	0.0335	0.784	4.47
		15.00	3.87	0.0337	0.784	4.51
		22.00	4.69	0.0340	0.783	4.54
		30.00	5.48	0.0342	0.783	4.57
		36.00	6.00	0.0343	0.782	4.58
		49.00	7.00	0.0347	0.782	4.63

LOAD INCREMENT DATA

TEST NAME CTB5-14E

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 12.00 tsf to 16.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	08:53:51	0.00	0.00	0.0348	0.781	4.64
		0.10	0.32	0.0374	0.775	5.00
		0.25	0.50	0.0380	0.773	5.08
		0.50	0.71	0.0384	0.772	5.12
		1.00	1.00	0.0388	0.771	5.18
		1.57	1.25	0.0393	0.770	5.25
		2.00	1.41	0.0395	0.769	5.28
		2.25	1.50	0.0395	0.769	5.28
		3.07	1.75	0.0399	0.769	5.32
		4.00	2.00	0.0401	0.768	5.35
		6.25	2.50	0.0406	0.767	5.42
		8.00	2.83	0.0408	0.766	5.45
		12.25	3.50	0.0414	0.765	5.52
		15.00	3.87	0.0416	0.764	5.55
		22.00	4.69	0.0421	0.763	5.62
		30.00	5.48	0.0424	0.762	5.66
		36.00	6.00	0.0426	0.762	5.69
		49.00	7.00	0.0431	0.760	5.75
		60.00	7.75	0.0433	0.760	5.79

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 17

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 16.00 tsf to 32.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	10:30:49	0.00	0.00	0.0441	0.758	5.89
		0.10	0.32	0.0512	0.740	6.83
		0.25	0.50	0.0624	0.712	8.33
		0.50	0.71	0.0656	0.704	8.76
		1.00	1.00	0.0676	0.699	9.03
		1.57	1.25	0.0686	0.697	9.16
		2.00	1.41	0.0692	0.695	9.24
		2.25	1.50	0.0694	0.695	9.27
		3.07	1.75	0.0701	0.693	9.37
		4.00	2.00	0.0707	0.692	9.44
		6.25	2.50	0.0717	0.689	9.58
		8.00	2.83	0.0722	0.688	9.64
		12.25	3.50	0.0731	0.686	9.77
		15.00	3.87	0.0736	0.684	9.83
		22.00	4.69	0.0744	0.682	9.94
		30.00	5.48	0.0751	0.681	10.03
		36.00	6.00	0.0756	0.680	10.09
		49.00	7.00	0.0763	0.678	10.18
		60.00	7.75	0.0766	0.677	10.23

LOAD INCREMENT DATA

TEST NAME CTB5-14E

PAGE NO: 18

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 32.00 tsf to 16.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	11:32:14	0.00	0.00	0.0766	0.677	10.23
		0.10	0.32	0.0750	0.681	10.01
		0.25	0.50	0.0728	0.686	9.72
		0.50	0.71	0.0727	0.687	9.70
		1.00	1.00	0.0727	0.687	9.70
		1.57	1.25	0.0727	0.687	9.70
		2.00	1.41	0.0727	0.687	9.70
		2.25	1.50	0.0727	0.687	9.70
		3.07	1.75	0.0727	0.687	9.70
		4.00	2.00	0.0727	0.687	9.70
		6.25	2.50	0.0727	0.687	9.70
		8.00	2.83	0.0727	0.687	9.70
		12.25	3.50	0.0726	0.687	9.69
		15.00	3.87	0.0726	0.687	9.69
		22.00	4.69	0.0726	0.687	9.69
		30.00	5.48	0.0726	0.687	9.69

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 19

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 16.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	12:04:08	0.00	0.00	0.0724	0.687	9.67
		0.10	0.32	0.0667	0.702	8.90
		0.25	0.50	0.0664	0.702	8.87
		0.50	0.71	0.0663	0.703	8.86
		1.00	1.00	0.0662	0.703	8.84
		1.57	1.25	0.0661	0.703	8.83
		2.00	1.41	0.0661	0.703	8.83
		2.25	1.50	0.0660	0.703	8.81
		3.07	1.75	0.0660	0.703	8.81
		4.00	2.00	0.0660	0.703	8.81
		6.25	2.50	0.0660	0.703	8.81
		8.00	2.83	0.0659	0.704	8.79
		12.25	3.50	0.0659	0.704	8.79
		15.00	3.87	0.0659	0.704	8.79
		22.00	4.69	0.0659	0.704	8.79
		30.00	5.48	0.0657	0.704	8.78
		36.00	6.00	0.0657	0.704	8.78

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 20

TEST NO: 3 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	12:52:18	0.00	0.00	0.0657	0.704	8.78
		0.10	0.32	0.0616	0.714	8.22
		0.25	0.50	0.0611	0.716	8.16
		0.50	0.71	0.0608	0.716	8.12
		1.00	1.00	0.0605	0.717	8.08
		1.57	1.25	0.0603	0.718	8.05
		2.00	1.41	0.0602	0.718	8.04
		2.25	1.50	0.0602	0.718	8.04
		3.07	1.75	0.0601	0.718	8.02
		4.00	2.00	0.0600	0.718	8.01
		6.25	2.50	0.0598	0.719	7.99
		8.00	2.83	0.0598	0.719	7.99
		12.25	3.50	0.0597	0.719	7.98
		15.00	3.87	0.0596	0.719	7.96
		22.00	4.69	0.0595	0.720	7.95
		30.00	5.48	0.0595	0.720	7.95
		36.00	6.00	0.0594	0.720	7.93
		49.00	7.00	0.0594	0.720	7.93
		60.00	7.75	0.0593	0.720	7.92

LOAD INCREMENT DATA

TEST NAME CTB5-14E PAGE NO: 21

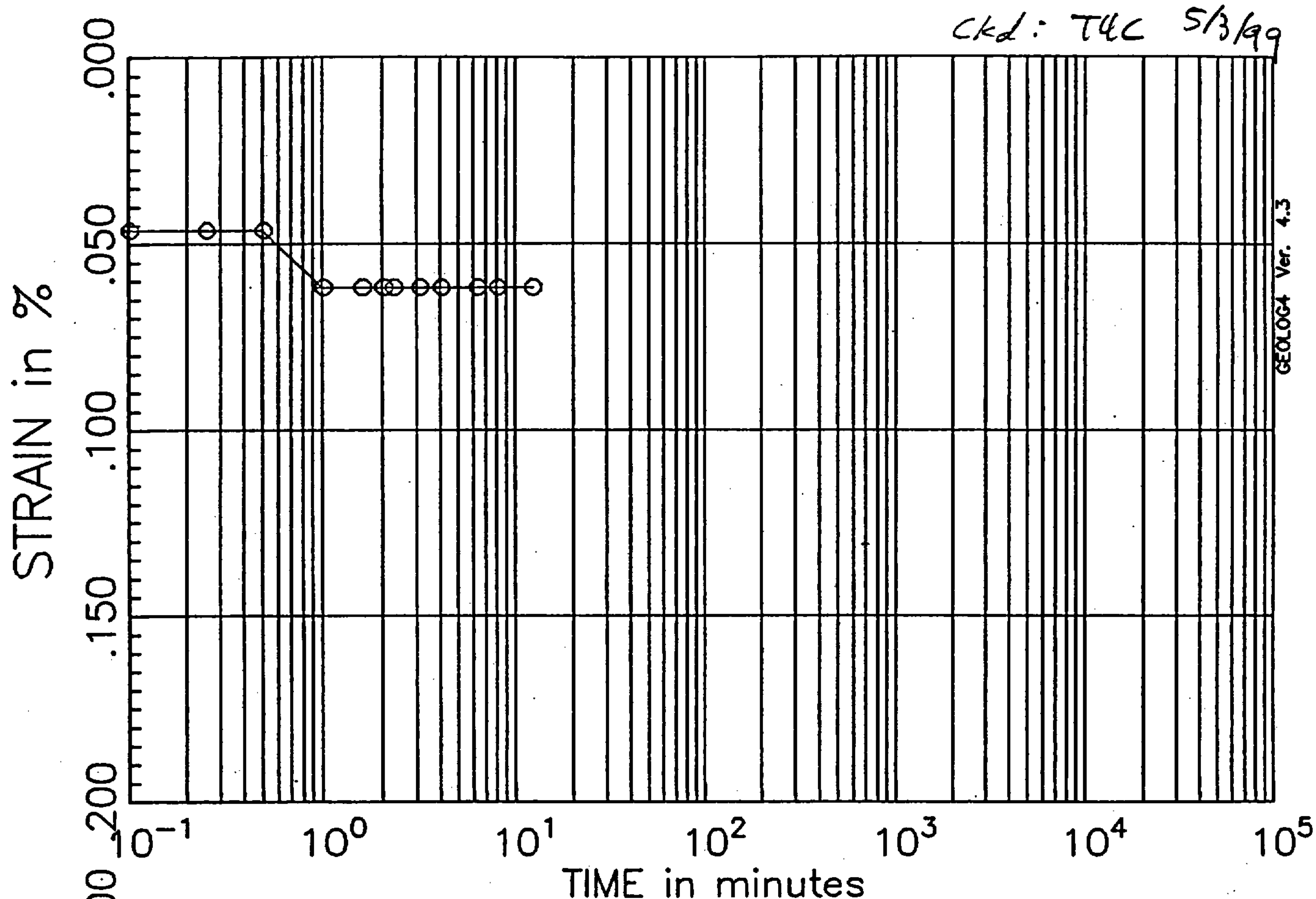
TEST NO: 3 TESTED BY: ACS

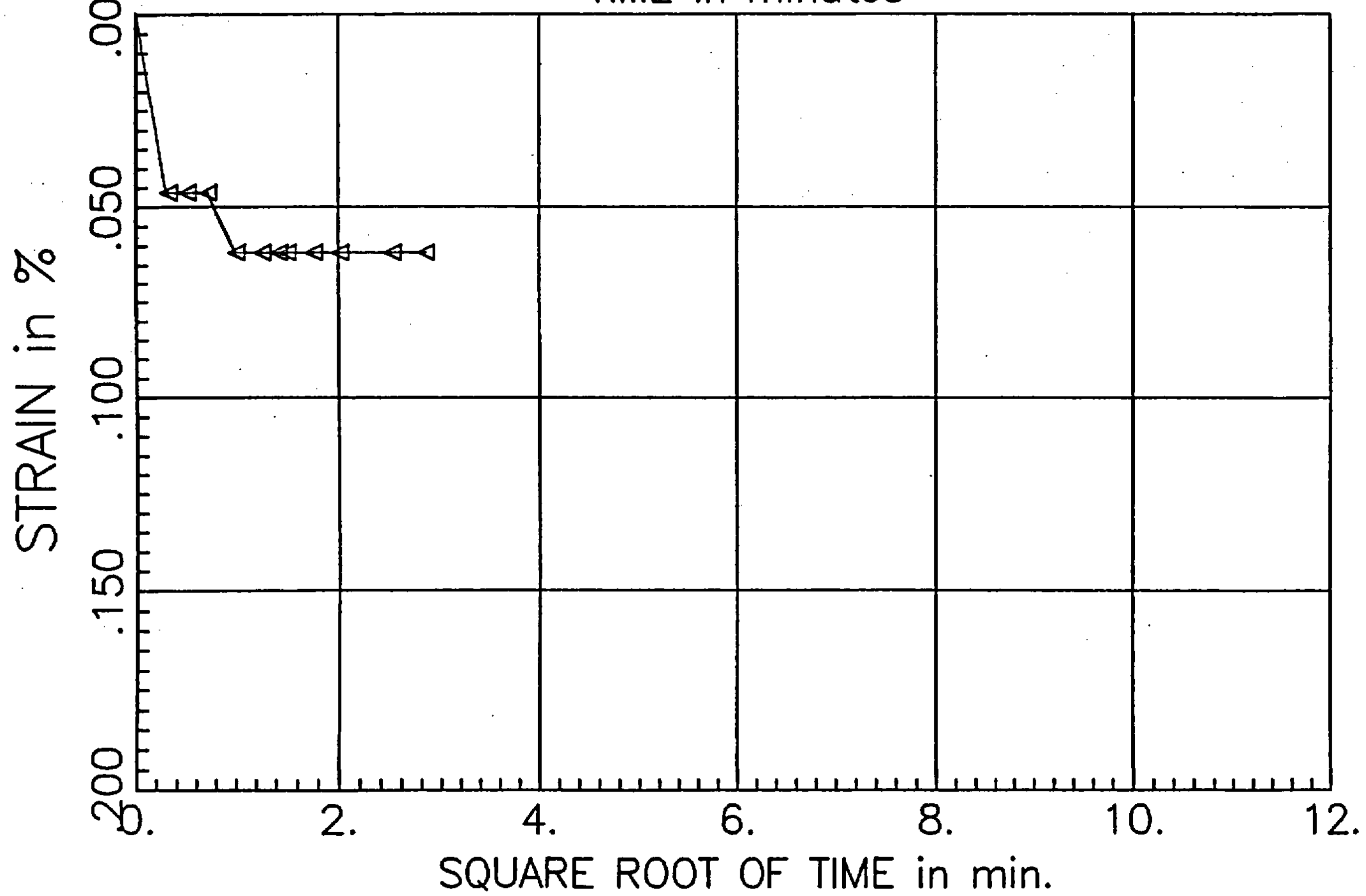
PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-16-99	14:04:16	0.00	0.00	0.0593	0.720	7.92
		0.10	0.32	0.0594	0.720	7.93
		0.25	0.50	0.0594	0.720	7.93
		0.50	0.71	0.0593	0.720	7.92
		1.00	1.00	0.0563	0.728	7.51
		1.57	1.25	0.0558	0.729	7.45
		2.00	1.41	0.0556	0.729	7.42
		2.25	1.50	0.0555	0.730	7.41
		3.07	1.75	0.0552	0.730	7.37
		4.00	2.00	0.0550	0.731	7.34
		6.25	2.50	0.0548	0.731	7.31
		8.00	2.83	0.0545	0.732	7.28
		12.25	3.50	0.0543	0.733	7.25
		15.00	3.87	0.0542	0.733	7.24
		22.00	4.69	0.0540	0.733	7.21
		30.00	5.48	0.0538	0.734	7.19
		36.00	6.00	0.0537	0.734	7.17
		49.00	7.00	0.0536	0.734	7.16
		60.00	7.75	0.0535	0.735	7.14
		87.00	9.33	0.0533	0.735	7.11

$d_{100} = 0.06\%$

Jo 05996.02
Calc: ACS 4/23/99
Ckd: T4C 5/3/99



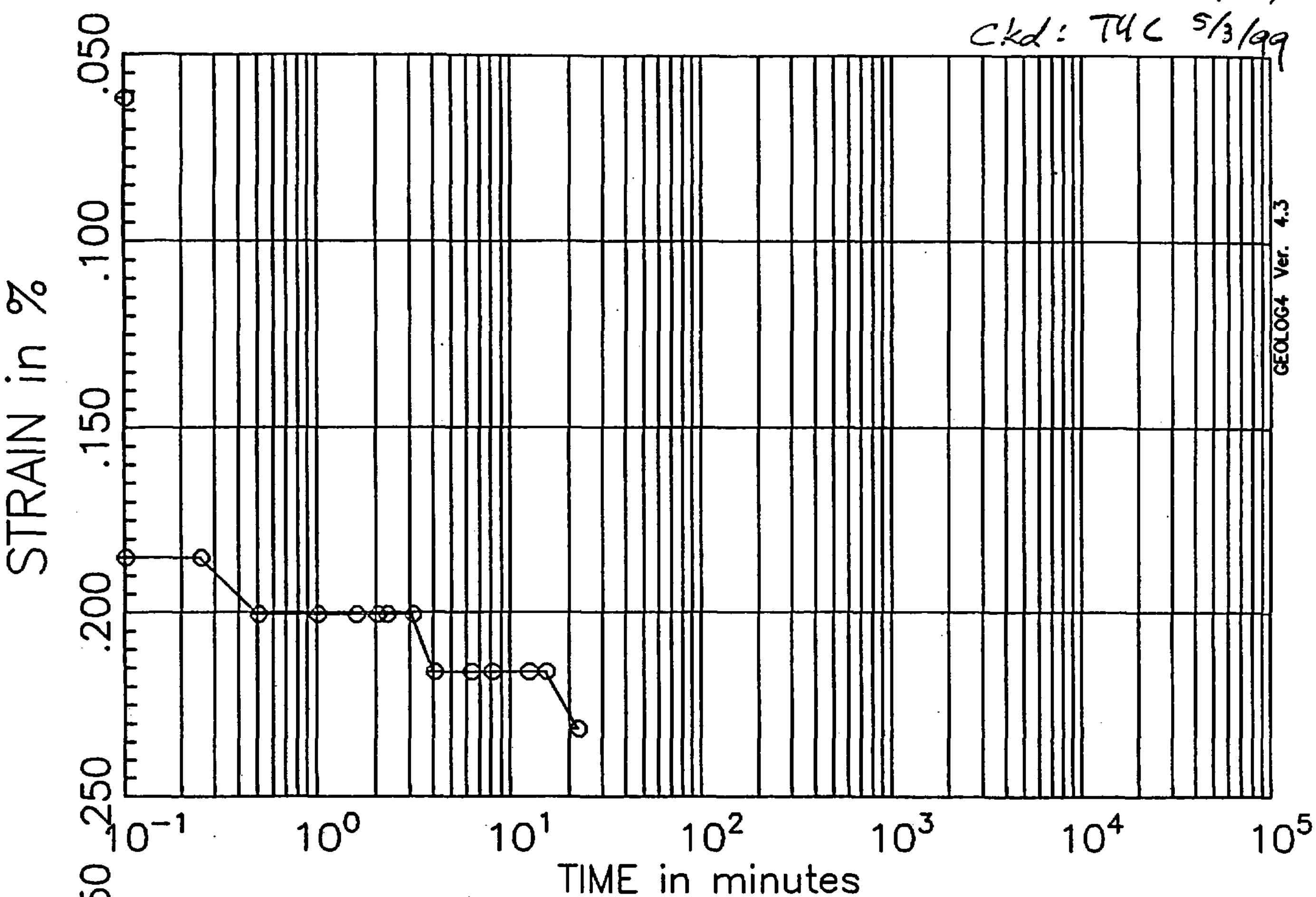


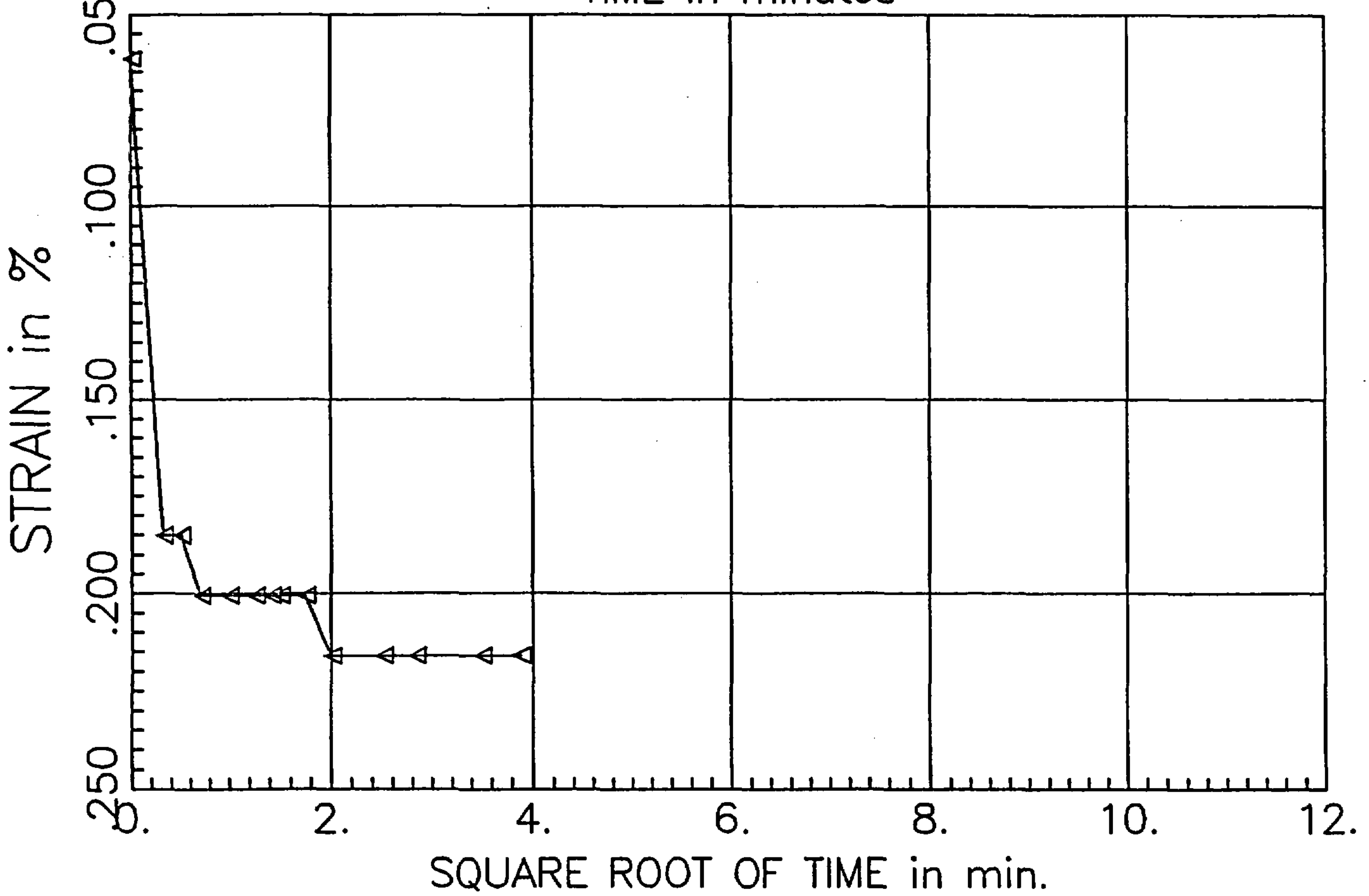
PRESSURE INCREMENT #1
from 0.00 tsf to 0.10 tsf

Test No: 3
Testname: CTB5-14E

JO 05776.02
Calc: ACS 4/23/99
Ckd: THC 5/3/99

$d_{100} = 0.20\%$



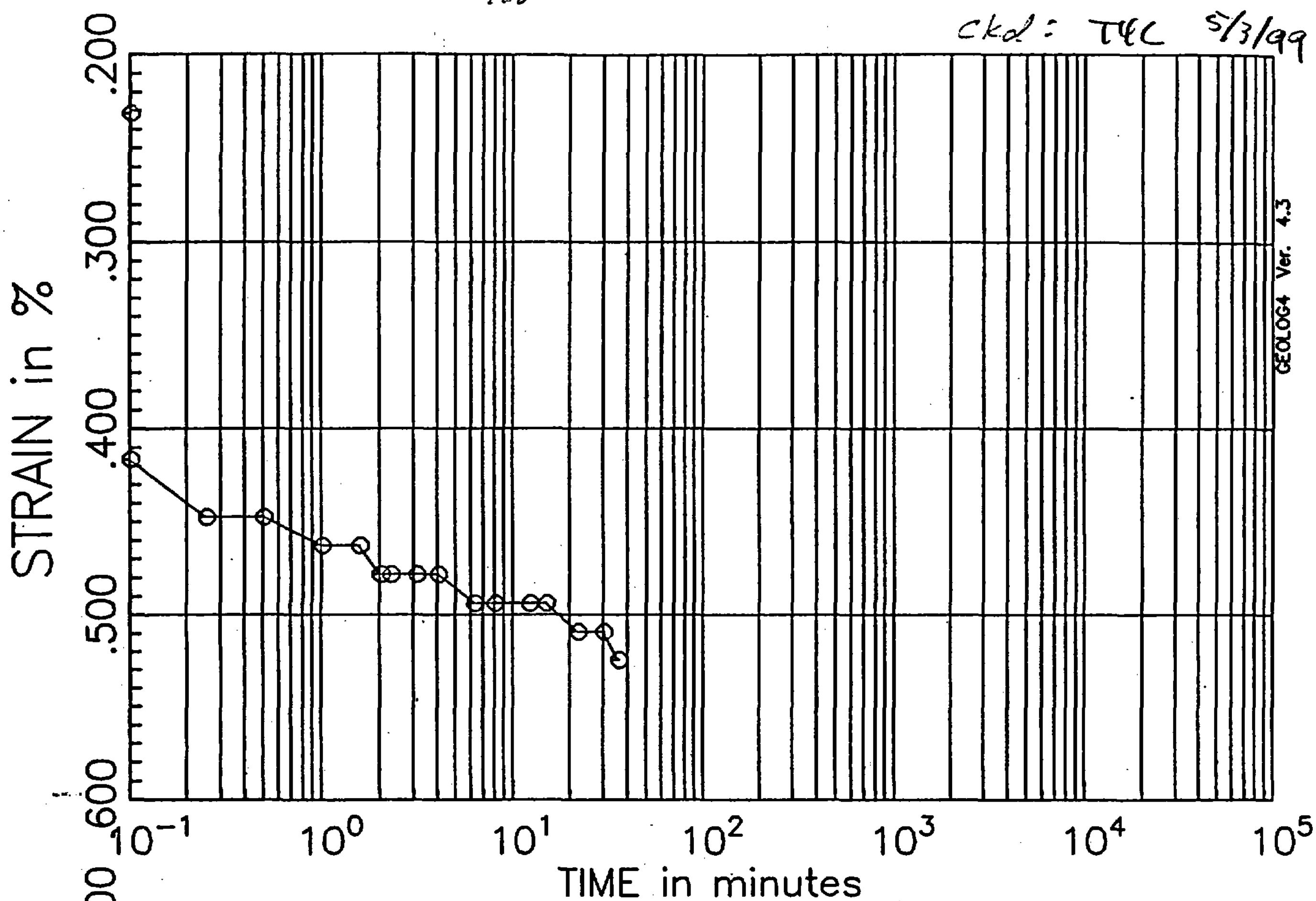


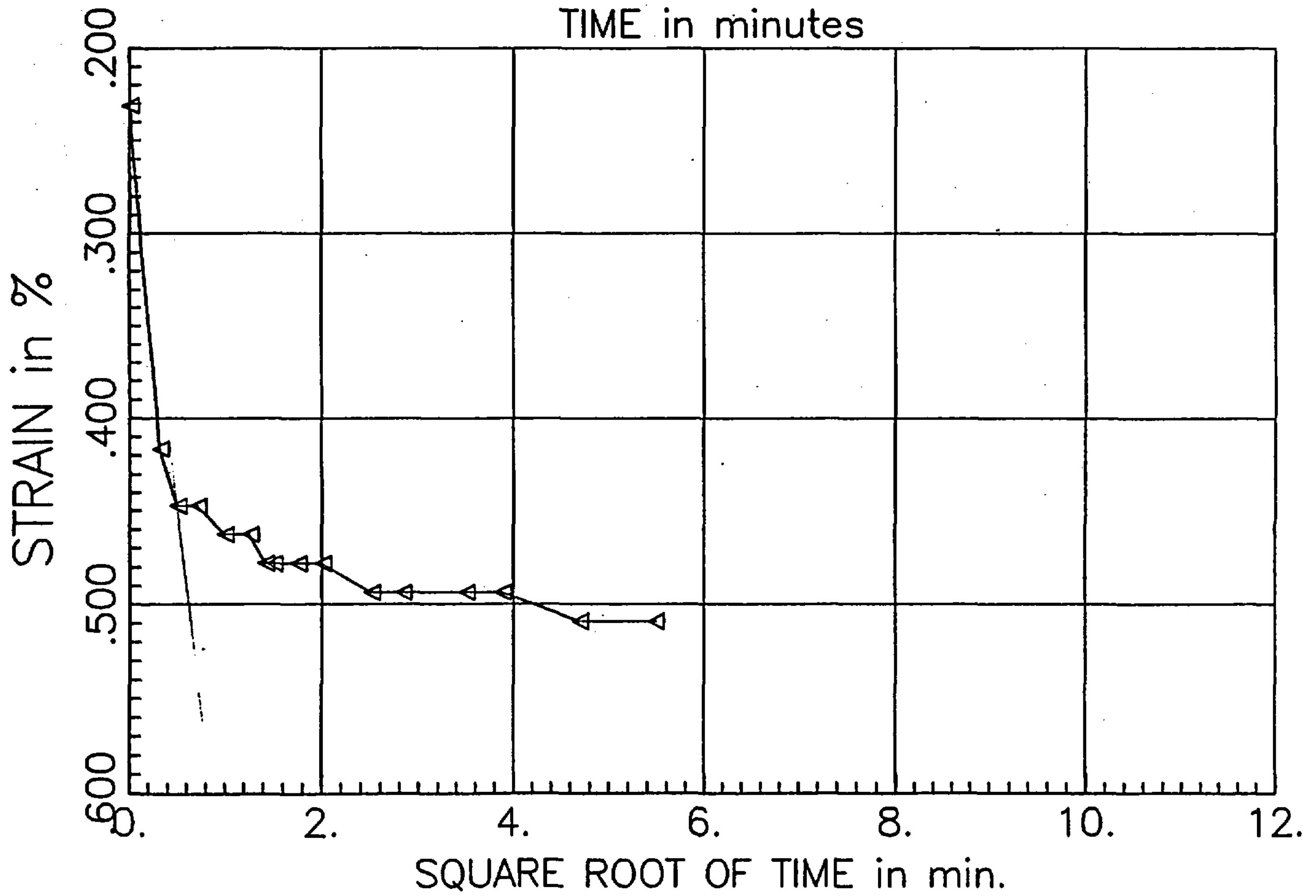
PRESSURE INCREMENT #2
from 0.10 tsf to 0.25 tsf

Test No: 3
Testname: CTB5-14E

J0 .05996.02
Calc: ACS 4/25/99
Ckd: TLC 5/3/99

$d_{100} = 0.46\%$





PRESSURE INCREMENT #3
from 0.25 tsf to 0.50 tsf

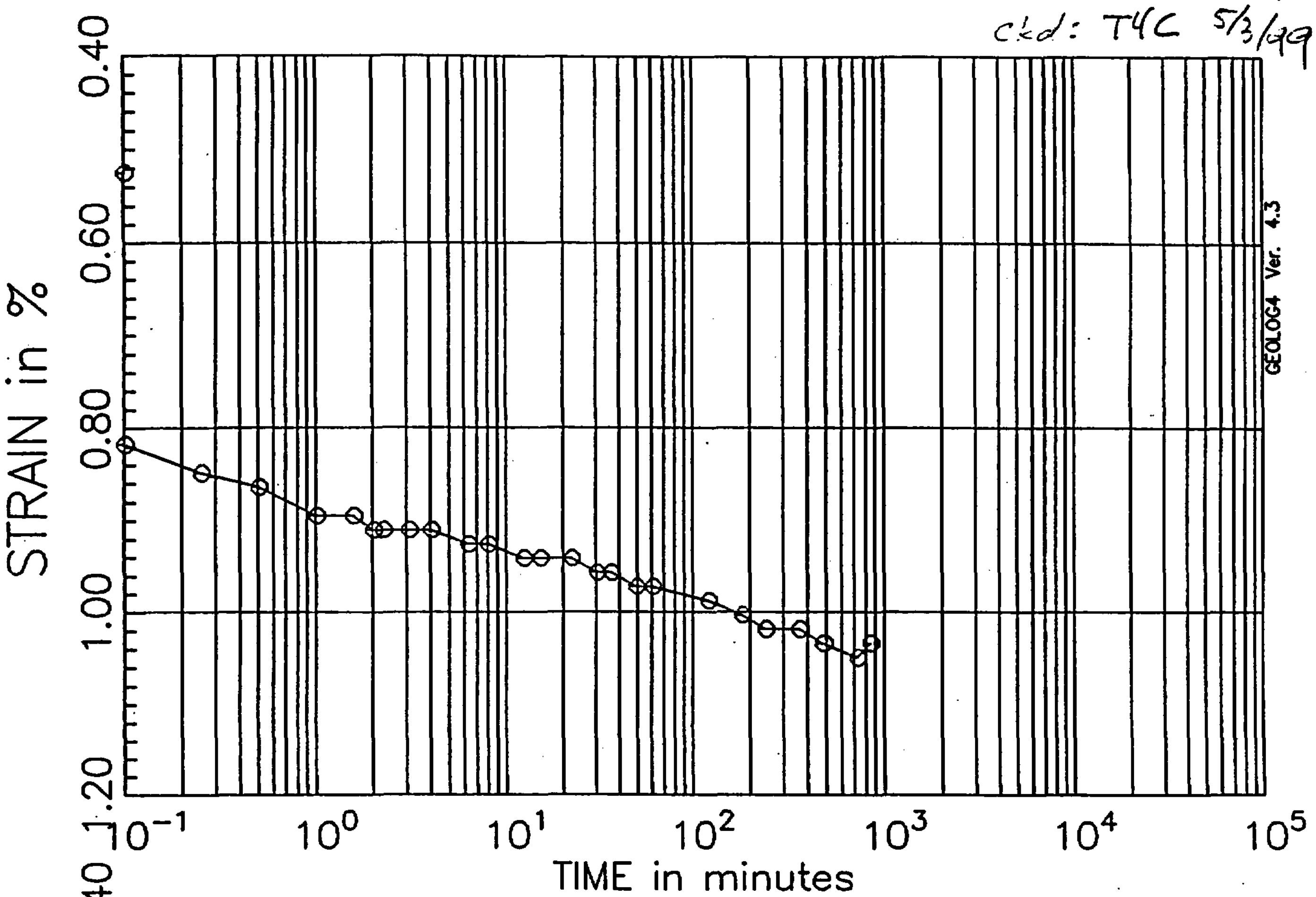
Test No: 3
Testname: CTB5-14E

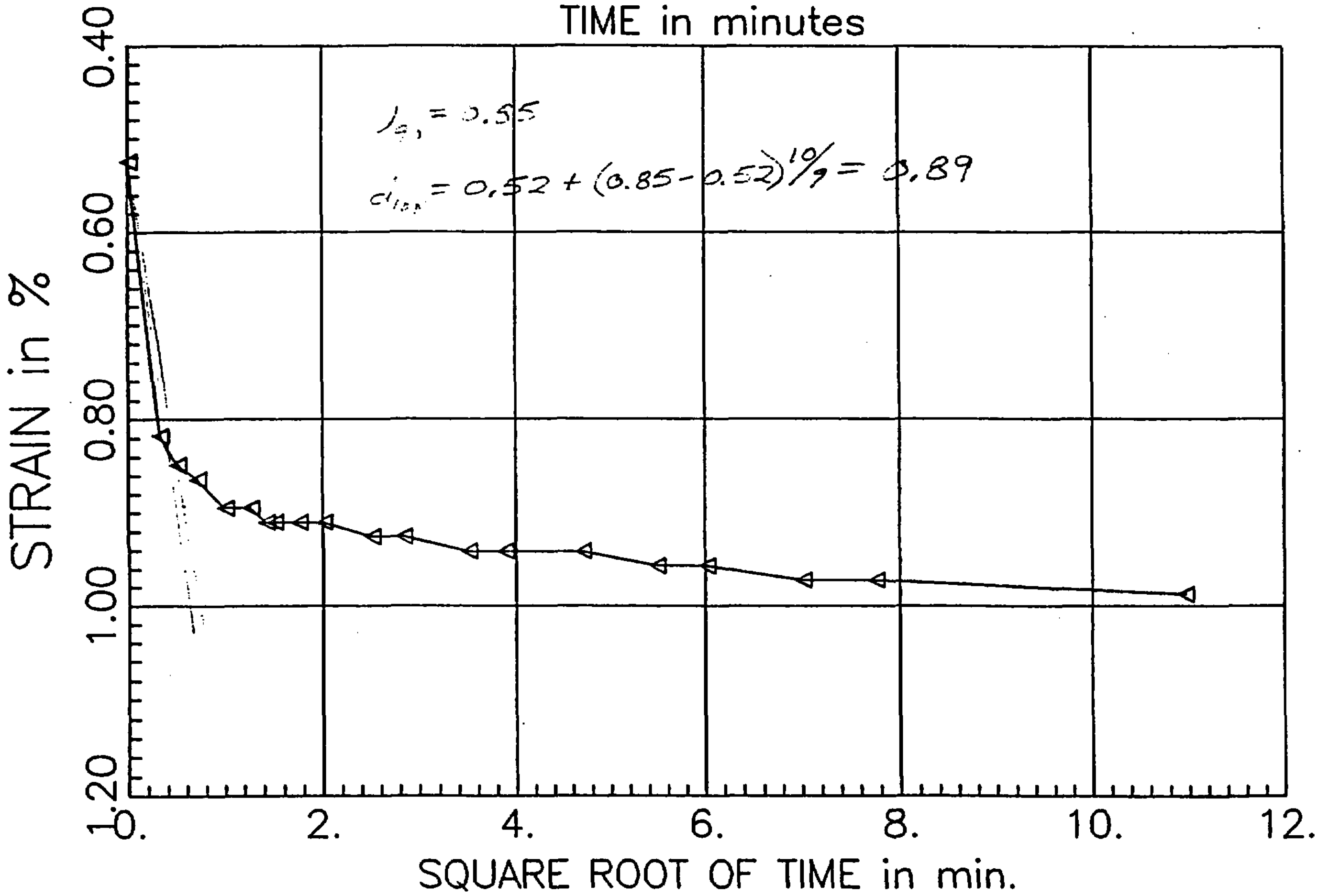
Jo 05996.02

Calc: AES 4/23/99

Ckd: T4C 5/3/99

$d_{100} = 0.89\%$



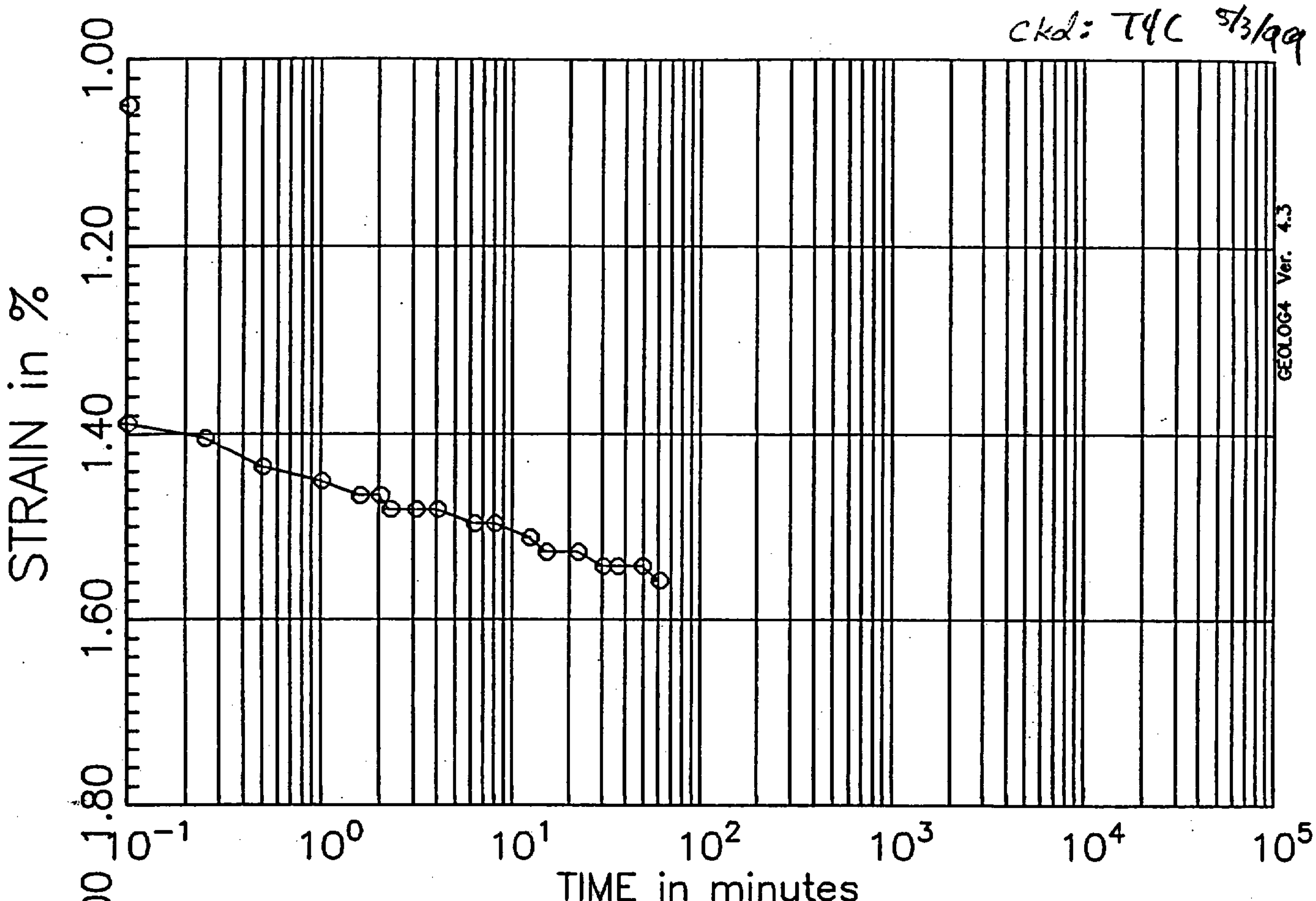


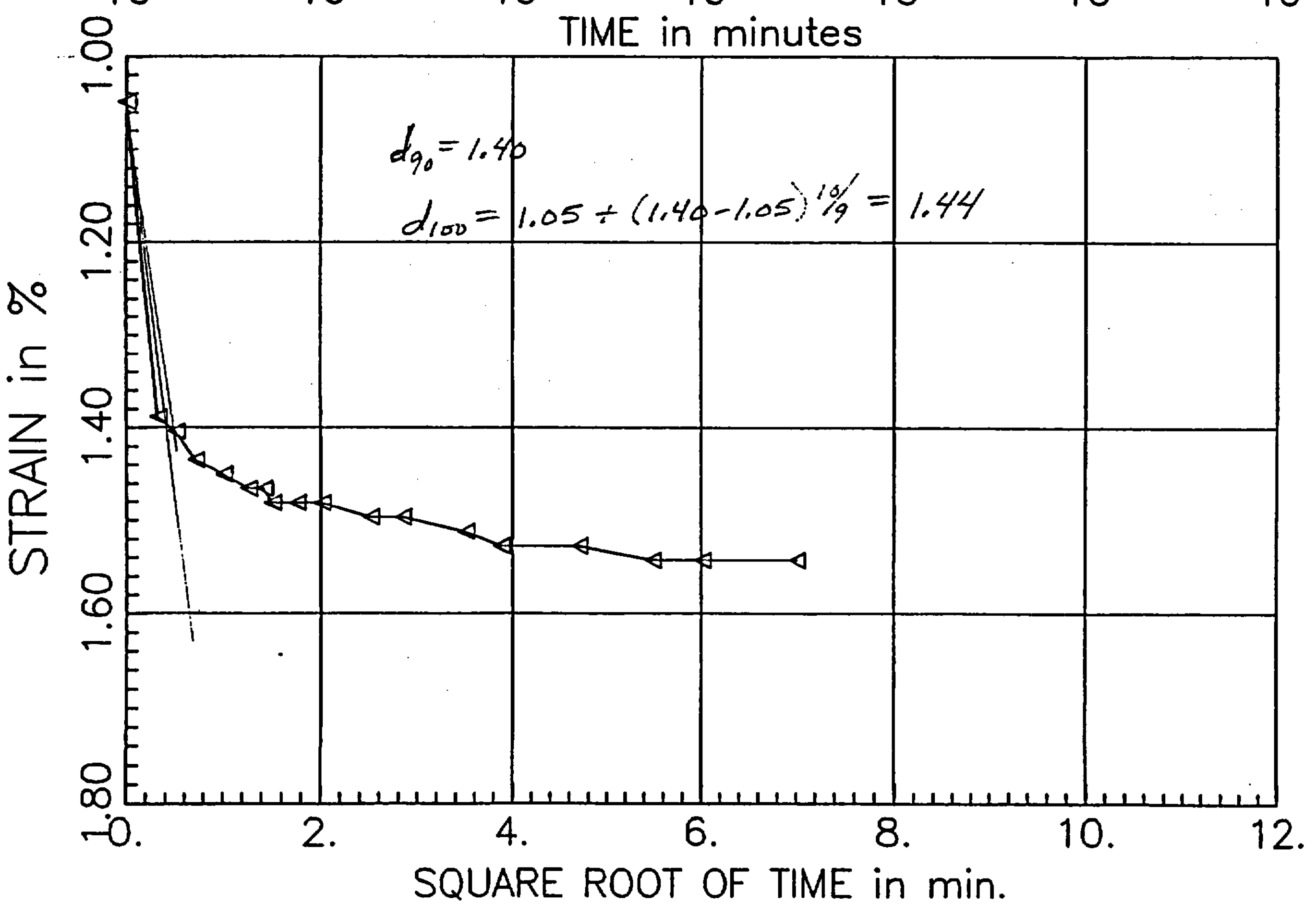
PRESSURE INCREMENT #4
from 0.50 tsf to 1.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02
Calc: ACE 4/23/99
Ckd: TYC 5/3/99

$d_{100} = 1.44\%$





PRESSURE INCREMENT #5
from 1.00 tsf to 2.00 tsf

Test No: 3
Testname: CTB5-14E

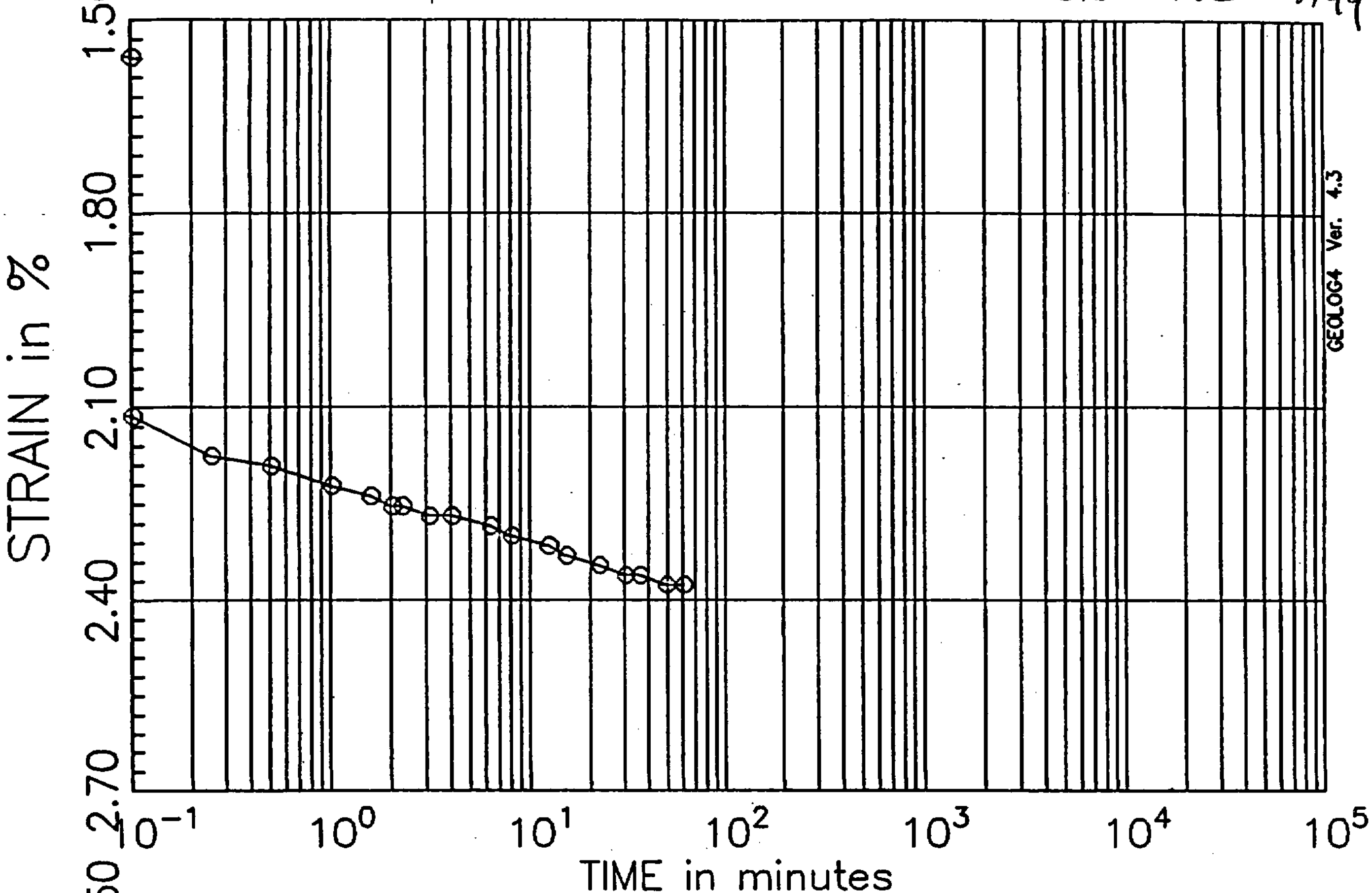
$d_{100} = 2.25\%$

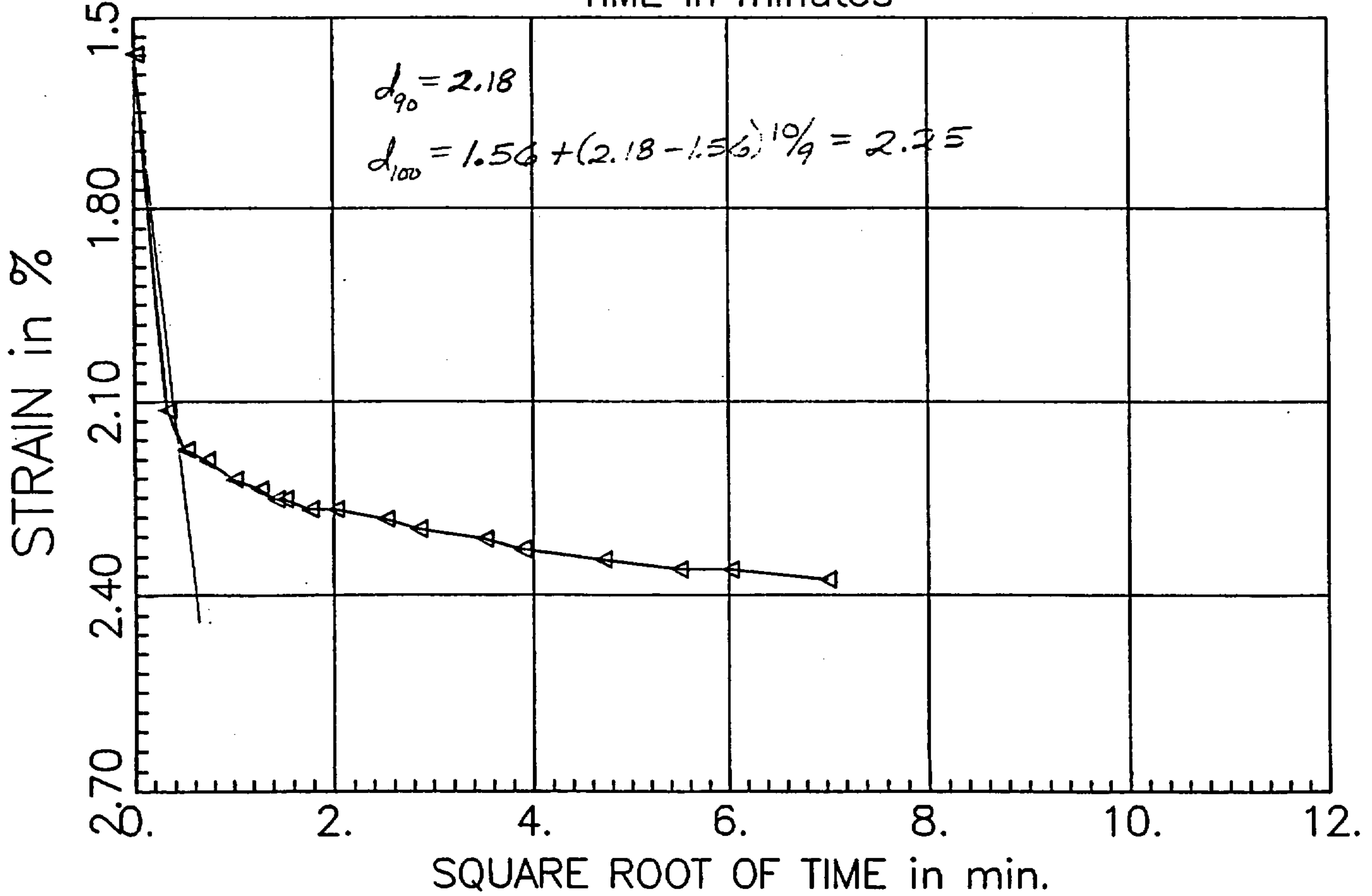
$d_f = 2.39\%$

JO 00110.02

Calc: ACS 4/23/99

Ckd: TVC 5/3/99



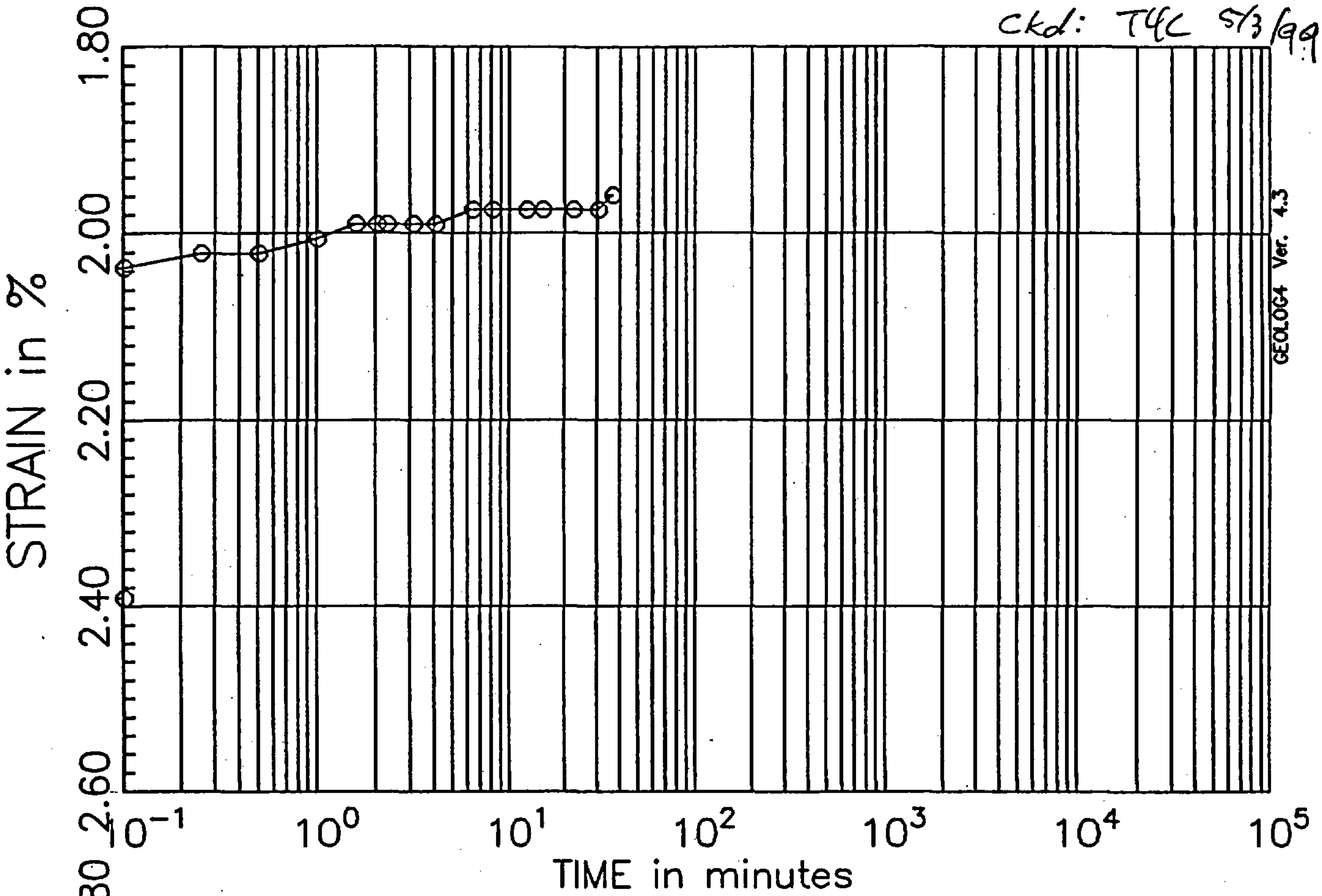


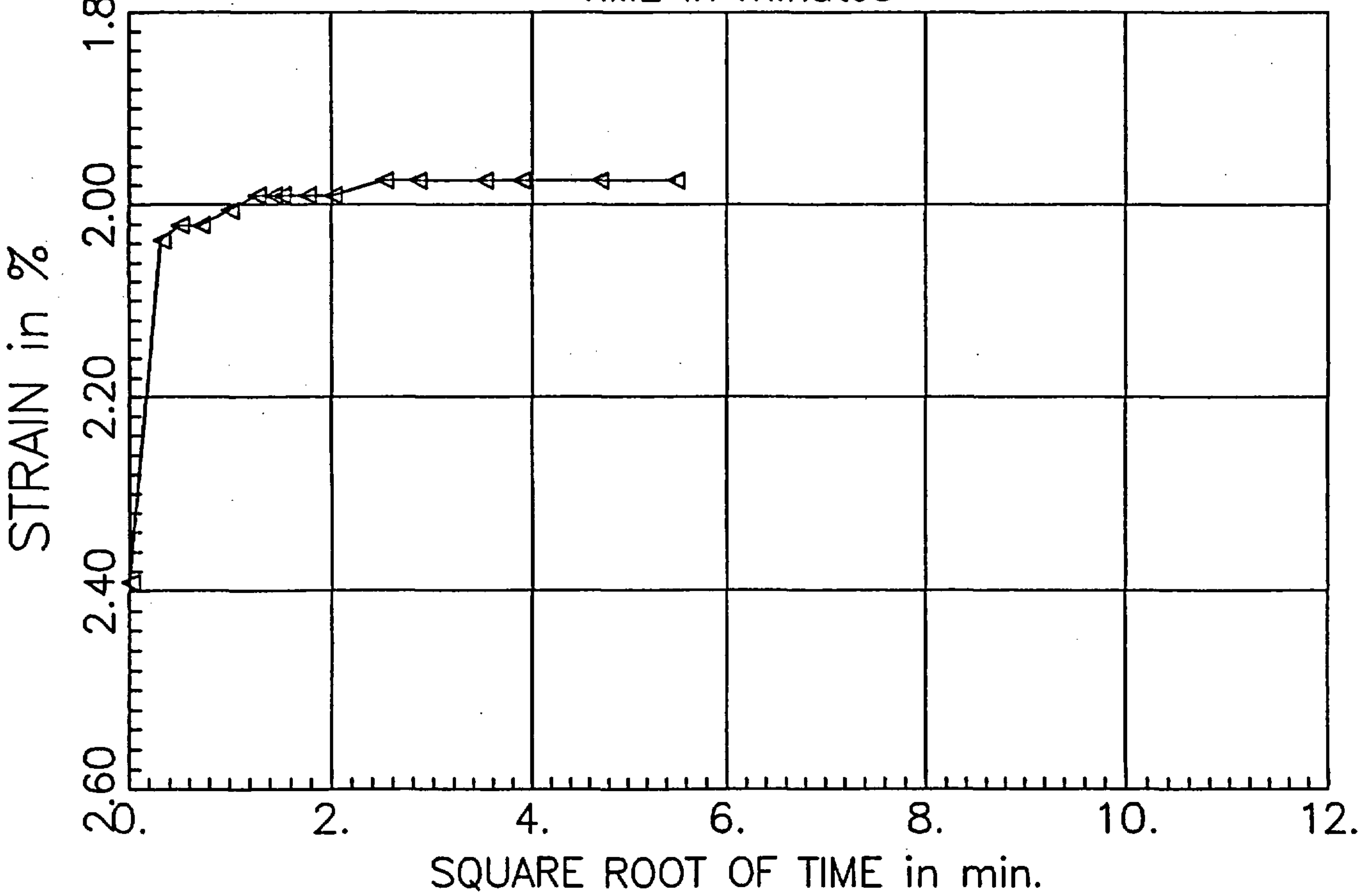
PRESSURE INCREMENT #6
from 2.00 tsf to 4.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02
Calc: ACS 4/23/99
Ckd: T4C 5/3/99

$d_{100} = 2.0\ \%$





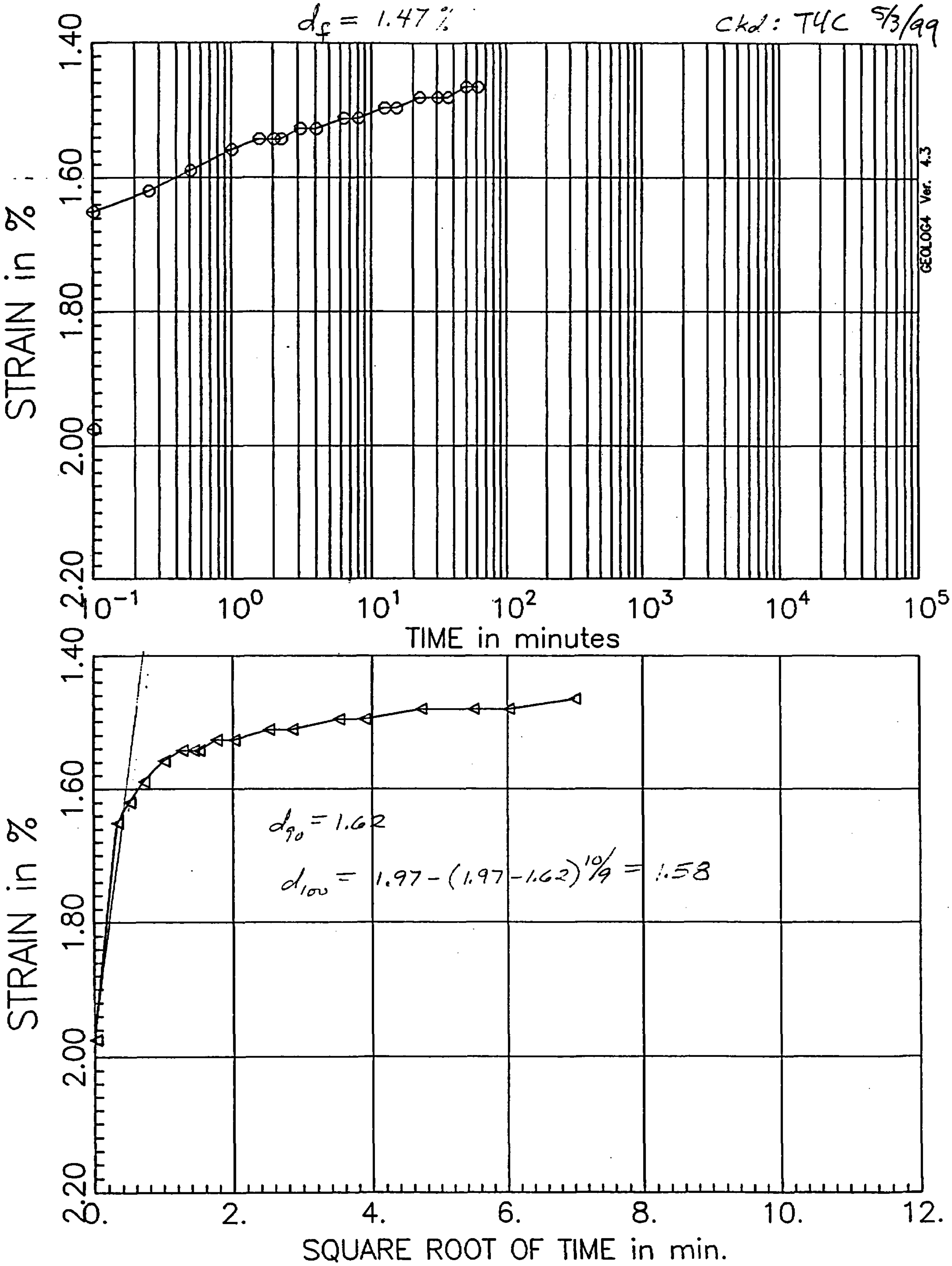
PRESSURE INCREMENT # 7
from 4.00 tsf to 1.00 tsf

Test No: 3
Testname: CTB5-14E

$d_{100} = 1.58\%$

$d_f = 1.47\%$

Calc: ACS 4/23/99
Ckd: THC 5/3/99

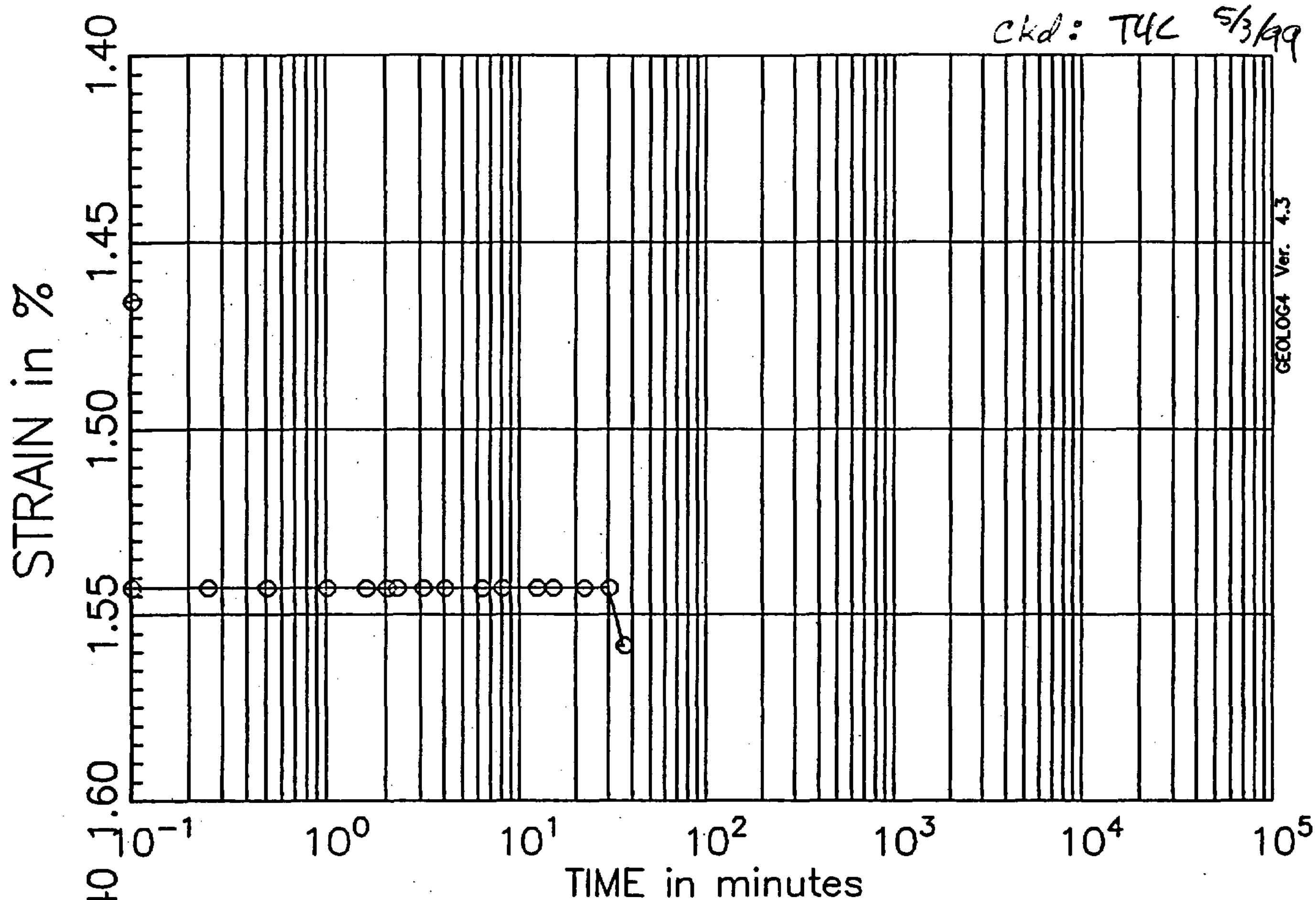


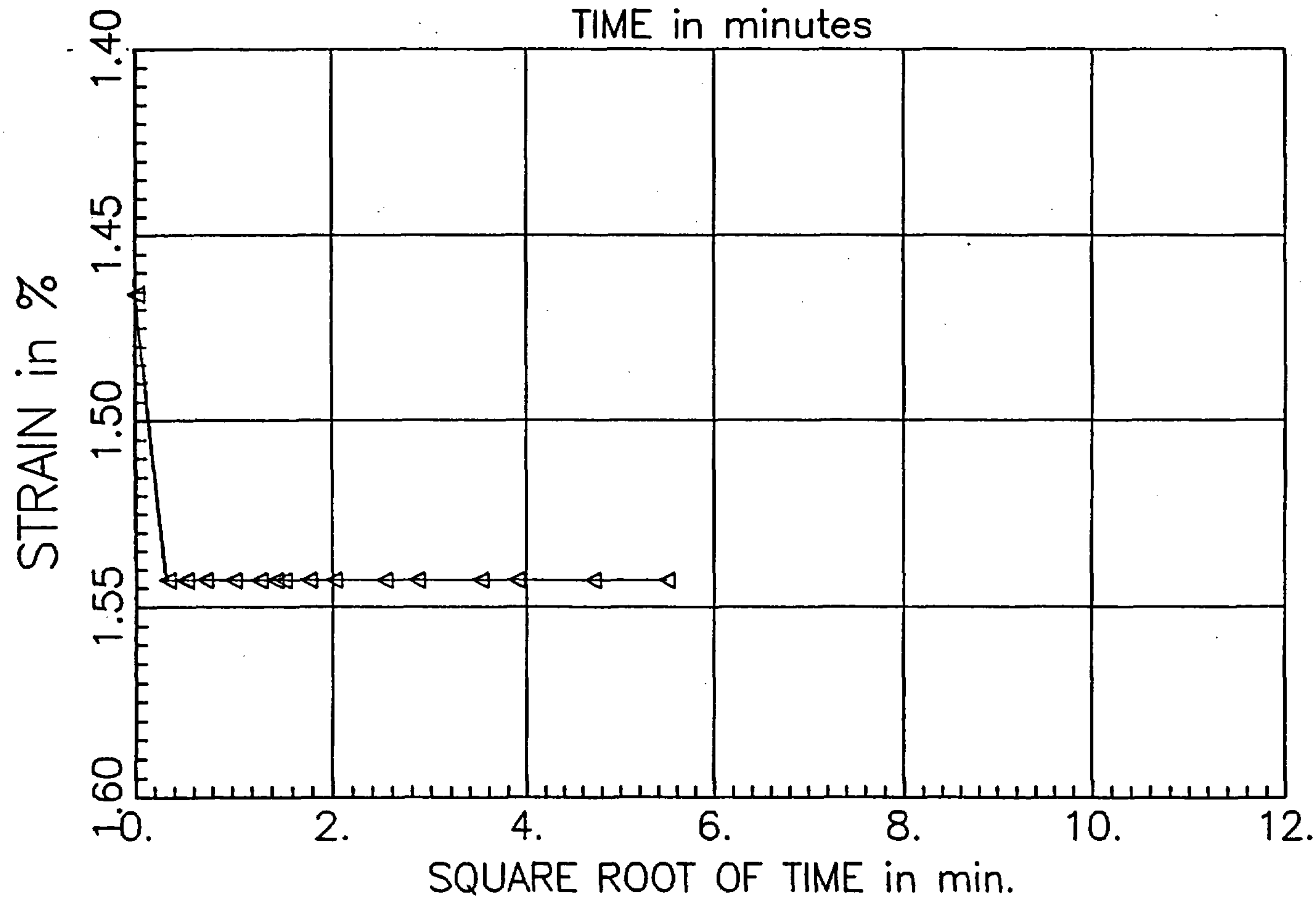
PRESSURE INCREMENT #8
from 1.00 tsf to 0.25 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02
Calc: RSS 4/23/99
Ckd: TLC 5/3/99

$d_{100} = 1.54\%$





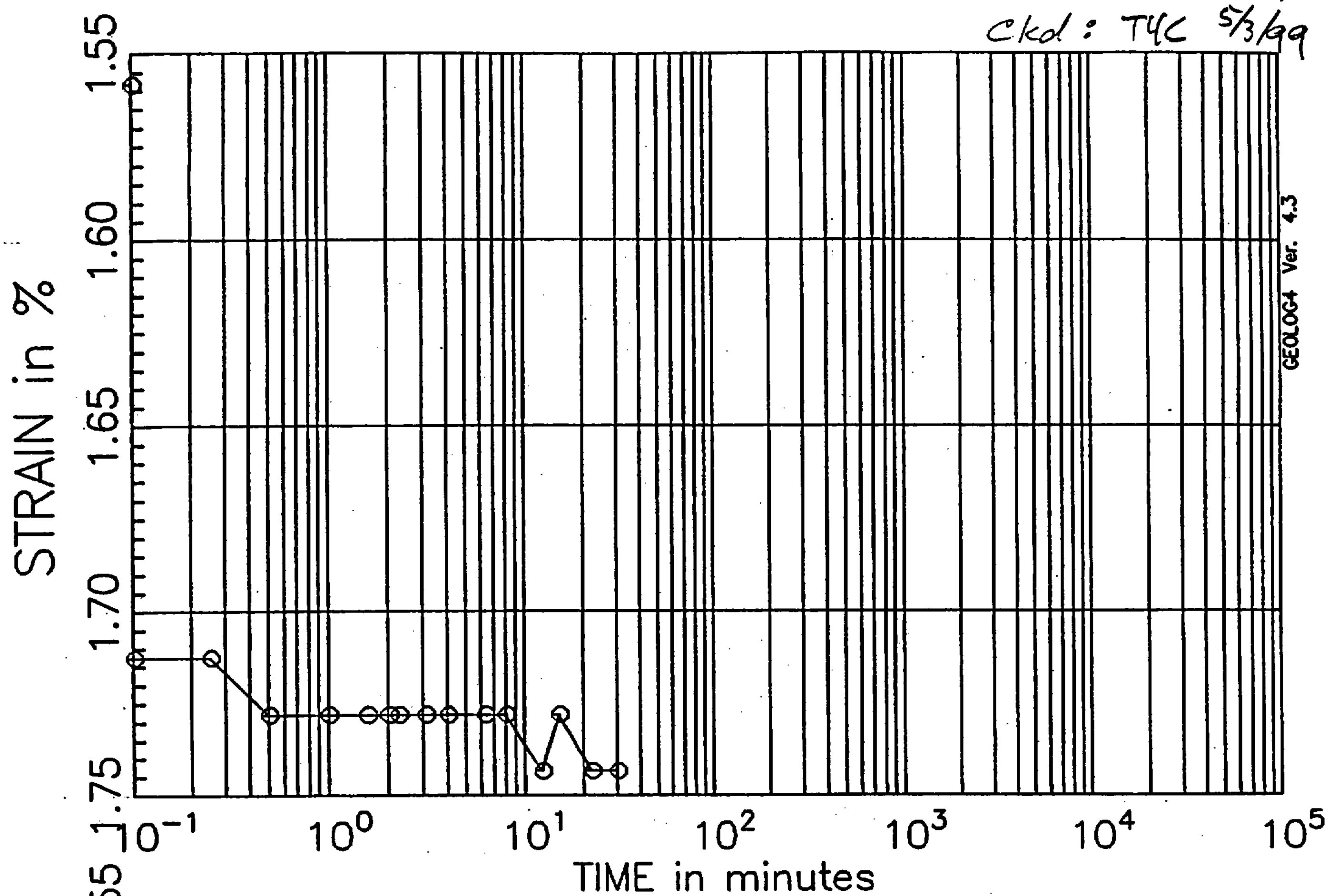
PRESSURE INCREMENT # 9
from 0.25 tsf to 0.50 tsf

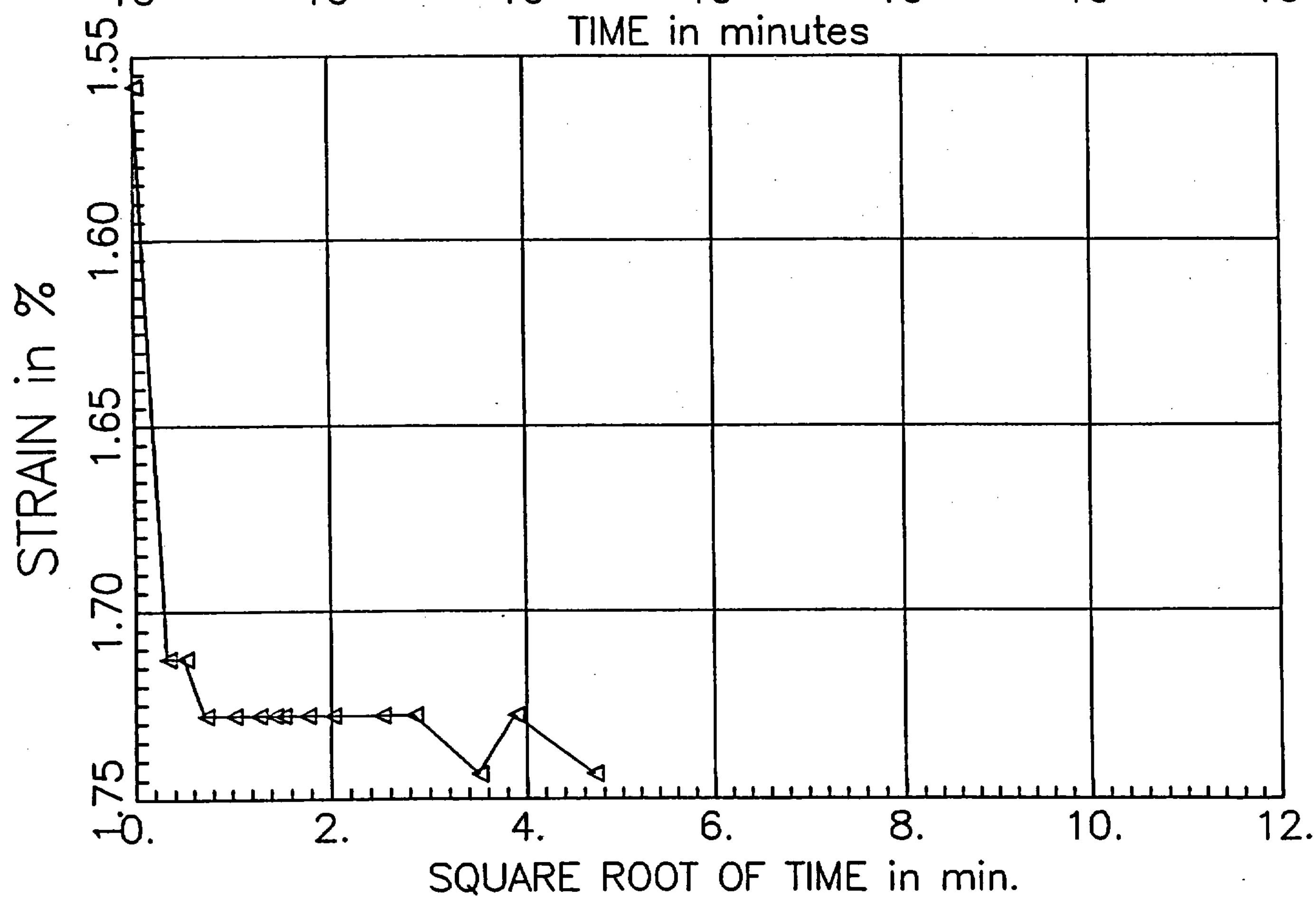
Test No: 3
Testname: CTB5-14E

JO 05996.02.

$d_{100} = 1.73\%$

Calc: ACS 4/23/99
Ckd: TYC 5/3/99



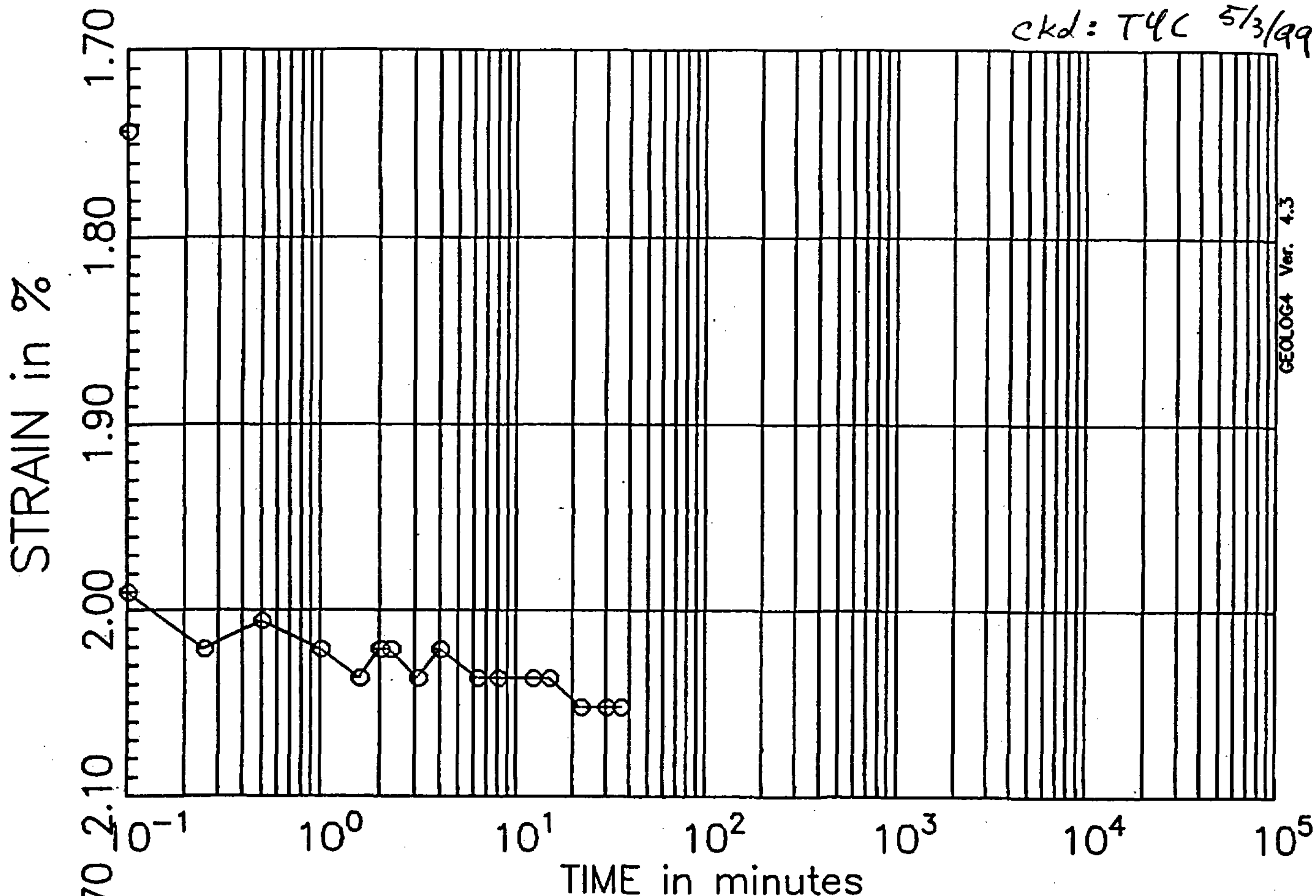


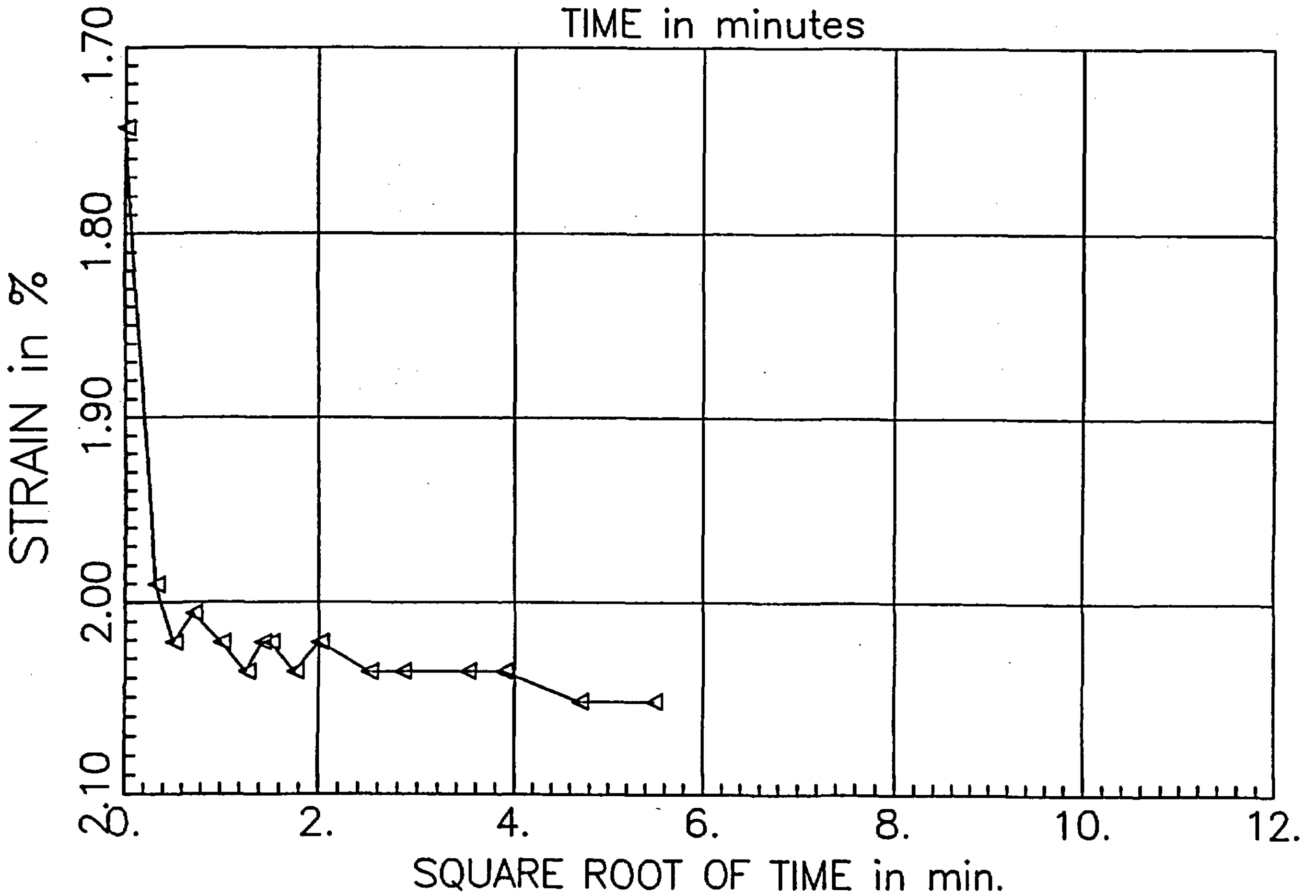
PRESSURE INCREMENT #10
from 0.50 tsf to 1.00 tsf

Test No: 3
Testname: CTB5-14E

Jo 05776.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

$d_{100} = 2.02\%$



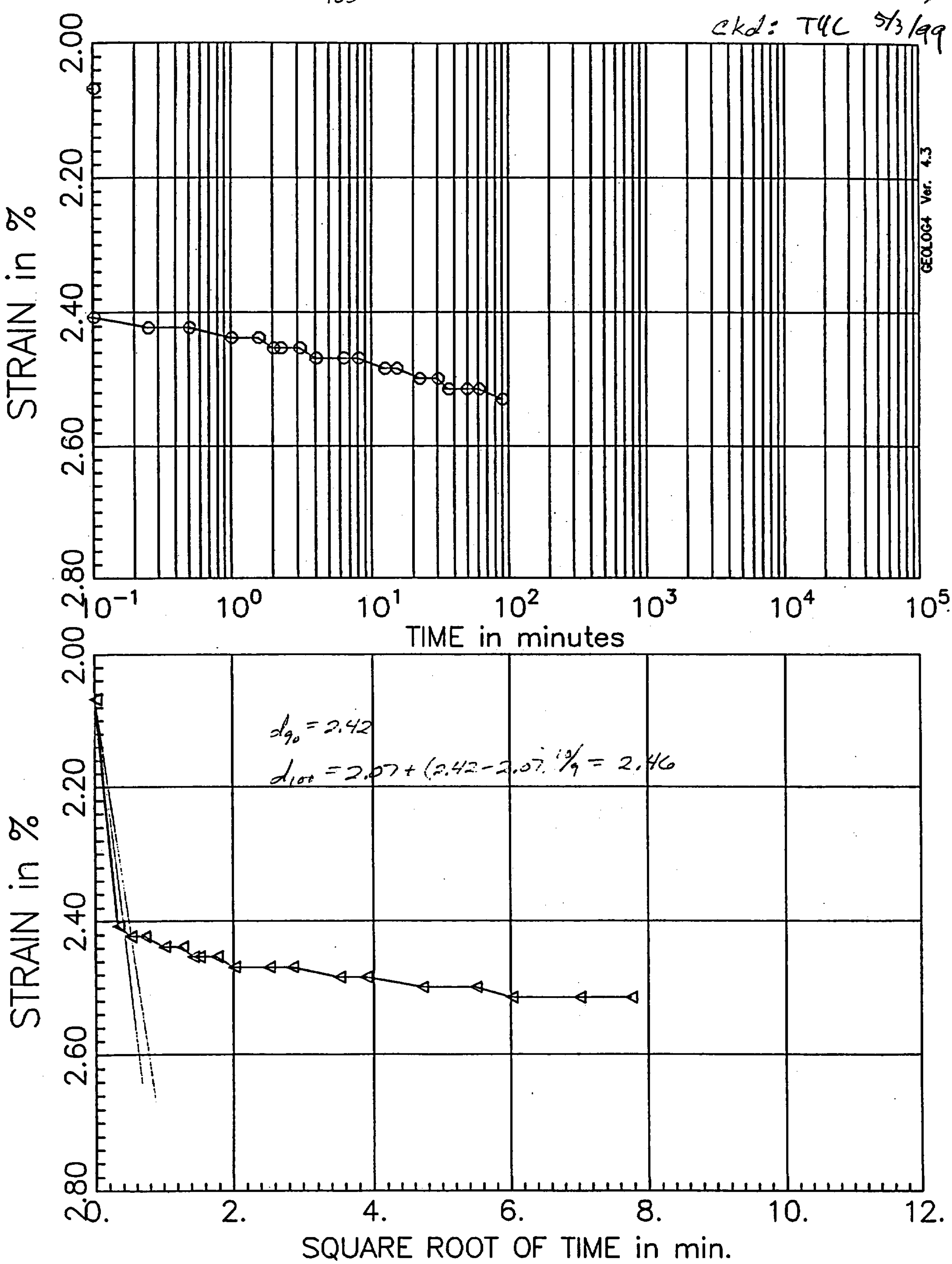


PRESSURE INCREMENT #11
from 1.00 tsf to 2.00 tsf

Test No: 3
Testname: CTB5-14E

Jo 05996.02
Calc: ACS 4/23/99
Ckd: TWC 5/3/99

$d_{100} = 2.46\%$

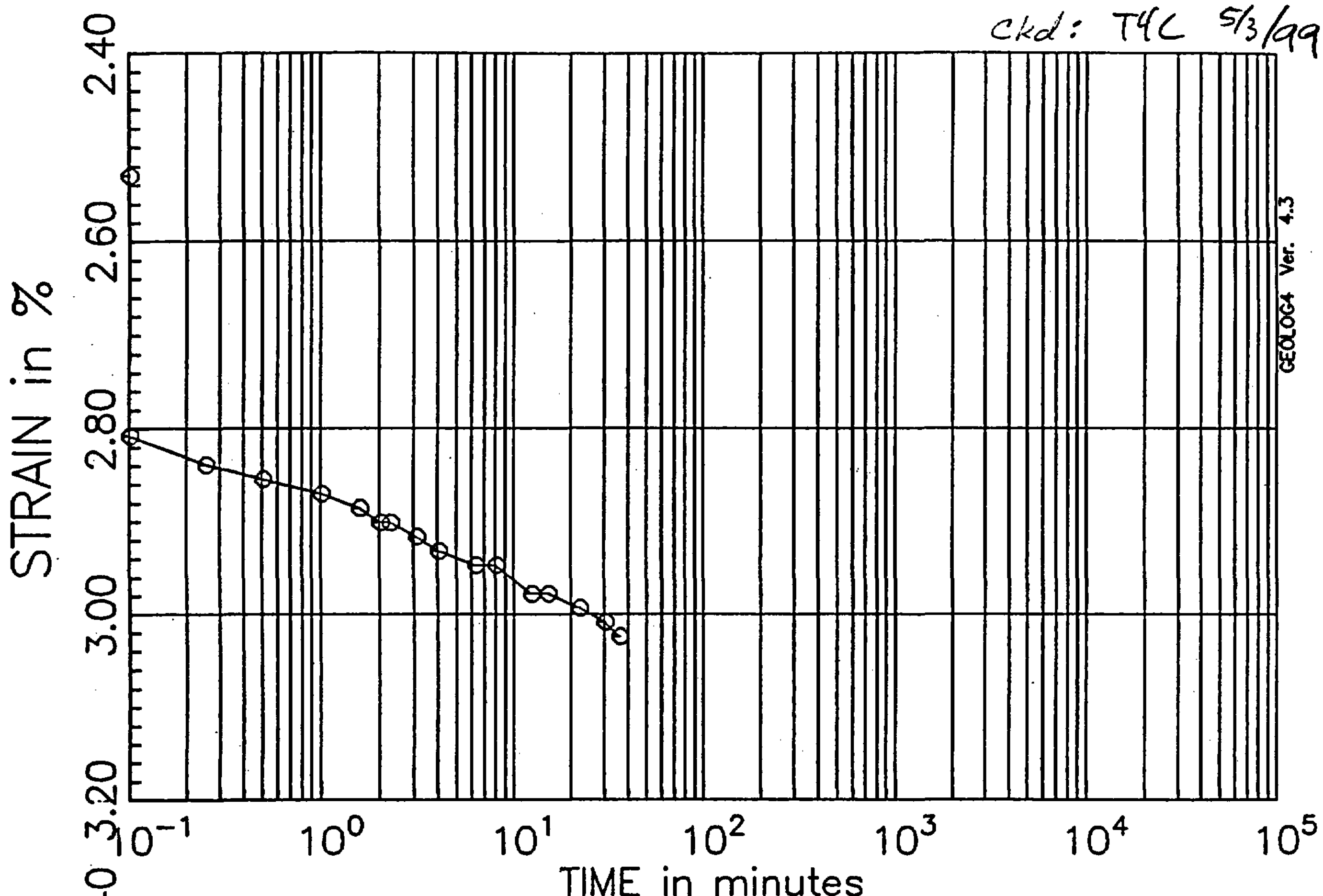


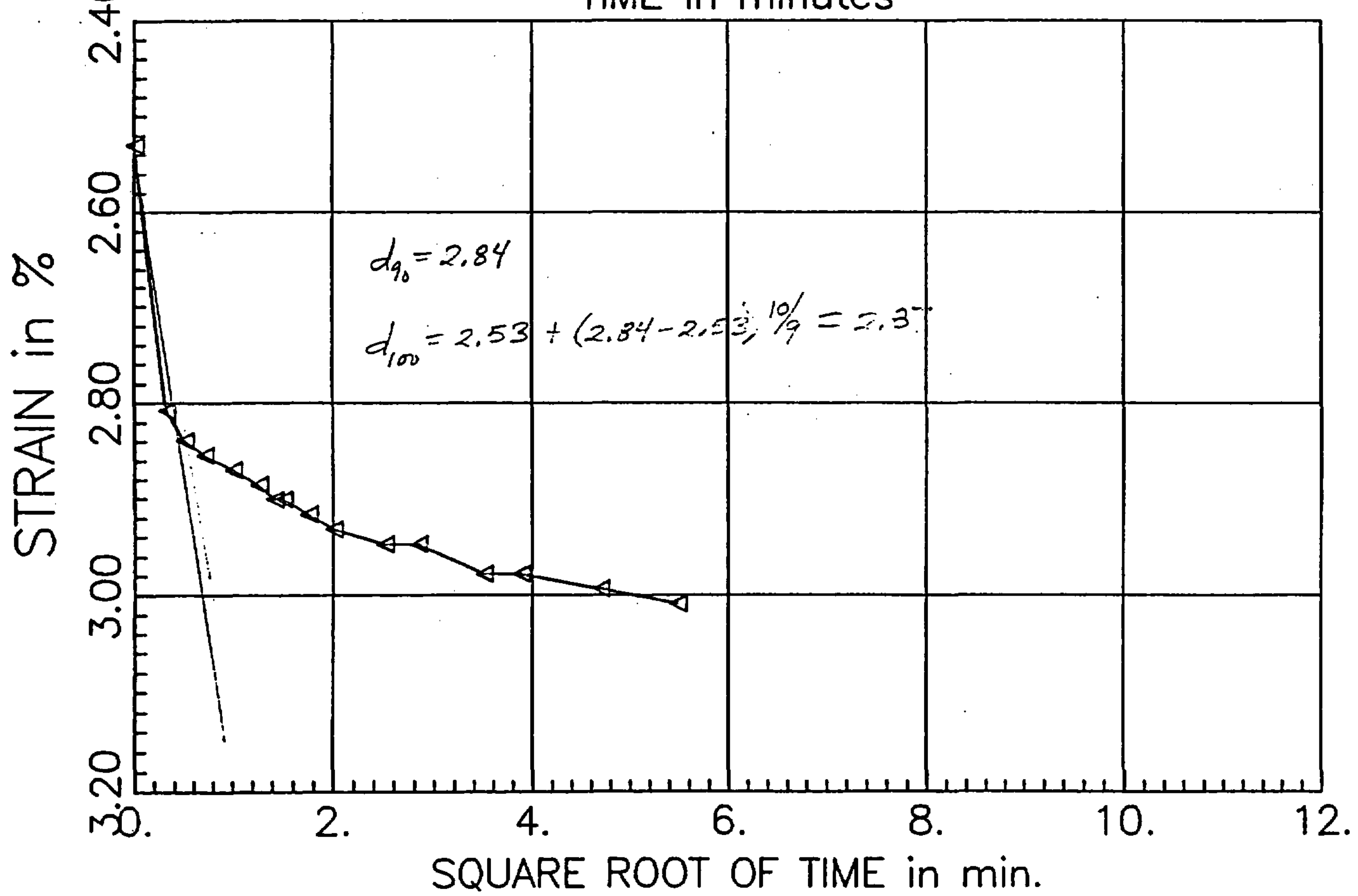
PRESSURE INCREMENT #12
from 2.00 tsf to 4.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02 .
Calc : ACS 4/23/99
Ckd: T4C 5/3/99

$d_{100} = 2.87\%$

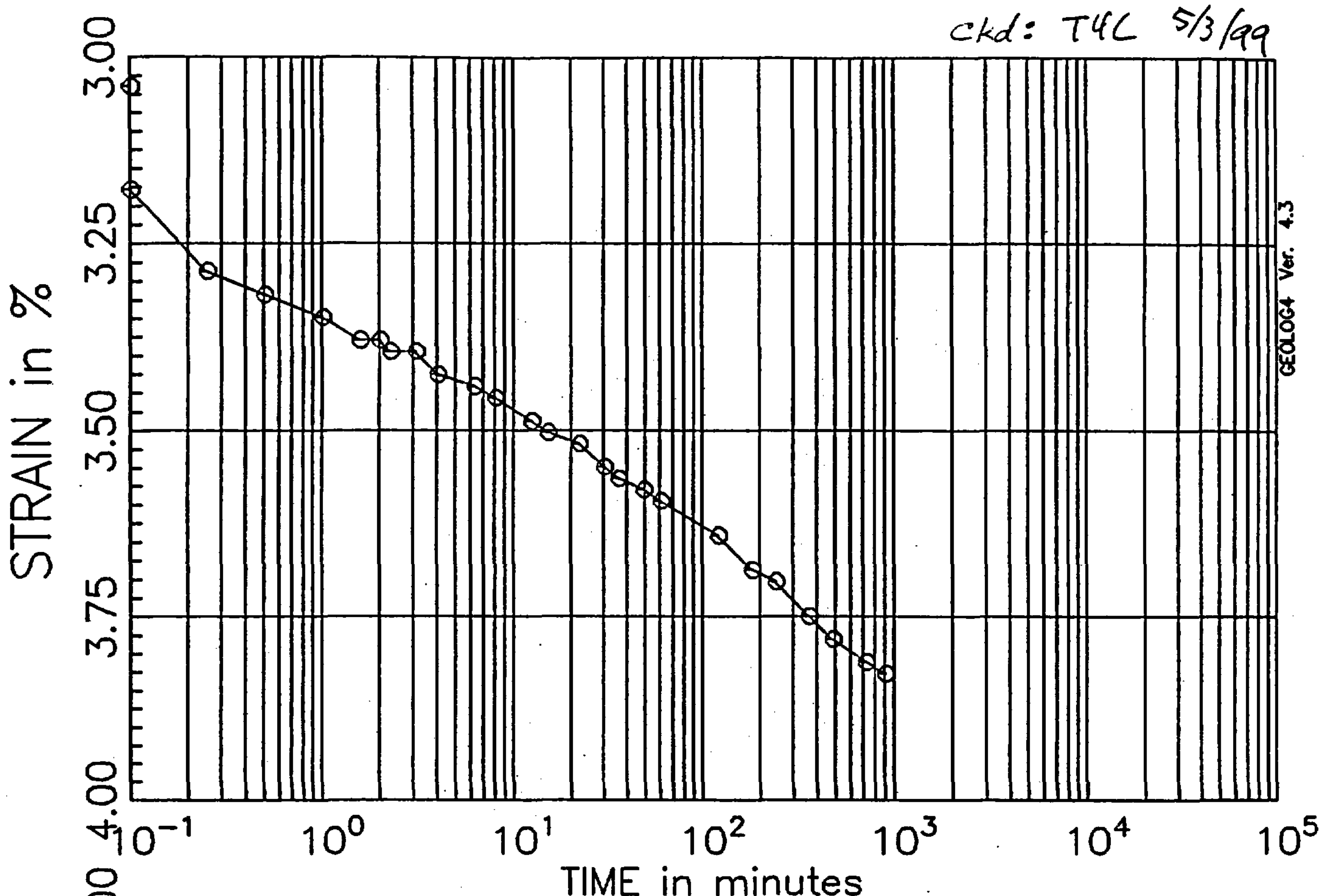


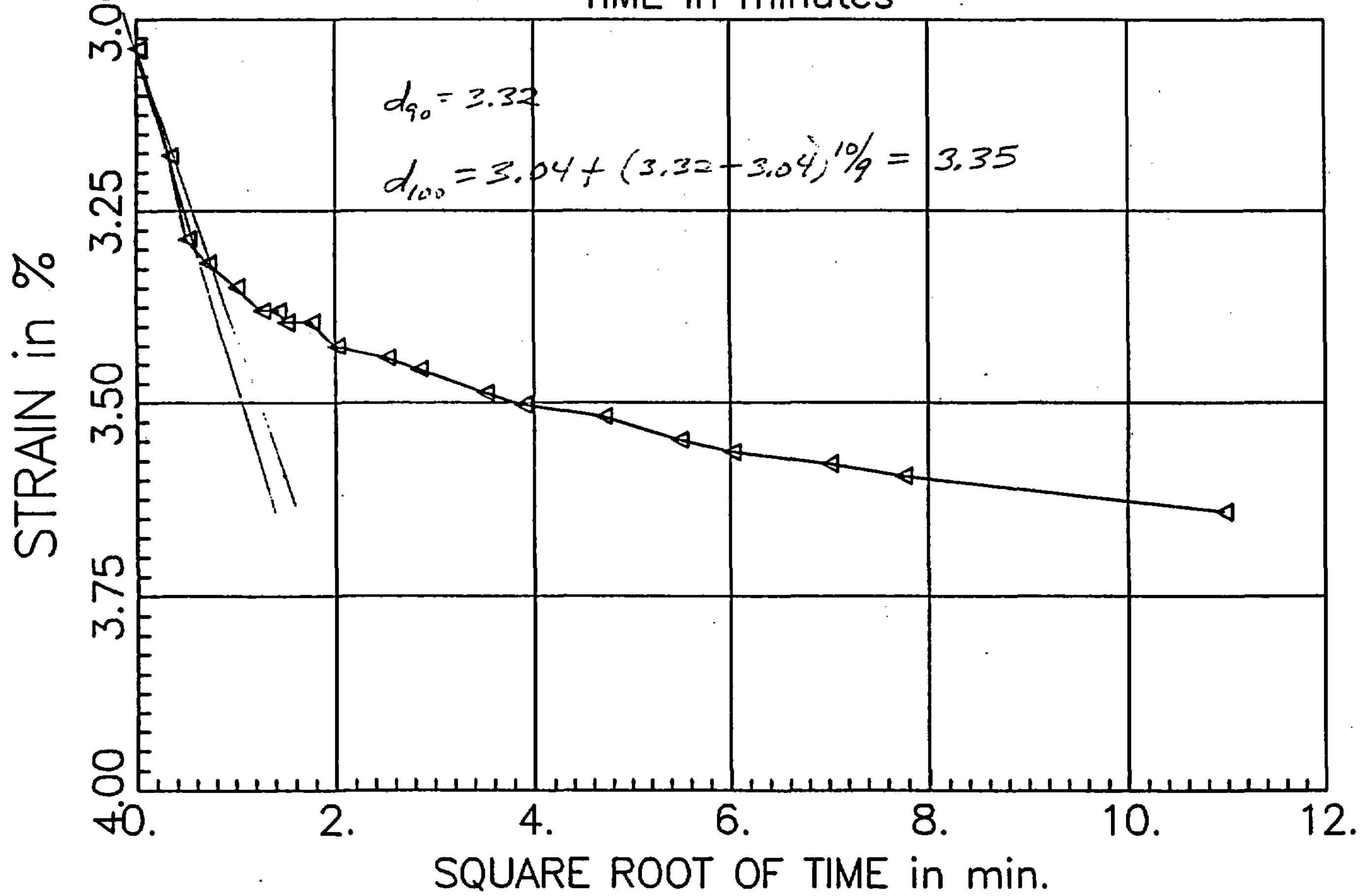


PRESSURE INCREMENT #13
from 4.00 tsf to 6.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02
$d_{100} = 3.35\%$
Calc: ACS 4/23/99
Ckd: TYL 5/3/99



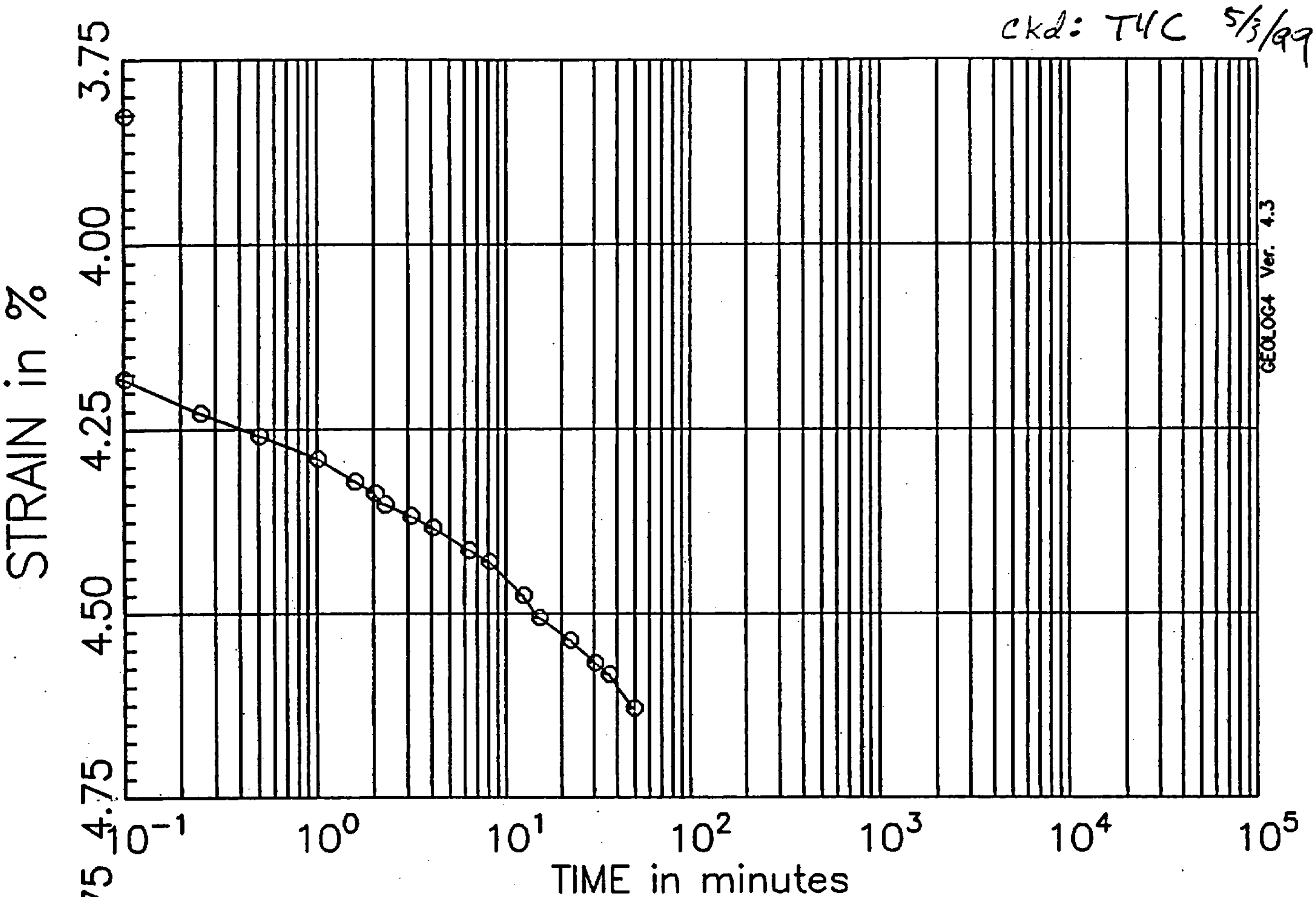


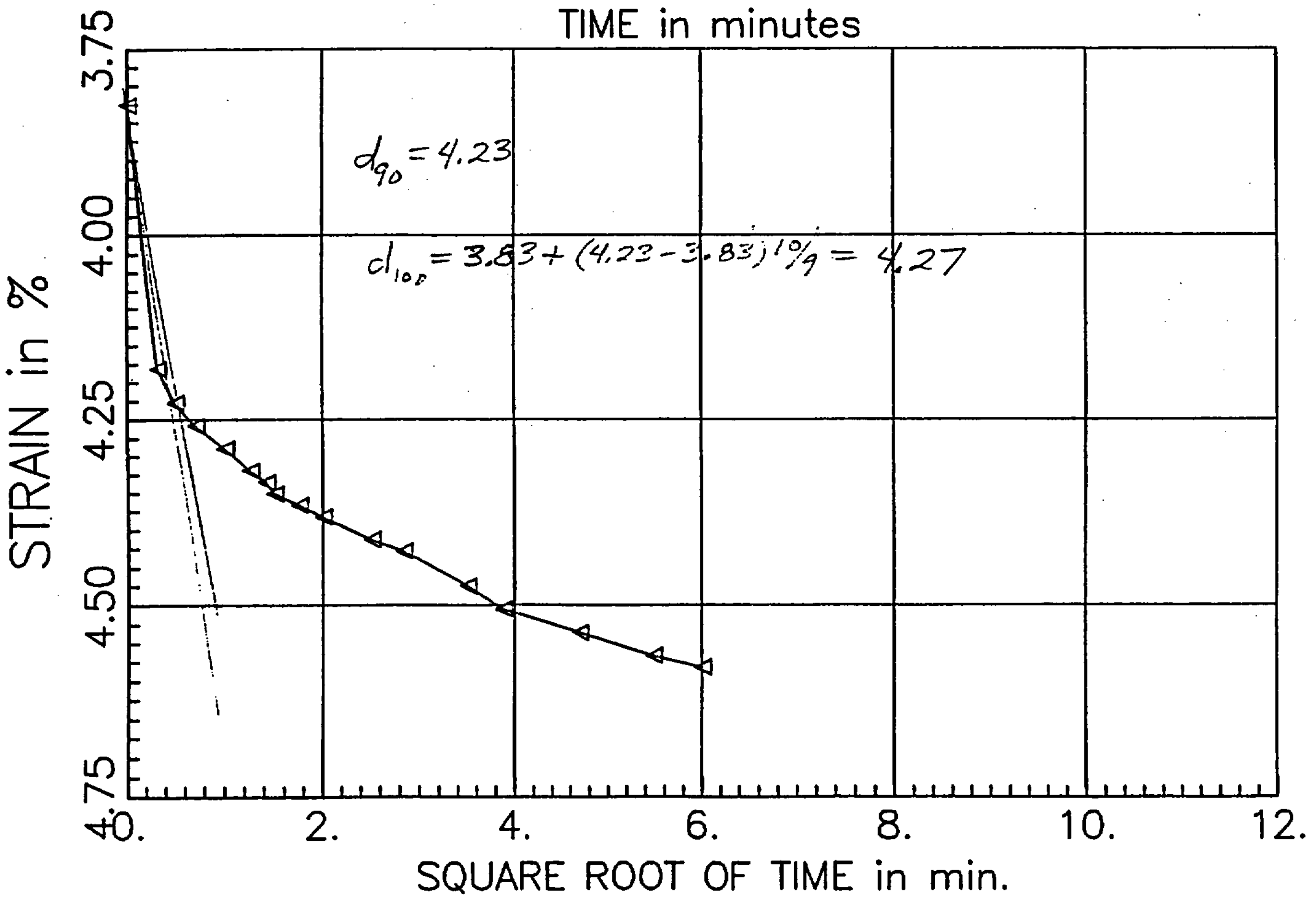
PRESSURE INCREMENT #14
from 6.00 tsf to 8.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05996.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

$d_{100} = 4.27\%$



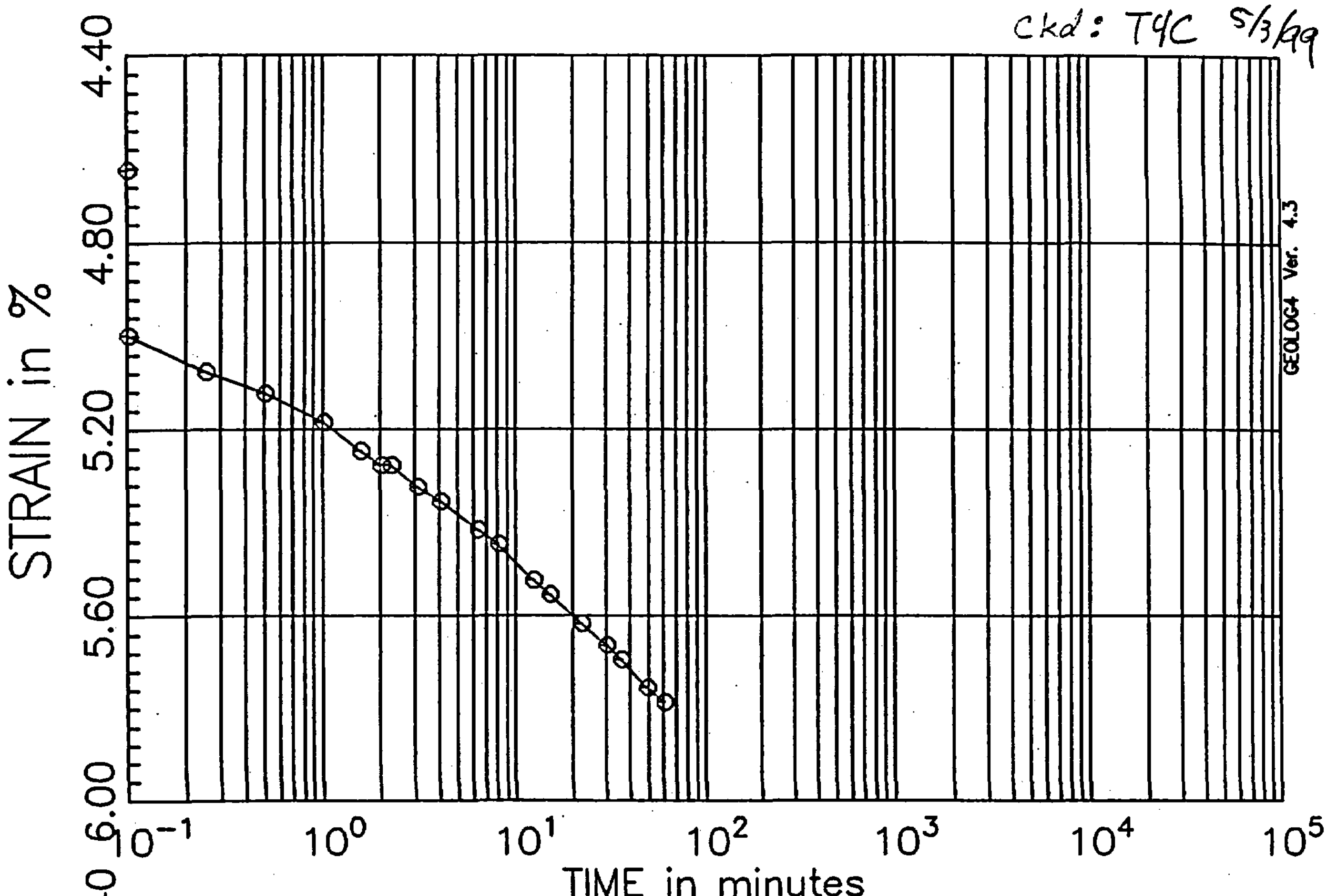


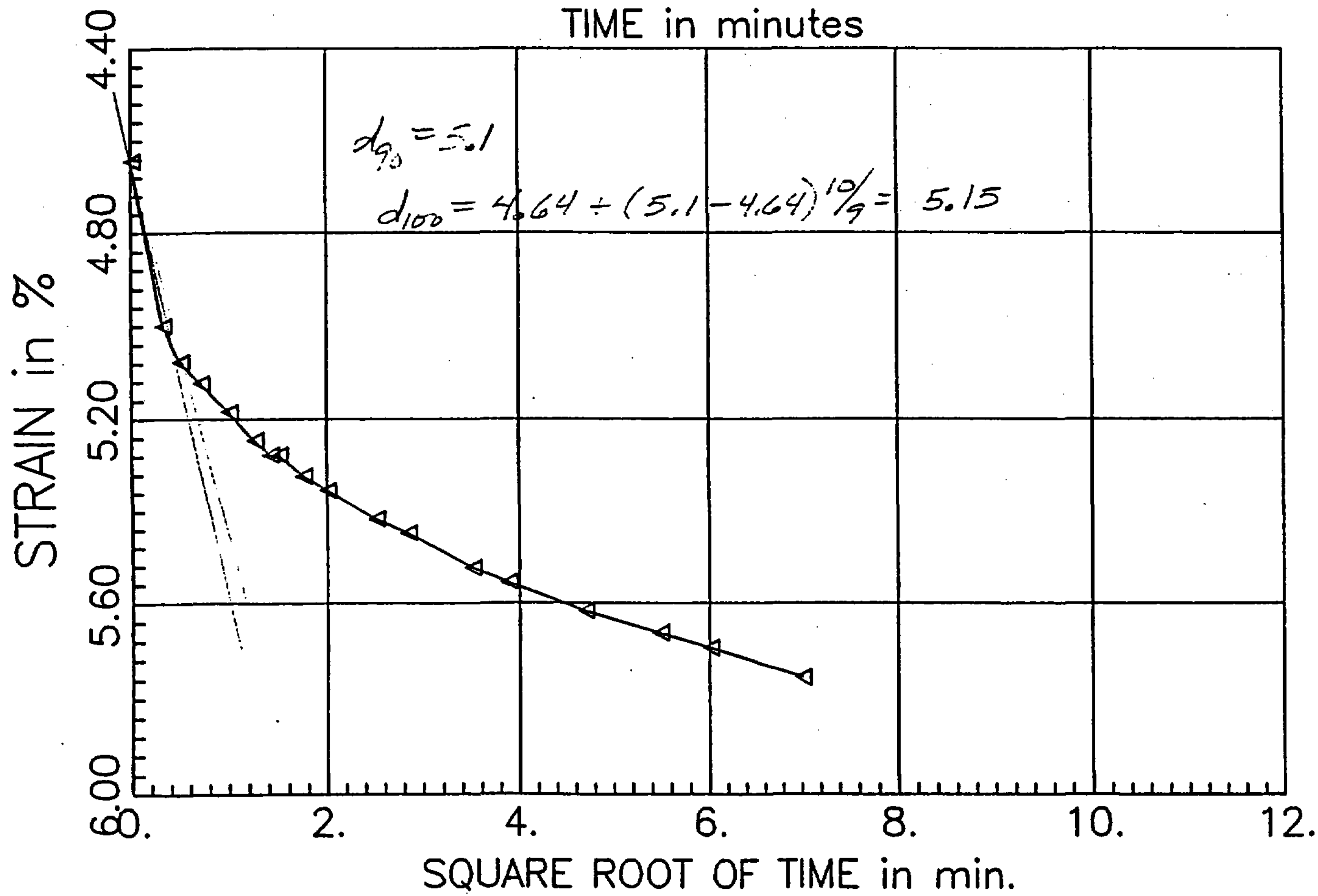
PRESSURE INCREMENT #15
from 8.00 tsf to 12.00 tsf

Test No: 3
Testname: CTB5-14E

$d_{100} = 5.15\ \%$

J0 05776.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99



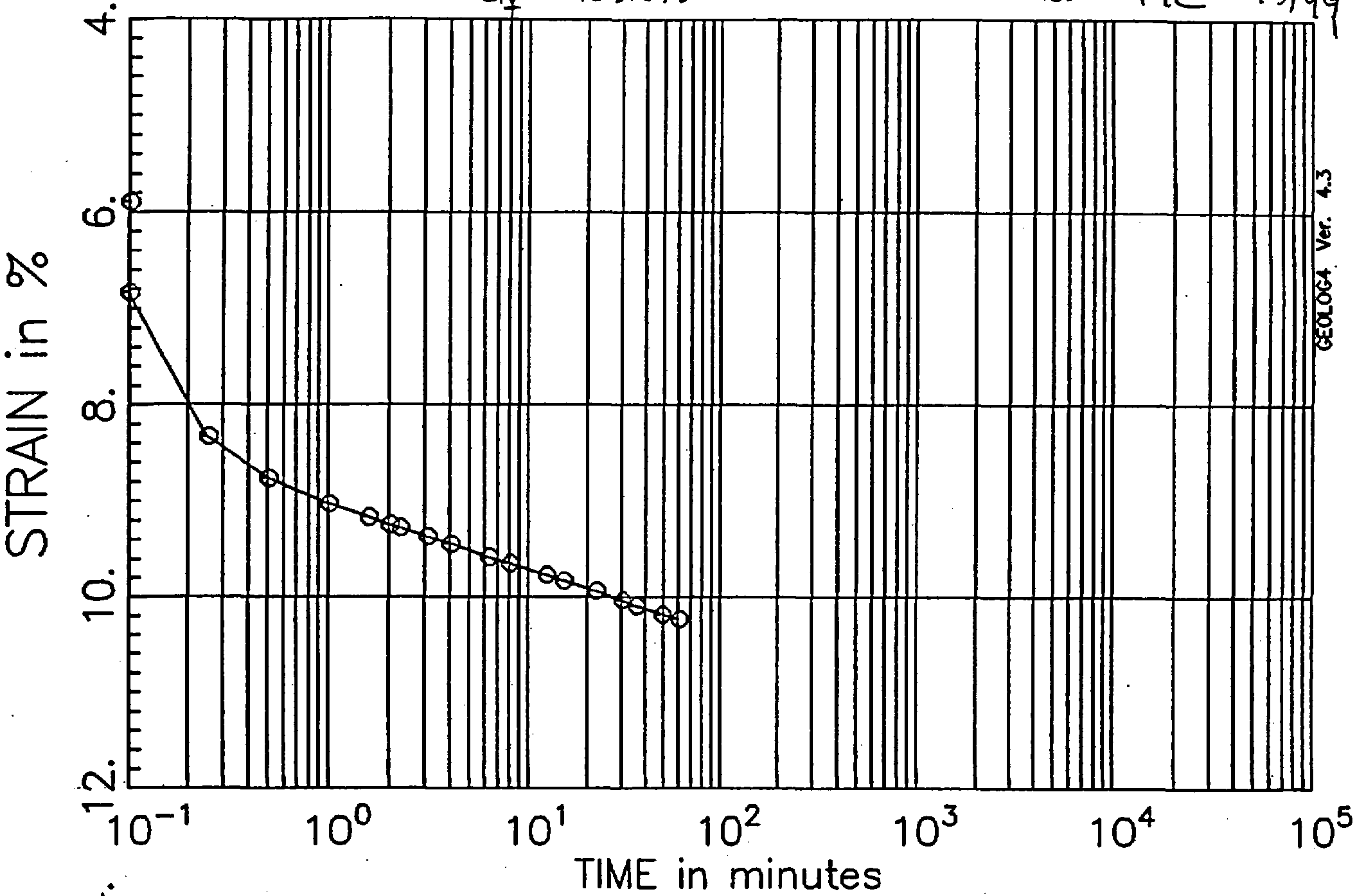


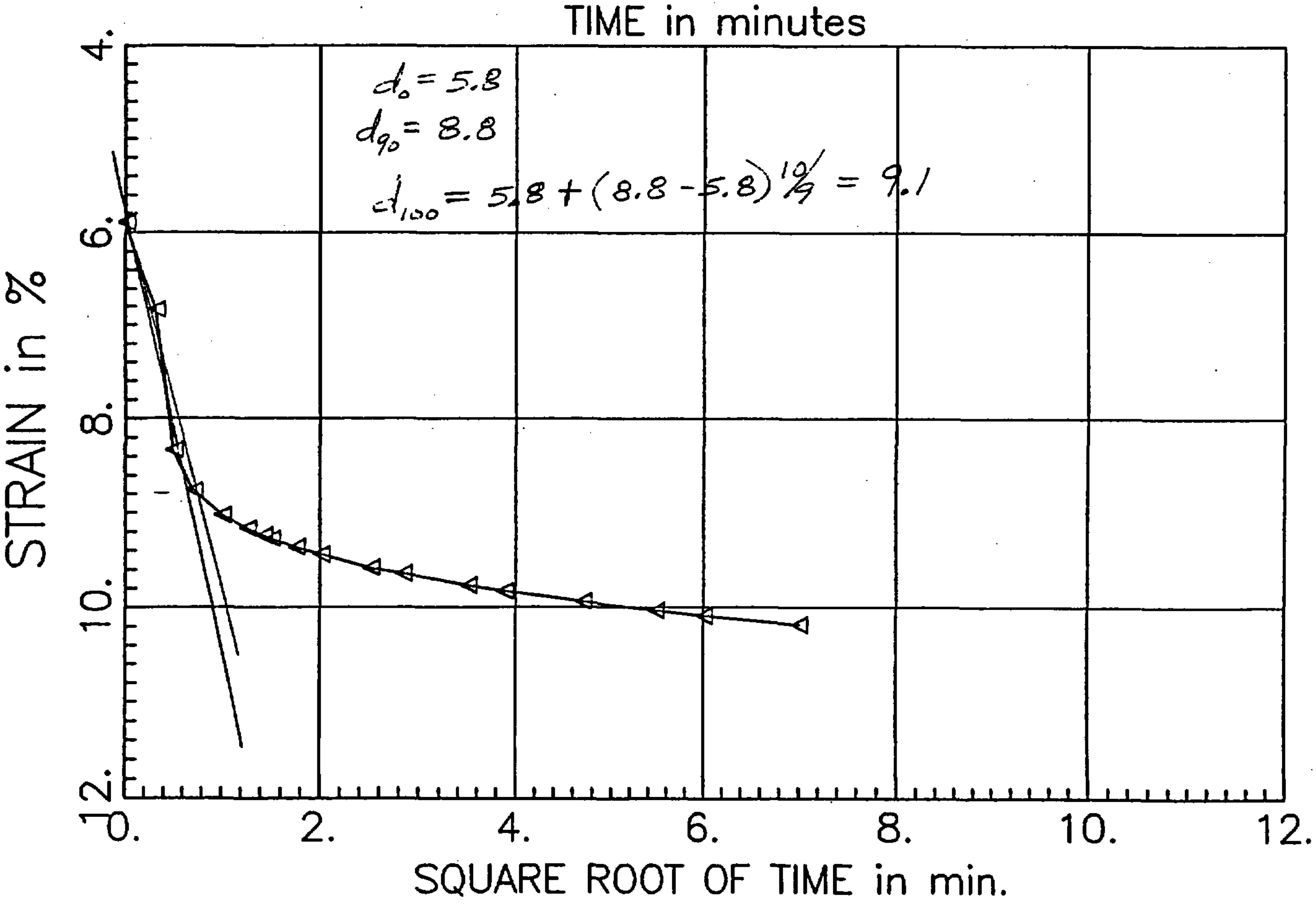
PRESSURE INCREMENT #16
from 12.00 tsf to 16.00 tsf

Test No: 3
Testname: CTB5-14E

Jo 05996.02
Calc: ACS 4/23/99
Ckd: T4C 5/3/99

$d_{100} = 9.1\%$
$d_f = 10.2\%$



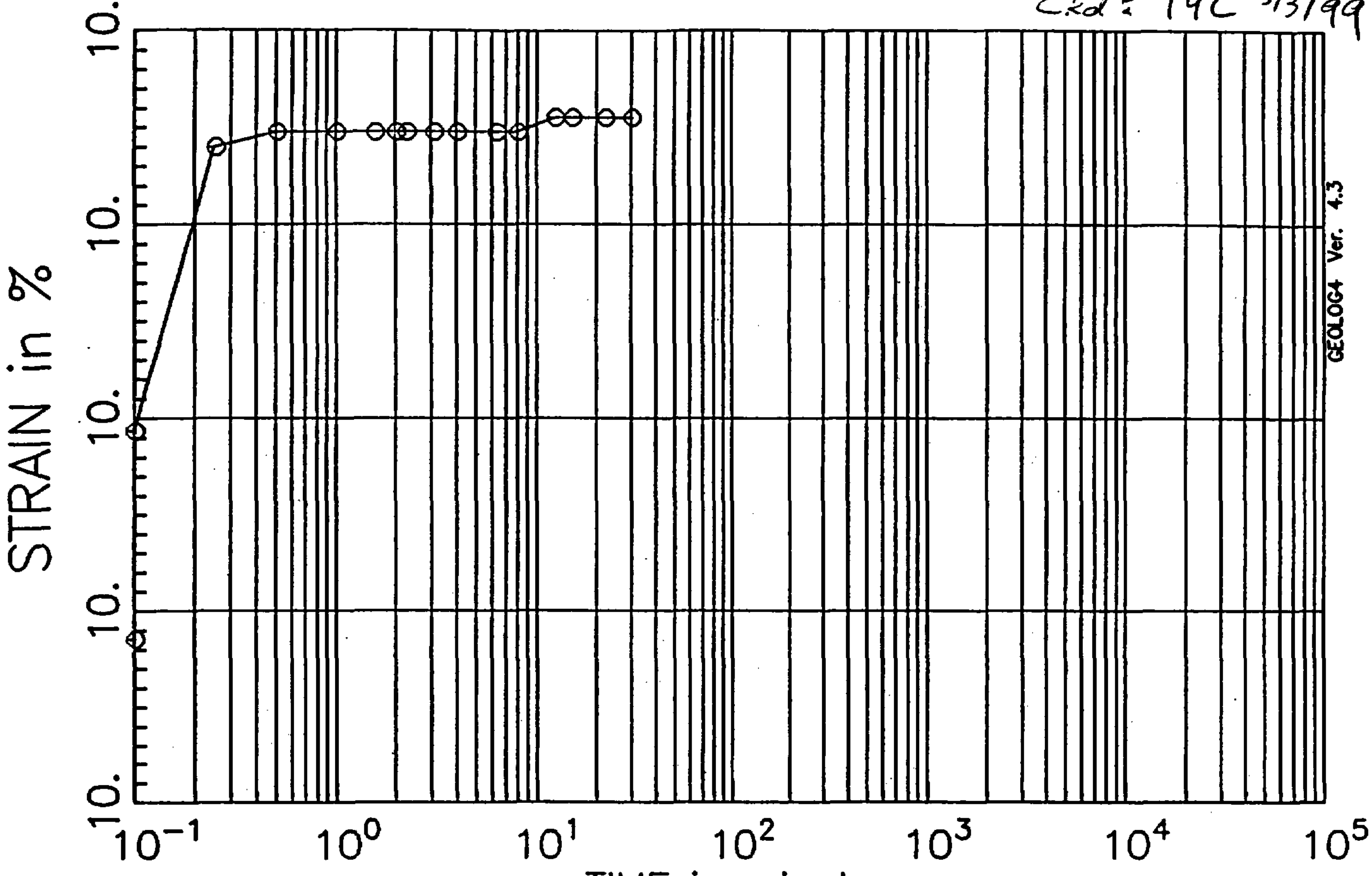


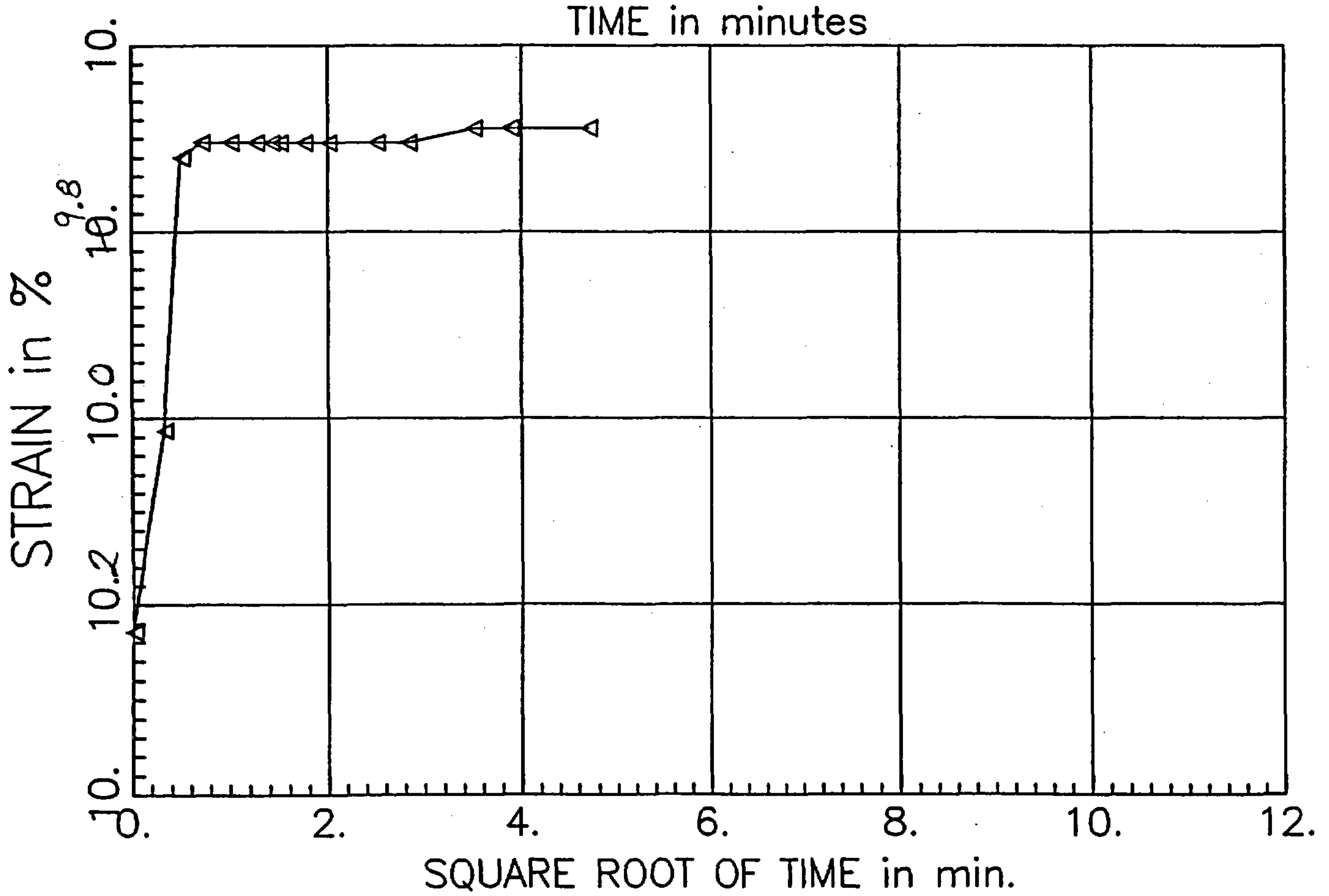
PRESSURE INCREMENT #17
from 16.00 tsf to 32.00 tsf

Test No: 3
Testname: CTB5-14E

JO 05746.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

d_{100} = 9.7 %



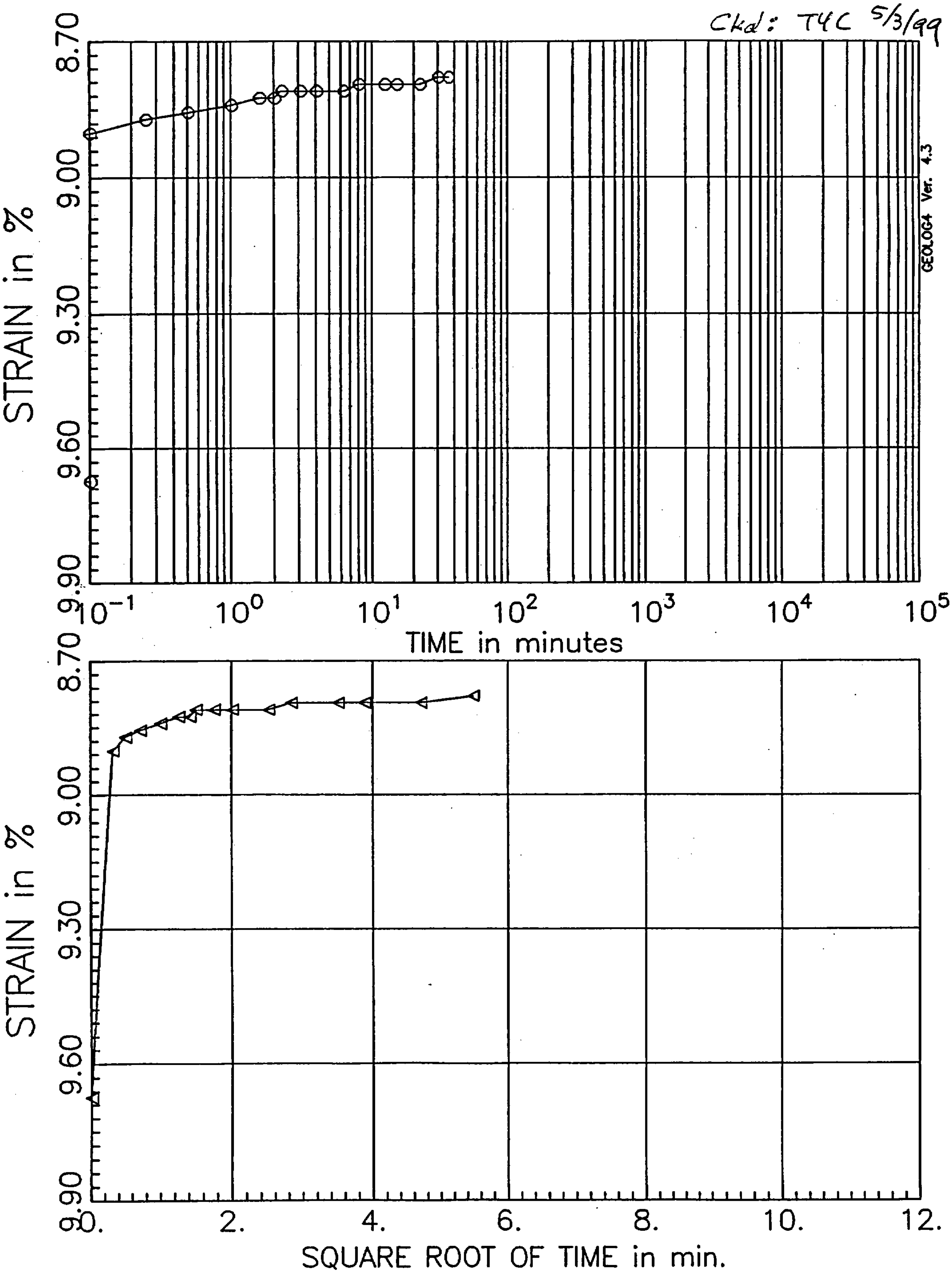


PRESSURE INCREMENT #18
from 32.00 tsf to 16.00 tsf

Test No: 3
Testname: CTB5-14E

$d_{100} = 8.85\%$

Jo 05996.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

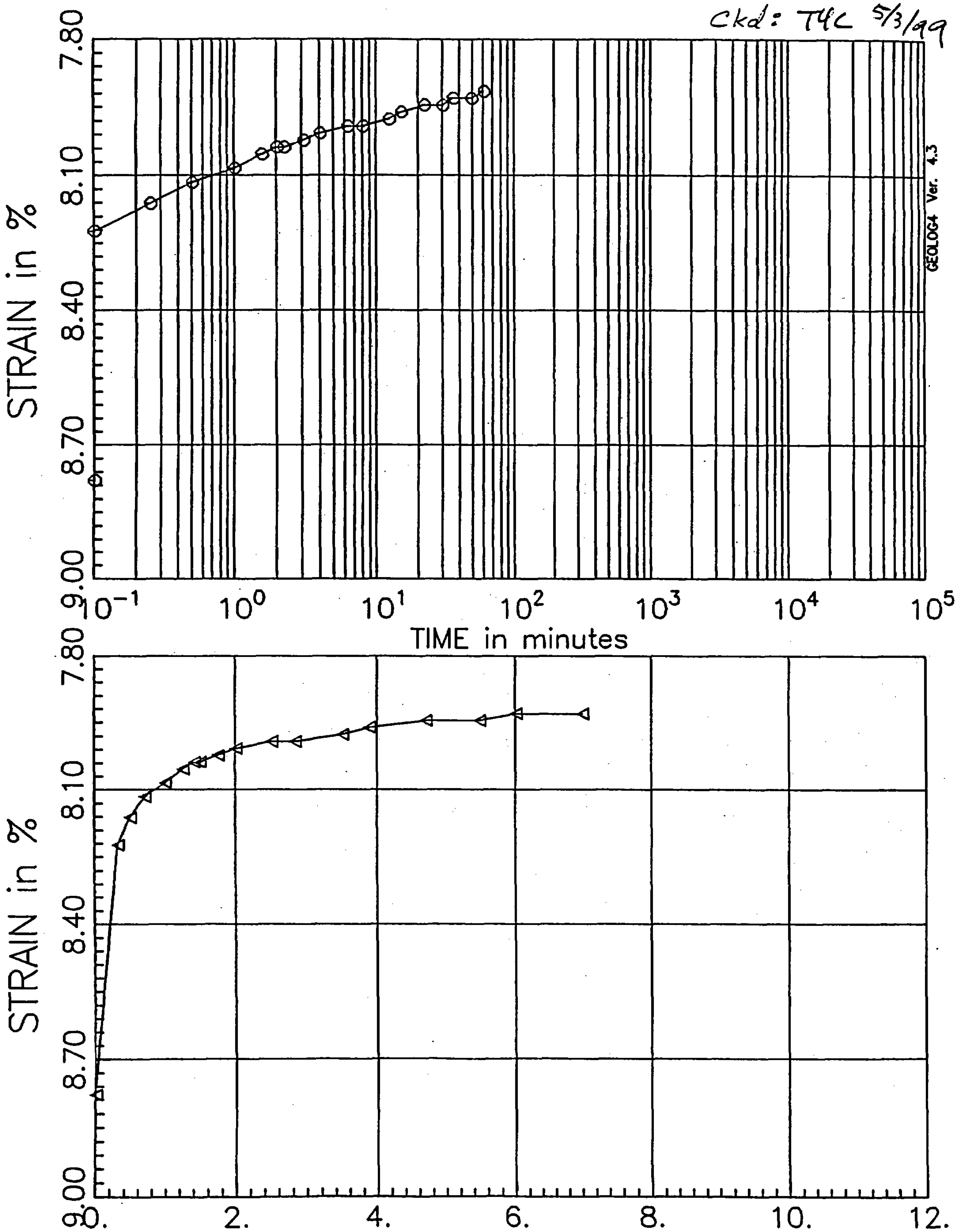


PRESSURE INCREMENT #19
from 16.00 tsf to 4.00 tsf

Test No: 3
Testname: CTB5-14E

$d_{100} = 8.1\%$

JO 05996.02.
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

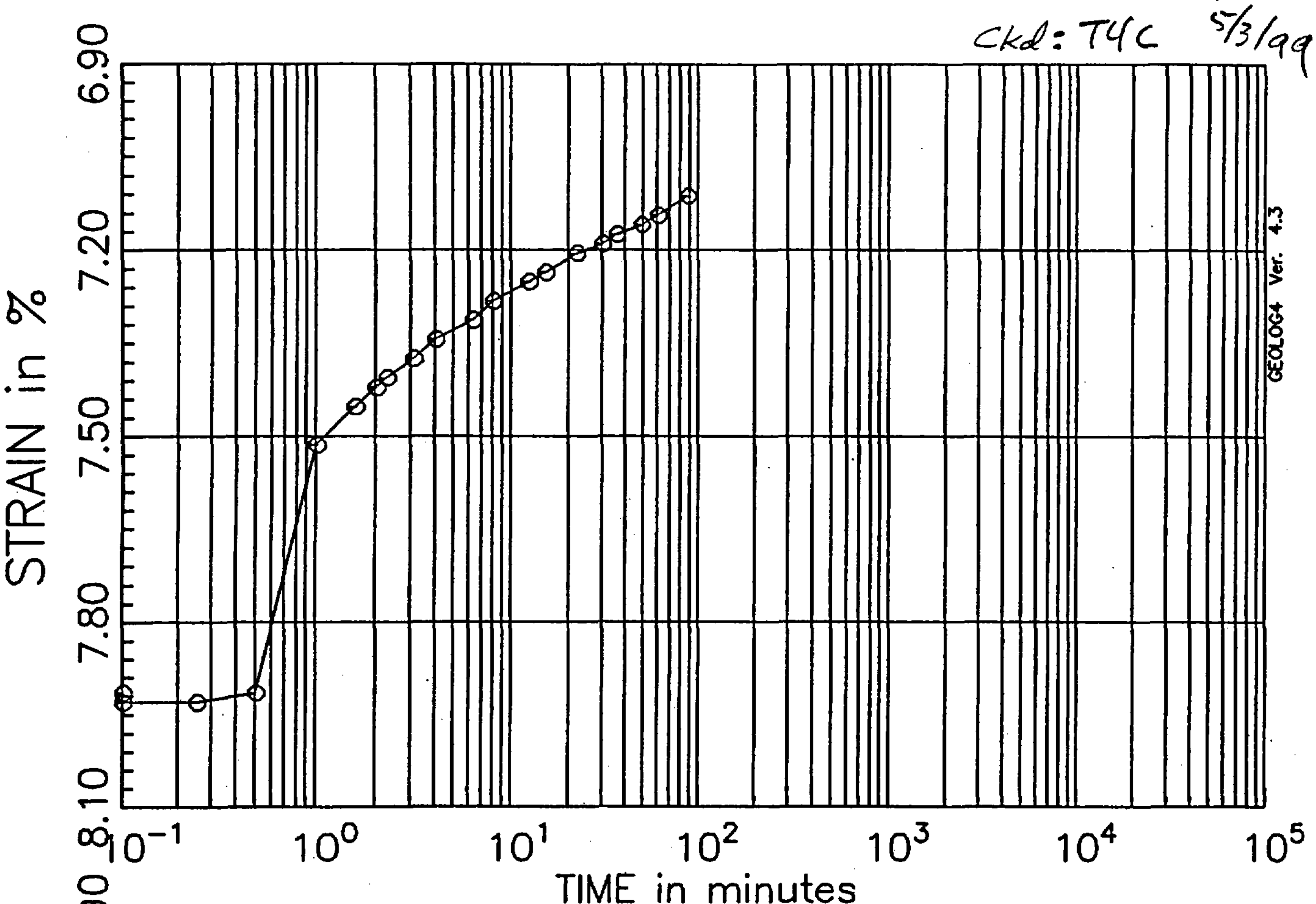


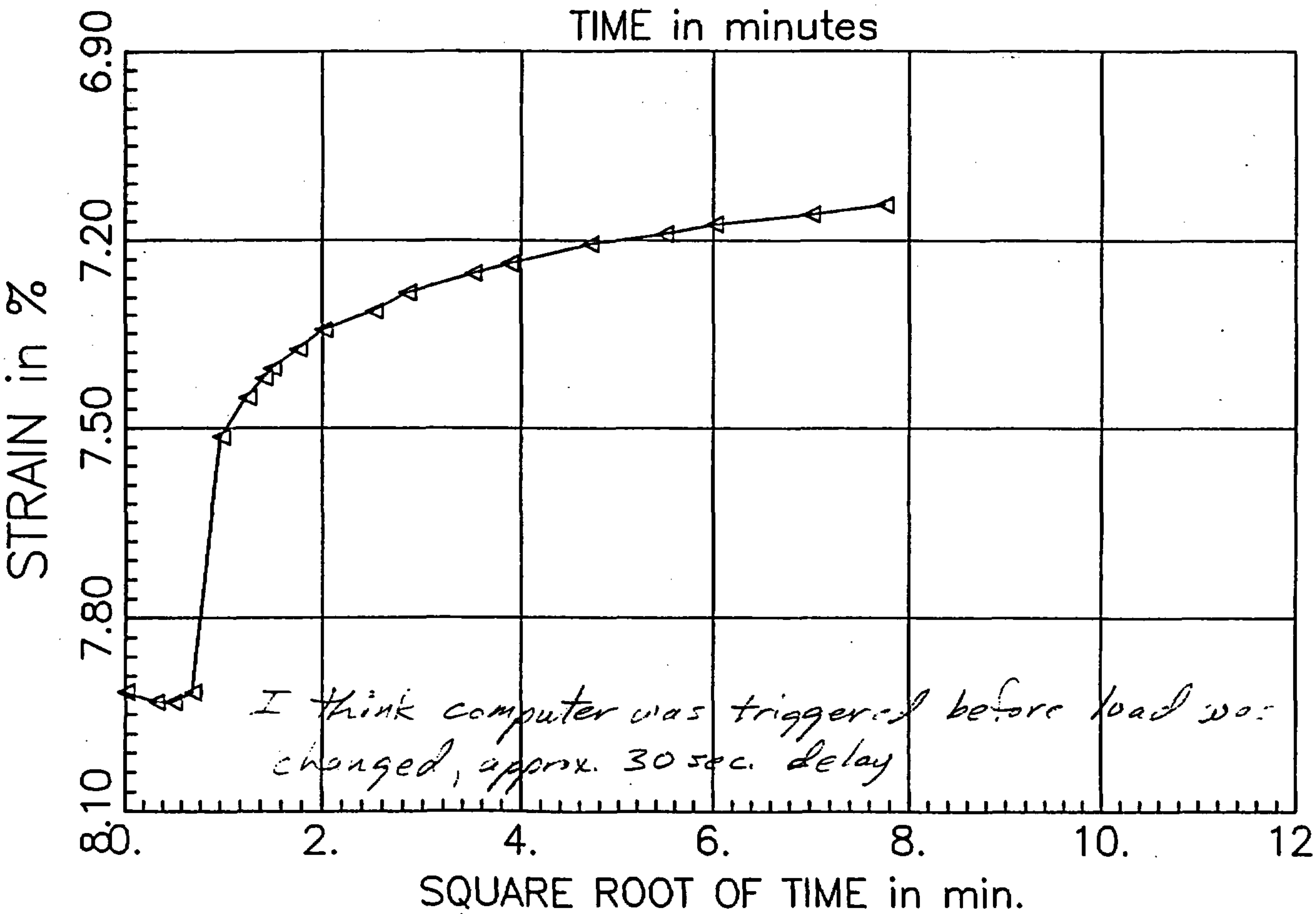
PRESSURE INCREMENT #20
from 4.00 tsf to 1.00 tsf

Test No: 3
Testname: CTB5-14E

$d_{100} = 7.4\%$

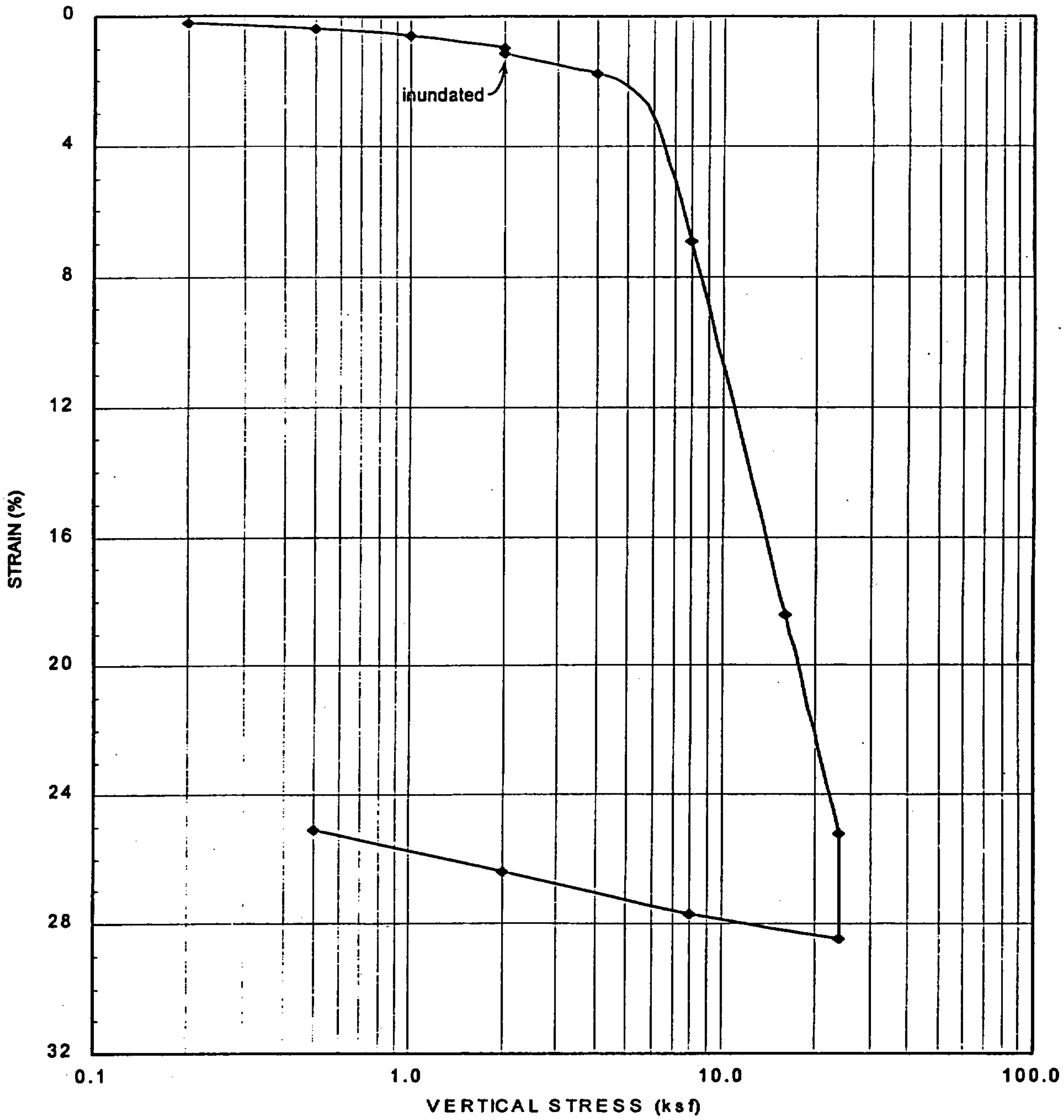
JO 05996.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99





PRESSURE INCREMENT #21
from 1.00 tsf to 0.25 tsf

Test No: 3
Testname: CTB5-14E



SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	4/12/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	8.6 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	63.0 %	60.6 %	2.72 (est)
DRY UNIT WEIGHT:	48.4 pcf	64.0 pcf	
VOID RATIO:	2.511	1.655	
SATURATION:	68.2 %	99.5 %	

NOTE: Sample was inundated 41 minutes after applying a vertical stress of 2 ksf

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING CTB-N, SAMPLE U-2D	JO 05996.02 April 1999

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	4/12/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	8.6 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	63.0 %	60.6 %
DRY UNIT WEIGHT:	48.4 pcf	64.0 pcf
VOID RATIO:	2.511	1.655
SATURATION:	68.2 %	99.5 %
HEIGHT:	1.894 cm	1.432 cm
AREA:	31.61 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE ksf	STRAIN %	
0.20	0.19	
0.50	0.38	
1.00	0.61	
2.00	0.95	
2.00	1.15	(after inundation)
4.00	1.78	
8.00	6.90	
16.00	18.41	
24.00	25.20	
24.00	28.46	
8.00	27.70	
2.00	26.40	
0.50	25.10	

LOAD INCREMENT DATA

TEST NAME   CTBN-U2D   PAGE NO: 1
TEST NO: 1   TESTED BY: ACS

PRESSURE INCREMENT FROM   0.00 tsf to   0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	15:25:16	0.10	0.32	0.0013	2.505	0.17
		0.25	0.50	0.0013	2.505	0.17
		0.50	0.71	0.0014	2.504	0.19
		1.00	1.00	0.0014	2.504	0.19
		1.57	1.25	0.0013	2.505	0.17
		2.00	1.41	0.0014	2.504	0.19
		2.25	1.50	0.0013	2.505	0.17
		3.07	1.75	0.0014	2.504	0.19
		4.00	2.00	0.0014	2.504	0.19
		6.25	2.50	0.0014	2.504	0.19
		8.00	2.83	0.0014	2.504	0.19
		12.25	3.50	0.0014	2.504	0.19
		15.00	3.87	0.0016	2.504	0.21
		22.00	4.69	0.0016	2.504	0.21
		30.00	5.48	0.0017	2.503	0.23

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 2
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:01:19	0.00	0.00	0.0017	2.503	0.23
		0.10	0.32	0.0027	2.498	0.36
		0.25	0.50	0.0027	2.498	0.36
		0.50	0.71	0.0027	2.498	0.36
		1.00	1.00	0.0027	2.498	0.36
		1.57	1.25	0.0028	2.498	0.38
		2.00	1.41	0.0028	2.498	0.38
		2.25	1.50	0.0028	2.498	0.38
		3.07	1.75	0.0028	2.498	0.38
		4.00	2.00	0.0028	2.498	0.38
		6.25	2.50	0.0028	2.498	0.38
		8.00	2.83	0.0028	2.498	0.38
		12.25	3.50	0.0030	2.497	0.40
		15.00	3.87	0.0030	2.497	0.40
		22.00	4.69	0.0030	2.497	0.40

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 3
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	16:29:04	0.00	0.00	0.0030	2.497	0.40
		0.10	0.32	0.0043	2.491	0.57
		0.25	0.50	0.0044	2.490	0.59
		0.50	0.71	0.0044	2.490	0.59
		1.00	1.00	0.0045	2.490	0.61
		1.57	1.25	0.0044	2.490	0.59
		2.00	1.41	0.0045	2.490	0.61
		2.25	1.50	0.0044	2.490	0.59
		3.07	1.75	0.0045	2.490	0.61
		4.00	2.00	0.0045	2.490	0.61
		6.25	2.50	0.0045	2.490	0.61
		8.00	2.83	0.0045	2.490	0.61
		12.25	3.50	0.0047	2.489	0.62
		15.00	3.87	0.0047	2.489	0.62
		22.00	4.69	0.0048	2.488	0.64
		30.00	5.48	0.0048	2.488	0.64
		36.00	6.00	0.0048	2.488	0.64

LOAD INCREMENT DATA

TEST NAME CTBN-U2D

TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-14-99	17:07:33	0.00	0.00	0.0048	2.488	0.64
		0.10	0.32	0.0068	2.479	0.92
		0.25	0.50	0.0068	2.479	0.92
		0.50	0.71	0.0070	2.478	0.94
		1.00	1.00	0.0071	2.478	0.95
		1.57	1.25	0.0071	2.478	0.95
		2.00	1.41	0.0072	2.477	0.97
		2.25	1.50	0.0072	2.477	0.97
		3.07	1.75	0.0072	2.477	0.97
		4.00	2.00	0.0072	2.477	0.97
		6.25	2.50	0.0074	2.476	0.99
		8.00	2.83	0.0074	2.476	0.99
		12.25	3.50	0.0075	2.476	1.00
		15.00	3.87	0.0075	2.476	1.00
		22.00	4.69	0.0075	2.476	1.00
		30.00	5.48	0.0076	2.475	1.02
		36.00	6.00	0.0076	2.475	1.02
		49.00	7.00	0.0076	2.475	1.02
		60.00	7.75	0.0076	2.475	1.02
		120.00	10.95	0.0079	2.474	1.06
		180.00	13.42	0.0080	2.473	1.07
		240.00	15.49	0.0081	2.473	1.09
		360.00	18.97	0.0083	2.472	1.11
04-15-99	01:07:33	480.00	21.91	0.0084	2.471	1.13
		720.00	26.83	0.0087	2.470	1.16
		840.00	28.98	0.0085	2.471	1.14

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 5
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	07:57:17	0.00	0.00	0.0085	2.471	1.14
		0.10	0.32	0.0121	2.454	1.63
		0.25	0.50	0.0127	2.451	1.70
		0.50	0.71	0.0130	2.450	1.75
		1.00	1.00	0.0134	2.448	1.80
		1.57	1.25	0.0138	2.446	1.85
		2.00	1.41	0.0141	2.445	1.89
		2.25	1.50	0.0141	2.445	1.89
		3.07	1.75	0.0143	2.443	1.92
		4.00	2.00	0.0146	2.442	1.96
		6.25	2.50	0.0150	2.440	2.01
		8.00	2.83	0.0152	2.439	2.04
		12.25	3.50	0.0158	2.437	2.11
		15.00	3.87	0.0160	2.436	2.15
		22.00	4.69	0.0165	2.433	2.22
		30.00	5.48	0.0169	2.431	2.27
		36.00	6.00	0.0171	2.431	2.29
		49.00	7.00	0.0176	2.428	2.36
		60.00	7.75	0.0180	2.426	2.41

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 6
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	09:03:05	0.00	0.00	0.0181	2.426	2.43
		0.10	0.32	0.0407	2.319	5.46
		0.25	0.50	0.0461	2.294	6.18
		0.50	0.71	0.0496	2.277	6.65
		1.00	1.00	0.0530	2.262	7.10
		1.57	1.25	0.0552	2.251	7.40
		2.00	1.41	0.0563	2.246	7.55
		2.25	1.50	0.0570	2.243	7.64
		3.07	1.75	0.0585	2.235	7.85
		4.00	2.00	0.0599	2.229	8.04
		6.25	2.50	0.0623	2.218	8.35
		8.00	2.83	0.0637	2.211	8.54
		12.25	3.50	0.0660	2.200	8.85
		15.00	3.87	0.0672	2.195	9.01
		22.00	4.69	0.0695	2.184	9.32
		30.00	5.48	0.0713	2.175	9.56
		36.00	6.00	0.0723	2.170	9.70
		49.00	7.00	0.0743	2.161	9.96
		60.00	7.75	0.0754	2.156	10.12
		70.40	8.39	0.0766	2.150	10.27

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 7

TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	10:13:30	0.00	0.00	0.0766	2.150	10.27
		0.10	0.32	0.1167	1.962	15.64
		0.25	0.50	0.1274	1.911	17.08
		0.50	0.71	0.1337	1.881	17.93
		1.00	1.00	0.1390	1.857	18.64
		1.57	1.25	0.1422	1.841	19.07
		2.00	1.41	0.1439	1.833	19.30
		2.25	1.50	0.1447	1.830	19.40
		3.07	1.75	0.1468	1.820	19.68
		4.00	2.00	0.1484	1.812	19.91
		6.25	2.50	0.1514	1.798	20.30
		8.00	2.83	0.1530	1.791	20.51
		12.25	3.50	0.1557	1.778	20.88
		15.00	3.87	0.1570	1.772	21.05
		22.00	4.69	0.1593	1.761	21.36
		30.00	5.48	0.1612	1.752	21.62
		36.00	6.00	0.1624	1.746	21.78
		49.00	7.00	0.1642	1.738	22.02

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 8

TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 8.00 tsf to 12.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	11:05:16	0.00	0.00	0.1646	1.736	22.07
		0.10	0.32	0.1775	1.675	23.80
		0.25	0.50	0.1820	1.654	24.41
		0.50	0.71	0.1856	1.637	24.90
		1.00	1.00	0.1894	1.619	25.40
		1.57	1.25	0.1918	1.608	25.73
		2.00	1.41	0.1931	1.602	25.90
		2.25	1.50	0.1938	1.599	25.99
		3.07	1.75	0.1955	1.591	26.21
		4.00	2.00	0.1969	1.584	26.40
		6.25	2.50	0.1993	1.572	26.73
		8.00	2.83	0.2006	1.566	26.90
		12.25	3.50	0.2029	1.555	27.22
		15.00	3.87	0.2041	1.550	27.37
		22.00	4.69	0.2060	1.541	27.63
		30.00	5.48	0.2079	1.532	27.88
		36.00	6.00	0.2088	1.528	28.00
		49.00	7.00	0.2104	1.520	28.22
		60.00	7.75	0.2115	1.515	28.36
		70.00	8.37	0.2122	1.512	28.46

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 9
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 12.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:15:40	0.00	0.00	0.2122	1.512	28.46
		0.10	0.32	0.2067	1.538	27.72
		0.25	0.50	0.2066	1.538	27.70
		0.50	0.71	0.2064	1.539	27.68
		1.00	1.00	0.2063	1.540	27.67
		1.57	1.25	0.2063	1.540	27.67
		2.00	1.41	0.2062	1.540	27.65
		2.25	1.50	0.2062	1.540	27.65
		3.07	1.75	0.2062	1.540	27.65
		4.00	2.00	0.2062	1.540	27.65
		6.25	2.50	0.2062	1.540	27.65
		8.00	2.83	0.2060	1.541	27.63
		12.25	3.50	0.2060	1.541	27.63
		15.00	3.87	0.2060	1.541	27.63
		22.00	4.69	0.2060	1.541	27.63
		30.00	5.48	0.2060	1.541	27.63

LOAD INCREMENT DATA

TEST NAME CTBN-U2D PAGE NO: 10
TEST NO: 1 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	12:48:27	0.00	0.00	0.2059	1.541	27.62
		0.10	0.32	0.1991	1.574	26.70
		0.25	0.50	0.1982	1.578	26.58
		0.50	0.71	0.1975	1.581	26.49
		1.00	1.00	0.1970	1.583	26.42
		1.57	1.25	0.1967	1.585	26.39
		2.00	1.41	0.1966	1.585	26.37
		2.25	1.50	0.1965	1.586	26.35
		3.07	1.75	0.1964	1.586	26.33
		4.00	2.00	0.1964	1.586	26.33
		6.25	2.50	0.1961	1.588	26.30
		8.00	2.83	0.1961	1.588	26.30
		12.25	3.50	0.1960	1.588	26.28
		15.00	3.87	0.1958	1.589	26.26
		22.00	4.69	0.1957	1.589	26.25
		30.00	5.48	0.1957	1.589	26.25

LOAD INCREMENT DATA

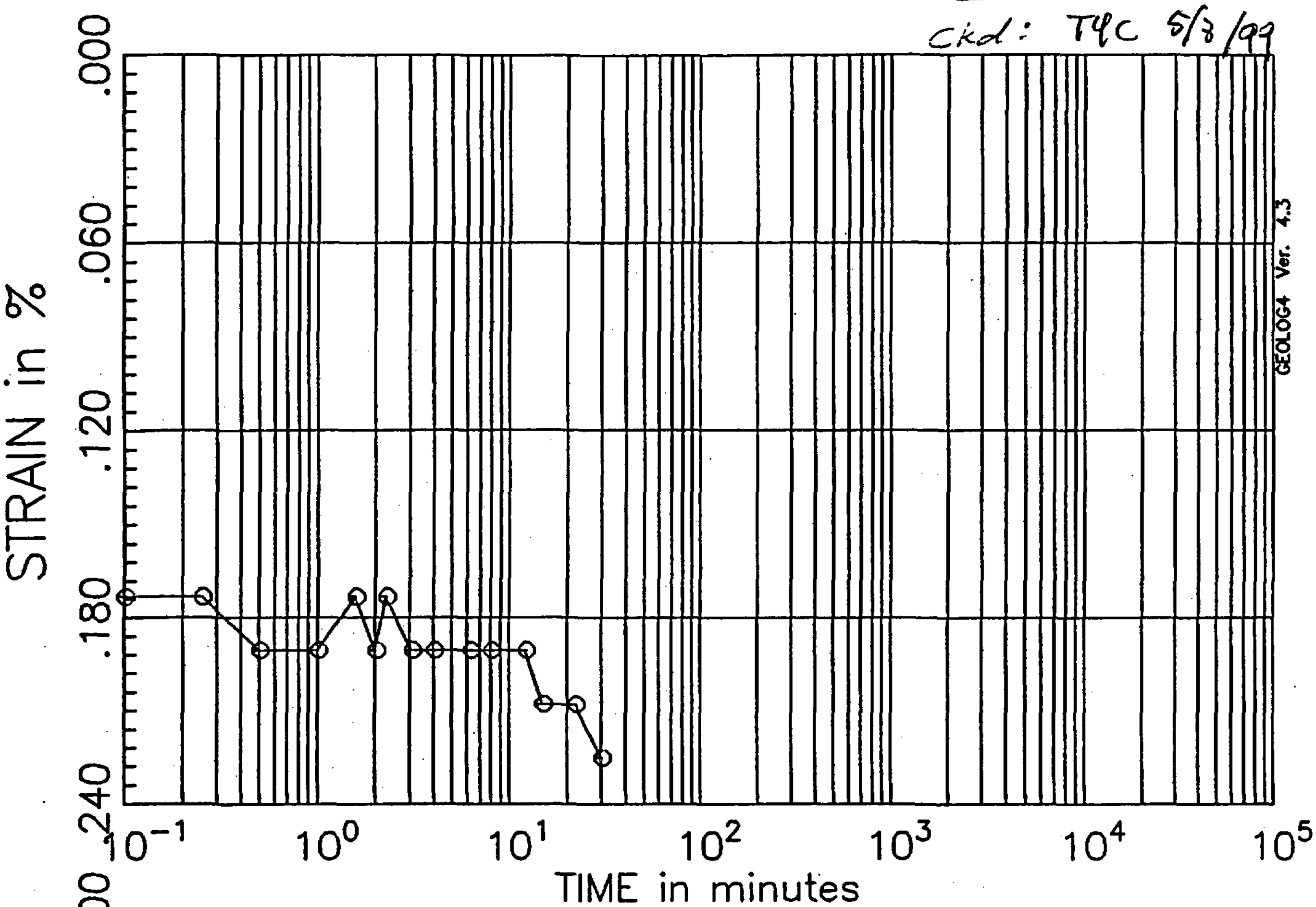
TEST NAME CTBN-U2D PAGE NO: 11
TEST NO: 1 TESTED BY: ACS

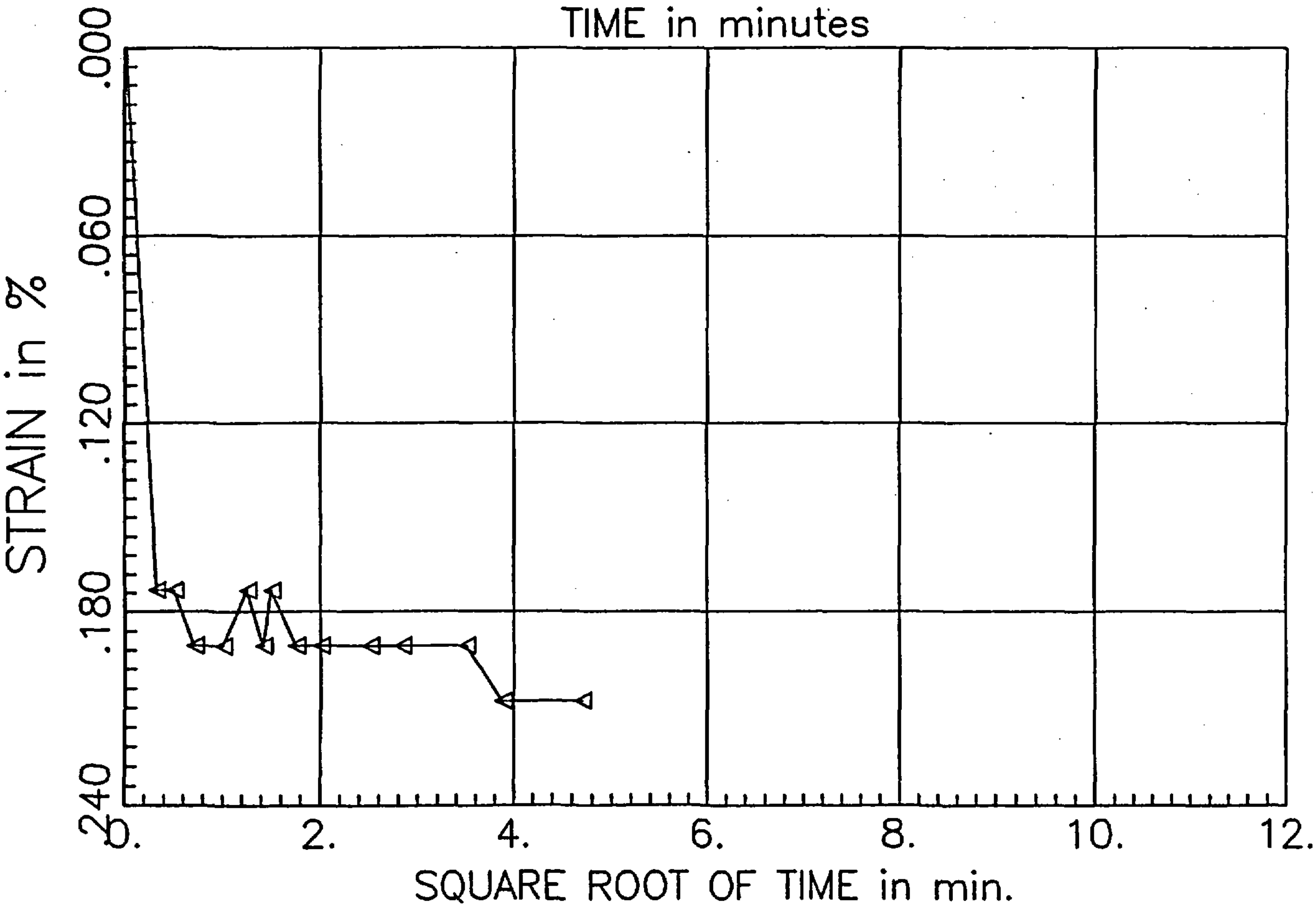
PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-15-99	13:24:11	0.00	0.00	0.1956	1.590	26.23
		0.10	0.32	0.1909	1.612	25.61
		0.25	0.50	0.1894	1.619	25.40
		0.50	0.71	0.1880	1.626	25.21
		1.00	1.00	0.1864	1.633	25.00
		1.57	1.25	0.1856	1.637	24.90
		2.00	1.41	0.1851	1.639	24.83
		2.25	1.50	0.1850	1.640	24.81
		3.07	1.75	0.1846	1.642	24.76
		4.00	2.00	0.1843	1.643	24.72
		6.25	2.50	0.1838	1.645	24.65
		8.00	2.83	0.1836	1.647	24.62
		12.25	3.50	0.1833	1.648	24.58
		15.00	3.87	0.1832	1.648	24.57
		22.00	4.69	0.1829	1.650	24.53
		30.00	5.48	0.1827	1.651	24.50
		36.00	6.00	0.1825	1.652	24.48
		49.00	7.00	0.1824	1.652	24.46
		60.00	7.75	0.1823	1.653	24.44
		120.00	10.95	0.1820	1.654	24.41
		180.00	13.42	0.1818	1.655	24.38
		196.00	14.00	0.1818	1.655	24.38

$d_{100} = 0.19\%$

Cak: ACS 4/23/99
Ckd: TYC 5/3/99





PRESSURE INCREMENT #1
from 0.00 tsf to 0.10 tsf

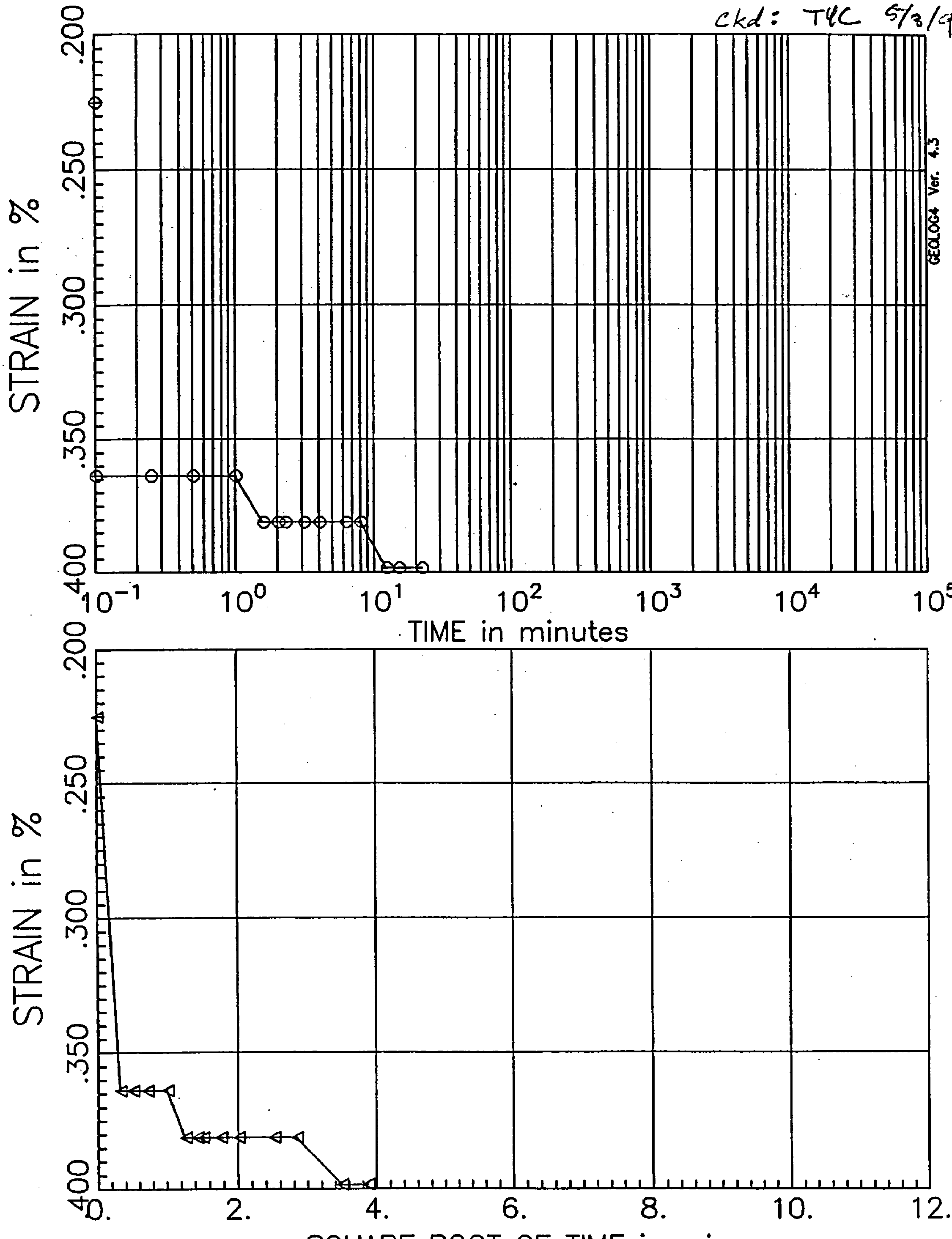
Test No: 1
Testname: CTBN-U2D

JO 05996.02

$d_{100} = 0.38\%$

Calc: ACS 4/23/99

Ckd: TYC 5/3/99



PRESSURE INCREMENT #2
from 0.10 tsf to 0.25 tsf

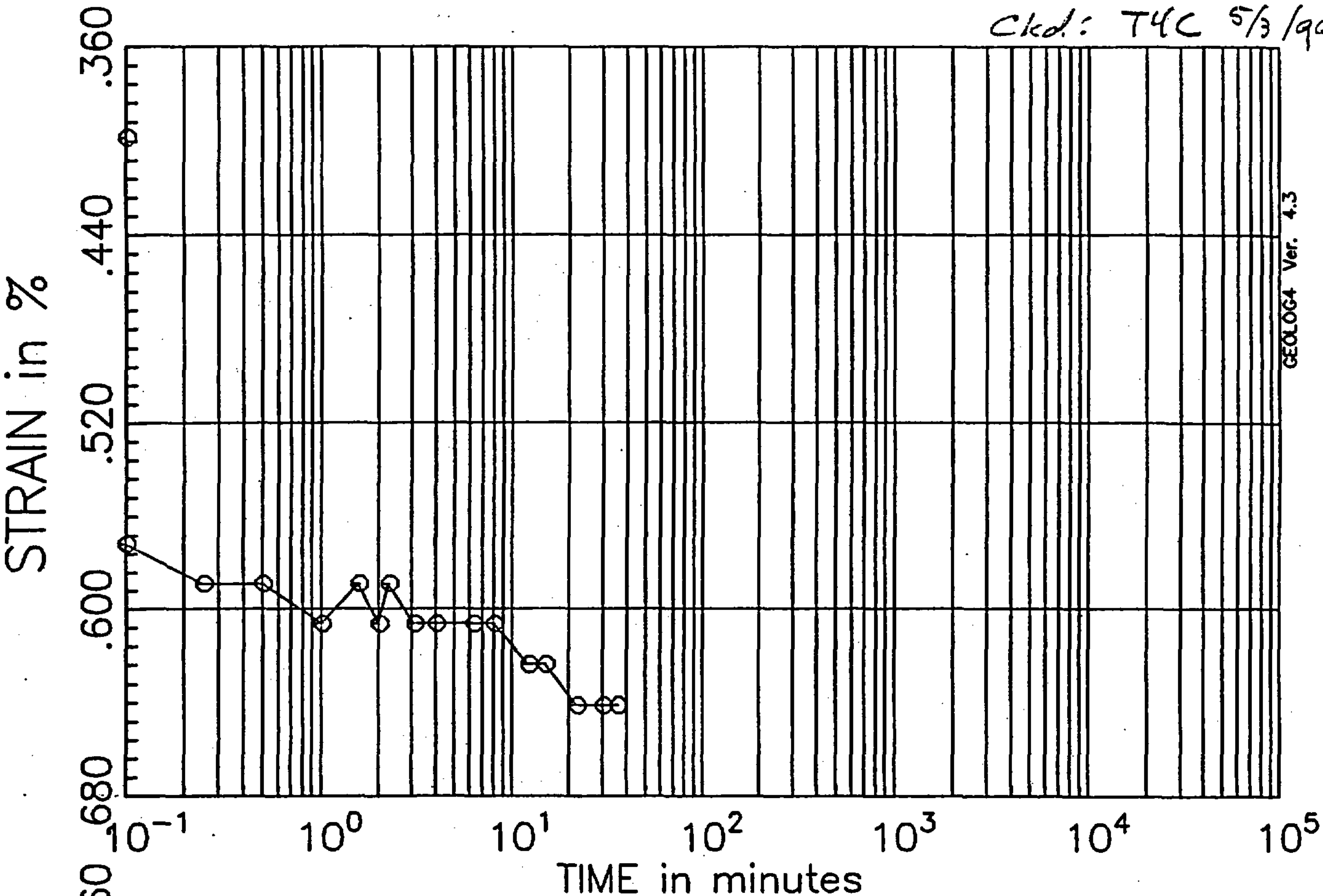
Test No: 1
Testname: CTBN-U2D

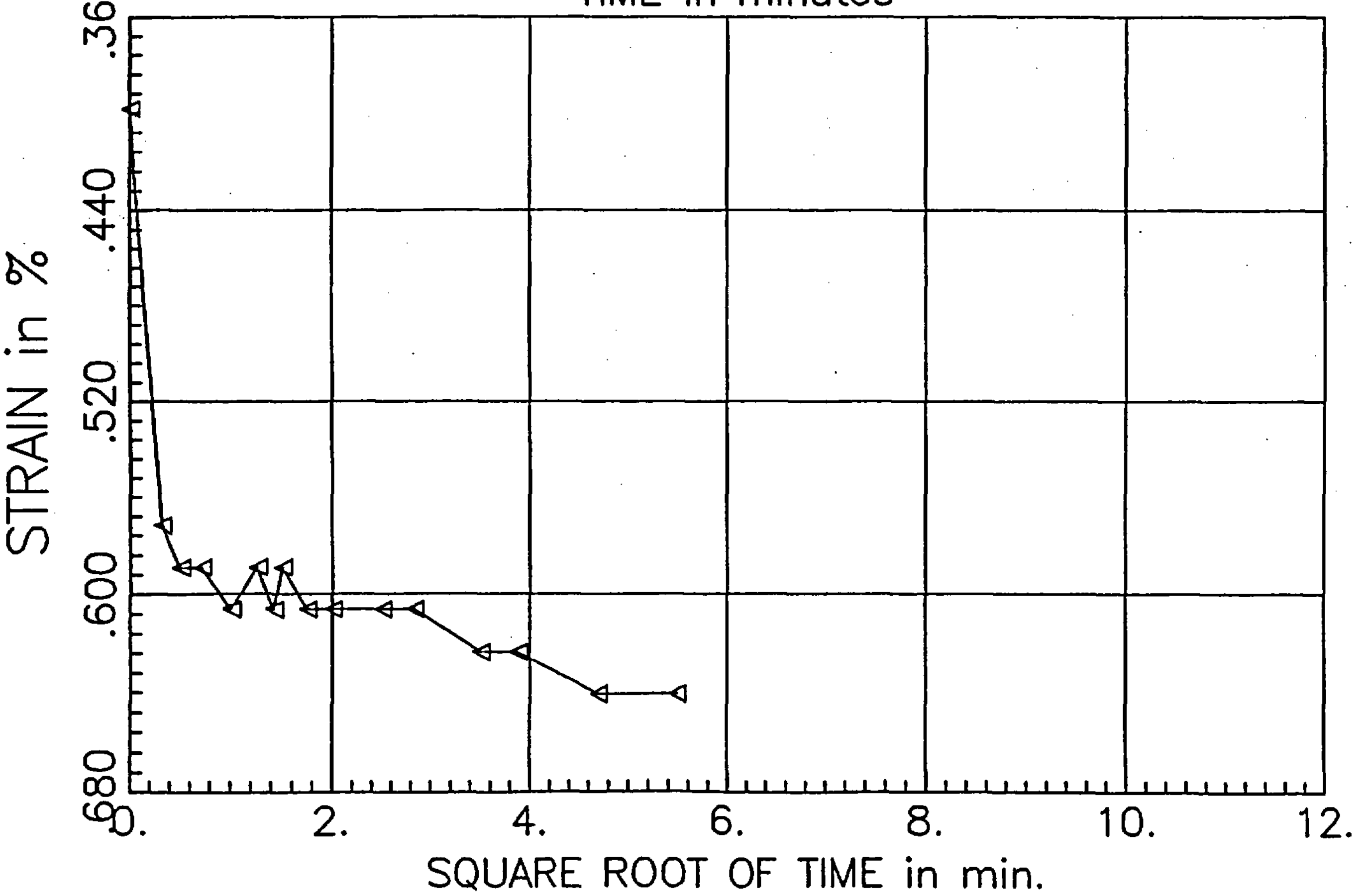
Jo 05996.02

$d_{100} = 0.61\%$

Calc: ACS 4/23/99

Ckd: T4C 5/3/99





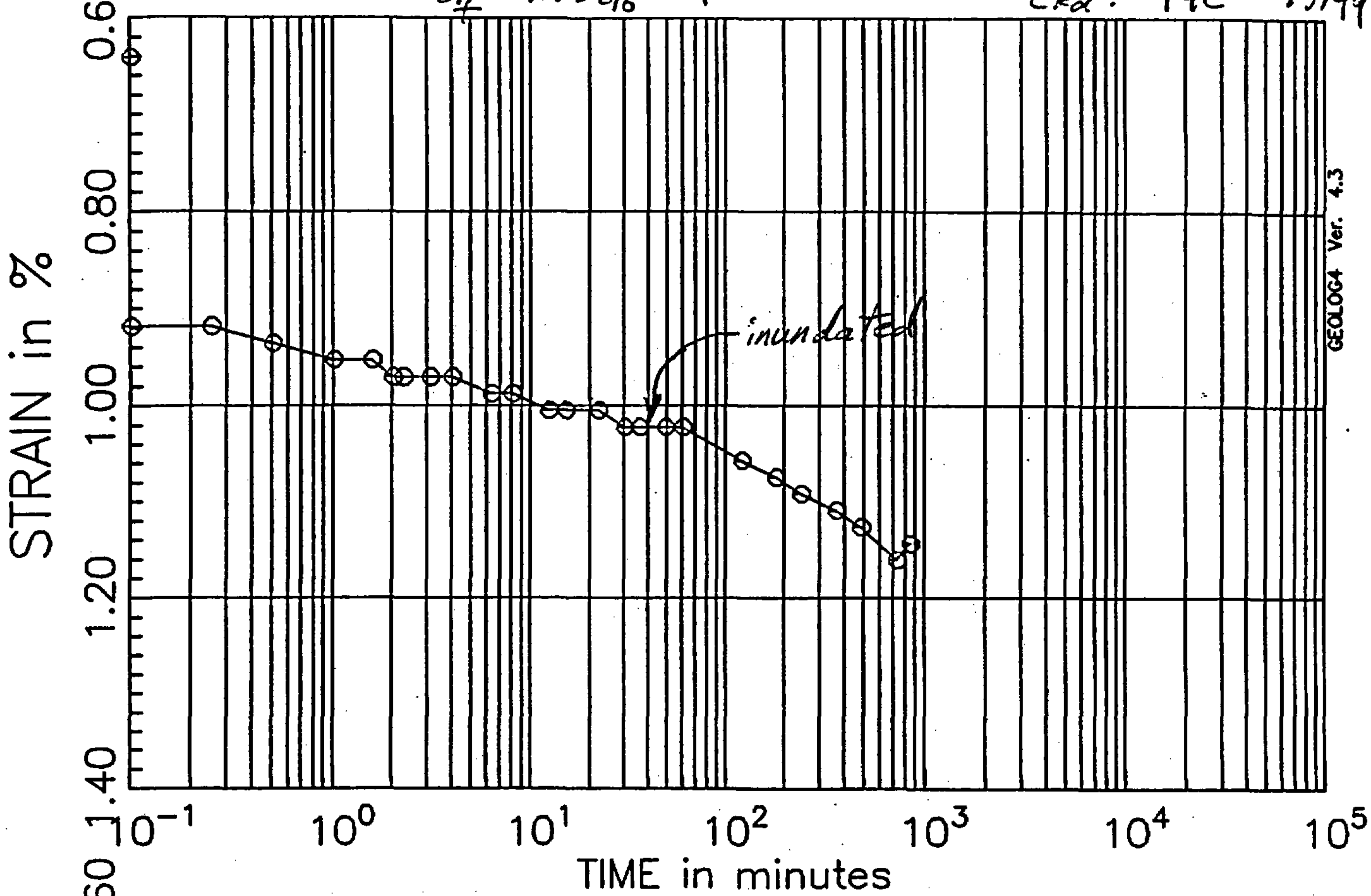
PRESSURE INCREMENT #3
from 0.25 tsf to 0.50 tsf

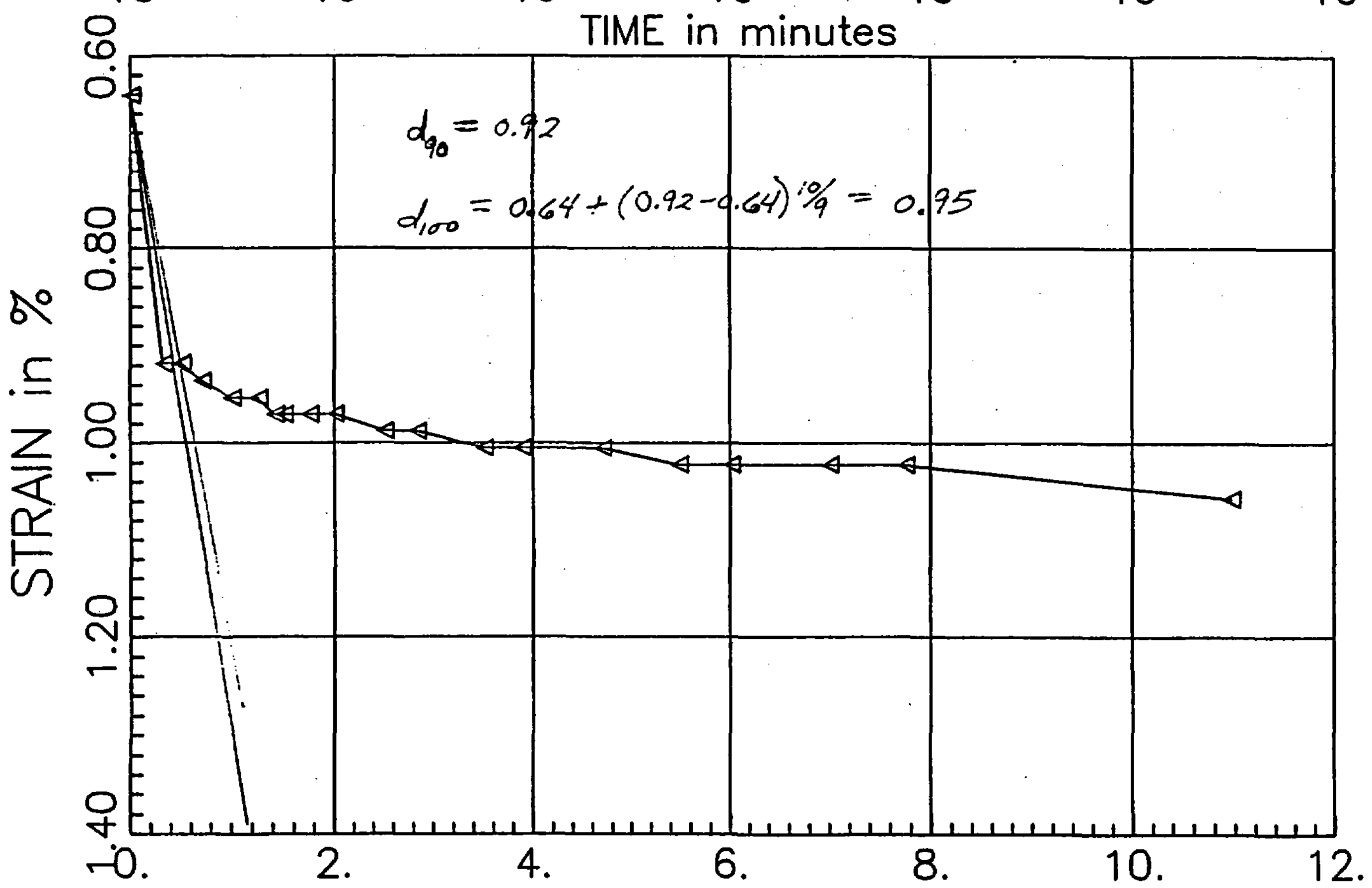
Test No: 1
Testname: CTBN-U2D

Jo 05996.02.
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

$d_{100} = 0.95\%$

$d_f = 1.15\%$ after inundation





PRESSURE INCREMENT #4
from 0.50 tsf to 1.00 tsf

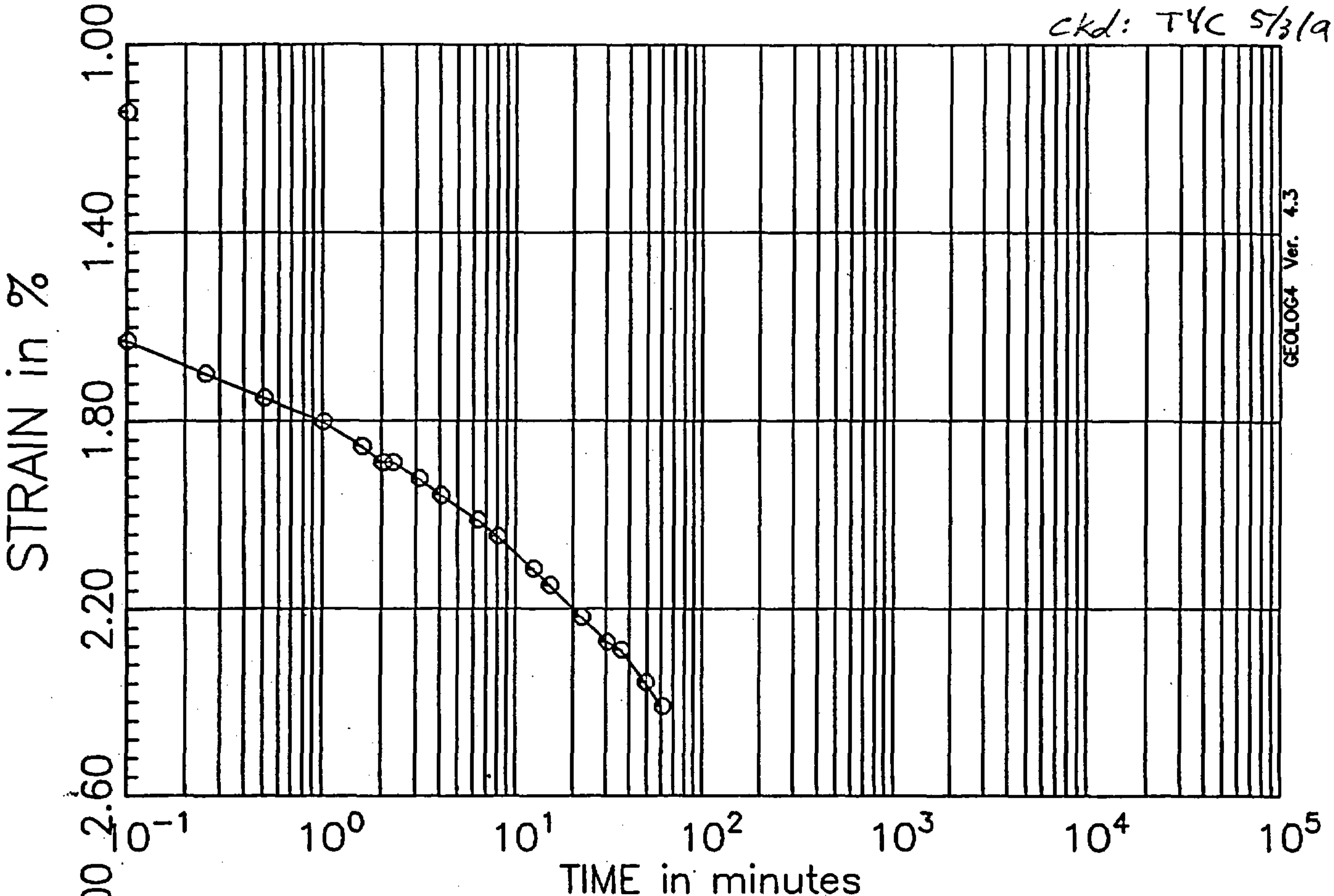
Test No: 1
Testname: CTBN–U2D

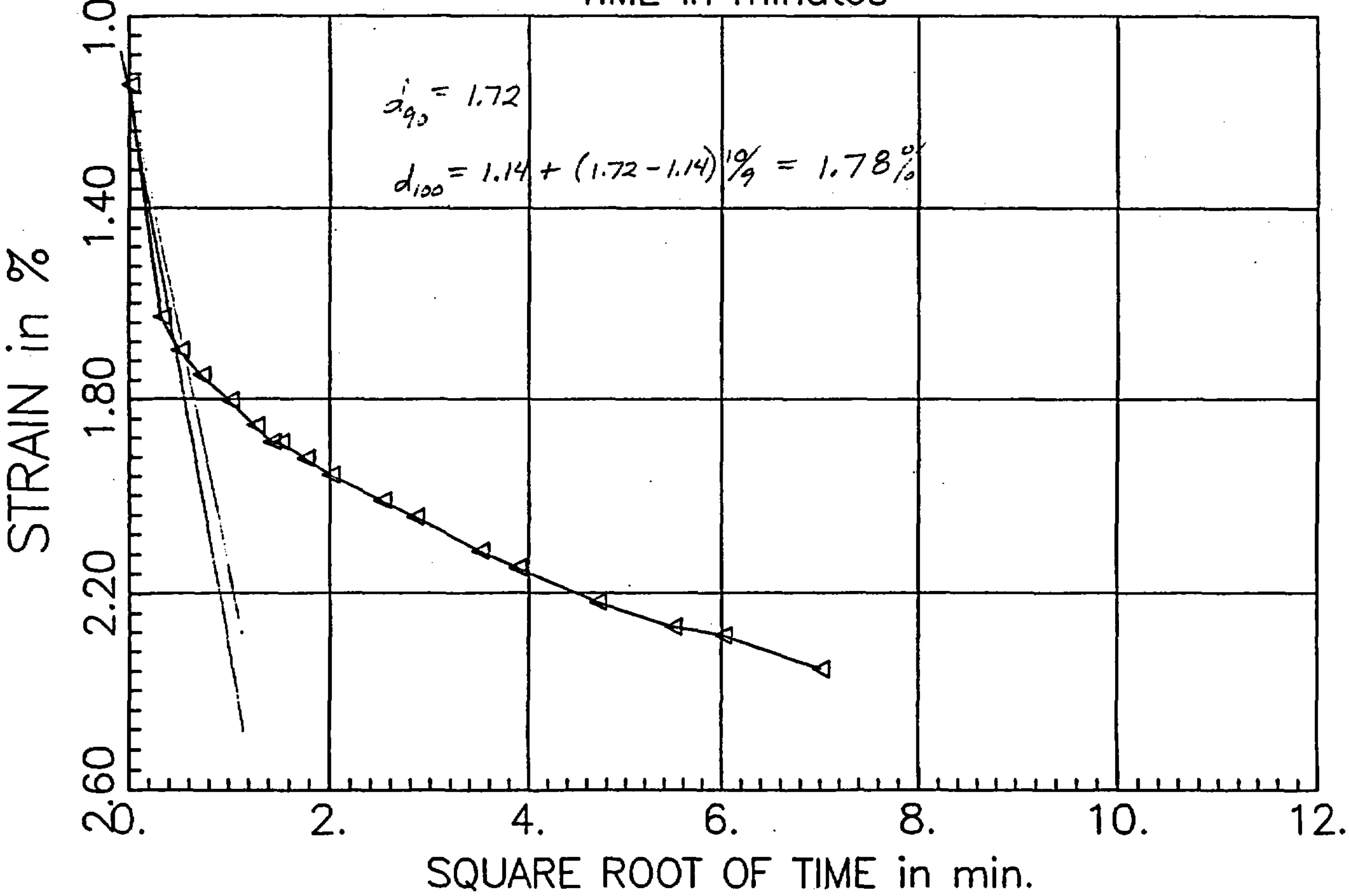
Jo 05996.02

Calc: ACS 4/23/99

Ckd: TYC 5/3/99

$d_{100} = 1.78\%$



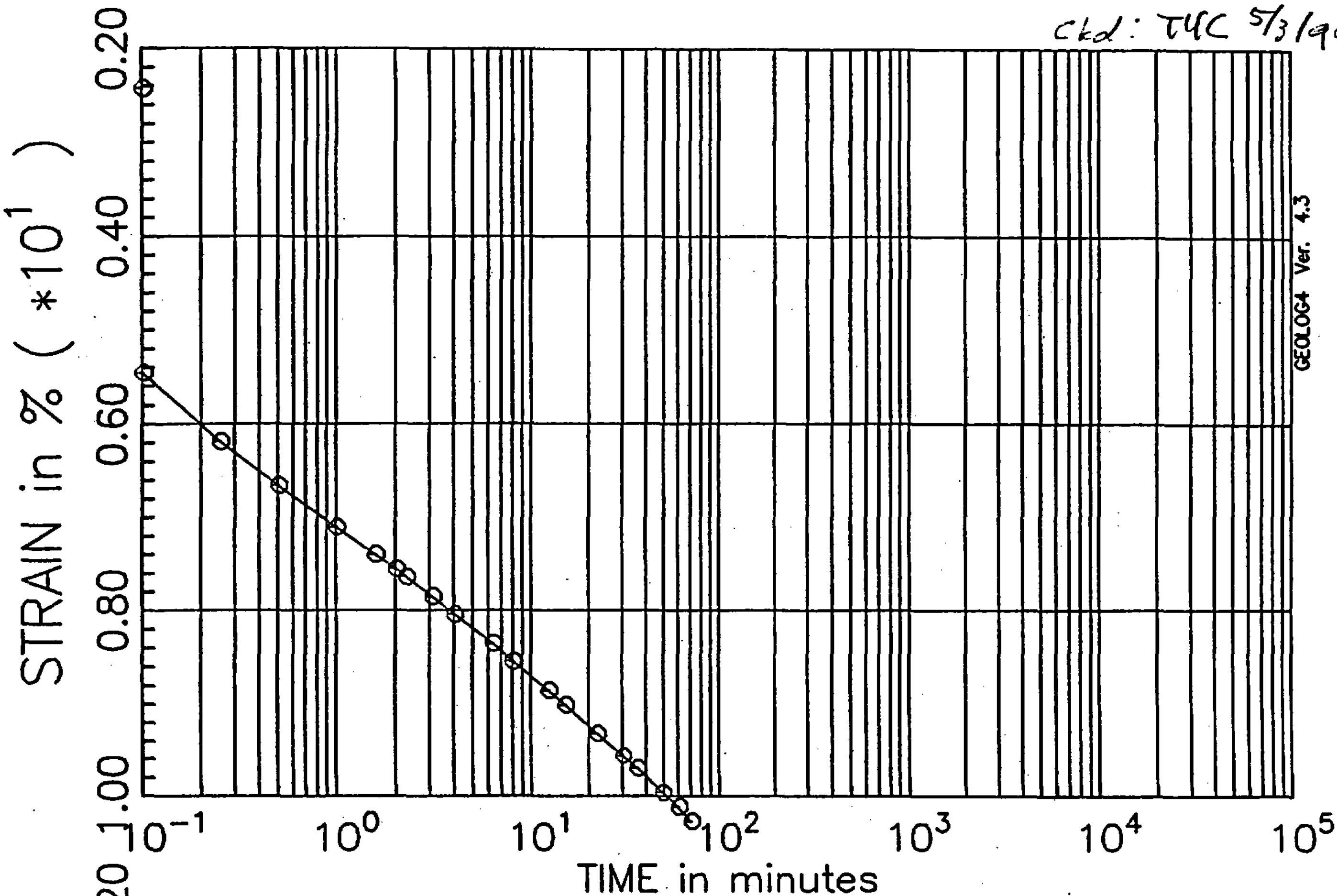


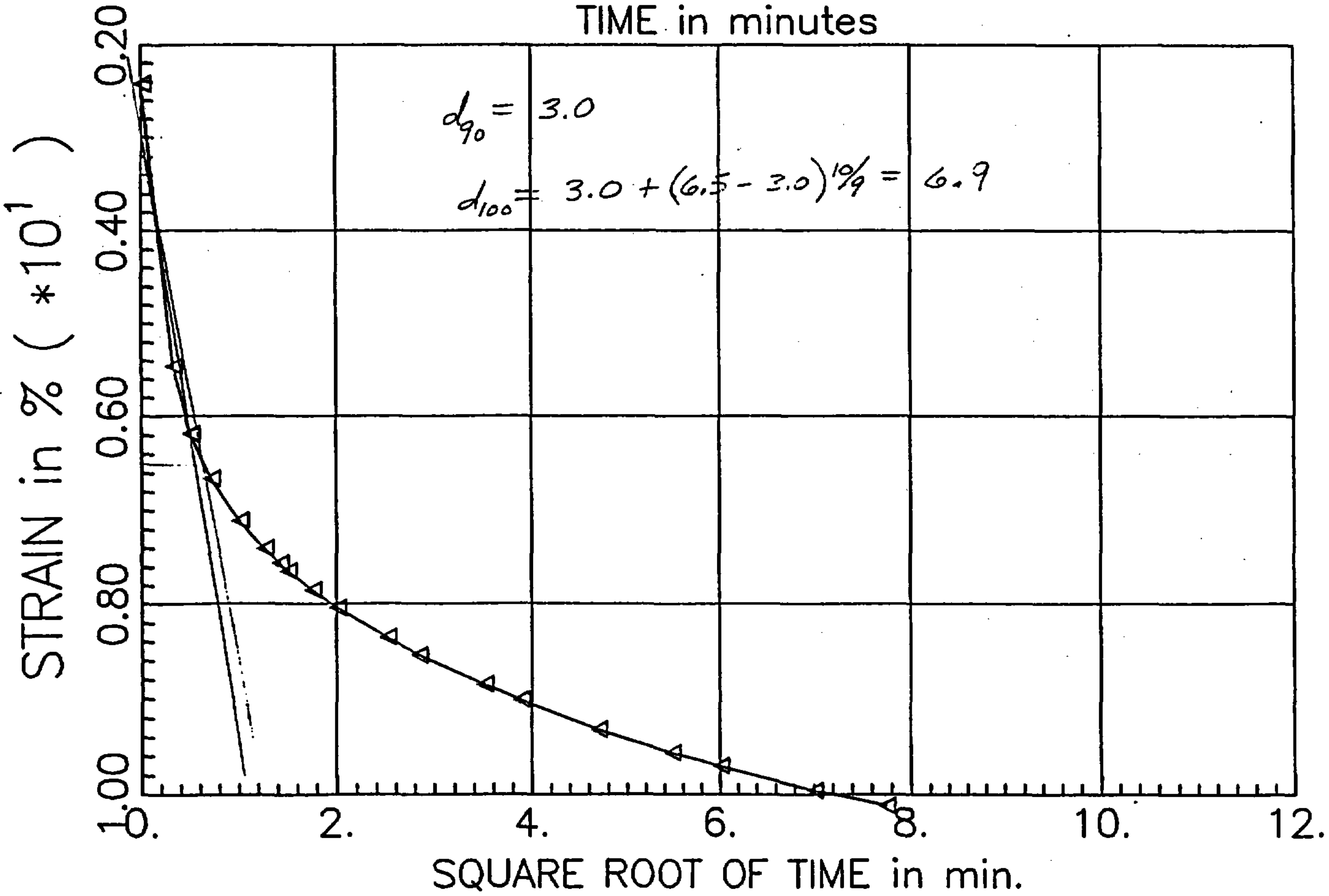
PRESSURE INCREMENT #5
from 1.00 tsf to 2.00 tsf

Test No: 1
Testname: CTBN–U2D

Jo 05996.02
Calc: ACS 4/23/99
Ckd: TMC 5/3/99

$d_{100} = 6.9\ \%$



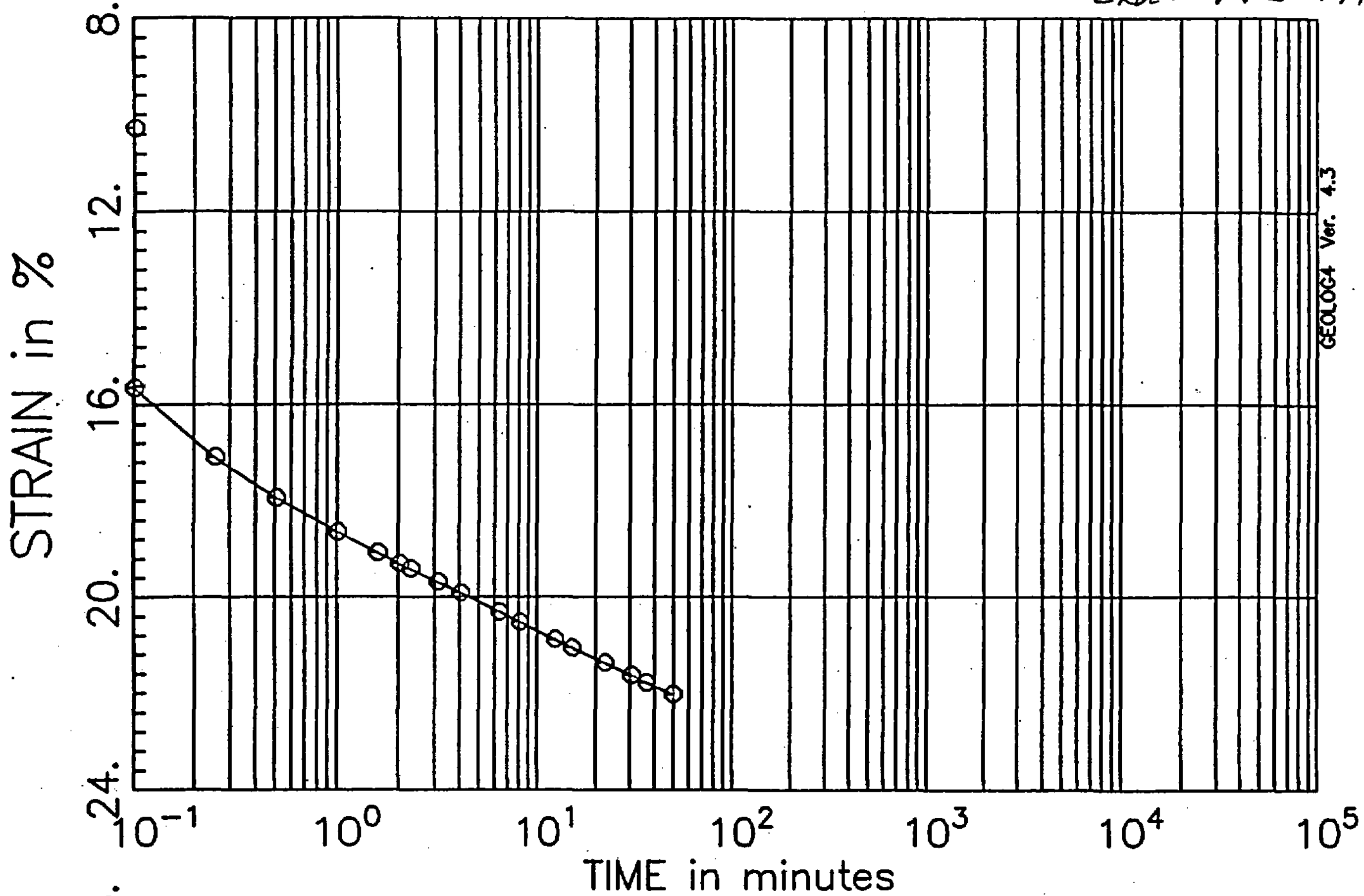


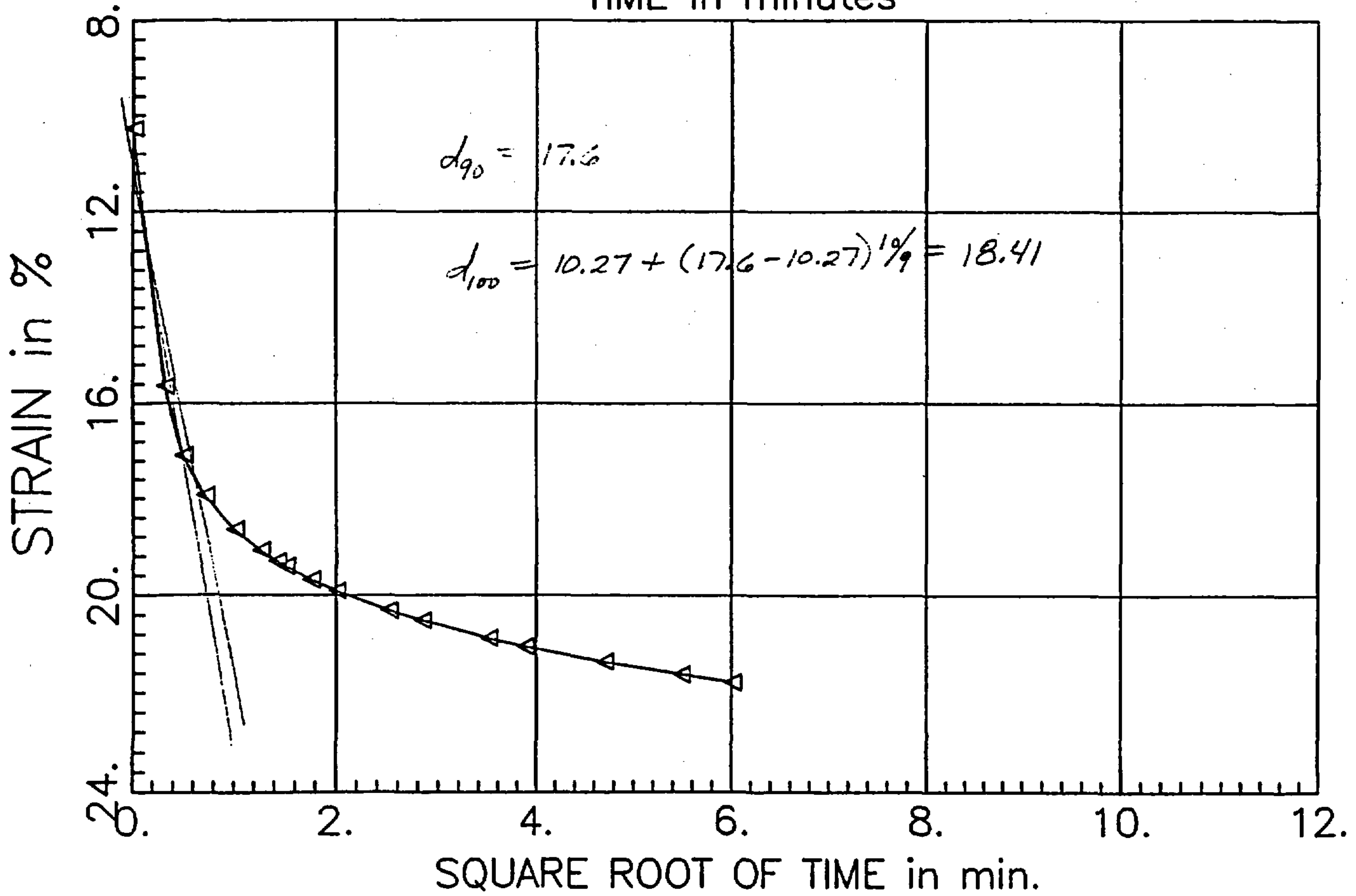
PRESSURE INCREMENT #6
from 2.00 tsf to 4.00 tsf

Test No: 1
Testname: CTBN-U2D

JO 05996.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

$d_{100} = 18.41\%$





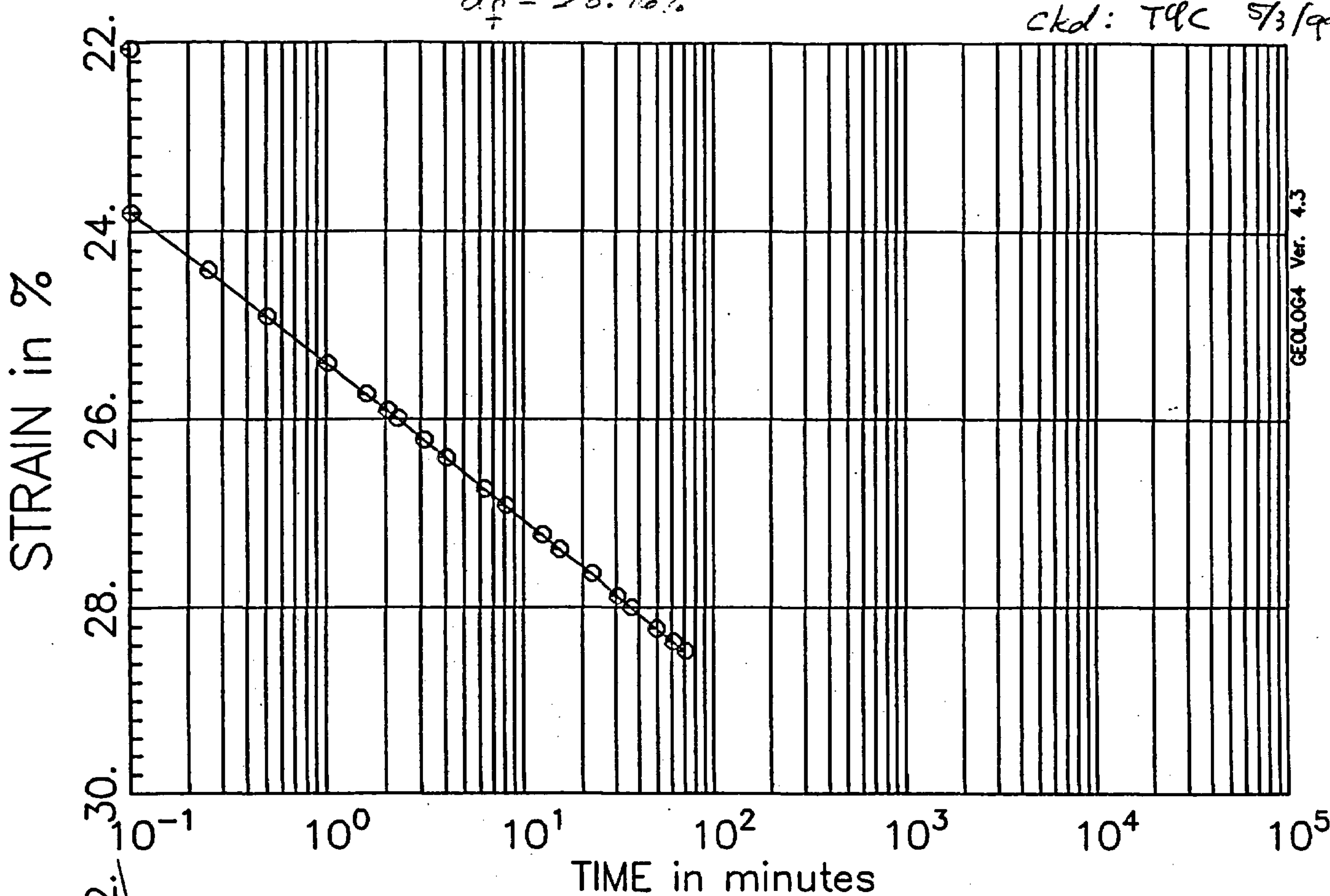
PRESSURE INCREMENT #7
from 4.00 tsf to 8.00 tsf

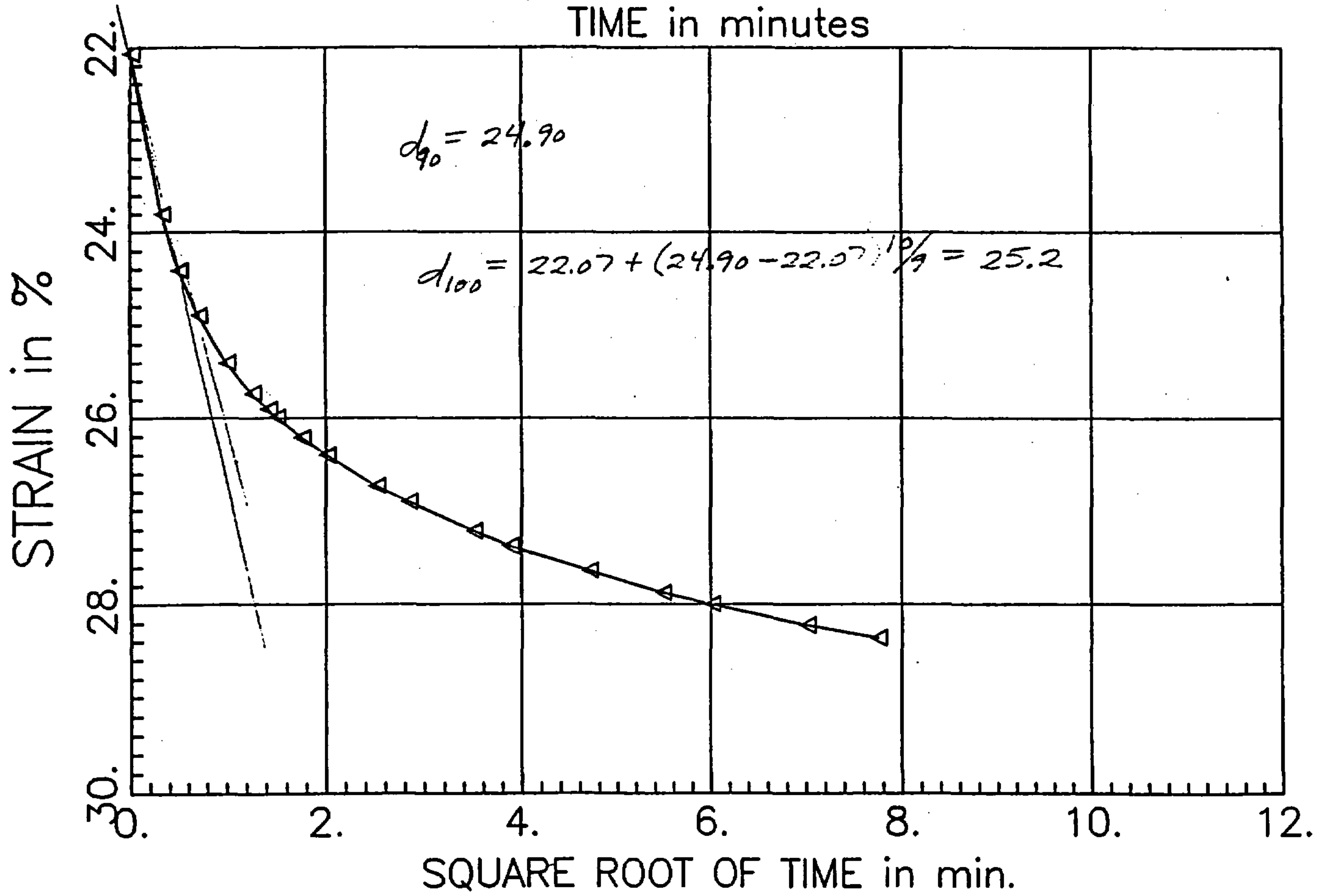
Test No: 1
Testname: CTBN–U2D

$d_{100} = 25.2\%$

$d_f = 28.46\%$

JO 05996.02
Calc: ACS 4/23/99
Ckd: TQC 5/3/99



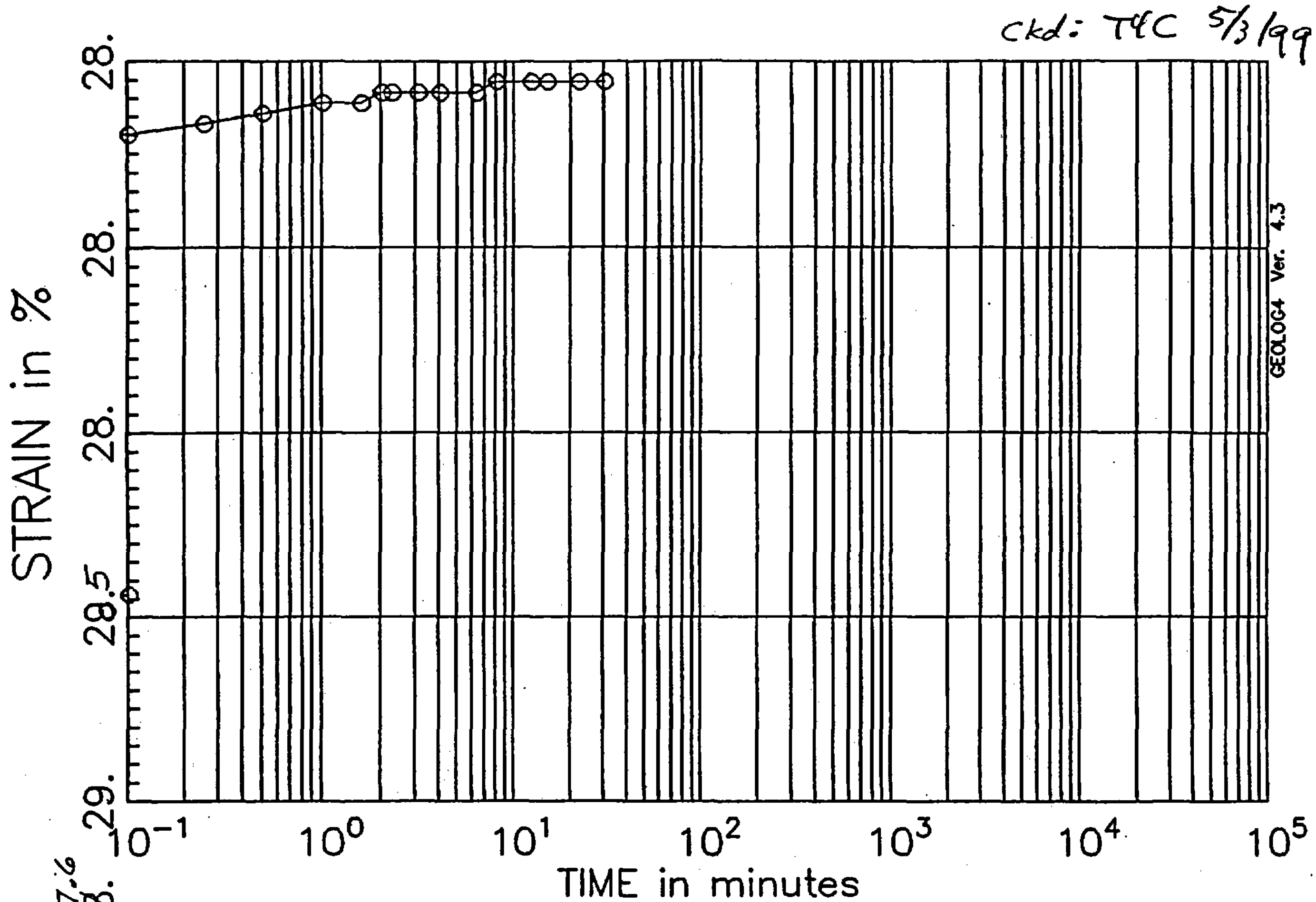


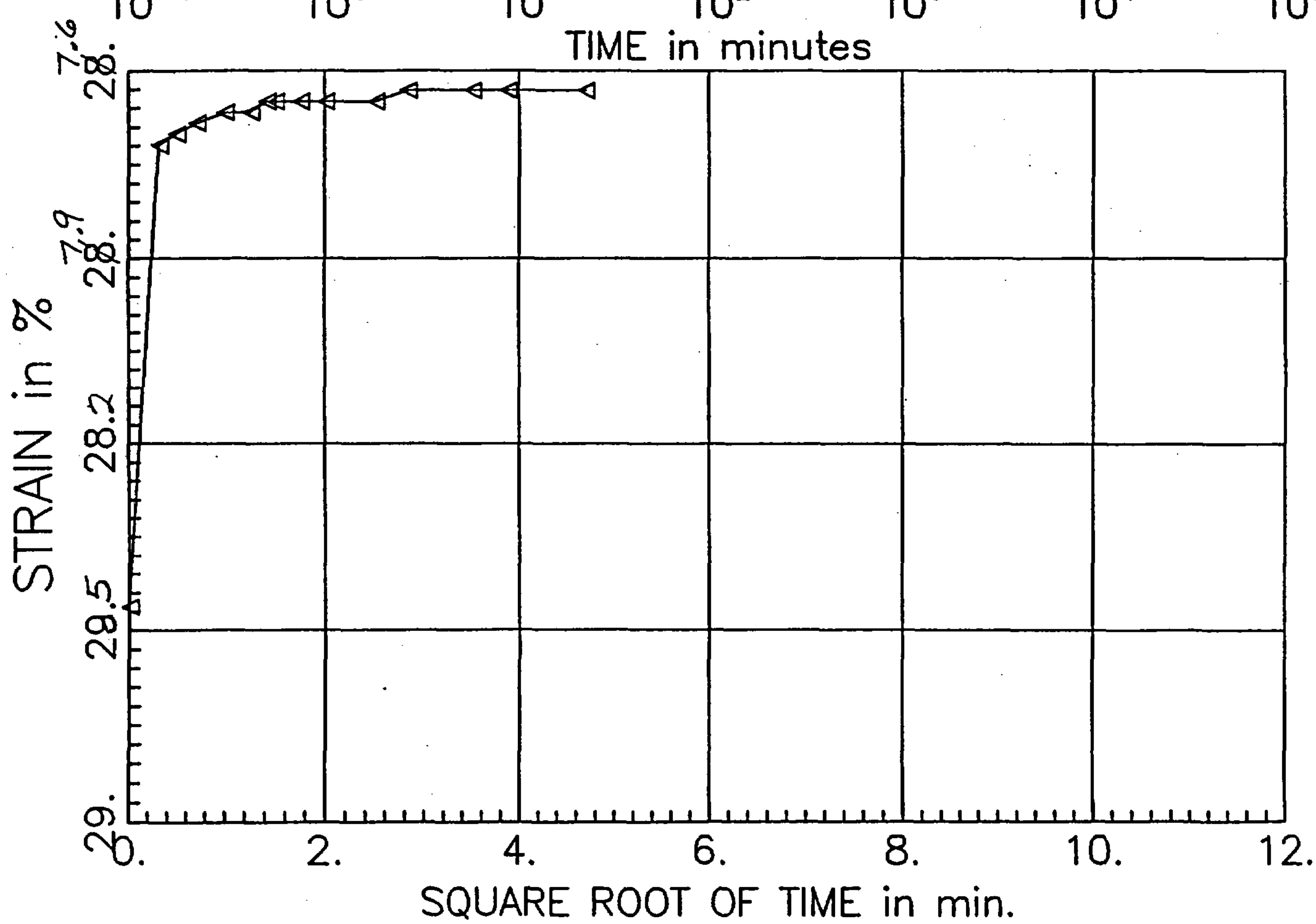
PRESSURE INCREMENT #8
from 8.00 tsf to 12.00 tsf

Test No: 1
Testname: CTBN-U2D

Jo 05996.02
Calc: ACS 4/23/99
Ckd: T&C 5/3/99

$d_{100} = 27.7\%$





PRESSURE INCREMENT #9
from 12.00 tsf to 4.00 tsf

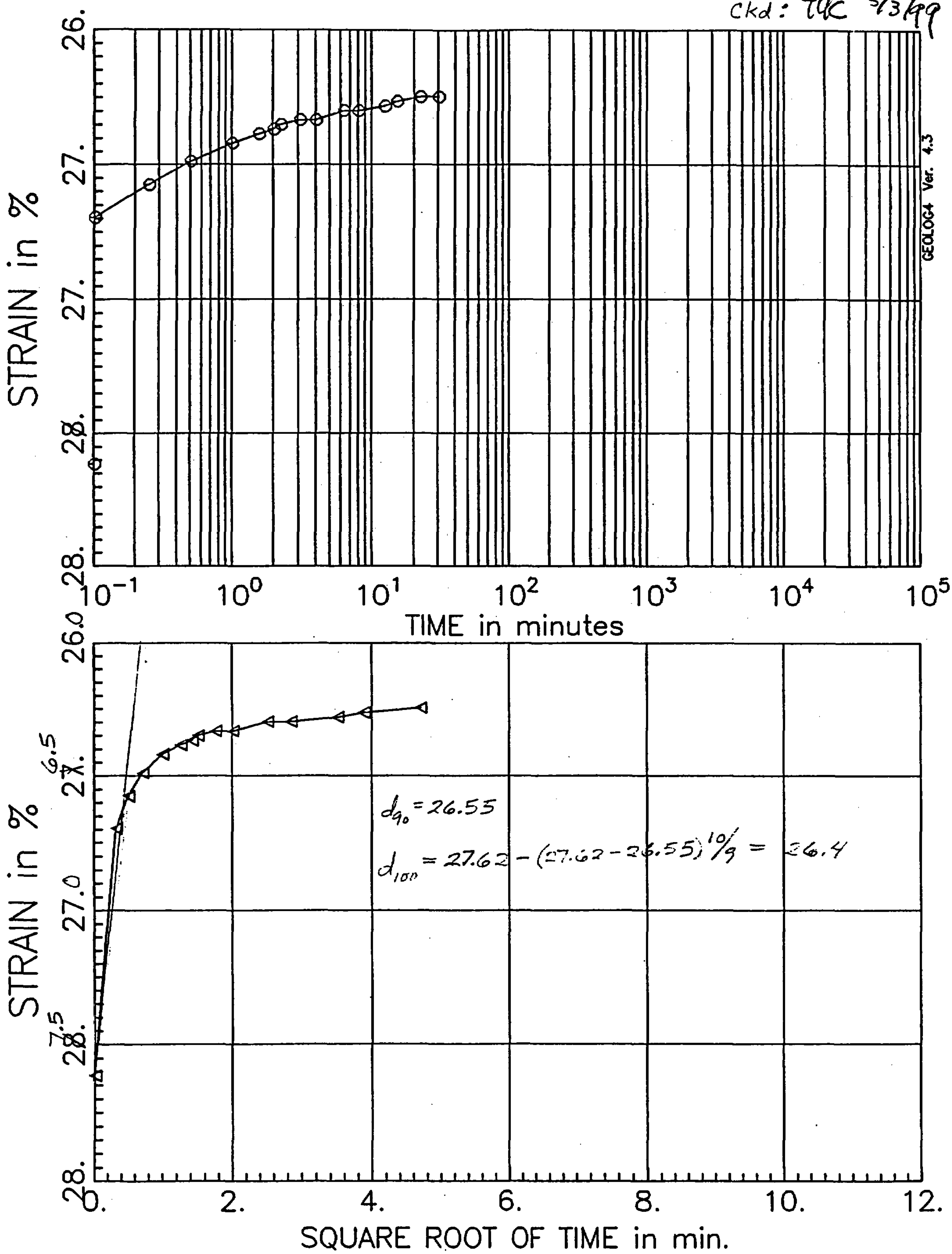
Test No: 1
Testname: CTBN-U2D

JO 05776.02

$d_{100} = 26.7\%$

Calc: ACS 4/23/99

Ckd: TKC 5/3/99



$d_{90} = 26.55$

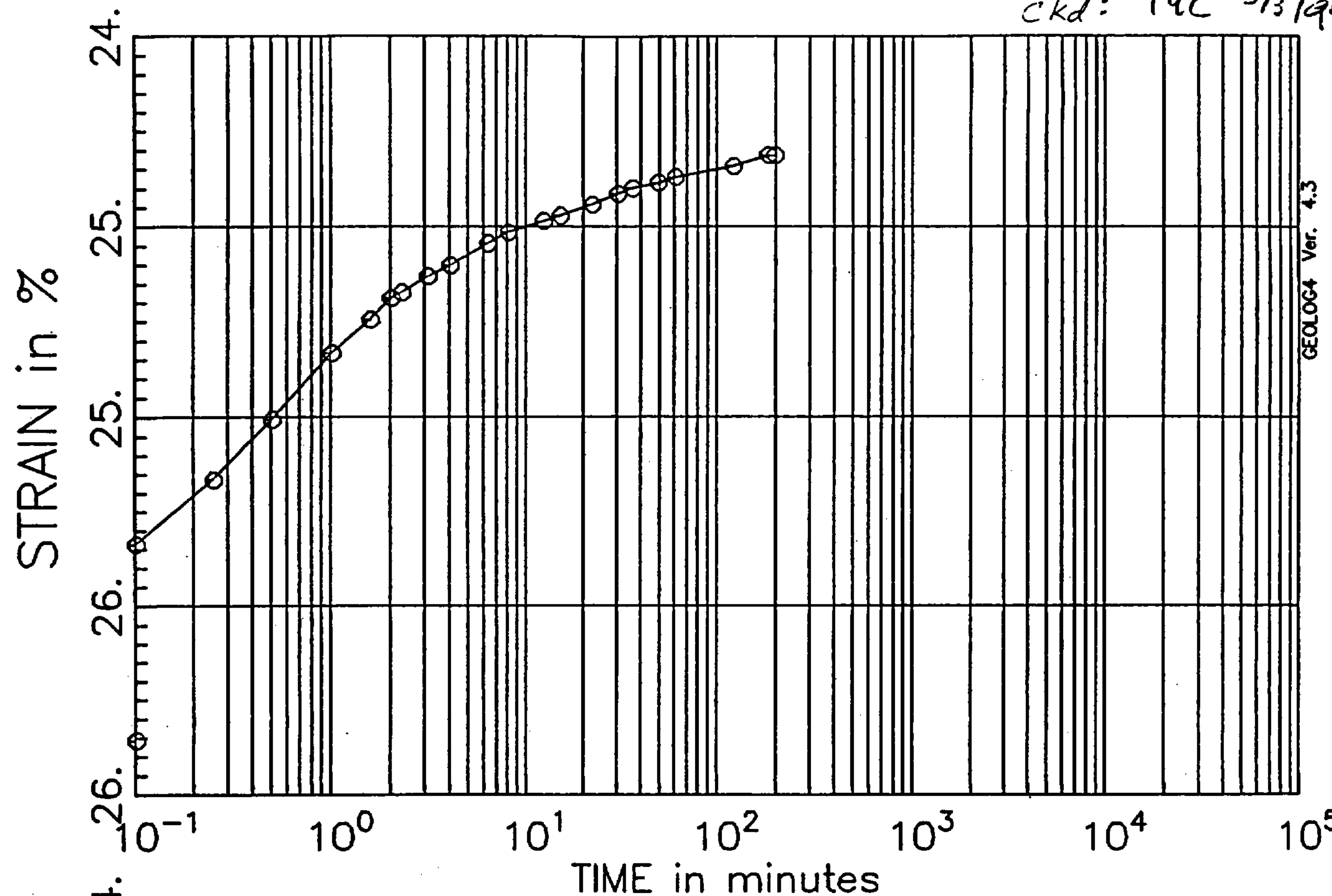
$d_{100} = 27.62 - (27.62 - 26.55)\, ^{10}/_{9} = 26.4$

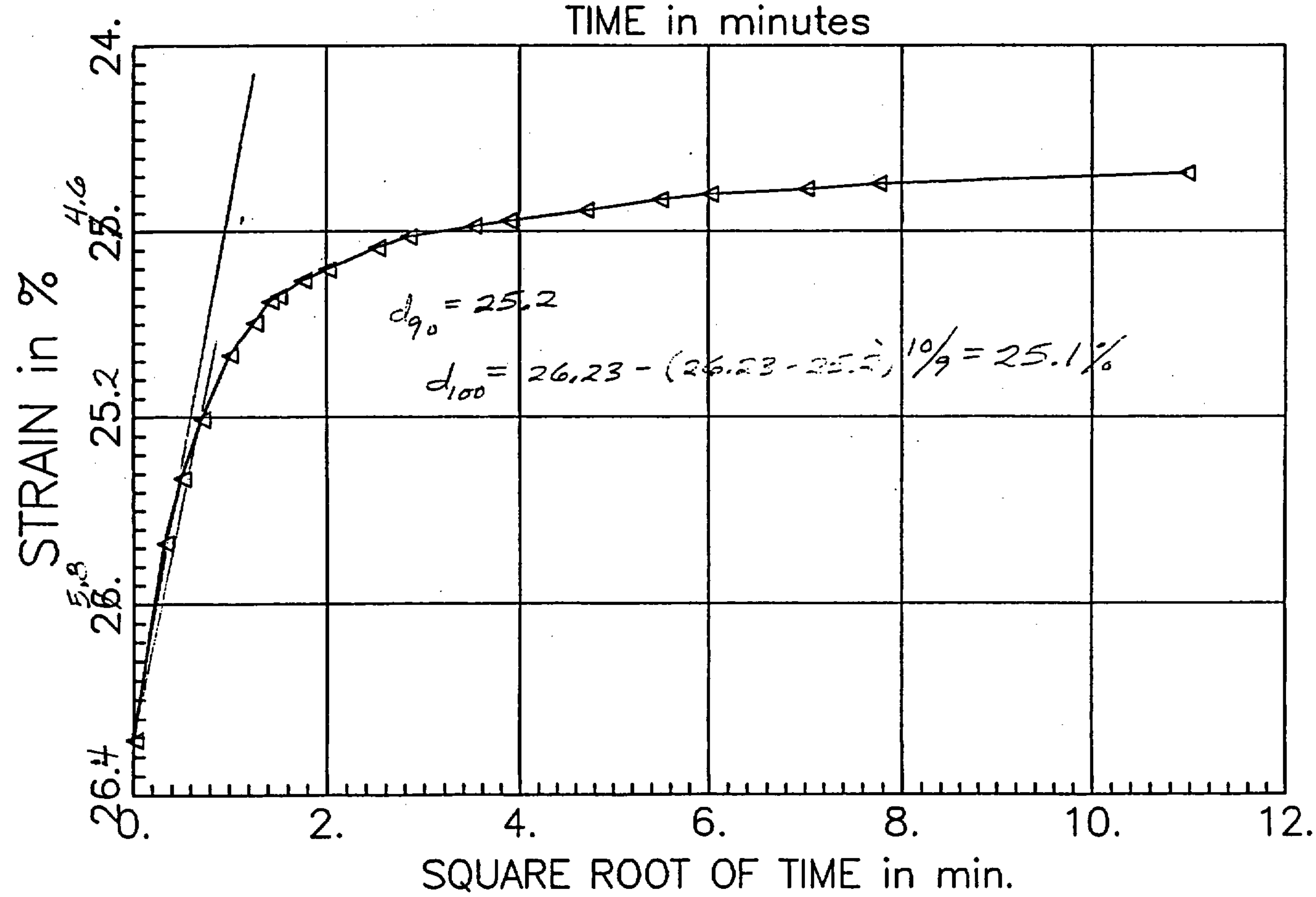
PRESSURE INCREMENT #10
from 4.00 tsf to 1.00 tsf

Test No: 1
Testname: CTBN-U2D

Jo 05996.02
Calc: ACS 4/23/99
Ckd: TYC 5/3/99

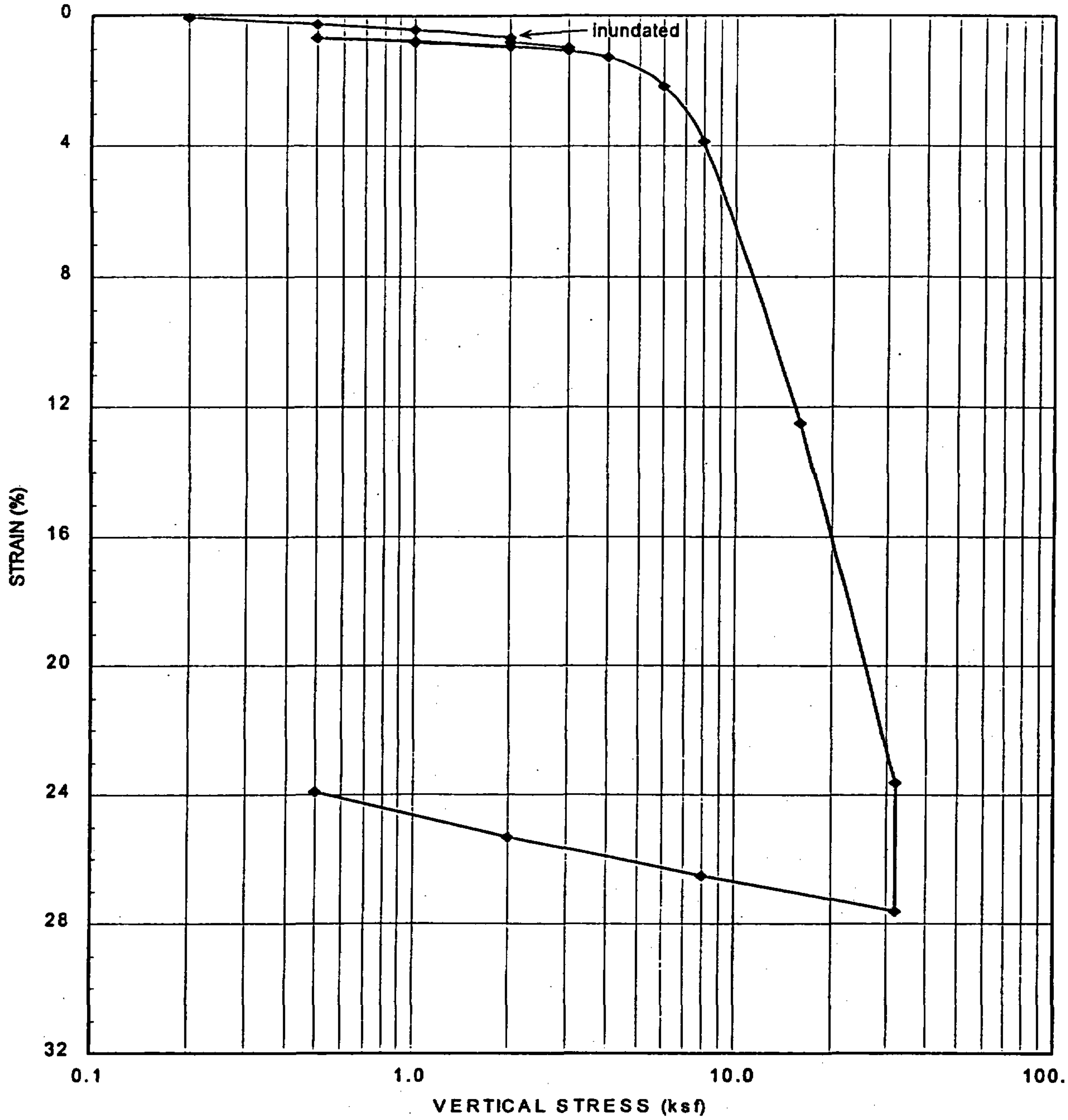
$d_{100} = 25.1\%$





PRESSURE INCREMENT #11
from 1.00 tsf to 0.25 tsf

Test No: 1
Testname: CTBN-U2D



SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	4/21/99
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	10 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	72.2 %	54.4 %
DRY UNIT WEIGHT:	51.9 pcf	67.4 pcf
VOID RATIO:	2.269	1.519
SATURATION:	86.6 %	97.4 %

SPECIFIC GRAVITY: 2.72 (est)

NOTE: Sample was inundated 34 minutes after applying a vertical stress of 2 ksf

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATION TEST RESULTS BORING CTB-S, SAMPLE U-3C	JO 05996.02 April 1999

CONSOLIDATION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	4/21/99
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	10.0 ft	CHECKED:	TYC
DESCRIPTION:	SILT (MH)		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	72.2 %	54.4 %
DRY UNIT WEIGHT:	51.9 pcf	67.4 pcf
VOID RATIO:	2.269	1.519
SATURATION:	86.6 %	97.4 %
HEIGHT:	1.894 cm	1.459 cm
AREA:	31.61 sq cm	
SP. GRAVITY :	2.72 (est)	

TEST DATA:

APPLIED PRESSURE ksf	STRAIN %	
0.20	0.10	
0.50	0.26	
1.00	0.43	
2.00	0.68	
2.00	0.80	(after inundation)
3.00	0.96	
3.00	1.06	
1.00	0.80	
0.50	0.68	
1.00	0.76	
2.00	0.92	
4.00	1.29	
6.00	2.18	
8.00	3.86	
16.00	12.53	
32.00	23.63	
32.00	27.62	
8.00	26.50	
2.00	25.32	
0.50	23.90	

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 1
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.00 tsf to 0.10 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	14:29:27	0.10	0.32	0.0006	2.266	0.09
		0.25	0.50	0.0006	2.266	0.09
		0.50	0.71	0.0008	2.266	0.10
		1.00	1.00	0.0008	2.266	0.10
		1.57	1.25	0.0008	2.266	0.10
		2.00	1.41	0.0008	2.266	0.10
		2.25	1.50	0.0008	2.266	0.10
		3.07	1.75	0.0008	2.266	0.10
		4.00	2.00	0.0008	2.266	0.10
		6.25	2.50	0.0008	2.266	0.10
		8.00	2.83	0.0008	2.266	0.10
		12.25	3.50	0.0008	2.266	0.10
		15.00	3.87	0.0008	2.266	0.10

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 2

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.10 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	14:50:03	0.00	0.00	0.0009	2.265	0.12
		0.10	0.32	0.0018	2.261	0.24
		0.25	0.50	0.0018	2.261	0.24
		0.50	0.71	0.0018	2.261	0.24
		1.00	1.00	0.0019	2.261	0.26
		1.57	1.25	0.0018	2.261	0.24
		2.00	1.41	0.0018	2.261	0.24
		2.25	1.50	0.0018	2.261	0.24
		3.07	1.75	0.0018	2.261	0.24
		4.00	2.00	0.0018	2.261	0.24
		6.25	2.50	0.0019	2.261	0.26
		8.00	2.83	0.0019	2.261	0.26
		12.25	3.50	0.0019	2.261	0.26
		15.00	3.87	0.0019	2.261	0.26
		22.00	4.69	0.0019	2.261	0.26
		30.00	5.48	0.0019	2.261	0.26

LOAD INCREMENT DATA

TEST NAME CTBS-U3C

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	15:26:46	0.00	0.00	0.0019	2.261	0.26
		0.10	0.32	0.0030	2.256	0.40
		0.25	0.50	0.0031	2.255	0.42
		0.50	0.71	0.0031	2.255	0.42
		1.00	1.00	0.0031	2.255	0.42
		1.57	1.25	0.0031	2.255	0.42
		2.00	1.41	0.0031	2.255	0.42
		2.25	1.50	0.0031	2.255	0.42
		3.07	1.75	0.0031	2.255	0.42
		4.00	2.00	0.0031	2.255	0.42
		6.25	2.50	0.0032	2.255	0.43
		8.00	2.83	0.0032	2.255	0.43
		12.25	3.50	0.0032	2.255	0.43
		15.00	3.87	0.0032	2.255	0.43
		22.00	4.69	0.0032	2.255	0.43
		30.00	5.48	0.0034	2.254	0.45
		36.00	6.00	0.0034	2.254	0.45
		49.00	7.00	0.0034	2.254	0.45

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 4
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-22-99	16:18:33	0.00	0.00	0.0034	2.254	0.45
		0.10	0.32	0.0048	2.248	0.64
		0.25	0.50	0.0049	2.247	0.66
		0.50	0.71	0.0049	2.247	0.66
		1.00	1.00	0.0050	2.247	0.68
		1.57	1.25	0.0050	2.247	0.68
		2.00	1.41	0.0050	2.247	0.68
		2.25	1.50	0.0050	2.247	0.68
		3.07	1.75	0.0052	2.246	0.69
		4.00	2.00	0.0052	2.246	0.69
		6.25	2.50	0.0052	2.246	0.69
		8.00	2.83	0.0052	2.246	0.69
		12.25	3.50	0.0053	2.246	0.71
		15.00	3.87	0.0053	2.246	0.71
		22.00	4.69	0.0053	2.246	0.71
		30.00	5.48	0.0054	2.245	0.73
		36.00	6.00	0.0054	2.245	0.73
		49.00	7.00	0.0054	2.245	0.73
		60.00	7.75	0.0054	2.245	0.73
		120.00	10.95	0.0056	2.245	0.74
		180.00	13.42	0.0056	2.245	0.74
		240.00	15.49	0.0057	2.244	0.76
		360.00	18.97	0.0058	2.244	0.78
04-23-99	00:18:33	480.00	21.91	0.0058	2.244	0.78
		720.00	26.83	0.0059	2.243	0.80
		900.00	30.00	0.0059	2.243	0.80

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 5
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 1.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	07:54:28	0.00	0.00	0.0059	2.243	0.80
		0.10	0.32	0.0070	2.238	0.94
		0.25	0.50	0.0070	2.238	0.94
		0.50	0.71	0.0070	2.238	0.94
		1.00	1.00	0.0071	2.238	0.95
		1.57	1.25	0.0072	2.237	0.97
		2.00	1.41	0.0072	2.237	0.97
		2.25	1.50	0.0072	2.237	0.97
		3.07	1.75	0.0072	2.237	0.97
		4.00	2.00	0.0072	2.237	0.97
		6.25	2.50	0.0074	2.237	0.99
		8.00	2.83	0.0074	2.237	0.99
		12.25	3.50	0.0075	2.236	1.00
		15.00	3.87	0.0075	2.236	1.00
		22.00	4.69	0.0076	2.236	1.02
		30.00	5.48	0.0076	2.236	1.02
		36.00	6.00	0.0078	2.235	1.04
		49.00	7.00	0.0078	2.235	1.04

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 6

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.50 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	08:48:45	0.00	0.00	0.0079	2.234	1.06
		0.10	0.32	0.0061	2.242	0.81
		0.25	0.50	0.0061	2.242	0.81
		0.50	0.71	0.0059	2.243	0.80
		1.00	1.00	0.0059	2.243	0.80
		1.57	1.25	0.0059	2.243	0.80
		2.00	1.41	0.0059	2.243	0.80
		2.25	1.50	0.0059	2.243	0.80
		3.07	1.75	0.0059	2.243	0.80
		4.00	2.00	0.0059	2.243	0.80
		6.25	2.50	0.0059	2.243	0.80
		8.00	2.83	0.0059	2.243	0.80
		12.25	3.50	0.0059	2.243	0.80
		15.00	3.87	0.0059	2.243	0.80
		22.00	4.69	0.0059	2.243	0.80

LOAD INCREMENT DATA

TEST NAME CTBS-U3C

PAGE NO: 7

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	09:17:33	0.00	0.00	0.0058	2.244	0.78
		0.10	0.32	0.0052	2.246	0.69
		0.25	0.50	0.0052	2.246	0.69
		0.50	0.71	0.0050	2.247	0.68
		1.00	1.00	0.0052	2.246	0.69
		1.57	1.25	0.0052	2.246	0.69
		2.00	1.41	0.0050	2.247	0.68
		2.25	1.50	0.0050	2.247	0.68
		3.07	1.75	0.0050	2.247	0.68
		4.00	2.00	0.0050	2.247	0.68
		6.25	2.50	0.0050	2.247	0.68
		8.00	2.83	0.0050	2.247	0.68
		12.25	3.50	0.0050	2.247	0.68
		15.00	3.87	0.0050	2.247	0.68
		22.00	4.69	0.0050	2.247	0.68

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 8

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.25 tsf to 0.50 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	09:43:07	0.00	0.00	0.0050	2.247	0.68
		0.10	0.32	0.0057	2.244	0.76
		0.25	0.50	0.0057	2.244	0.76
		0.50	0.71	0.0057	2.244	0.76
		1.00	1.00	0.0057	2.244	0.76
		1.57	1.25	0.0057	2.244	0.76
		2.00	1.41	0.0057	2.244	0.76
		2.25	1.50	0.0057	2.244	0.76
		3.07	1.75	0.0057	2.244	0.76
		4.00	2.00	0.0057	2.244	0.76
		6.25	2.50	0.0057	2.244	0.76
		8.00	2.83	0.0057	2.244	0.76
		12.25	3.50	0.0057	2.244	0.76
		15.00	3.87	0.0057	2.244	0.76
		22.00	4.69	0.0057	2.244	0.76

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 9
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 0.50 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	10:07:02	0.00	0.00	0.0057	2.244	0.76
		0.10	0.32	0.0067	2.240	0.90
		0.25	0.50	0.0068	2.239	0.92
		0.50	0.71	0.0067	2.240	0.90
		1.00	1.00	0.0068	2.239	0.92
		1.57	1.25	0.0068	2.239	0.92
		2.00	1.41	0.0068	2.239	0.92
		2.25	1.50	0.0068	2.239	0.92
		3.07	1.75	0.0068	2.239	0.92
		4.00	2.00	0.0068	2.239	0.92
		6.25	2.50	0.0068	2.239	0.92
		8.00	2.83	0.0068	2.239	0.92
		12.25	3.50	0.0068	2.239	0.92
		15.00	3.87	0.0068	2.239	0.92
		22.00	4.69	0.0068	2.239	0.92

LOAD INCREMENT DATA

TEST NAME CTBS-U3C

PAGE NO: 10

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 1.00 tsf to 2.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	10:37:05	0.00	0.00	0.0068	2.239	0.92
		0.10	0.32	0.0092	2.229	1.23
		0.25	0.50	0.0093	2.228	1.25
		0.50	0.71	0.0094	2.228	1.26
		1.00	1.00	0.0096	2.227	1.28
		1.57	1.25	0.0097	2.227	1.30
		2.00	1.41	0.0098	2.226	1.32
		2.25	1.50	0.0098	2.226	1.32
		3.07	1.75	0.0099	2.225	1.33
		4.00	2.00	0.0099	2.225	1.33
		6.25	2.50	0.0101	2.225	1.35
		8.00	2.83	0.0102	2.224	1.37
		12.25	3.50	0.0103	2.224	1.39
		15.00	3.87	0.0105	2.223	1.40
		22.00	4.69	0.0107	2.222	1.44
		30.00	5.48	0.0109	2.221	1.46
		36.00	6.00	0.0110	2.221	1.47
		49.00	7.00	0.0111	2.220	1.49
		60.00	7.75	0.0112	2.220	1.51

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 11

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 2.00 tsf to 3.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	11:47:04	0.00	0.00	0.0115	2.219	1.54
		0.10	0.32	0.0145	2.206	1.94
		0.25	0.50	0.0151	2.203	2.03
		0.50	0.71	0.0160	2.199	2.15
		1.00	1.00	0.0168	2.195	2.25
		1.57	1.25	0.0172	2.194	2.30
		2.00	1.41	0.0174	2.193	2.34
		2.25	1.50	0.0176	2.192	2.36
		3.07	1.75	0.0180	2.190	2.41
		4.00	2.00	0.0182	2.189	2.44
		6.25	2.50	0.0189	2.186	2.53
		8.00	2.83	0.0192	2.185	2.58
		12.25	3.50	0.0199	2.182	2.67
		15.00	3.87	0.0202	2.181	2.70
		22.00	4.69	0.0208	2.178	2.79
		30.00	5.48	0.0214	2.175	2.88
		36.00	6.00	0.0217	2.174	2.91
		49.00	7.00	0.0223	2.171	3.00
		60.00	7.75	0.0229	2.169	3.07

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 12
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 3.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	12:55:16	0.00	0.00	0.0231	2.168	3.10
		0.10	0.32	0.0266	2.152	3.57
		0.25	0.50	0.0276	2.148	3.71
		0.50	0.71	0.0285	2.144	3.83
		1.00	1.00	0.0296	2.139	3.97
		1.57	1.25	0.0305	2.135	4.09
		2.00	1.41	0.0310	2.133	4.16
		2.25	1.50	0.0313	2.132	4.19
		3.07	1.75	0.0319	2.129	4.28
		4.00	2.00	0.0326	2.126	4.37
		6.25	2.50	0.0337	2.121	4.52
		8.00	2.83	0.0345	2.118	4.63
		12.25	3.50	0.0358	2.112	4.80
		15.00	3.87	0.0366	2.109	4.90
		22.00	4.69	0.0379	2.103	5.08
		30.00	5.48	0.0390	2.098	5.23
		36.00	6.00	0.0398	2.095	5.34
		49.00	7.00	0.0411	2.089	5.51
		60.00	7.75	0.0420	2.085	5.63
		91.00	9.54	0.0439	2.076	5.89

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 13

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 8.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	14:27:15	0.00	0.00	0.0439	2.076	5.89
		0.10	0.32	0.0766	1.933	10.27
		0.25	0.50	0.0849	1.897	11.38
		0.50	0.71	0.0911	1.870	12.21
		1.00	1.00	0.0969	1.844	12.99
		1.57	1.25	0.1005	1.828	13.48
		2.00	1.41	0.1024	1.820	13.74
		2.25	1.50	0.1032	1.816	13.84
		3.07	1.75	0.1054	1.807	14.14
		4.00	2.00	0.1072	1.799	14.38
		6.25	2.50	0.1102	1.786	14.78
		8.00	2.83	0.1119	1.779	15.00
		12.25	3.50	0.1146	1.767	15.37
		15.00	3.87	0.1159	1.761	15.54
		22.00	4.69	0.1186	1.749	15.90
		30.00	5.48	0.1205	1.741	16.16
		36.00	6.00	0.1218	1.735	16.34
		49.00	7.00	0.1239	1.726	16.61
		60.00	7.75	0.1253	1.720	16.80

LOAD INCREMENT DATA

TEST NAME CTBS-U3C PAGE NO: 14

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 8.00 tsf to 16.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	15:46:00	0.00	0.00	0.1274	1.711	17.08
		0.10	0.32	0.1590	1.572	21.33
		0.25	0.50	0.1685	1.530	22.59
		0.50	0.71	0.1748	1.503	23.44
		1.00	1.00	0.1805	1.478	24.20
		1.57	1.25	0.1837	1.464	24.64
		2.00	1.41	0.1854	1.456	24.86
		2.25	1.50	0.1862	1.453	24.96
		3.07	1.75	0.1881	1.444	25.22
		4.00	2.00	0.1896	1.438	25.43
		6.25	2.50	0.1922	1.426	25.78
		8.00	2.83	0.1936	1.420	25.97
		12.25	3.50	0.1961	1.409	26.30
		15.00	3.87	0.1973	1.404	26.45
		22.00	4.69	0.1993	1.395	26.73
		30.00	5.48	0.2010	1.388	26.96
		36.00	6.00	0.2020	1.383	27.10
		49.00	7.00	0.2036	1.376	27.30
		60.00	7.75	0.2048	1.371	27.46
		75.00	8.66	0.2059	1.366	27.62

LOAD INCREMENT DATA

TEST NAME CTBS-U3C
TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 16.00 tsf to 4.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	17:01:44	0.00	0.00	0.2059	1.366	27.62
		0.10	0.32	0.1982	1.400	26.58
		0.25	0.50	0.1979	1.401	26.54
		0.50	0.71	0.1977	1.403	26.51
		1.00	1.00	0.1975	1.403	26.49
		1.57	1.25	0.1974	1.404	26.47
		2.00	1.41	0.1973	1.404	26.45
		2.25	1.50	0.1973	1.404	26.45
		3.07	1.75	0.1973	1.404	26.45
		4.00	2.00	0.1973	1.404	26.45
		6.25	2.50	0.1971	1.405	26.44
		8.00	2.83	0.1971	1.405	26.44
		12.25	3.50	0.1970	1.405	26.42
		15.00	3.87	0.1970	1.405	26.42
		22.00	4.69	0.1969	1.406	26.40
		30.00	5.48	0.1969	1.406	26.40

LOAD INCREMENT DATA

TEST NAME CTBS-U3C

TEST NO: 4 TESTED BY: ACS

PRESSURE INCREMENT FROM 4.00 tsf to 1.00 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	17:34:53	0.00	0.00	0.1969	1.406	26.40
		0.10	0.32	0.1904	1.434	25.54
		0.25	0.50	0.1896	1.438	25.43
		0.50	0.71	0.1887	1.442	25.31
		1.00	1.00	0.1880	1.445	25.21
		1.57	1.25	0.1874	1.447	25.14
		2.00	1.41	0.1872	1.448	25.10
		2.25	1.50	0.1871	1.449	25.09
		3.07	1.75	0.1868	1.450	25.05
		4.00	2.00	0.1865	1.451	25.02
		6.25	2.50	0.1863	1.452	24.98
		8.00	2.83	0.1862	1.453	24.96
		12.25	3.50	0.1859	1.454	24.93
		15.00	3.87	0.1858	1.455	24.91
		22.00	4.69	0.1856	1.455	24.90
		30.00	5.48	0.1855	1.456	24.88

LOAD INCREMENT DATA

TEST NAME CTBS-U3C

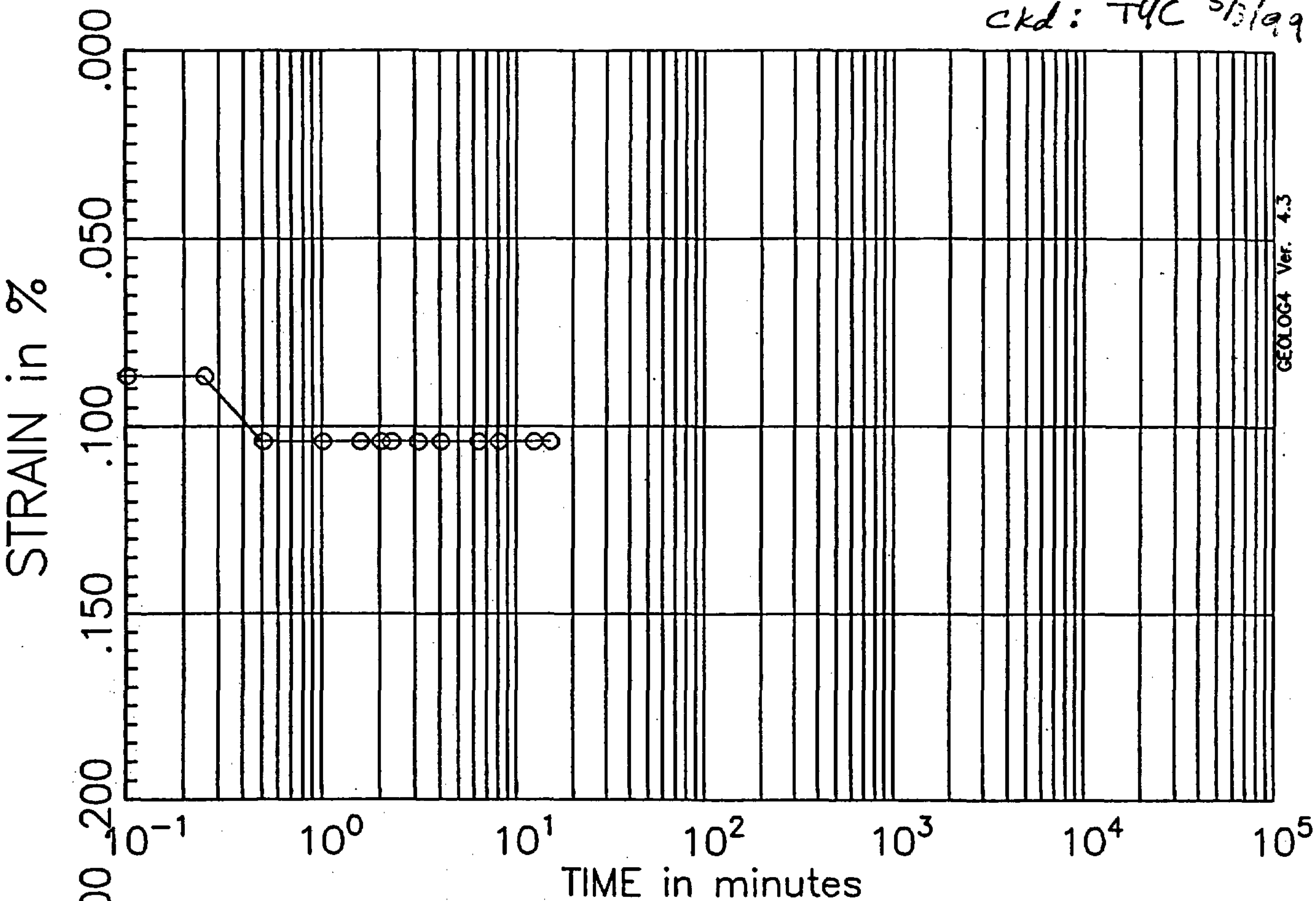
TEST NO: 4 TESTED BY: ACS

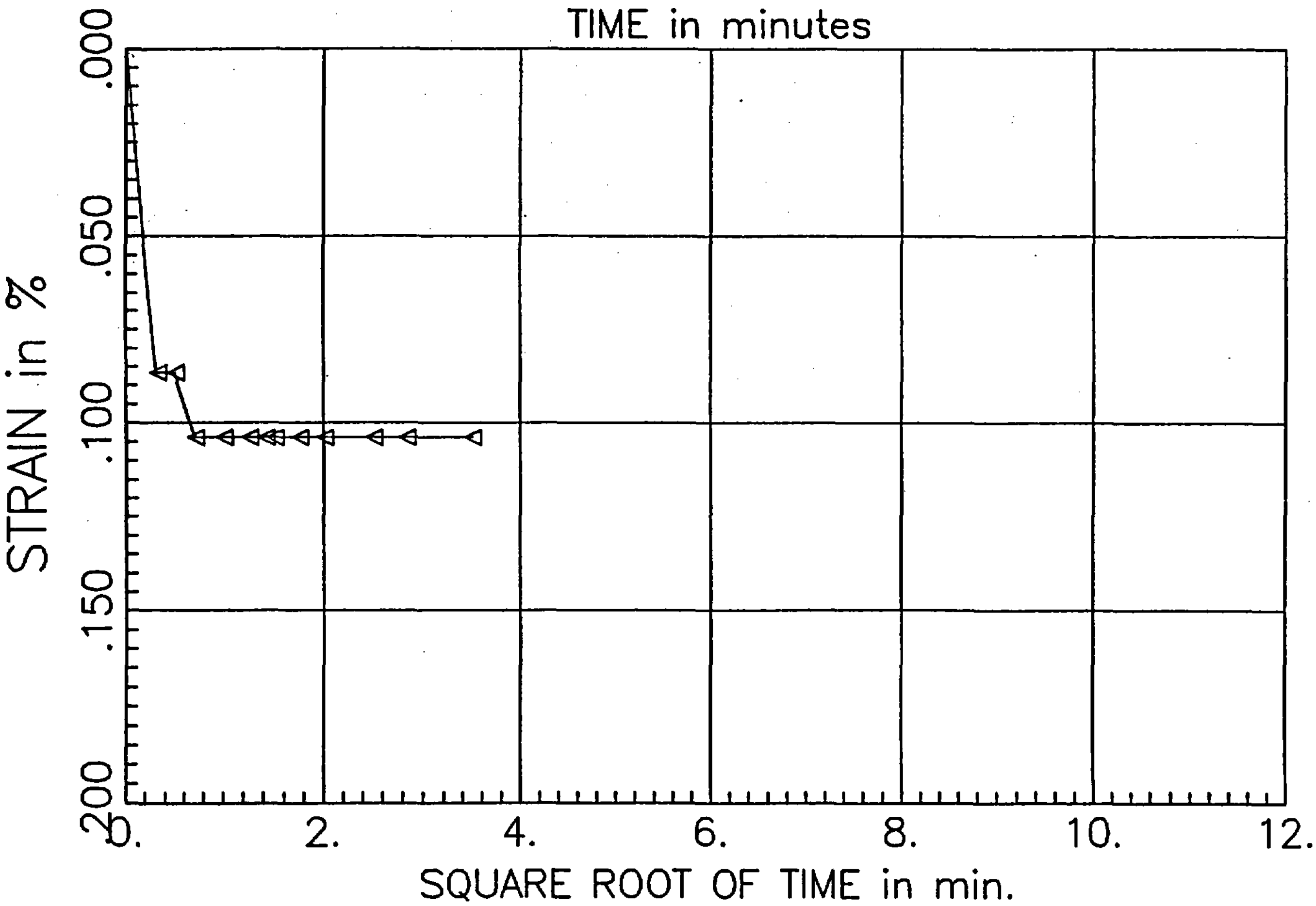
PRESSURE INCREMENT FROM 1.00 tsf to 0.25 tsf

DATE	TIME	ELAPSED TIME (min)	SQ RT OF TIME(min)	CHANGE IN HEIGHT in	VOID RATIO	STRAIN in %
04-23-99	18:09:59	0.00	0.00	0.1855	1.456	24.88
		0.10	0.32	0.1818	1.472	24.38
		0.25	0.50	0.1807	1.477	24.24
		0.50	0.71	0.1796	1.482	24.08
		1.00	1.00	0.1781	1.488	23.89
		1.57	1.25	0.1771	1.493	23.75
		2.00	1.41	0.1765	1.495	23.67
		2.25	1.50	0.1762	1.497	23.63
		3.07	1.75	0.1754	1.500	23.53
		4.00	2.00	0.1749	1.502	23.46
		6.25	2.50	0.1740	1.506	23.34
		8.00	2.83	0.1736	1.508	23.28
		12.25	3.50	0.1730	1.511	23.20
		15.00	3.87	0.1727	1.512	23.16
		22.00	4.69	0.1723	1.514	23.11
		30.00	5.48	0.1719	1.515	23.06
		36.00	6.00	0.1717	1.516	23.02
		49.00	7.00	0.1714	1.517	22.99
		60.00	7.75	0.1712	1.519	22.95

Jo 05996.02
Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 0.10\%$



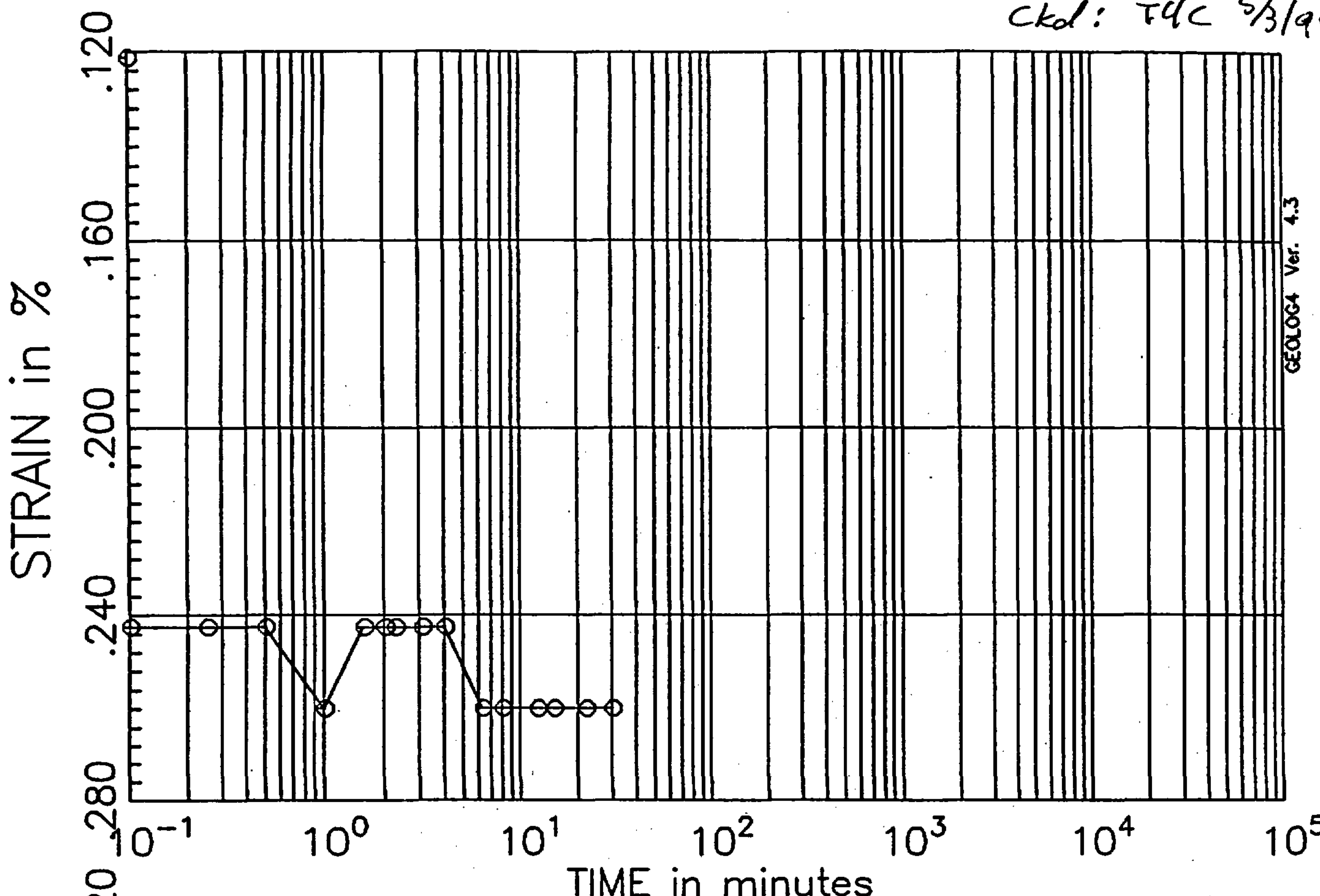


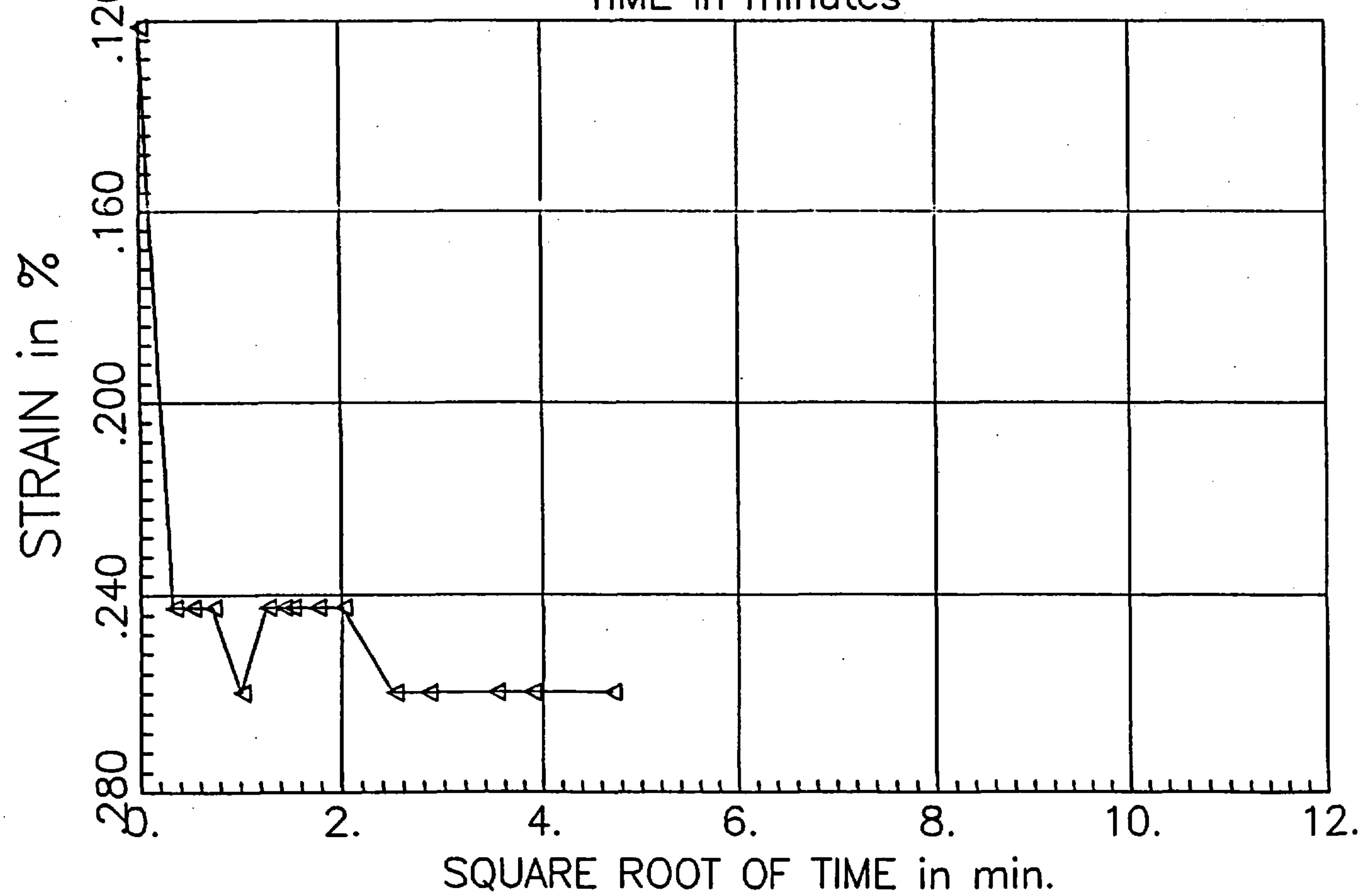
PRESSURE INCREMENT #1
from 0.00 tsf to 0.10 tsf

Test No: 4
Testname: CTBS-U3C

Calc: ACS 4/26/99
Ckd: TYC 5/3/99

$d_{100} = 0.26\%$



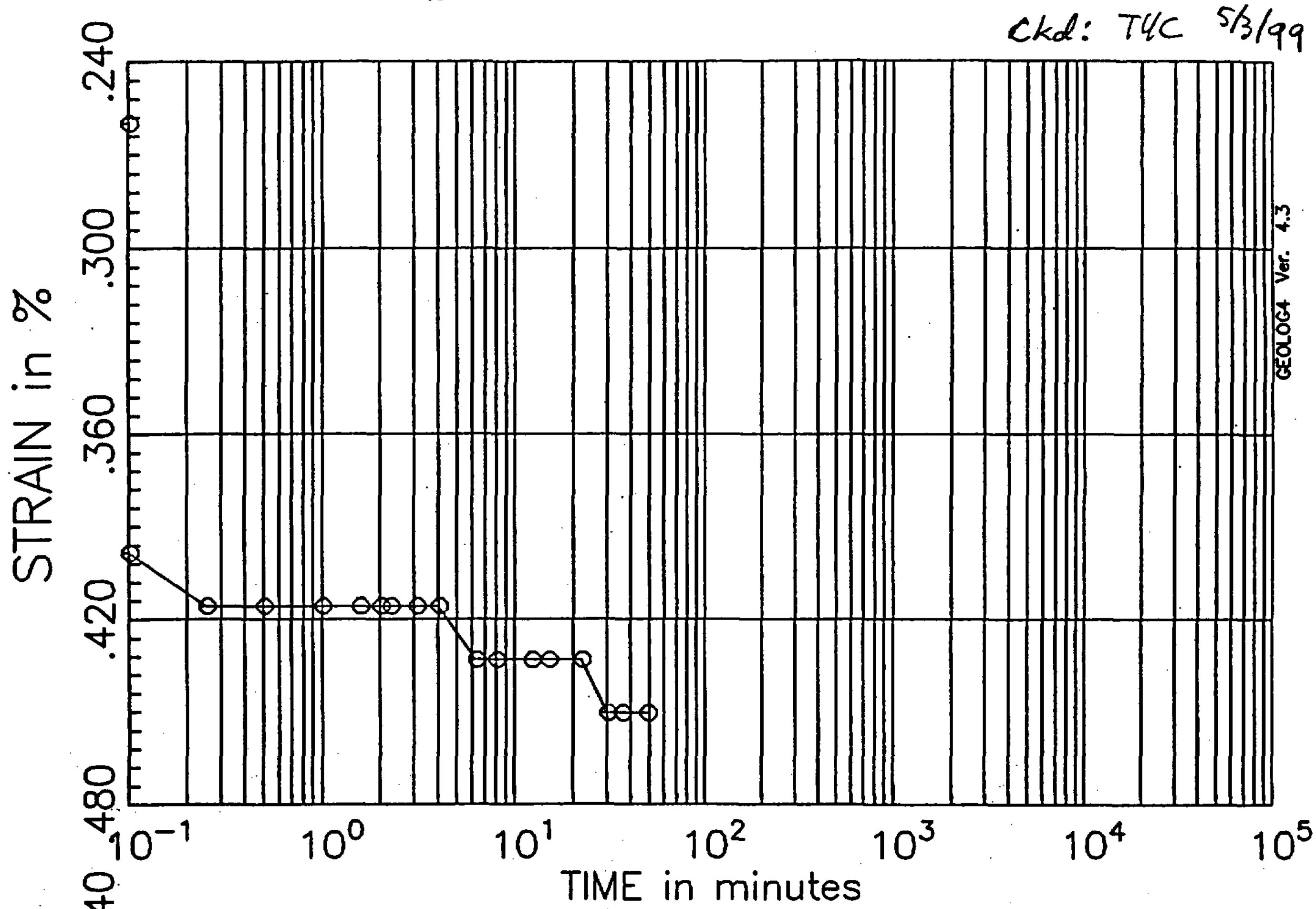


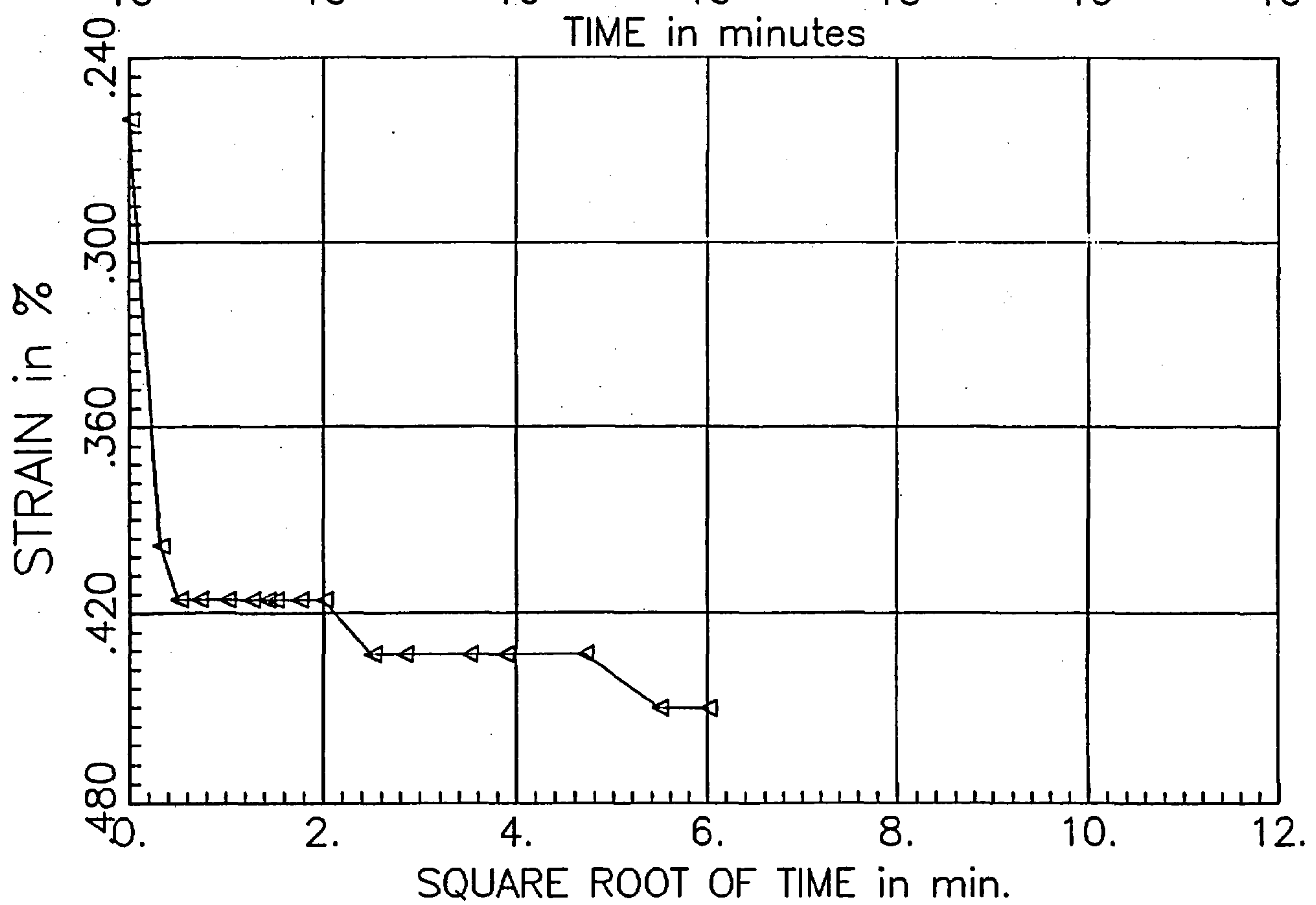
PRESSURE INCREMENT #2
from 0.10 tsf to 0.25 tsf

Test No: 4
Testname: CTBS-U3C

JO 05996.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 0.43\%$





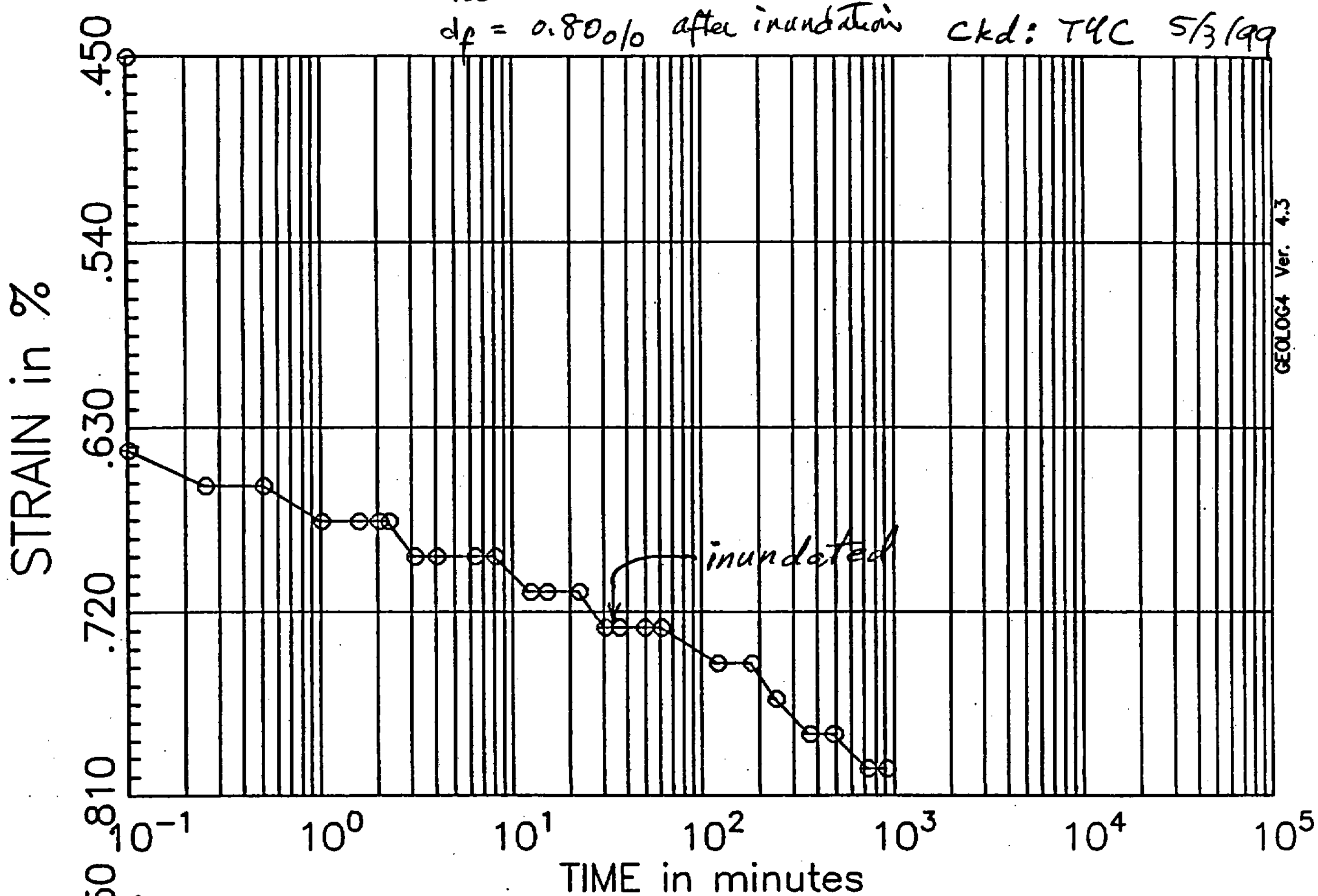
PRESSURE INCREMENT #3
from 0.25 tsf to 0.50 tsf

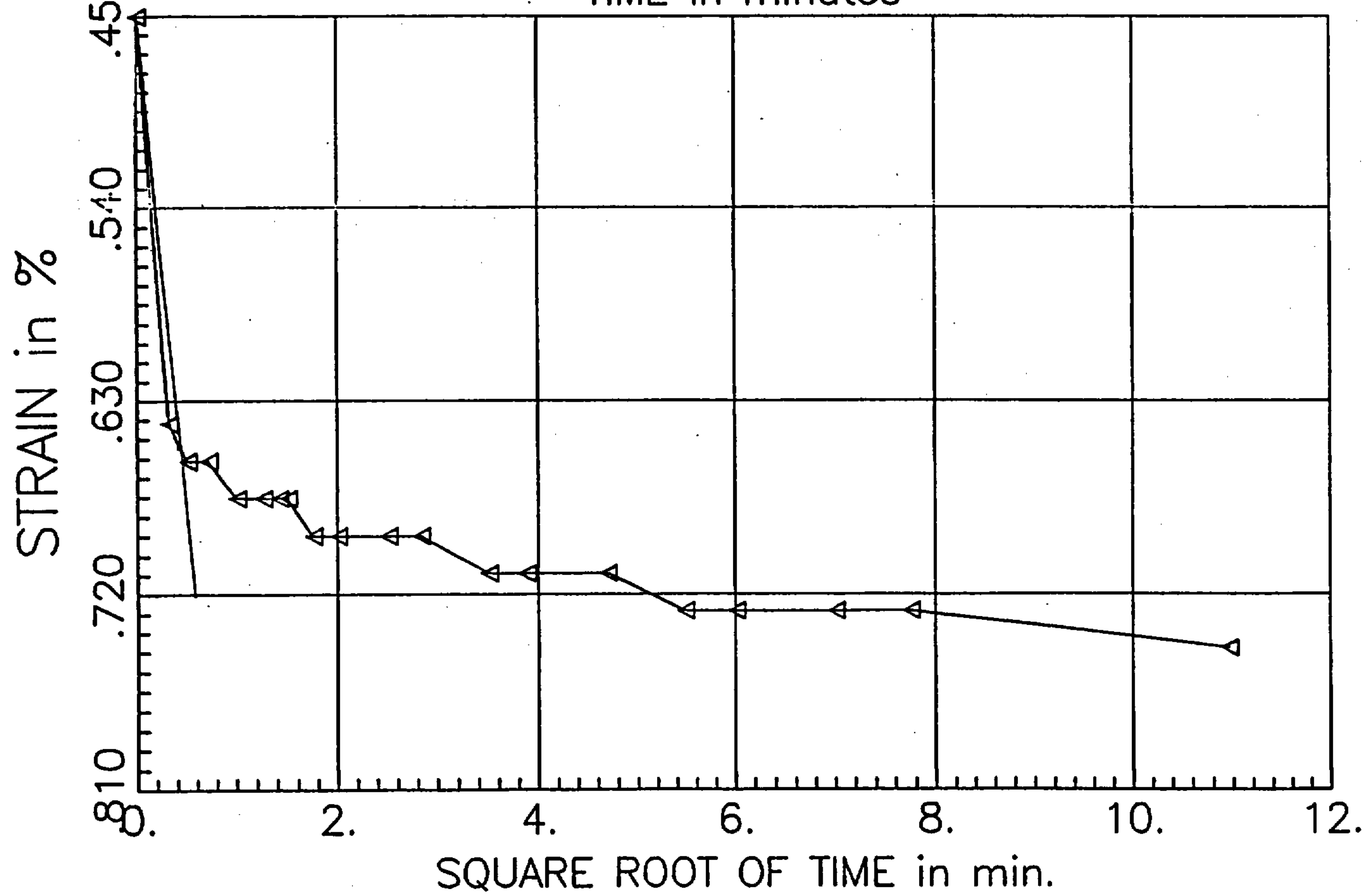
Test No: 4
Testname: CTBS-U3C

JO 05996.02

$d_{100} = 0.68\%$
$d_f = 0.80\%$ after inundation

Calc: ACS 4-26-99
Ckd: TYC 5/3/99





PRESSURE INCREMENT #4
from 0.50 tsf to 1.00 tsf

Test No: 4
Testname: CTBS-U3C

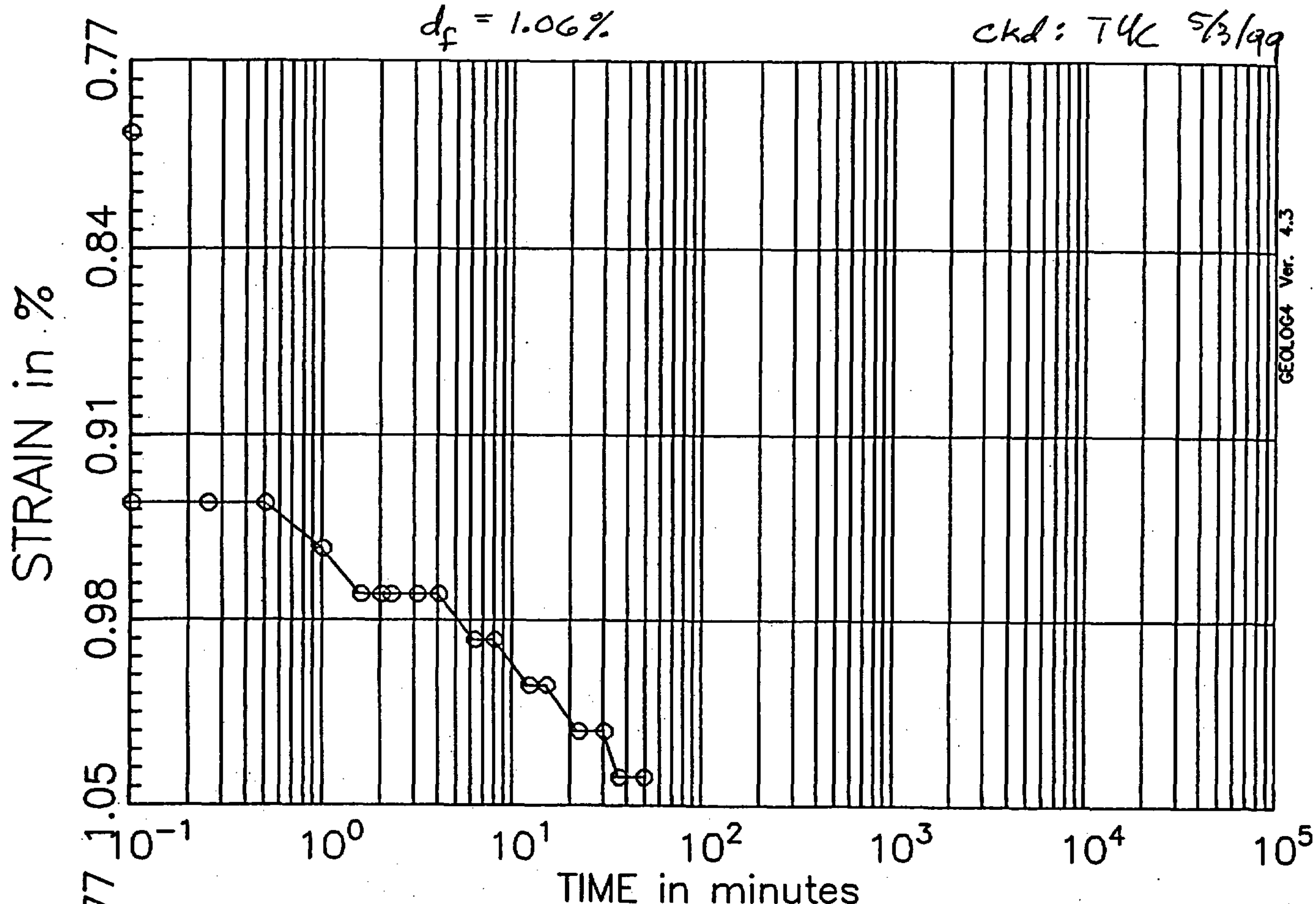
JO 05996.02

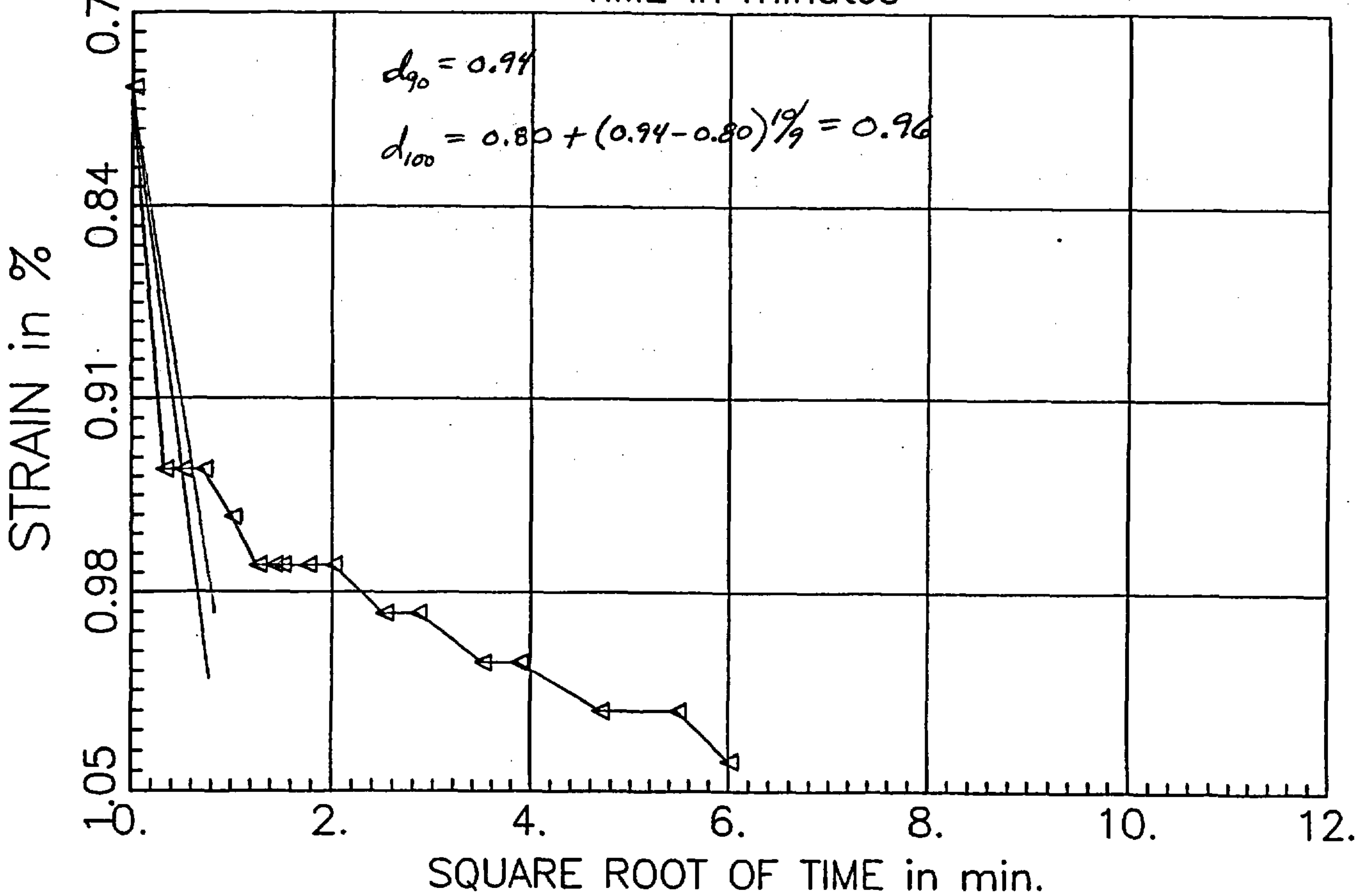
Calc: ACS 4-26-99

Ckd: TWC 5/3/99

$d_{100} = 0.96\%$

$d_f = 1.06\%$





PRESSURE INCREMENT #5
from 1.00 tsf to 1.50 tsf

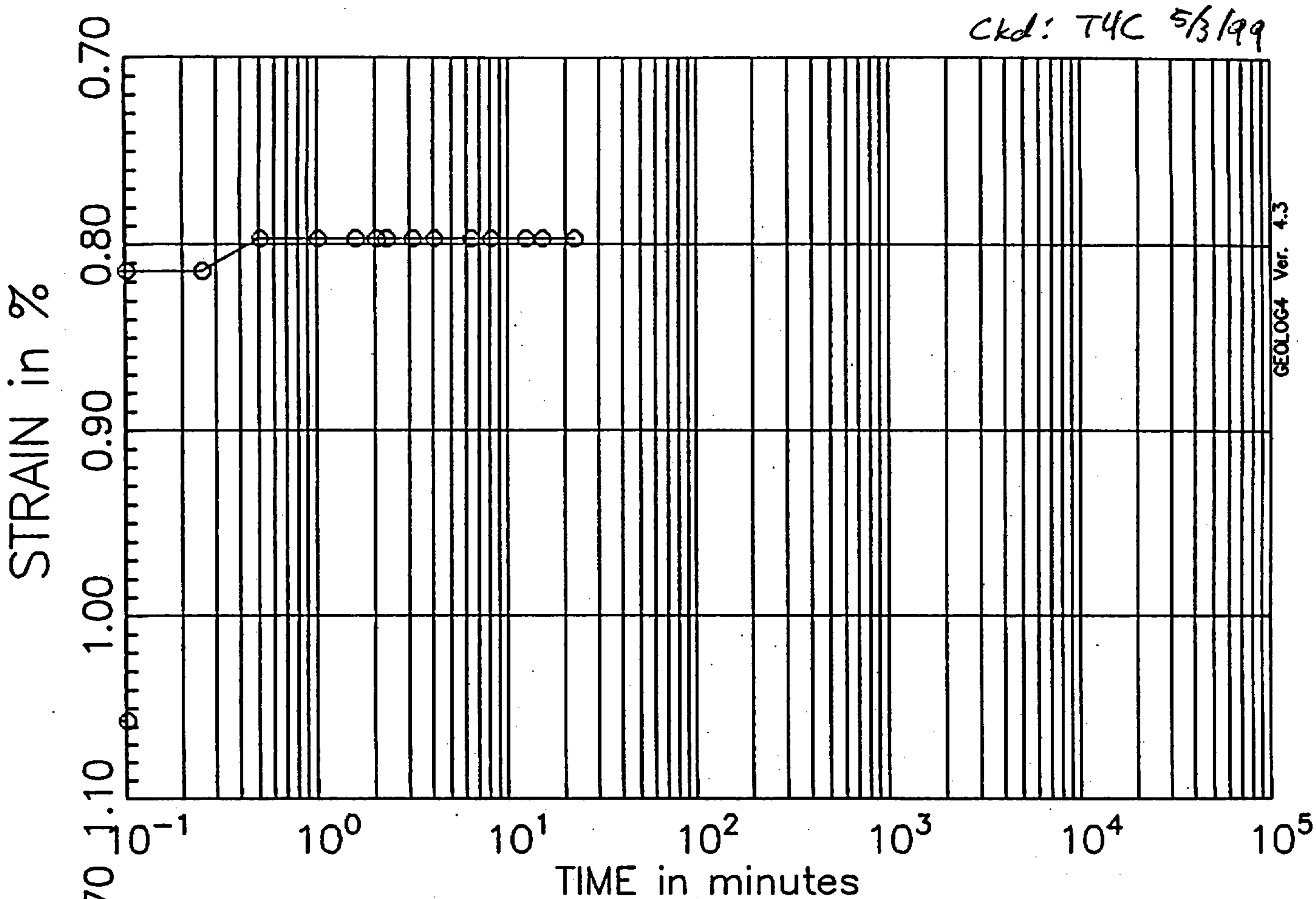
Test No: 4
Testname: CTBS-U3C

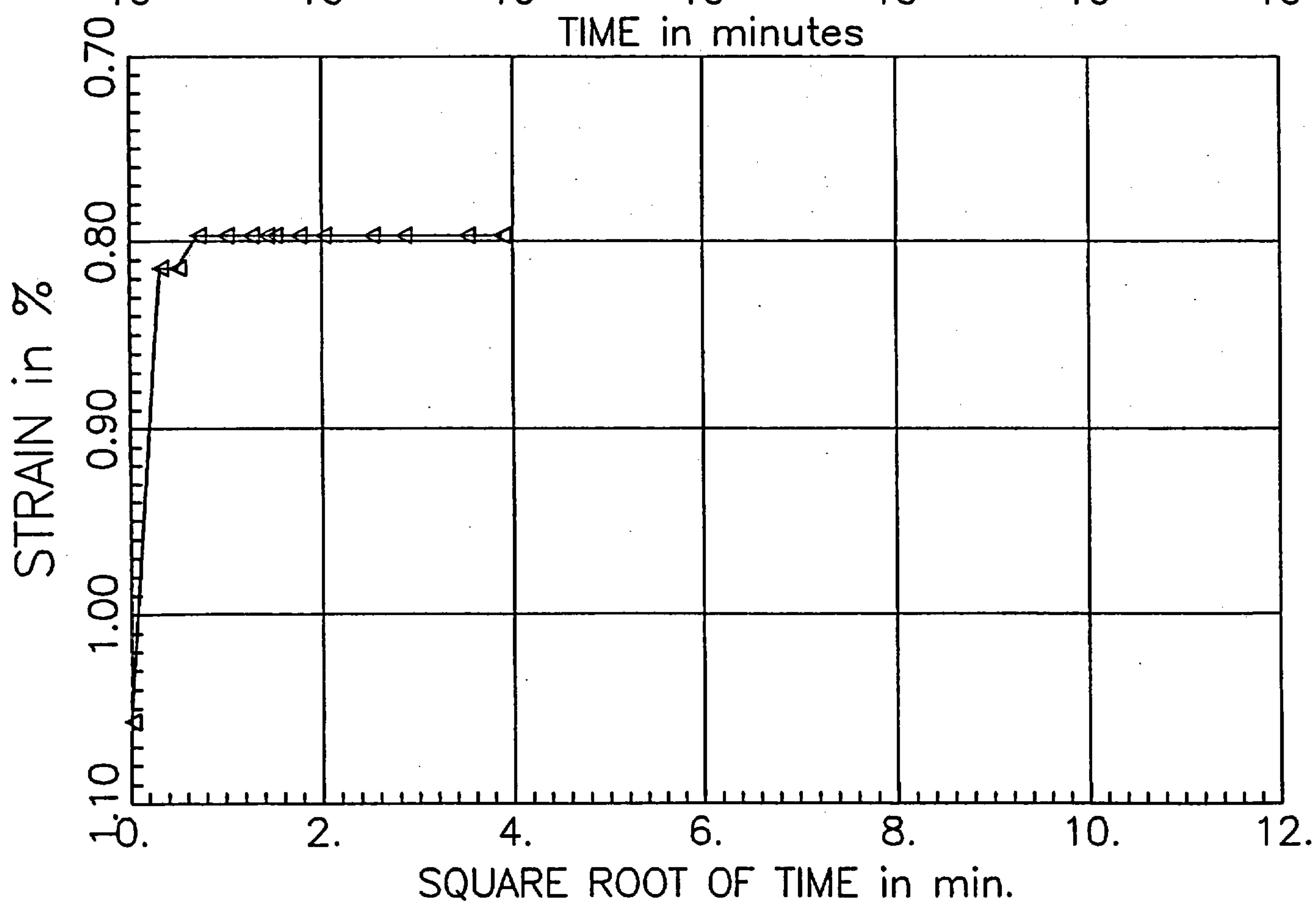
JO 05776.UK

d_{100} = 0.80%

Calc: ACS 4-26-99

Ckd: T4C 5/3/99





PRESSURE INCREMENT #6
from 1.50 tsf to 0.50 tsf

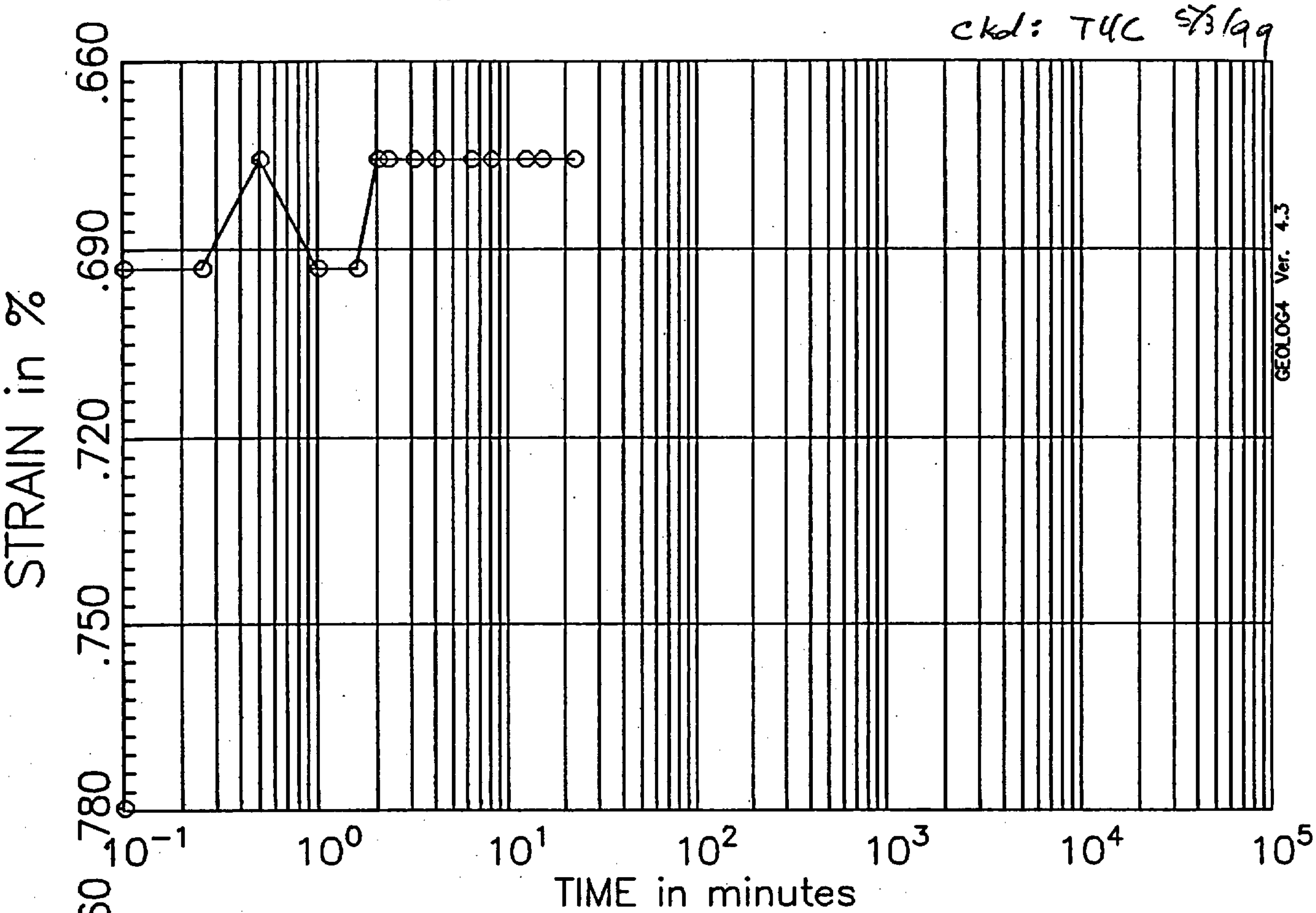
Test No: 4
Testname: CTBS–U3C

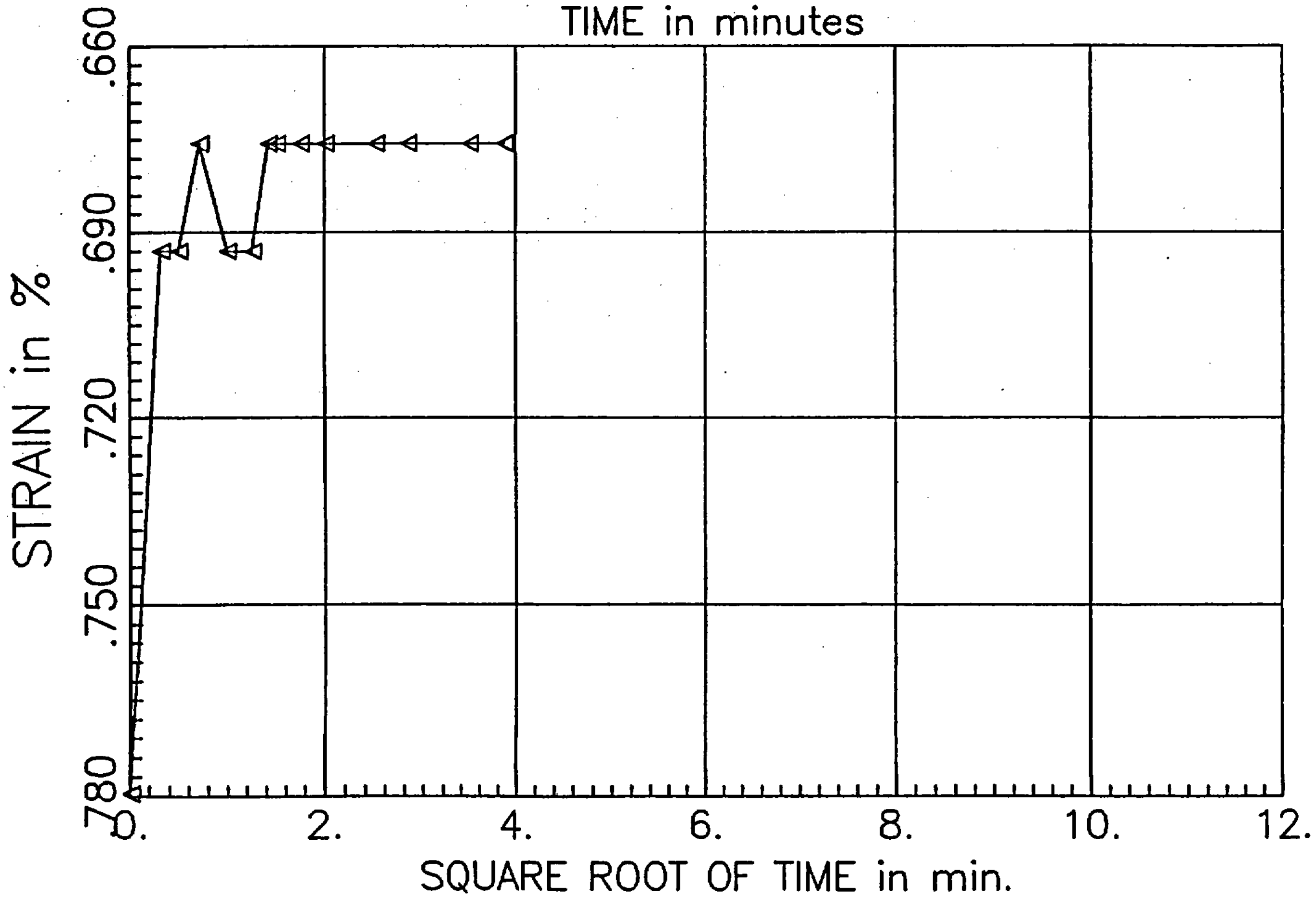
Jo 05996.02

d_{100} = 0.68%

Calc: ACS 4-26-99

Ckd: TUC 5/3/99





PRESSURE INCREMENT #7
from 0.50 tsf to 0.25 tsf

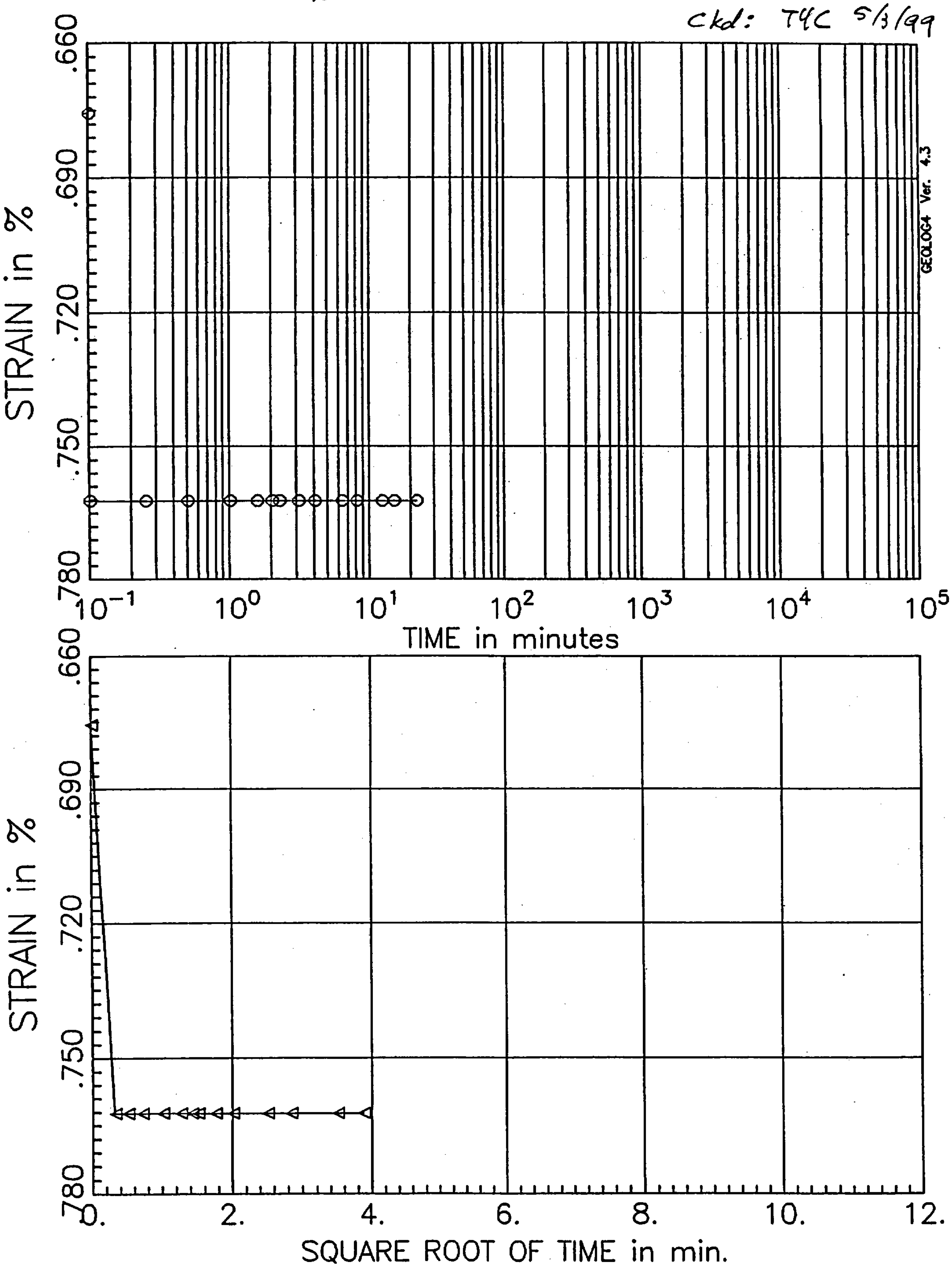
Test No: 4
Testname: CTBS-U3C

JO 05776.02

$d_{100} = 0.76\%$

Calc: ACS 4-26-99

Ckd: TYC 5/3/99

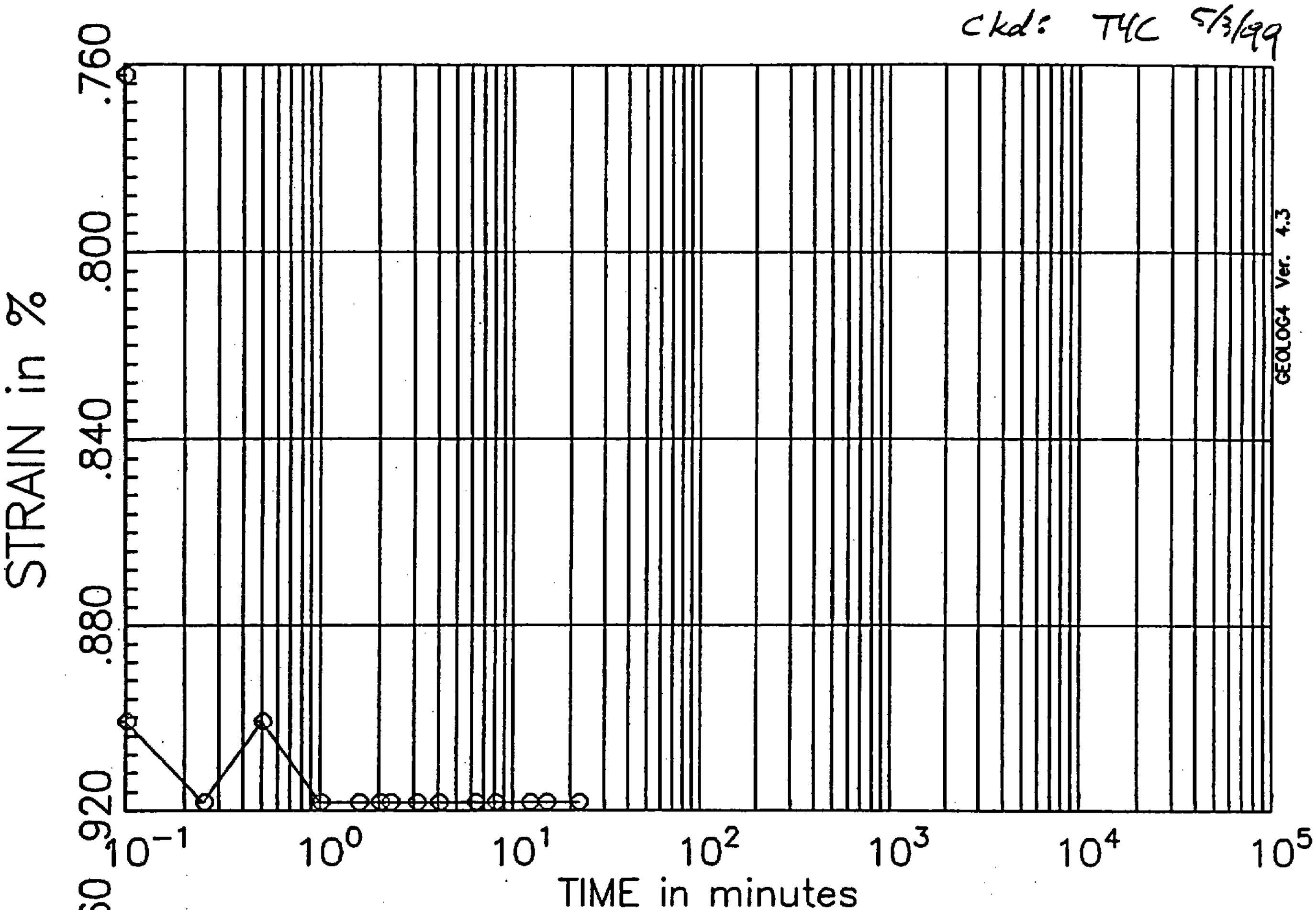


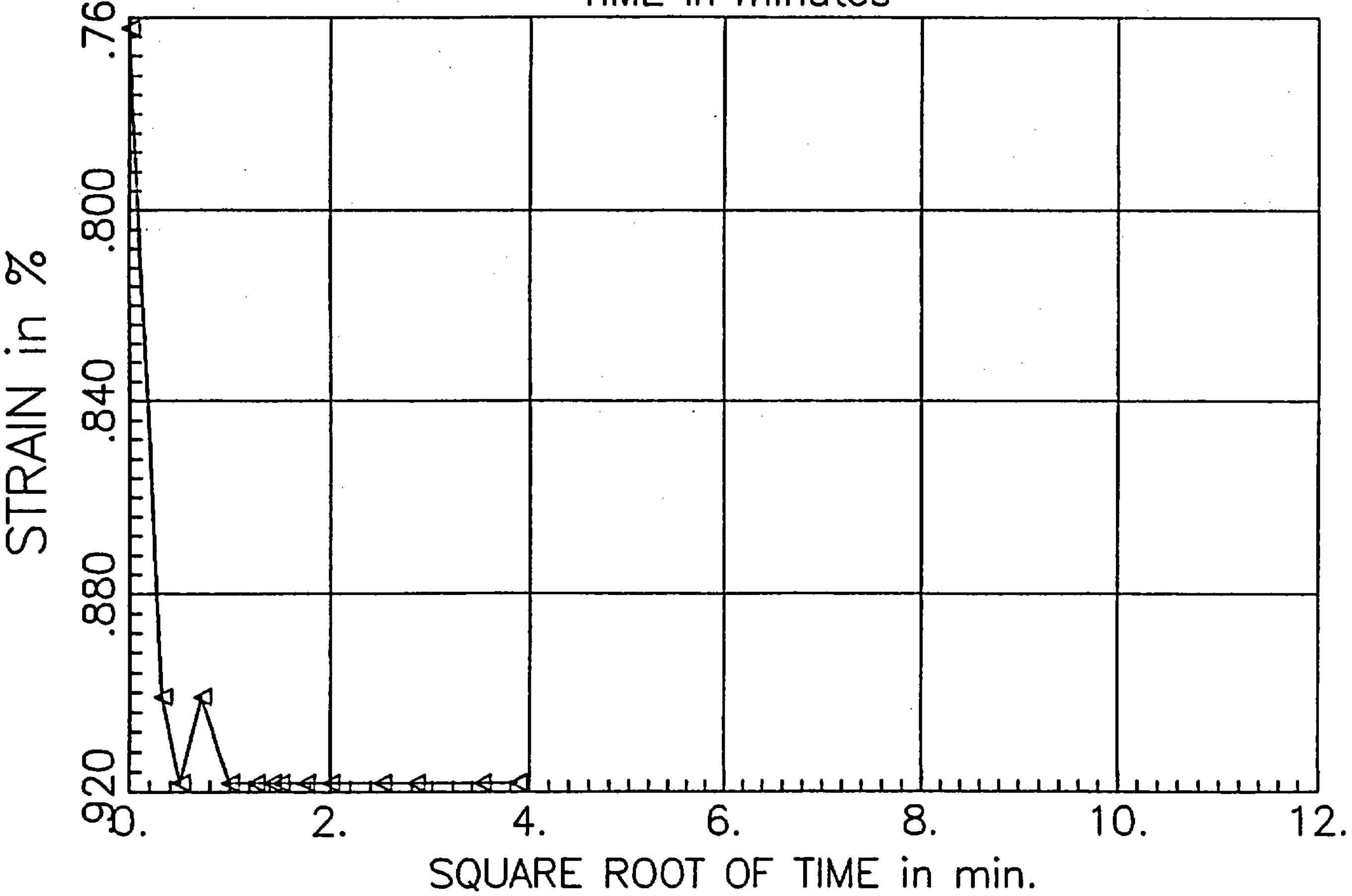
PRESSURE INCREMENT #8
from 0.25 tsf to 0.50 tsf

Test No: 4
Testname: CTBS-U3C

Jo 05996.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 0.92\%$



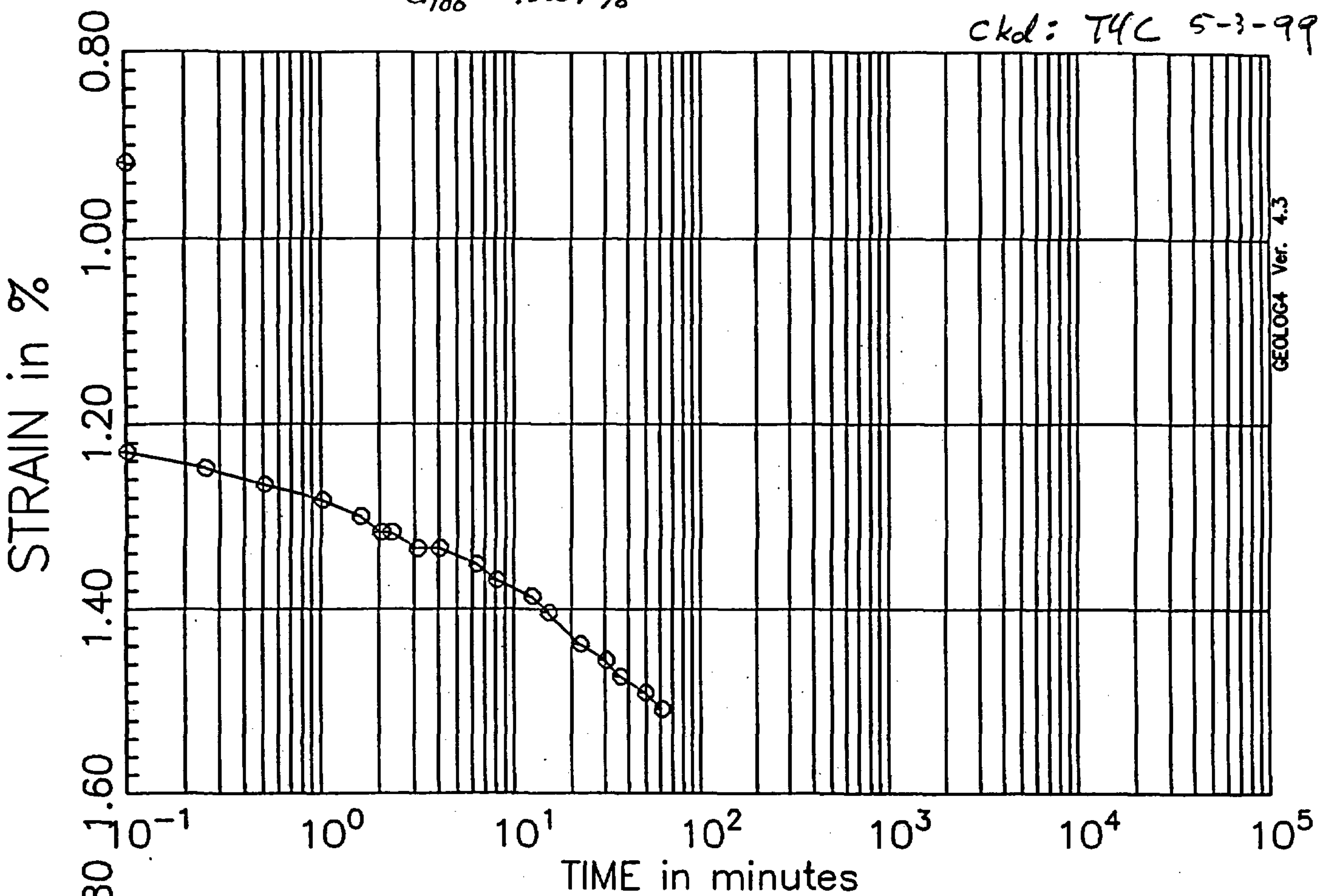


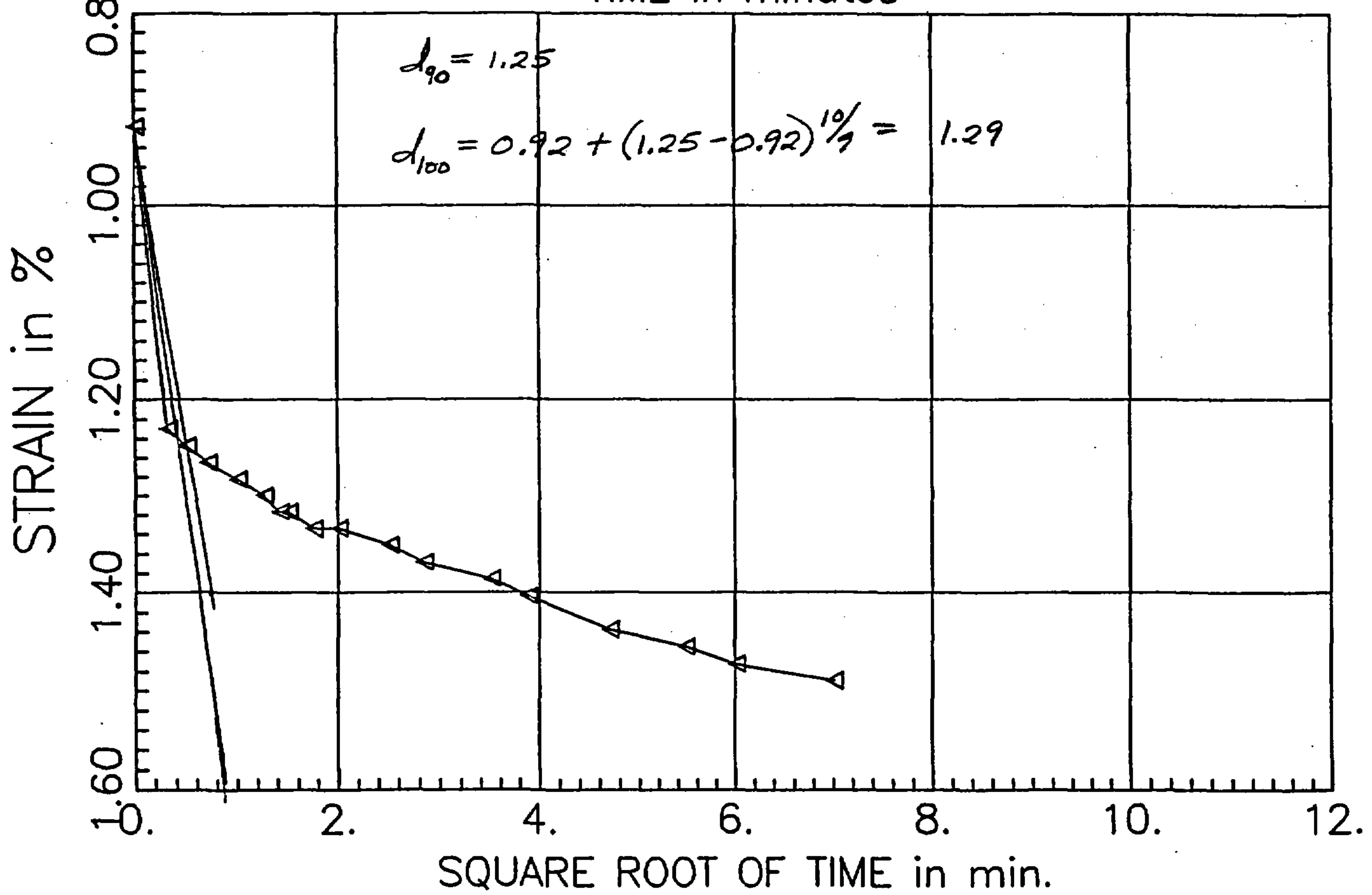
PRESSURE INCREMENT #9
from 0.50 tsf to 1.00 tsf

Test No: 4
Testname: CTBS–U3C

JO 05996.02
Calc: ACS 4-26-99
Ckd: T4C 5-3-99

$d_{100} = 1.29\%$





PRESSURE INCREMENT #10
from 1.00 tsf to 2.00 tsf

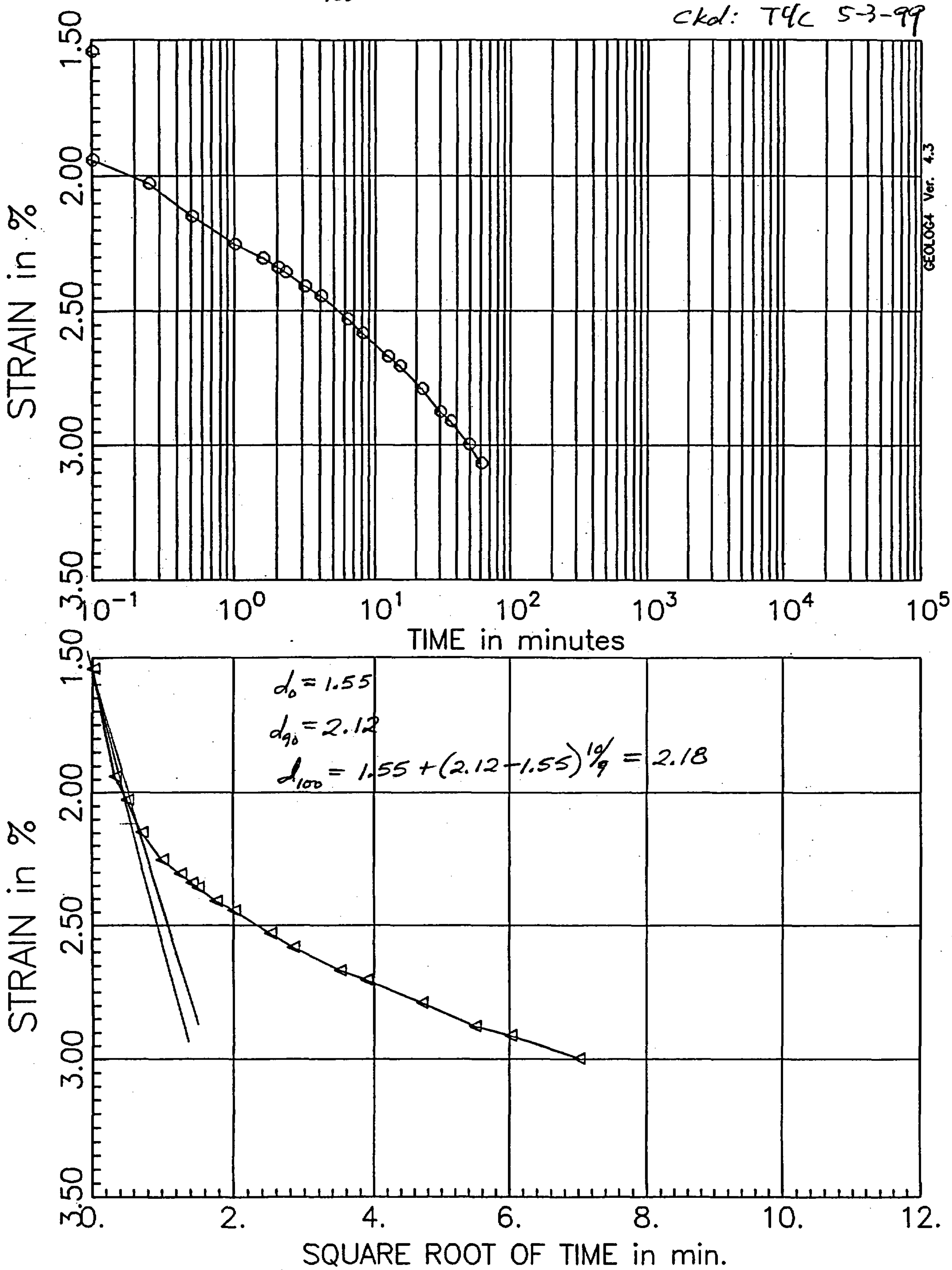
Test No: 4
Testname: CTBS–U3C

JO 05996.02

Calc: ACS 4-26-99

Ckd: T4C 5-3-99

$d_{100} = 2.18\%$



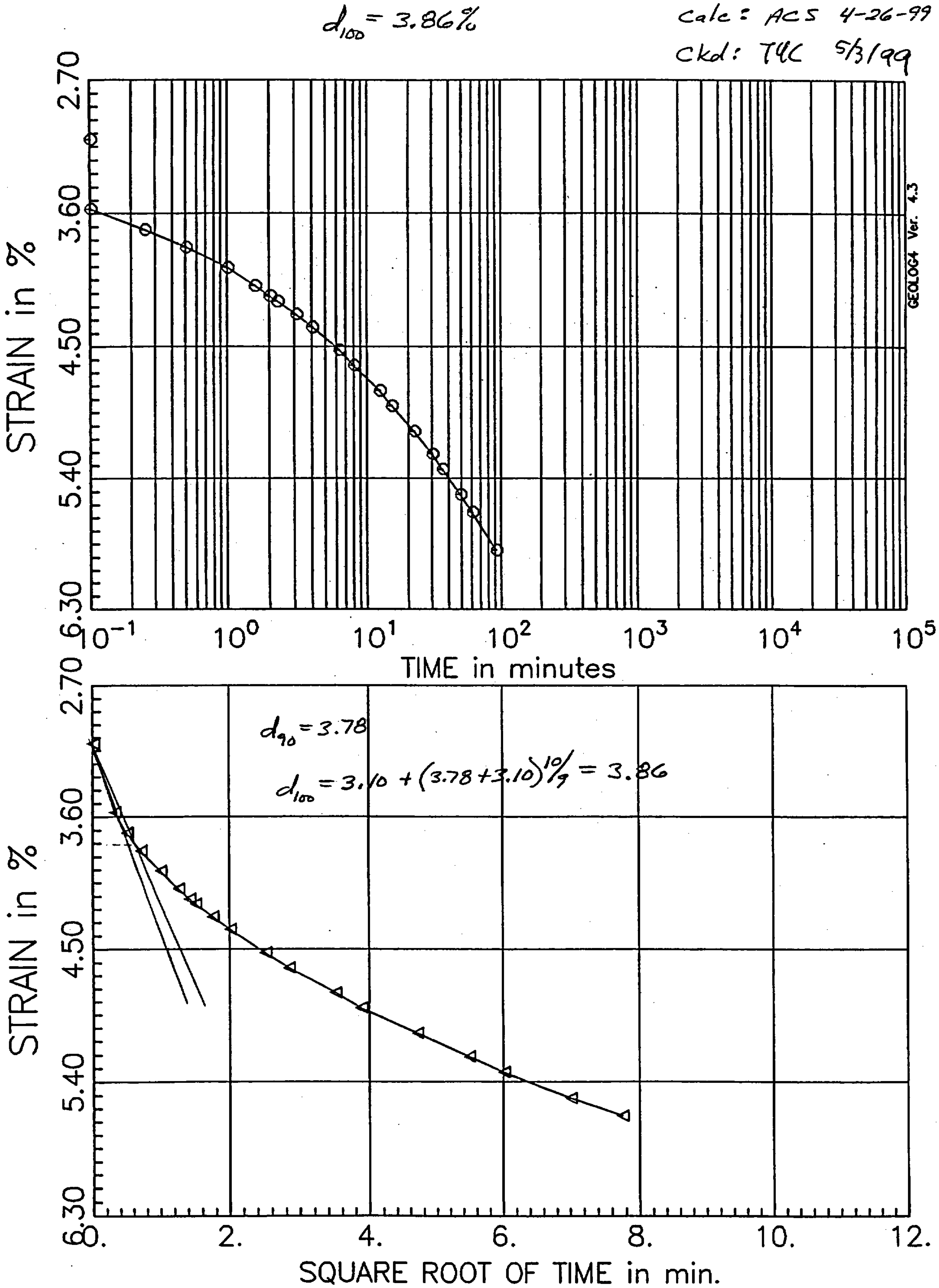
PRESSURE INCREMENT #11

from 2.00 tsf to 3.00 tsf

Test No: 4

Testname: CTBS-U3C

JO 05776.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

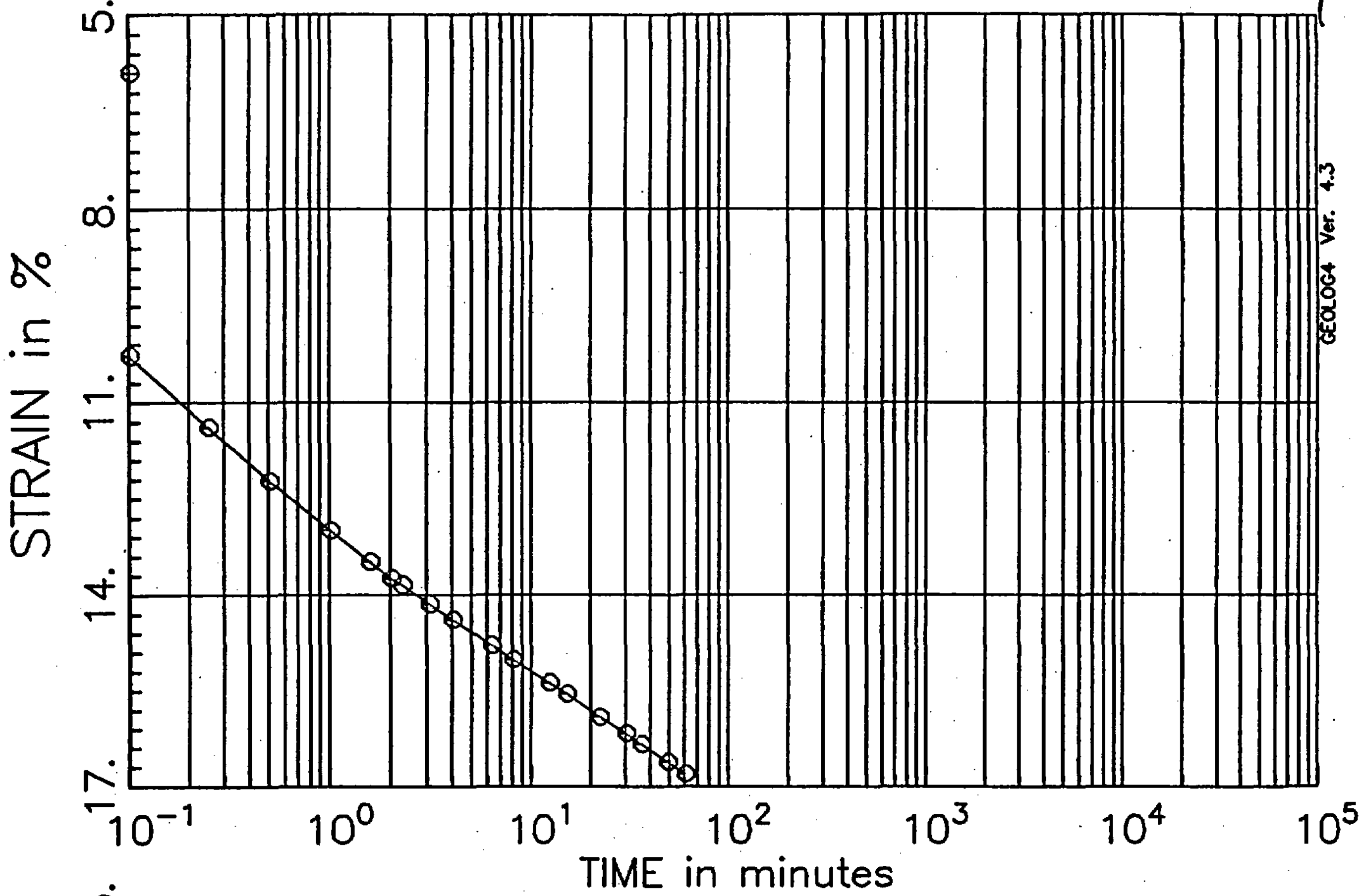


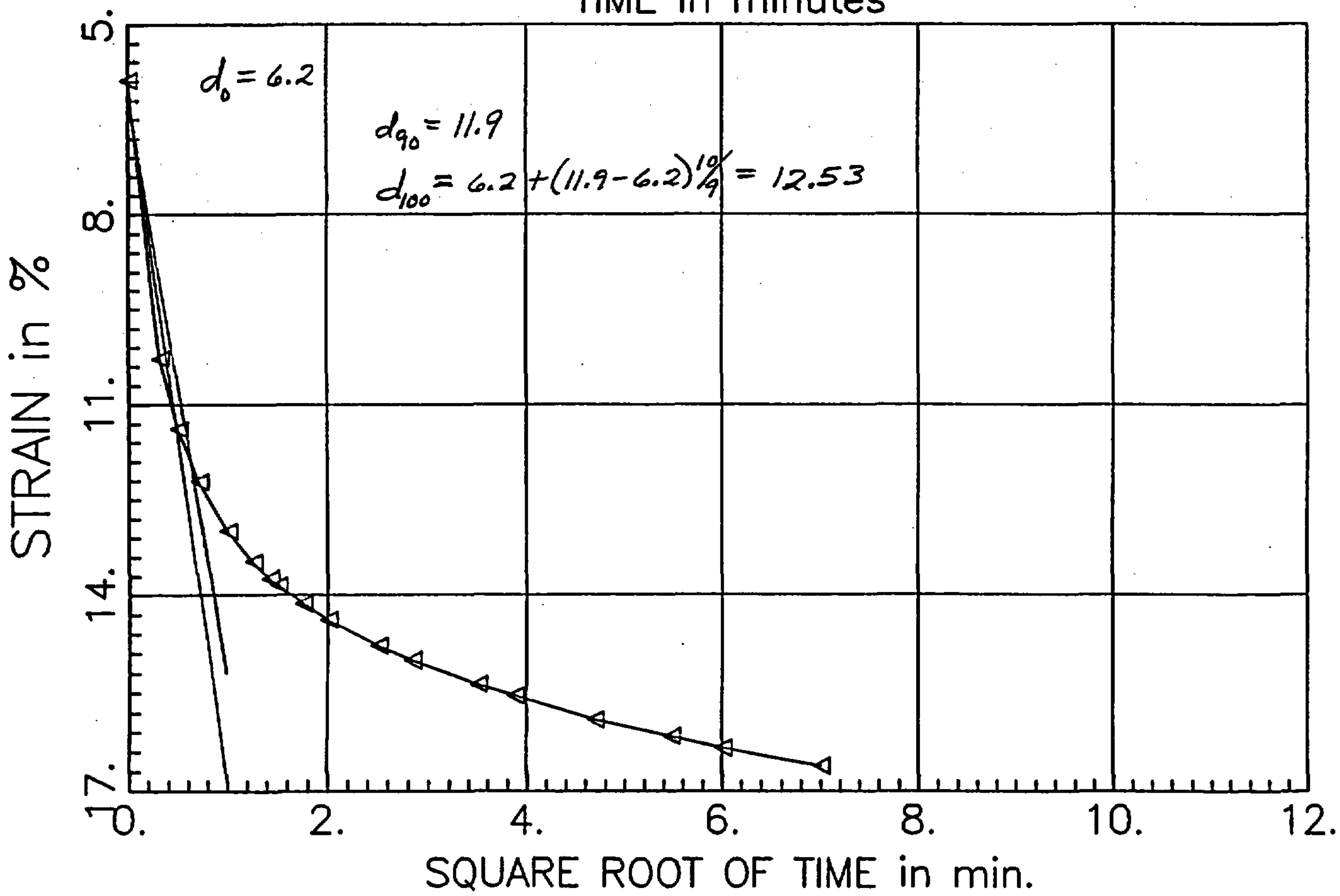
PRESSURE INCREMENT #12
from 3.00 tsf to 4.00 tsf

Test No: 4
Testname: CTBS-U3C

Jo 05996.02
Calc: ACS 4-26-99
Ckd: TYC 5/3/99

$d_{100} = 12.53\%$





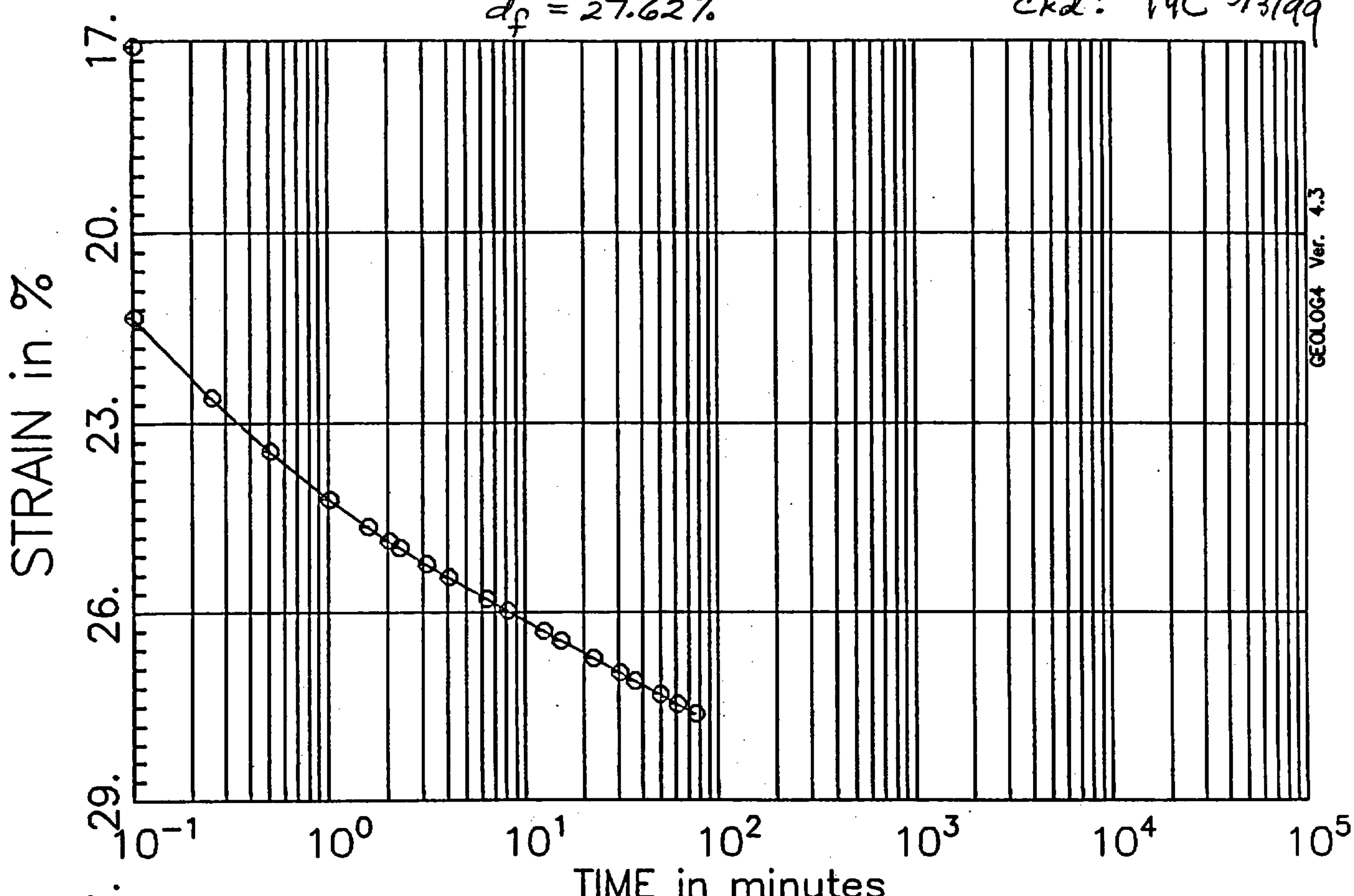
PRESSURE INCREMENT #13
from 4.00 tsf to 8.00 tsf

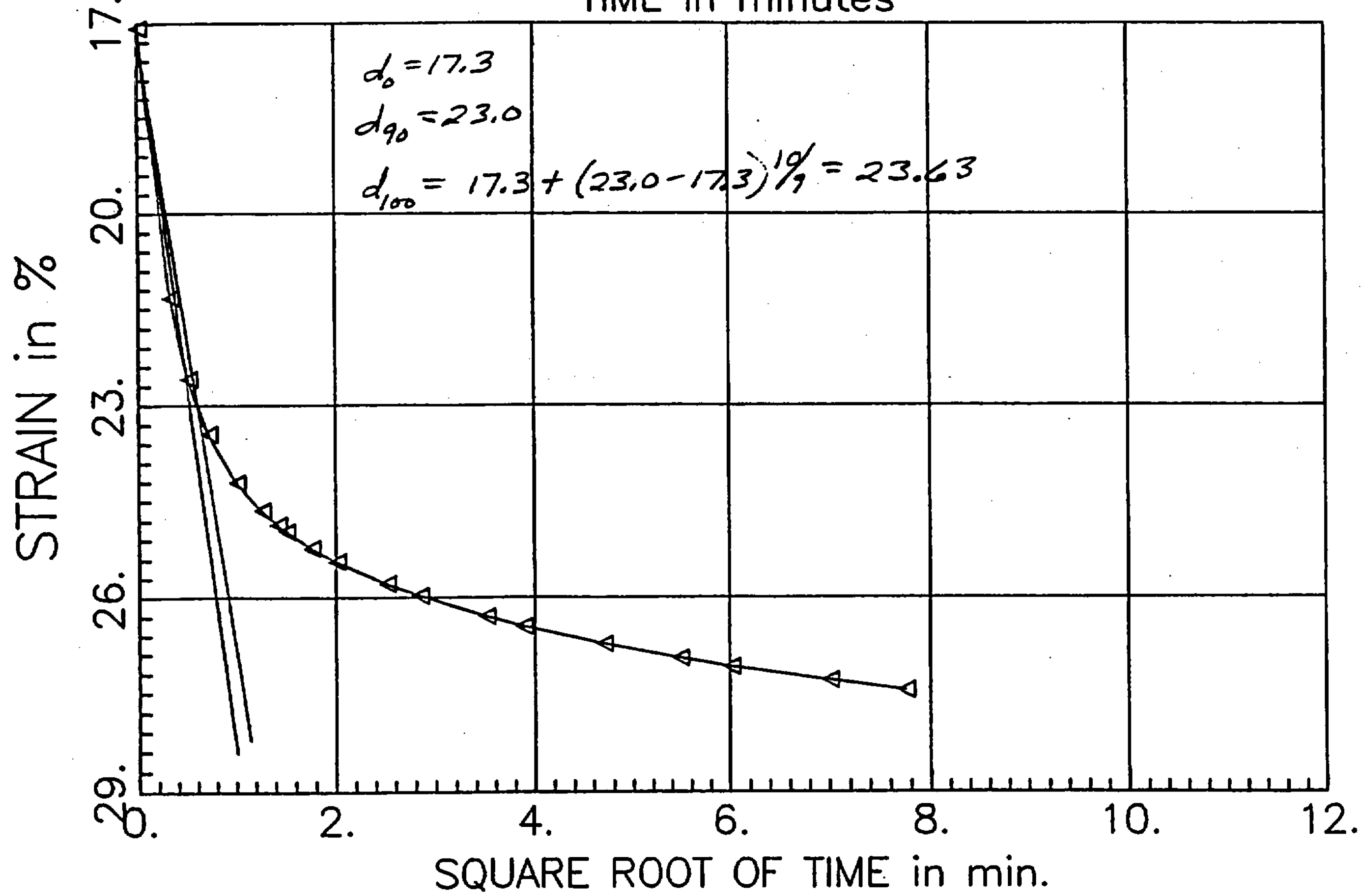
Test No: 4
Testname: CTBS-U3C

JO 05116.02
Calc: ACS 4-26-99
ckd: TYC 5/3/99

$d_{100} = 23.63\%$

$d_f = 27.62\%$



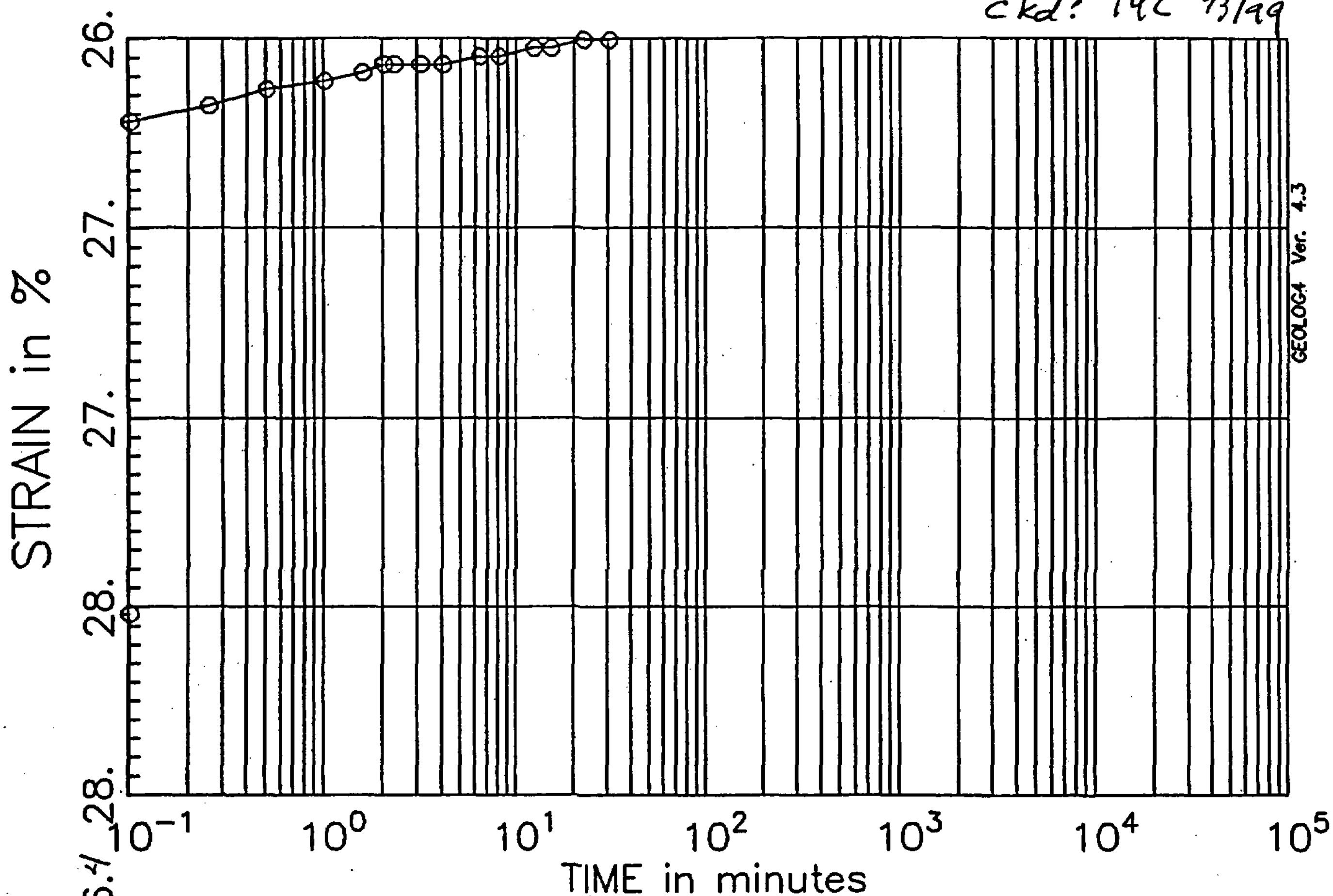


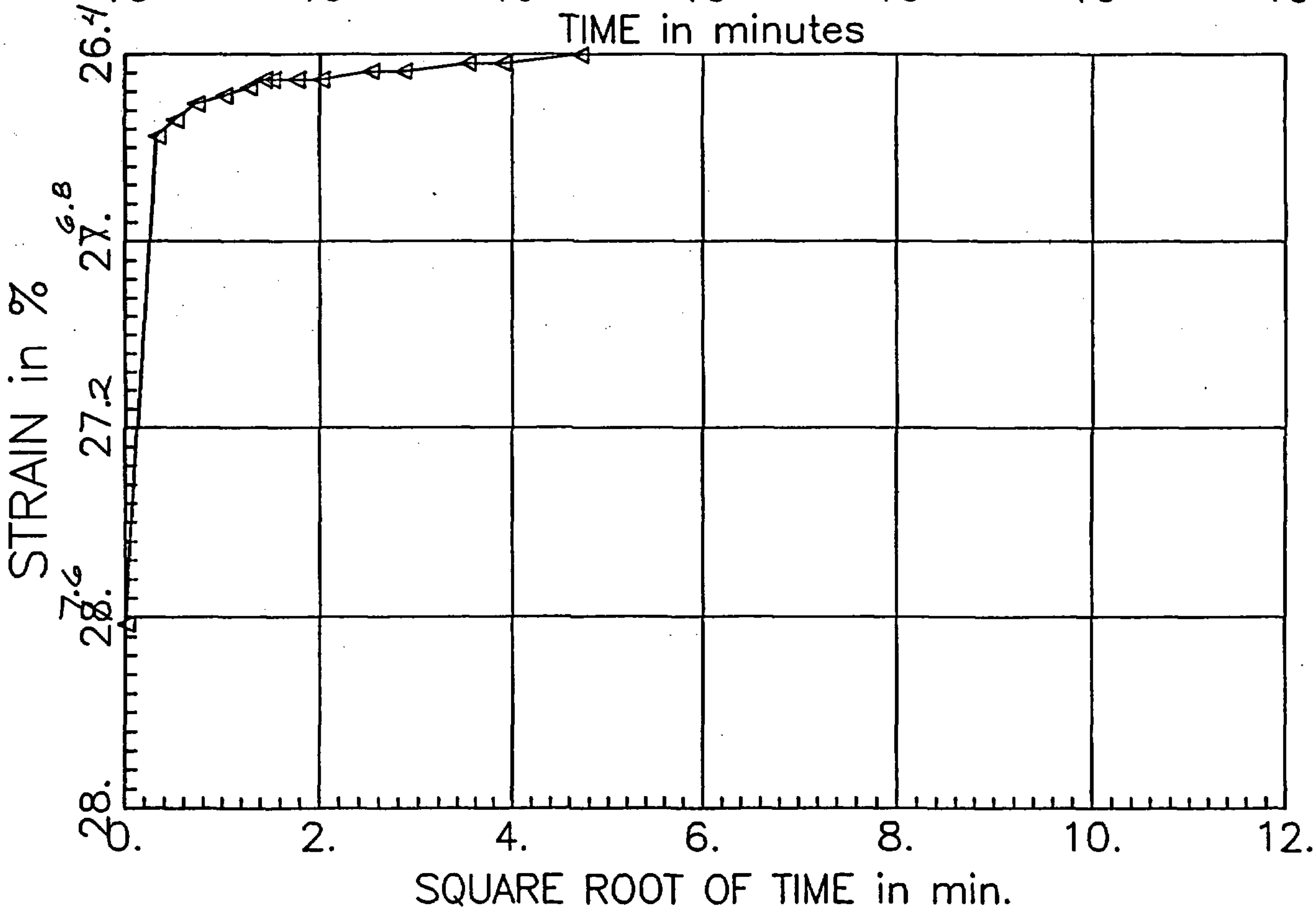
PRESSURE INCREMENT #14
from 8.00 tsf to 16.00 tsf

Test No: 4
Testname: CTBS–U3C

Jo 05996.02
Calc: AES 4-26-99
Ckd: TGC 5/3/99

$d_{100} = 26.5\ \%$





PRESSURE INCREMENT #15
from 16.00 tsf to 4.00 tsf

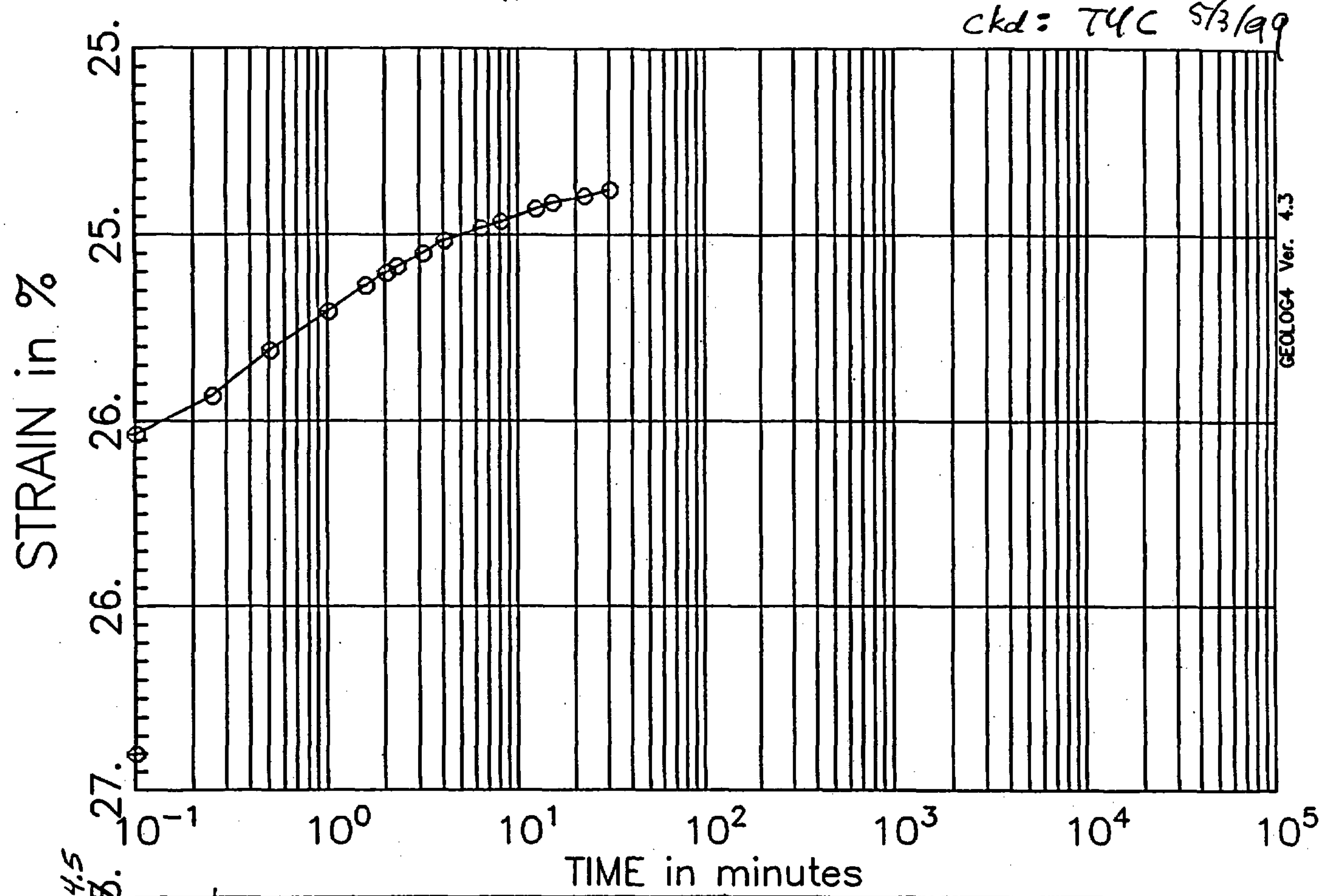
Test No: 4
Testname: CTBS-U3C

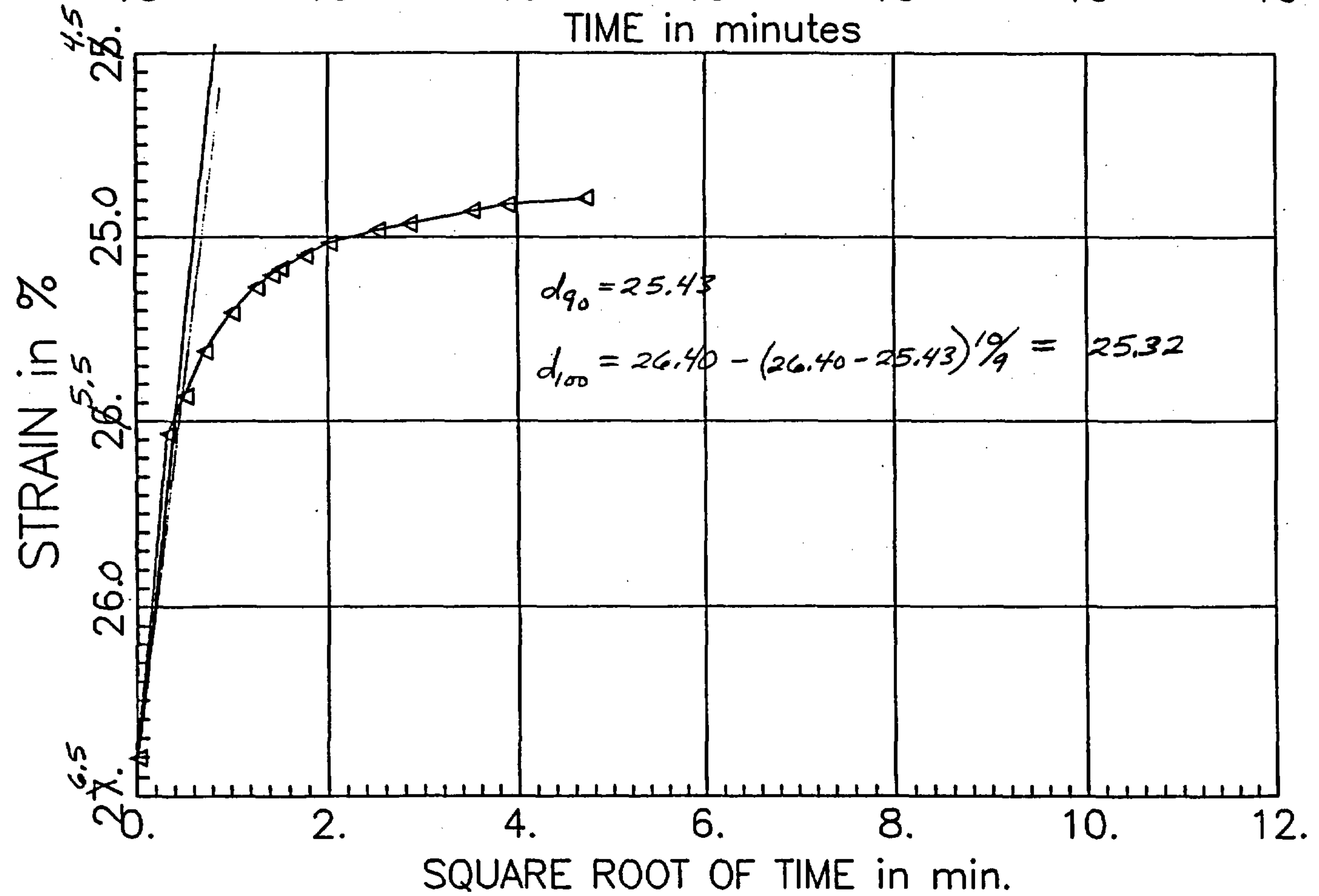
JO 05776.02

$d_{100} = 25.32\%$

Calc: ACS 4-26-99

Ckd: TYC 5/3/99





PRESSURE INCREMENT #16
from 4.00 tsf to 1.00 tsf

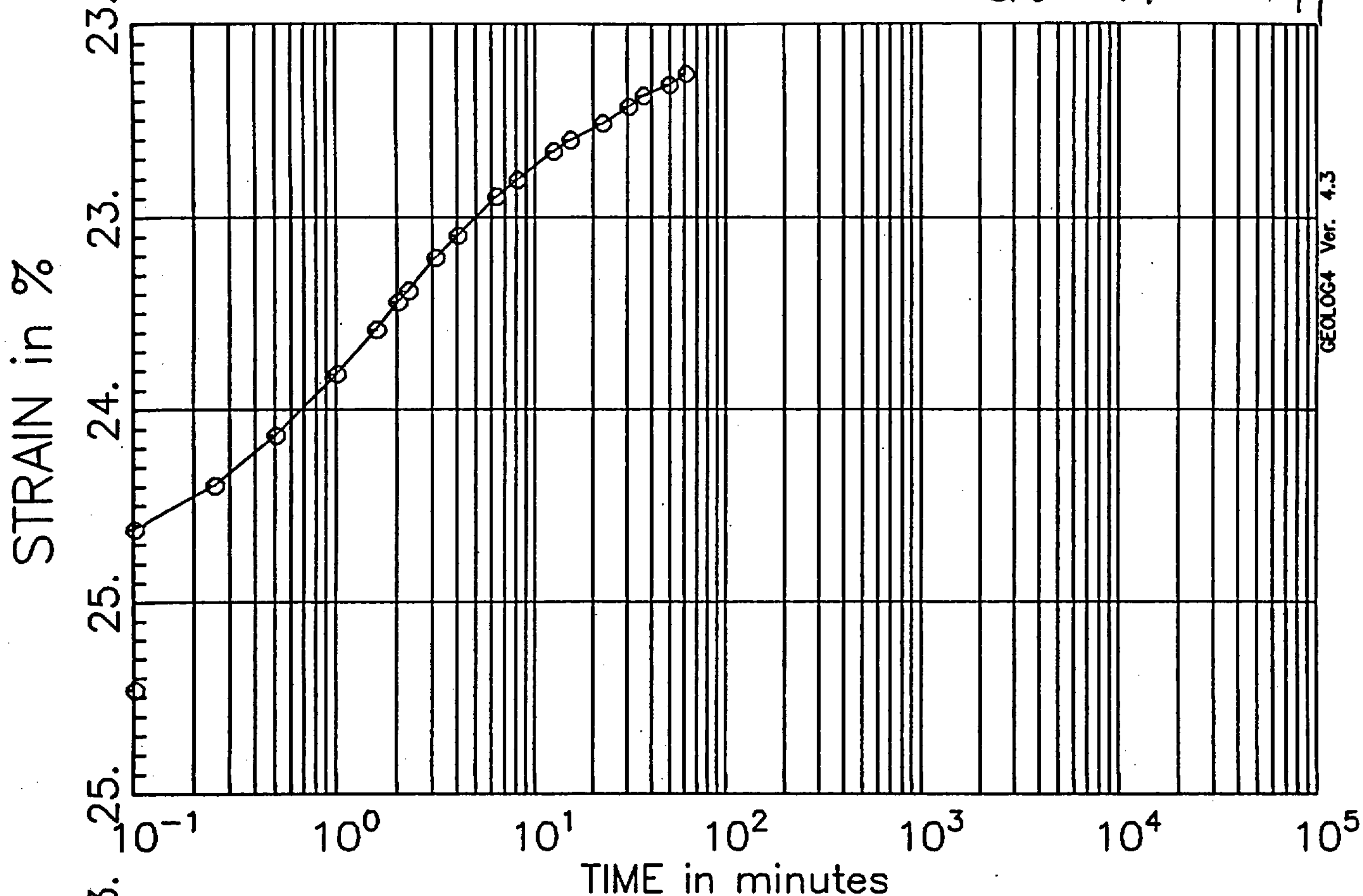
Test No: 4
Testname: CTBS-U3C

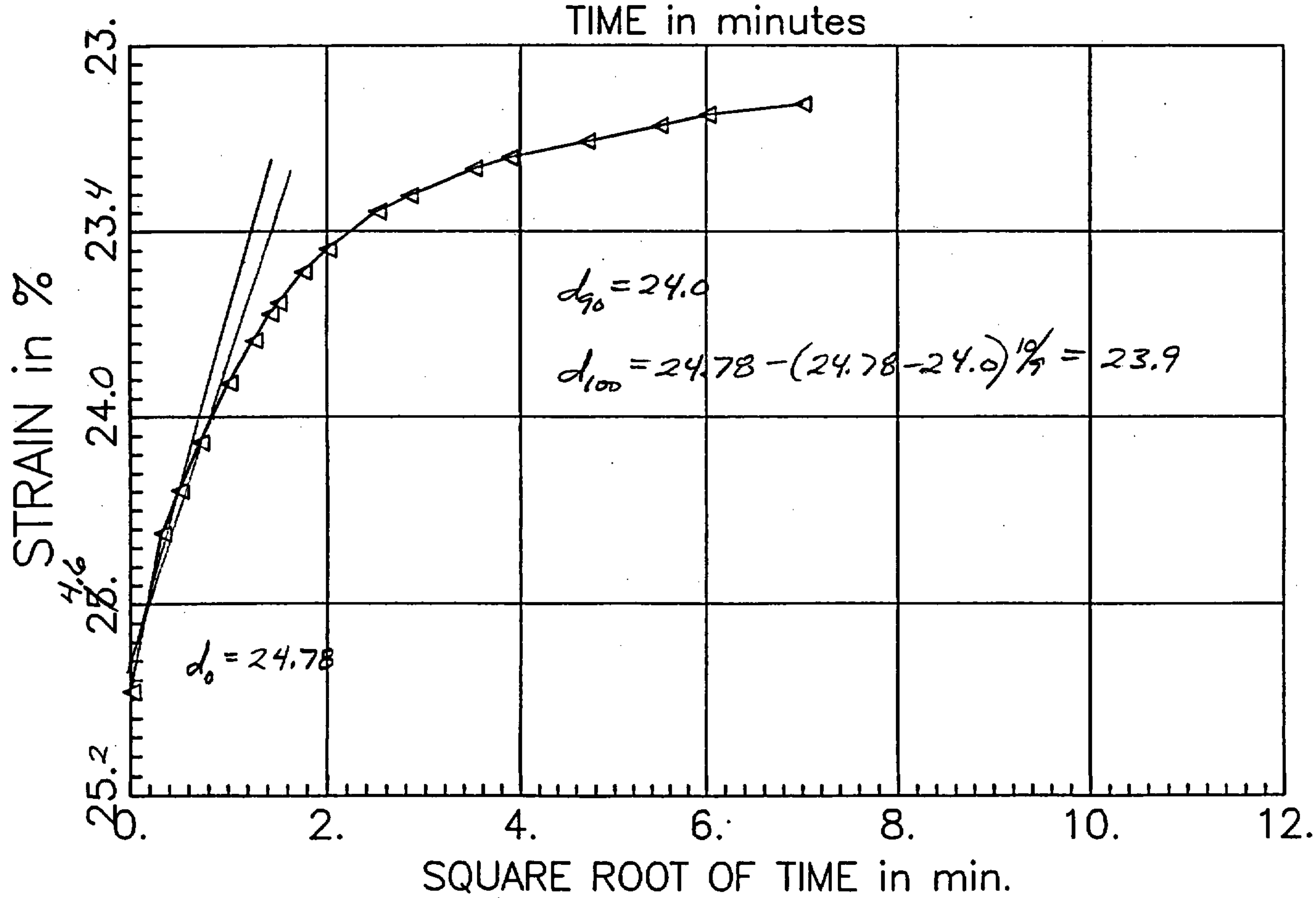
JO 05996.02

$d_{100} = 23.9$ %

Calc: ACS 4-26-99

Ckd: TWC 5/3/99





PRESSURE INCREMENT #17
from 1.00 tsf to 0.25 tsf

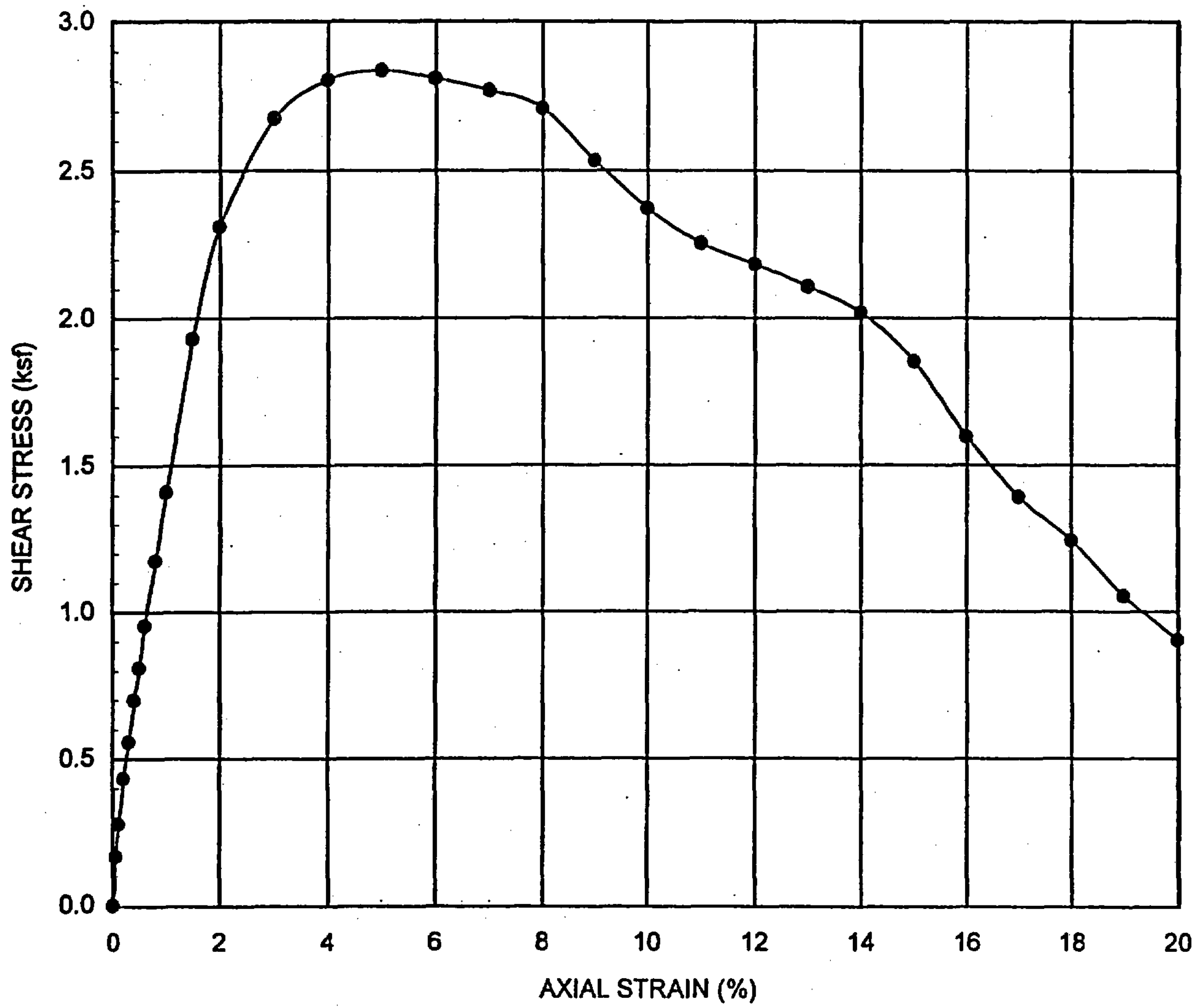
Test No: 4
Testname: CTBS-U3C

APPENDIX B

Triaxial Test Plots and Data

STONE & WEBSTER ENGINEERING CORPORATION

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-1	DATE:	04/30/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	8.4 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.546 ft	WATER CONTENT:	47.9 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	62.1 pcf
AREA:	0.0444 ft^2	INITIAL VOID RATIO:	1.73

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.84 ksf
COMPRESSIVE STRENGTH:	5.67 ksf
FAILURE STRAIN:	5.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-1	DATE:	04/30/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	8.4 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		

HEIGHT:	0.546 ft	WATER CONTENT:	47.9 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	62.1 pcf
AREA:	0.0444 ft^2	INITIAL VOID RATIO:	1.73

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.84 ksf
COMPRESSIVE STRENGTH:	5.67 ksf
FAILURE STRAIN:	5.0 %

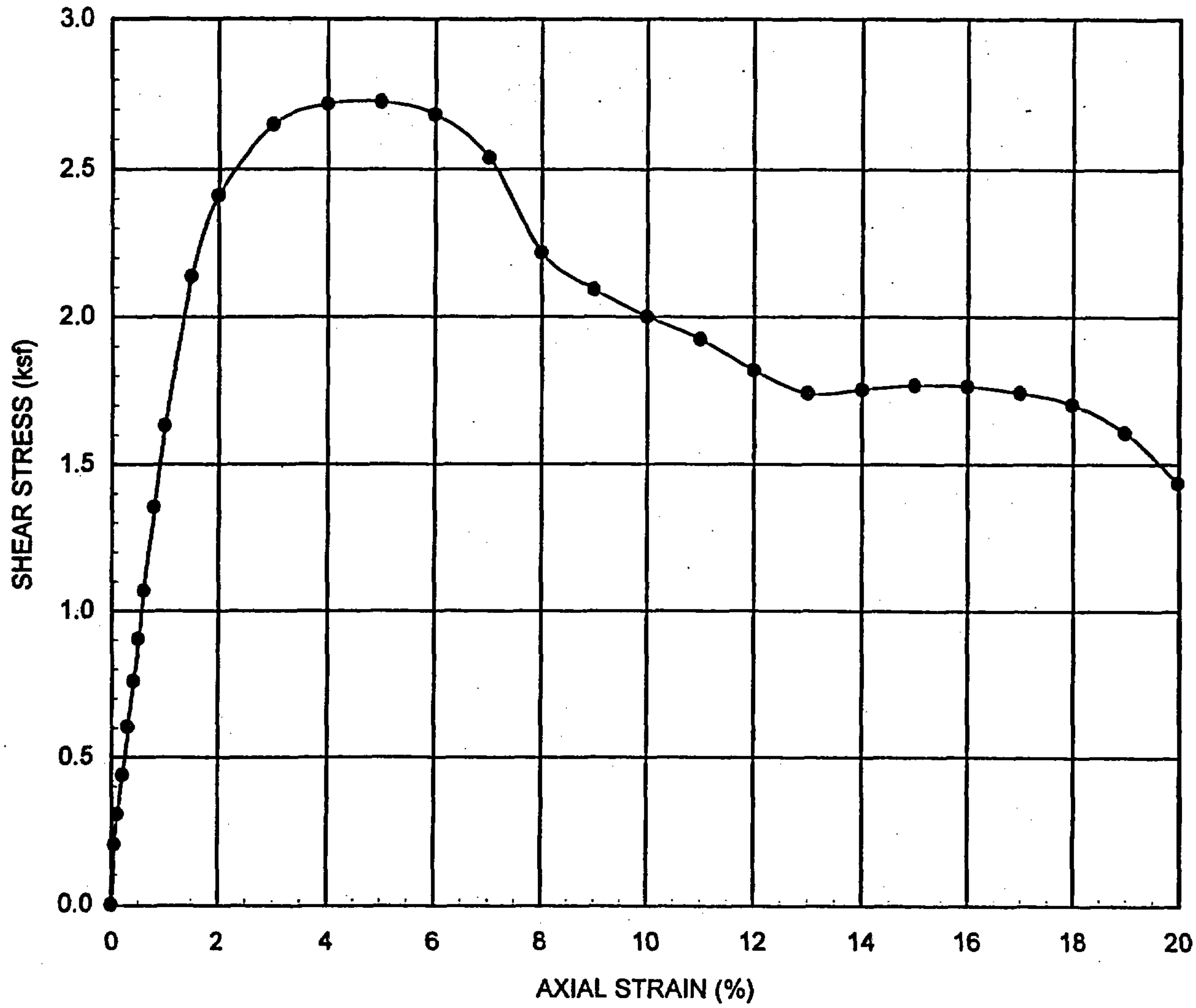
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.33	1.30	0.00	0.000	0.0444	0.00	0.00
1.41	6.04	0.05	0.015	0.0445	0.33	0.17
1.50	9.21	0.10	0.025	0.0445	0.55	0.28
1.66	13.71	0.20	0.039	0.0445	0.87	0.43
1.83	17.27	0.30	0.050	0.0446	1.11	0.56
2.00	21.36	0.40	0.062	0.0446	1.40	0.70
2.16	24.55	0.50	0.072	0.0447	1.62	0.81
2.33	28.70	0.60	0.085	0.0447	1.91	0.95
2.66	35.15	0.80	0.105	0.0448	2.35	1.18
2.99	42.00	1.00	0.127	0.0449	2.82	1.41
3.82	57.30	1.50	0.174	0.0451	3.86	1.93
4.66	68.70	2.00	0.210	0.0453	4.62	2.31
6.32	80.12	3.00	0.245	0.0458	5.35	2.68
7.98	84.78	4.00	0.260	0.0463	5.61	2.80
9.65	86.62	5.00	0.265	0.0468	5.67	2.84
11.31	86.72	6.00	0.266	0.0473	5.62	2.81
12.97	86.42	7.00	0.265	0.0478	5.54	2.77
14.63	85.48	8.00	0.262	0.0483	5.42	2.71
16.30	80.84	9.00	0.247	0.0488	5.07	2.53
17.96	76.66	10.00	0.234	0.0494	4.75	2.37
19.62	73.68	11.00	0.225	0.0499	4.51	2.25
21.29	72.20	12.00	0.220	0.0505	4.37	2.18
22.95	70.54	13.00	0.215	0.0511	4.22	2.11
24.61	68.40	14.00	0.209	0.0517	4.04	2.02
26.28	63.63	15.00	0.194	0.0523	3.71	1.85
27.94	55.65	16.00	0.169	0.0529	3.20	1.60
29.60	49.20	17.00	0.149	0.0535	2.78	1.39
31.27	44.60	18.00	0.135	0.0542	2.48	1.24
32.93	38.45	19.00	0.116	0.0549	2.11	1.05
34.59	33.56	20.00	0.100	0.0555	1.81	0.90

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-1	DATE:	05/03/99
SAMPLE:	U-7D	TESTED BY:	ACS
DEPTH:	21.4 ft	CHECKED:	JWM
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.547 ft	WATER CONTENT:	45.1 %
DIAMETER:	0.239 ft	INITIAL DRY UNIT WEIGHT:	62.9 pcf
AREA:	0.0447 ft^2	INITIAL VOID RATIO:	1.70

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.73 ksf
COMPRESSIVE STRENGTH:	5.45 ksf
FAILURE STRAIN:	5.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-1	DATE:	05/03/99
SAMPLE:	U-7D	TESTED BY:	ACS
DEPTH:	21.4 ft	CHECKED:	JWM
DESCRIPTION:	Clayey SILT		

HEIGHT:	0.547 ft	WATER CONTENT:	45.1 %
DIAMETER:	0.239 ft	INITIAL DRY UNIT WEIGHT	62.9 pcf
AREA:	0.0447 ft^2	INITIAL VOID RATIO:	1.70

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

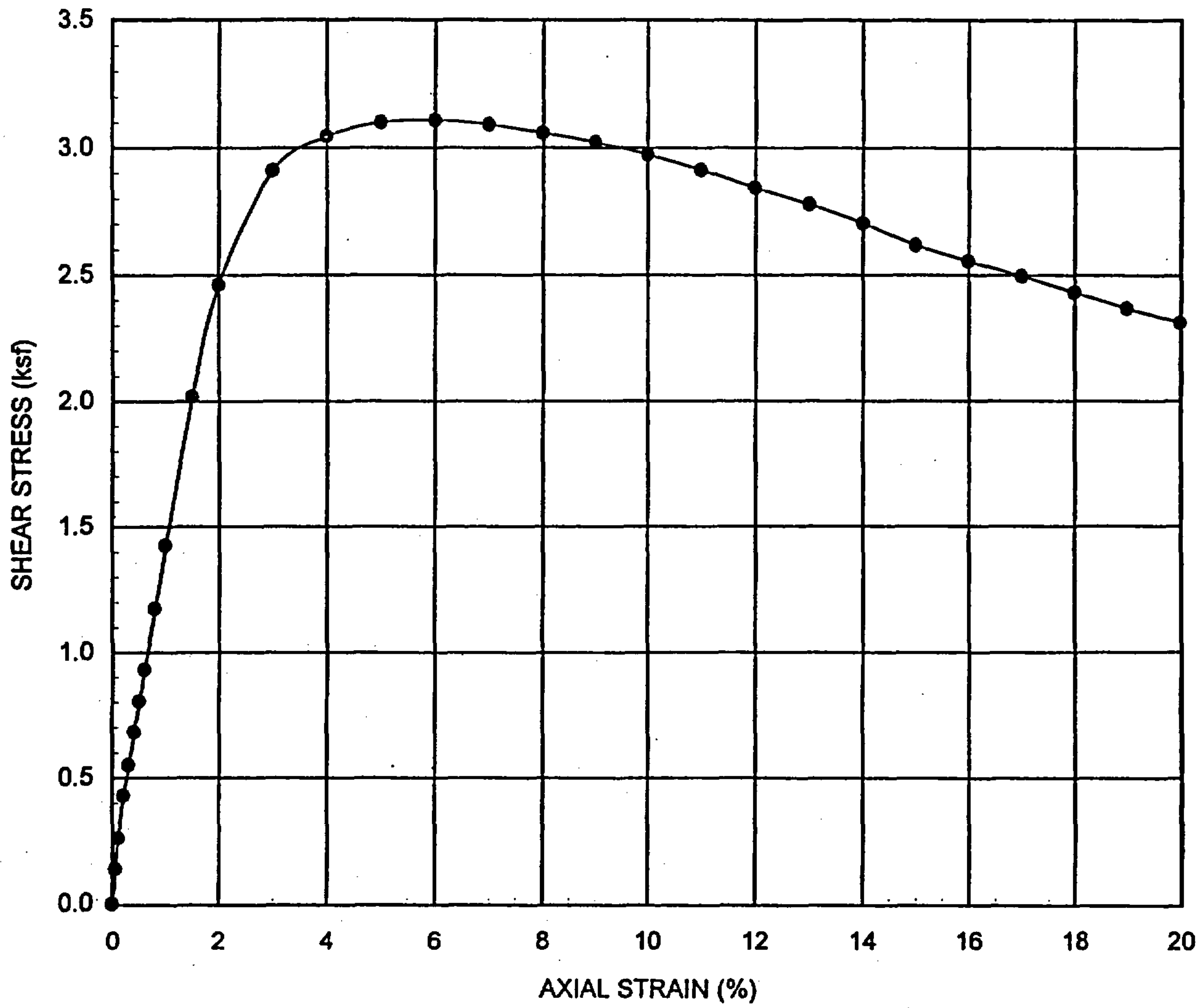
UNDRAINED SHEAR STRENGTH:	2.73 ksf
COMPRESSIVE STRENGTH:	5.45 ksf
FAILURE STRAIN:	5.0 %

DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.96	1.25	0.00	0.000	0.0447	0.00	0.00
1.04	7.09	0.05	0.018	0.0447	0.41	0.20
1.13	10.03	0.10	0.027	0.0447	0.61	0.31
1.29	13.91	0.20	0.039	0.0448	0.88	0.44
1.46	18.66	0.30	0.054	0.0448	1.21	0.60
1.63	23.16	0.40	0.068	0.0449	1.52	0.76
1.79	27.38	0.50	0.081	0.0449	1.81	0.90
1.96	32.14	0.60	0.096	0.0450	2.14	1.07
2.29	40.46	0.80	0.122	0.0450	2.71	1.35
2.63	48.65	1.00	0.147	0.0451	3.27	1.63
3.46	63.60	1.50	0.194	0.0454	4.27	2.14
4.29	71.97	2.00	0.220	0.0456	4.82	2.41
5.96	79.76	3.00	0.244	0.0461	5.30	2.65
7.63	82.62	4.00	0.253	0.0465	5.44	2.72
9.29	83.69	5.00	0.256	0.0470	5.45	2.73
10.96	83.27	6.00	0.255	0.0475	5.37	2.68
12.63	79.70	7.00	0.244	0.0480	5.08	2.54
14.29	70.52	8.00	0.215	0.0486	4.44	2.22
15.96	67.34	9.00	0.206	0.0491	4.19	2.09
17.63	65.14	10.00	0.199	0.0496	4.00	2.00
19.29	63.41	11.00	0.193	0.0502	3.85	1.93
20.96	60.63	12.00	0.185	0.0508	3.64	1.82
22.63	58.76	13.00	0.179	0.0514	3.48	1.74
24.30	59.85	14.00	0.182	0.0520	3.51	1.75
25.96	61.02	15.00	0.186	0.0526	3.54	1.77
27.63	61.62	16.00	0.188	0.0532	3.53	1.76
29.30	61.57	17.00	0.188	0.0538	3.48	1.74
30.96	60.92	18.00	0.186	0.0545	3.41	1.70
32.63	58.30	19.00	0.177	0.0552	3.22	1.61
34.30	52.83	20.00	0.160	0.0559	2.87	1.44

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC. JO: 05996.02
PFSF, Skull Valley, UT May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	04/29/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	9.2 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.552 ft	WATER CONTENT:	45.2 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	60.4 pcf
AREA:	0.0446 ft^2	INITIAL VOID RATIO:	1.81

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	3.11 ksf
COMPRESSIVE STRENGTH:	6.22 ksf
FAILURE STRAIN:	6.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	04/29/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	9.2 ft	CHECKED:	Jwm
DESCRIPTION:	CLAY		

HEIGHT:	0.552 ft	WATER CONTENT:	45.2 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	60.4 pcf
AREA:	0.0446 ft^2	INITIAL VOID RATIO:	1.81

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH: 3.11 ksf
COMPRESSIVE STRENGTH: 6.22 ksf
FAILURE STRAIN: 6.0 %

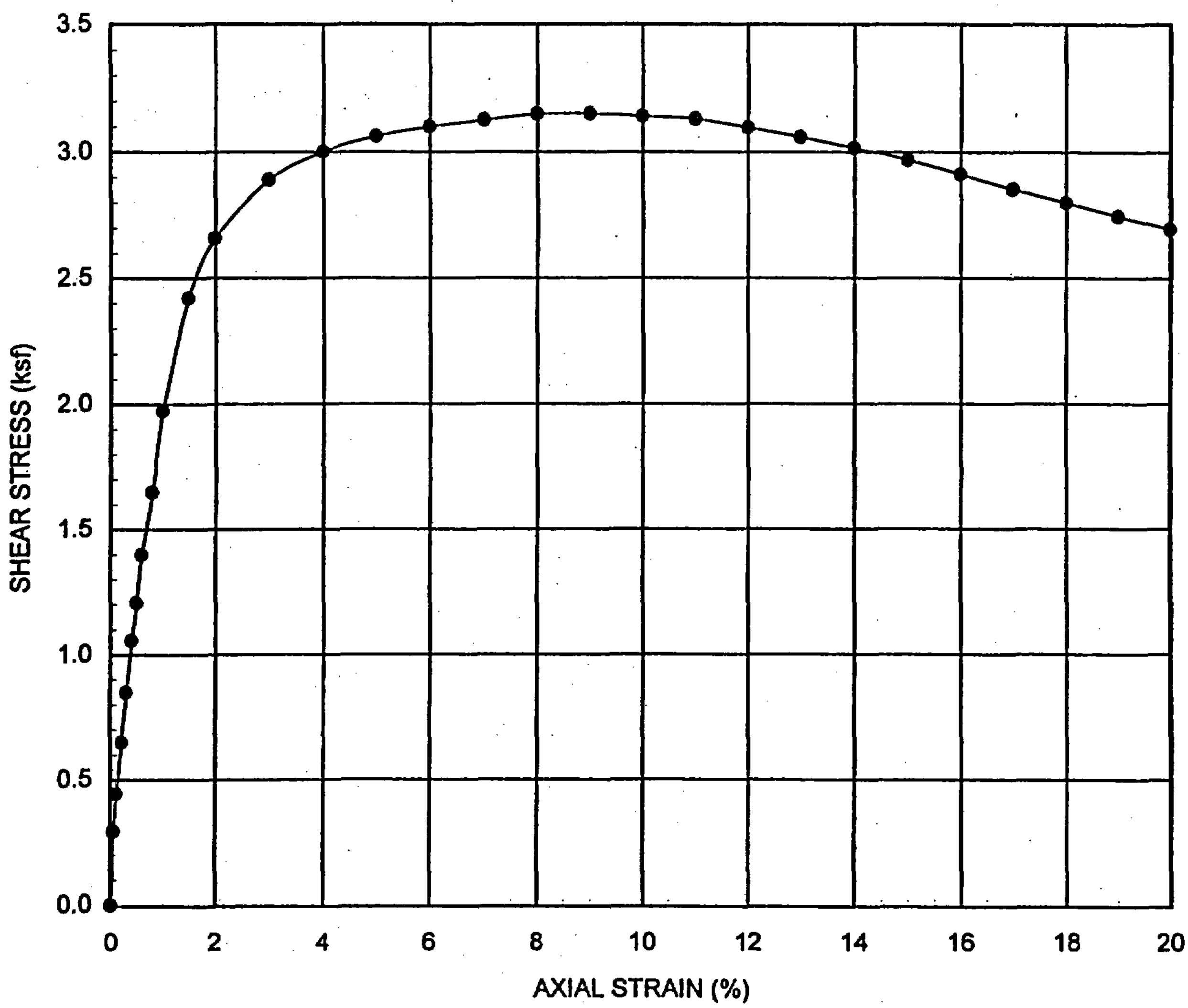
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.25	1.40	0.00	0.000	0.0446	0.00	0.00
1.33	5.36	0.05	0.012	0.0446	0.28	0.14
1.42	8.90	0.10	0.023	0.0446	0.52	0.26
1.59	13.75	0.20	0.038	0.0447	0.86	0.43
1.75	17.26	0.30	0.049	0.0447	1.10	0.55
1.92	21.01	0.40	0.061	0.0448	1.36	0.68
2.09	24.54	0.50	0.072	0.0448	1.61	0.80
2.26	28.26	0.60	0.084	0.0449	1.86	0.93
2.60	35.36	0.80	0.106	0.0449	2.35	1.17
2.93	42.70	1.00	0.128	0.0450	2.85	1.43
3.77	60.14	1.50	0.183	0.0453	4.04	2.02
4.61	73.43	2.00	0.224	0.0455	4.92	2.46
6.30	87.42	3.00	0.268	0.0460	5.82	2.91
7.98	92.33	4.00	0.283	0.0464	6.09	3.04
9.66	94.94	5.00	0.291	0.0469	6.20	3.10
11.34	96.20	6.00	0.295	0.0474	6.22	3.11
13.02	96.70	7.00	0.296	0.0479	6.18	3.09
14.70	96.70	8.00	0.296	0.0485	6.12	3.06
16.39	96.62	9.00	0.296	0.0490	6.04	3.02
18.07	96.11	10.00	0.295	0.0495	5.95	2.97
19.75	95.23	11.00	0.292	0.0501	5.83	2.91
21.43	94.00	12.00	0.288	0.0507	5.68	2.84
23.11	92.94	13.00	0.285	0.0512	5.56	2.78
24.79	91.51	14.00	0.280	0.0518	5.41	2.70
26.48	89.70	15.00	0.275	0.0525	5.24	2.62
28.16	88.58	16.00	0.271	0.0531	5.11	2.55
29.84	87.62	17.00	0.268	0.0537	4.99	2.50
31.52	86.39	18.00	0.264	0.0544	4.86	2.43
33.20	85.21	19.00	0.261	0.0550	4.74	2.37
34.88	84.27	20.00	0.258	0.0557	4.62	2.31

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	05/14/99
SAMPLE:	U-11D	TESTED BY:	ACS
DEPTH:	20.9 ft	CHECKED:	JWM
DESCRIPTION:	SILT		
HEIGHT:	0.551 ft	WATER CONTENT:	31.5 %
DIAMETER:	0.239 ft	INITIAL DRY UNIT WEIGHT:	68.4 pcf
AREA:	0.0447 ft^2	INITIAL VOID RATIO:	1.48

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	3.15 ksf
COMPRESSIVE STRENGTH:	6.30 ksf
FAILURE STRAIN:	8.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	05/14/99
SAMPLE:	U-11D	TESTED BY:	ACS
DEPTH:	20.9 ft	CHECKED:	Jwm
DESCRIPTION:	SILT		
HEIGHT:	0.551 ft	WATER CONTENT:	31.5 %
DIAMETER:	0.239 ft	INITIAL DRY UNIT WEIGHT	68.4 pcf
AREA:	0.0447 ft^2	INITIAL VOID RATIO:	1.48

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH: 3.15 ksf
COMPRESSIVE STRENGTH: 6.30 ksf
FAILURE STRAIN: 8.0 %

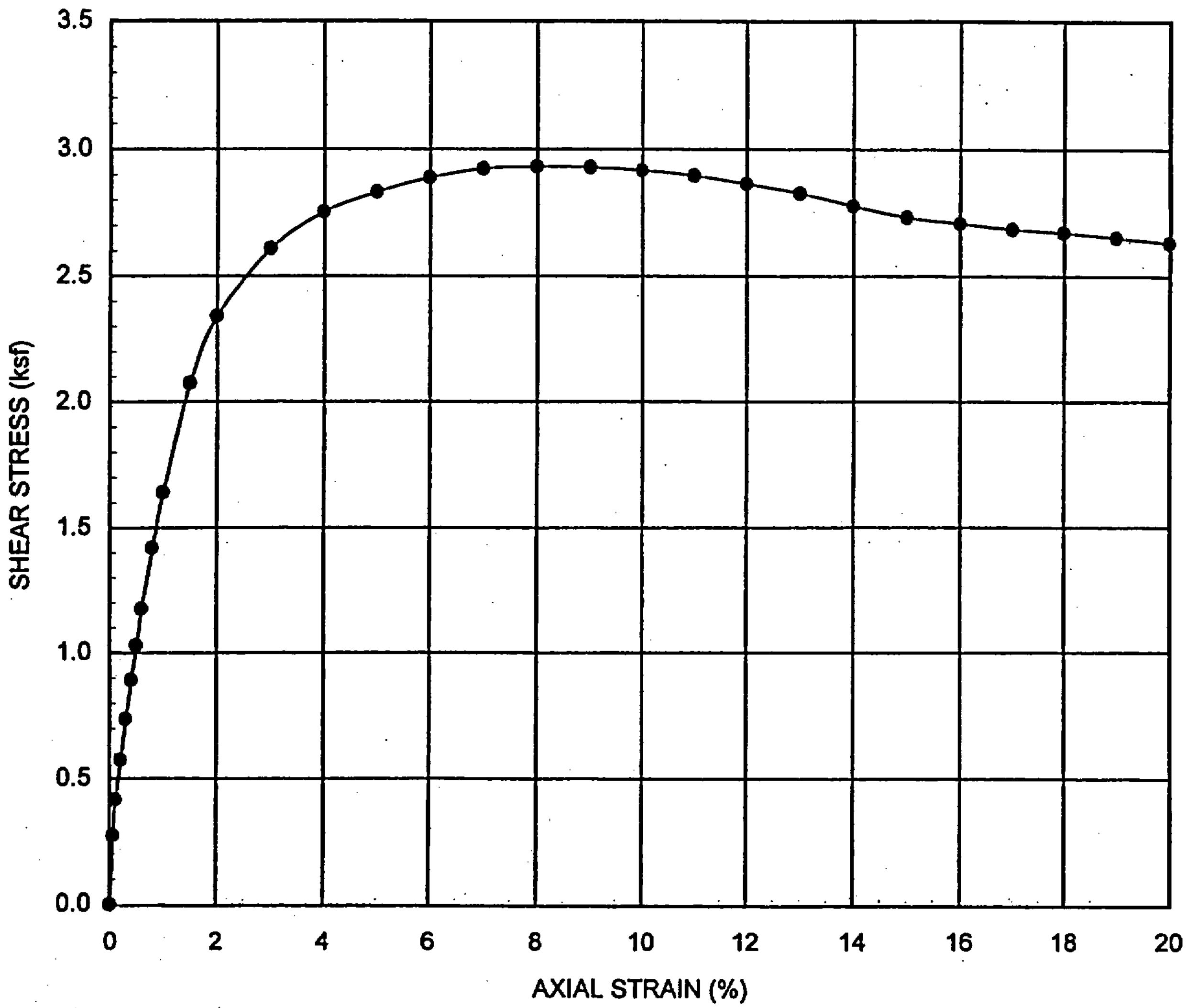
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.56	1.30	0.00	0.000	0.0447	0.00	0.00
0.64	9.74	0.05	0.026	0.0448	0.59	0.29
0.73	14.09	0.10	0.040	0.0448	0.89	0.44
0.90	20.00	0.20	0.058	0.0448	1.30	0.65
1.06	25.78	0.30	0.076	0.0449	1.70	0.85
1.23	31.80	0.40	0.095	0.0449	2.11	1.06
1.40	36.17	0.50	0.108	0.0450	2.41	1.21
1.57	41.78	0.60	0.126	0.0450	2.80	1.40
1.90	49.12	0.80	0.149	0.0451	3.30	1.65
2.24	58.67	1.00	0.178	0.0452	3.95	1.97
3.08	72.03	1.50	0.220	0.0454	4.84	2.42
3.92	79.36	2.00	0.243	0.0457	5.32	2.66
5.60	87.03	3.00	0.267	0.0461	5.78	2.89
7.27	91.25	4.00	0.280	0.0466	6.00	3.00
8.95	94.01	5.00	0.288	0.0471	6.12	3.06
10.63	96.20	6.00	0.295	0.0476	6.20	3.10
12.31	98.00	7.00	0.301	0.0481	6.25	3.13
13.99	99.80	8.00	0.306	0.0486	6.30	3.15
15.67	100.92	9.00	0.310	0.0492	6.30	3.15
17.34	101.74	10.00	0.312	0.0497	6.28	3.14
19.02	102.50	11.00	0.315	0.0503	6.26	3.13
20.70	102.55	12.00	0.315	0.0508	6.19	3.10
22.38	102.48	13.00	0.315	0.0514	6.12	3.06
24.06	102.20	14.00	0.314	0.0520	6.03	3.02
25.74	101.80	15.00	0.313	0.0526	5.94	2.97
27.41	101.05	16.00	0.310	0.0533	5.82	2.91
29.09	100.20	17.00	0.308	0.0539	5.71	2.85
30.77	99.56	18.00	0.306	0.0546	5.60	2.80
32.45	98.90	19.00	0.304	0.0552	5.50	2.75
34.13	98.30	20.00	0.302	0.0559	5.39	2.70

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	05/17/99
SAMPLE:	U-10D	TESTED BY:	ACS
DEPTH:	19.1 ft	CHECKED:	JWM
DESCRIPTION:	SILT		
HEIGHT:	0.545 ft	WATER CONTENT:	27.7 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT:	74.0 pcf
AREA:	0.0440 ft^2	INITIAL VOID RATIO:	1.29

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.93 ksf
COMPRESSIVE STRENGTH:	5.86 ksf
FAILURE STRAIN:	8.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	05/17/99
SAMPLE:	U-10D	TESTED BY:	ACS
DEPTH:	19.1 ft	CHECKED:	JWM
DESCRIPTION:	SILT		

HEIGHT:	0.545 ft	WATER CONTENT:	27.7 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT	74.0 pcf
AREA:	0.0440 ft^2	INITIAL VOID RATIO:	1.29

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.93 ksf
COMPRESSIVE STRENGTH:	5.86 ksf
FAILURE STRAIN:	8.0 %

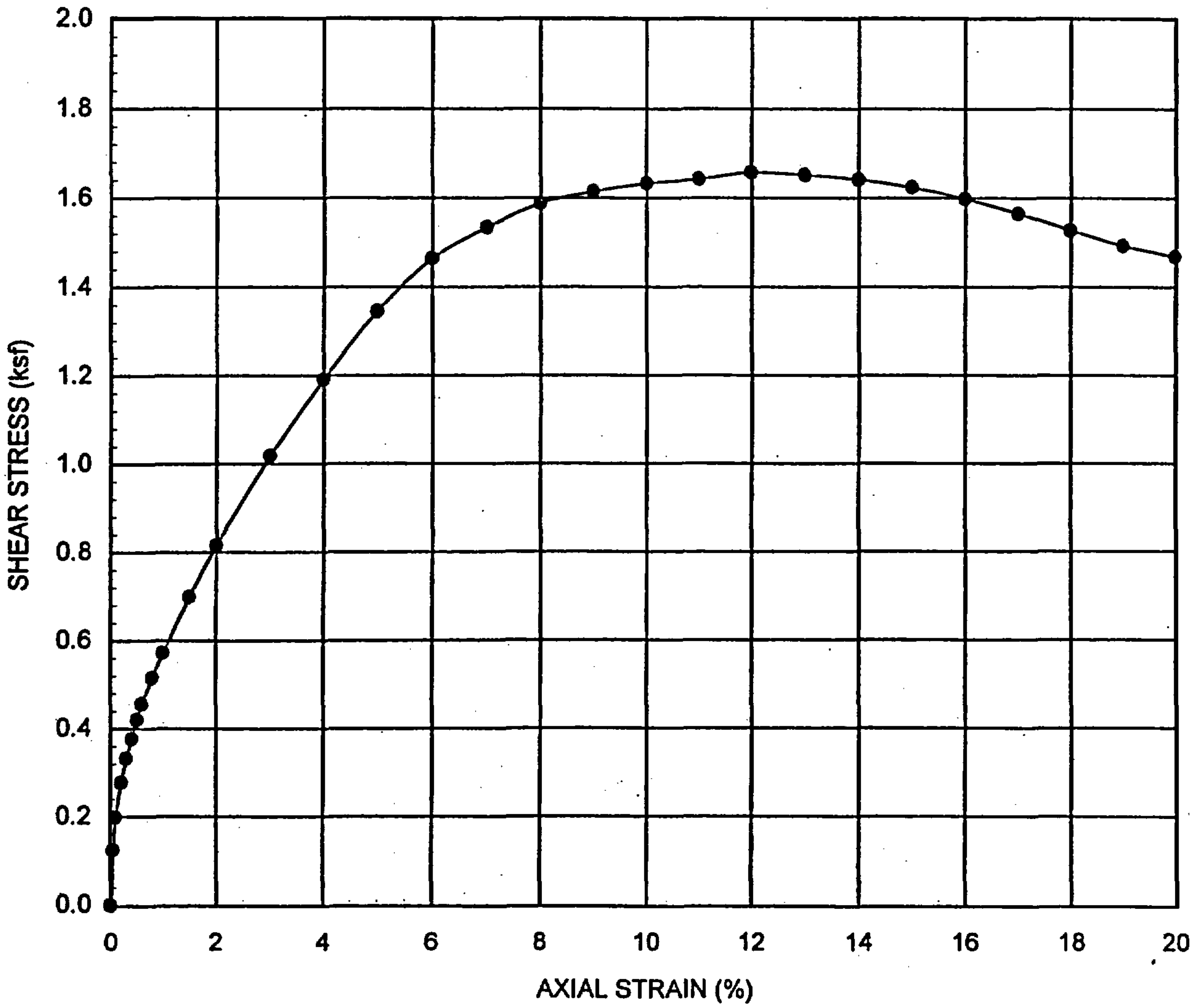
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.03	1.25	0.00	0.000	0.0440	0.00	0.00
1.11	9.00	0.05	0.024	0.0440	0.55	0.27
1.20	13.03	0.10	0.037	0.0441	0.83	0.42
1.36	17.52	0.20	0.051	0.0441	1.15	0.57
1.53	22.17	0.30	0.065	0.0441	1.47	0.74
1.69	26.59	0.40	0.079	0.0442	1.78	0.89
1.86	30.54	0.50	0.091	0.0442	2.06	1.03
2.03	34.74	0.60	0.104	0.0443	2.35	1.18
2.36	41.76	0.80	0.126	0.0444	2.84	1.42
2.69	48.15	1.00	0.146	0.0445	3.28	1.64
3.52	60.83	1.50	0.185	0.0447	4.15	2.07
4.35	68.85	2.00	0.210	0.0449	4.68	2.34
6.01	77.34	3.00	0.237	0.0454	5.22	2.61
7.67	82.40	4.00	0.252	0.0458	5.51	2.75
9.34	85.60	5.00	0.262	0.0463	5.66	2.83
11.00	88.20	6.00	0.270	0.0468	5.78	2.89
12.66	90.22	7.00	0.277	0.0473	5.85	2.92
14.32	91.45	8.00	0.281	0.0478	5.86	2.93
15.98	92.36	9.00	0.283	0.0484	5.86	2.93
17.64	93.00	10.00	0.285	0.0489	5.84	2.92
19.30	93.36	11.00	0.286	0.0494	5.79	2.90
20.96	93.36	12.00	0.286	0.0500	5.73	2.86
22.62	93.20	13.00	0.286	0.0506	5.65	2.83
24.28	92.65	14.00	0.284	0.0512	5.55	2.78
25.95	92.25	15.00	0.283	0.0518	5.47	2.73
27.61	92.52	16.00	0.284	0.0524	5.42	2.71
29.27	92.84	17.00	0.285	0.0530	5.37	2.69
30.93	93.50	18.00	0.287	0.0537	5.35	2.67
32.59	93.94	19.00	0.288	0.0543	5.31	2.65
34.25	94.38	20.00	0.290	0.0550	5.27	2.63

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	04/29/99
SAMPLE:	U-14D	TESTED BY:	ACS
DEPTH:	26.7 ft	CHECKED:	JWM
DESCRIPTION:	Silty CLAY		
HEIGHT:	0.550 ft	WATER CONTENT:	30.5 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	87.2 pcf
AREA:	0.0444 ft^2	INITIAL VOID RATIO:	0.95

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	1.66 ksf
COMPRESSIVE STRENGTH:	3.32 ksf
FAILURE STRAIN:	12.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	04/29/99
SAMPLE:	U-14D	TESTED BY:	ACS
DEPTH:	26.7 ft	CHECKED:	Jwm
DESCRIPTION:	Silty CLAY		
HEIGHT:	0.550 ft	WATER CONTENT:	30.5 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	87.2 pcf
AREA:	0.0444 ft^2	INITIAL VOID RATIO:	0.95

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH: 1.66 ksf
COMPRESSIVE STRENGTH: 3.32 ksf
FAILURE STRAIN: 12.0 %

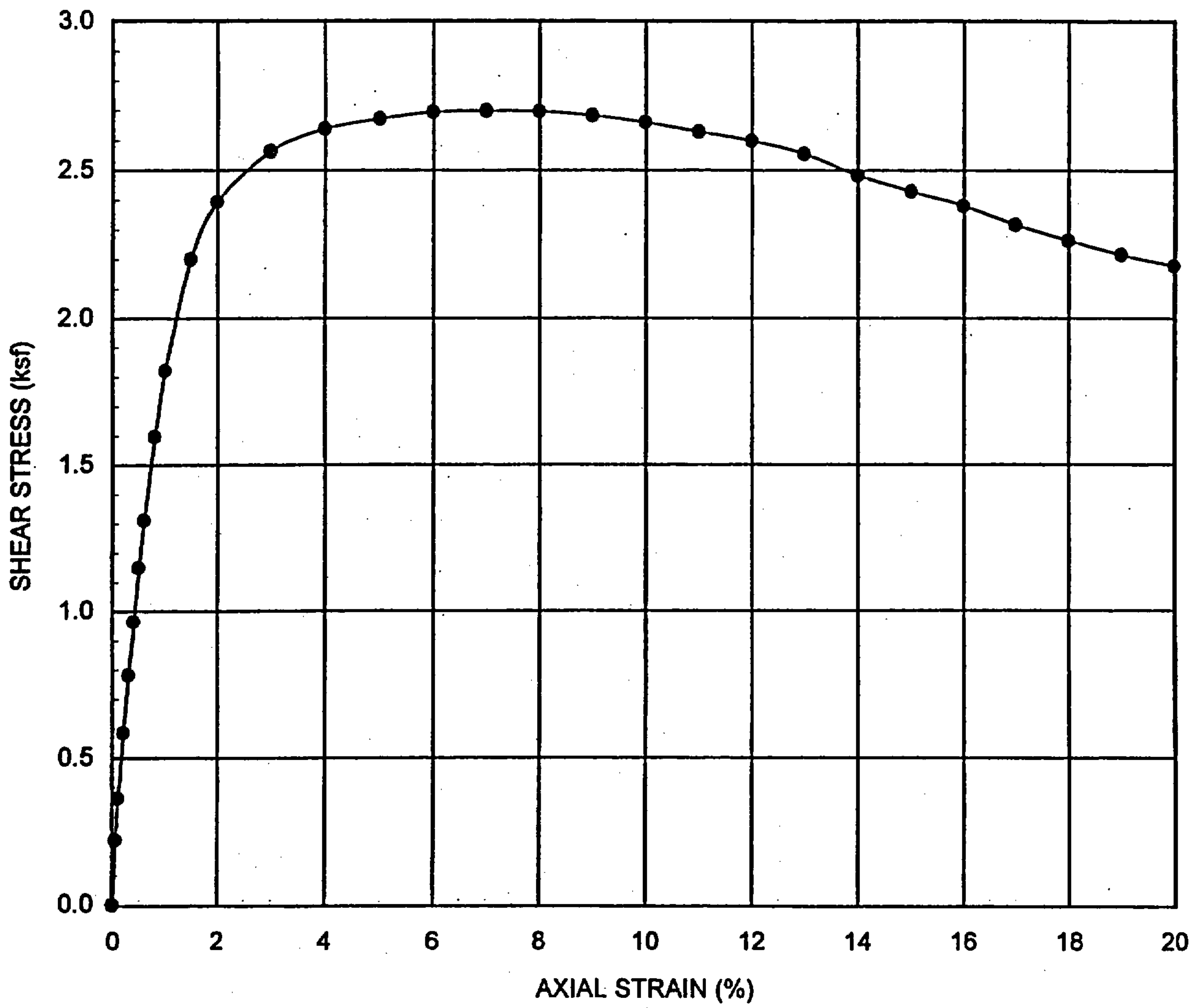
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.70	1.28	0.00	0.000	0.0444	0.00	0.00
1.78	4.84	0.05	0.011	0.0444	0.25	0.12
1.87	6.96	0.10	0.018	0.0444	0.40	0.20
2.04	9.21	0.20	0.025	0.0444	0.55	0.28
2.20	10.78	0.30	0.030	0.0445	0.66	0.33
2.37	12.08	0.40	0.034	0.0445	0.75	0.38
2.54	13.31	0.50	0.037	0.0446	0.84	0.42
2.71	14.36	0.60	0.041	0.0446	0.91	0.46
3.04	16.09	0.80	0.046	0.0447	1.03	0.52
3.38	17.78	1.00	0.051	0.0448	1.15	0.57
4.22	21.54	1.50	0.063	0.0450	1.40	0.70
5.05	24.99	2.00	0.074	0.0453	1.63	0.81
6.73	31.21	3.00	0.093	0.0457	2.04	1.02
8.41	36.67	4.00	0.110	0.0462	2.38	1.19
10.09	41.66	5.00	0.126	0.0467	2.69	1.35
11.76	45.70	6.00	0.138	0.0472	2.93	1.46
13.44	48.32	7.00	0.146	0.0477	3.07	1.53
15.12	50.51	8.00	0.153	0.0482	3.18	1.59
16.79	51.88	9.00	0.157	0.0487	3.23	1.61
18.47	53.05	10.00	0.161	0.0493	3.27	1.63
20.15	53.93	11.00	0.164	0.0498	3.29	1.64
21.83	55.01	12.00	0.167	0.0504	3.32	1.66
23.50	55.44	13.00	0.168	0.0510	3.30	1.65
25.18	55.72	14.00	0.169	0.0516	3.28	1.64
26.86	55.76	15.00	0.169	0.0522	3.25	1.62
28.53	55.55	16.00	0.169	0.0528	3.20	1.60
30.21	55.06	17.00	0.167	0.0534	3.13	1.57
31.89	54.38	18.00	0.165	0.0541	3.05	1.53
33.57	53.82	19.00	0.163	0.0548	2.98	1.49
35.24	53.62	20.00	0.163	0.0554	2.94	1.47

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	05/18/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	8 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.561 ft	WATER CONTENT:	52.7 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	56.2 pcf
AREA:	0.0445 ft^2	INITIAL VOID RATIO:	2.02

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.70 ksf
COMPRESSIVE STRENGTH:	5.40 ksf
FAILURE STRAIN:	7.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	05/18/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	8.0 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.561 ft	WATER CONTENT:	52.7 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	56.2 pcf
AREA:	0.0445 ft^2	INITIAL VOID RATIO:	2.02

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.70 ksf
COMPRESSIVE STRENGTH:	5.40 ksf
FAILURE STRAIN:	7.0 %

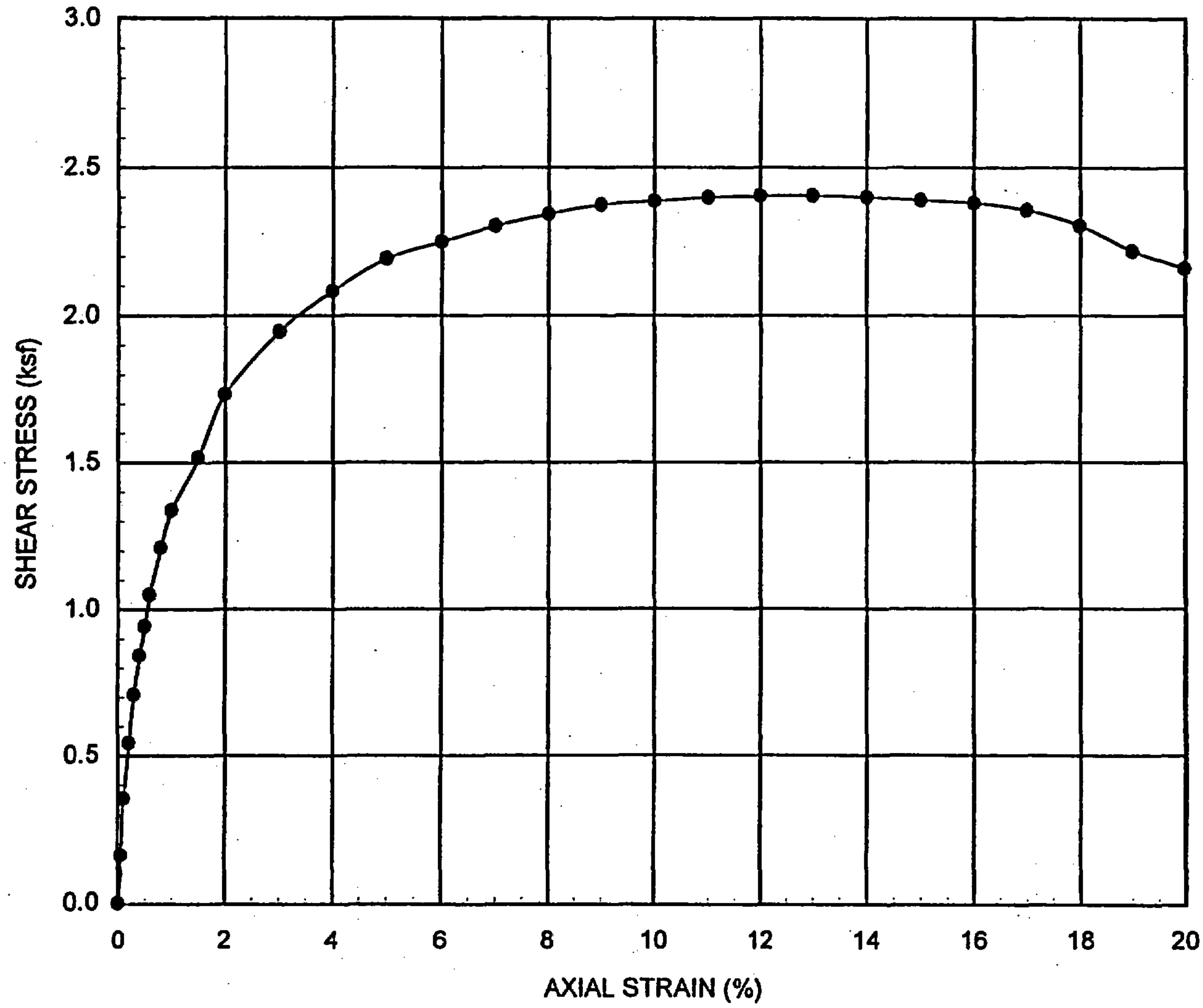
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.65	1.25	0.00	0.000	0.0445	0.00	0.00
0.74	7.59	0.05	0.020	0.0445	0.44	0.22
0.82	11.65	0.10	0.032	0.0446	0.73	0.36
0.99	18.04	0.20	0.052	0.0446	1.17	0.59
1.16	23.70	0.30	0.070	0.0446	1.56	0.78
1.33	28.98	0.40	0.086	0.0447	1.93	0.96
1.51	34.35	0.50	0.103	0.0447	2.30	1.15
1.68	39.03	0.60	0.117	0.0448	2.62	1.31
2.02	47.35	0.80	0.143	0.0449	3.20	1.60
2.36	53.88	1.00	0.164	0.0450	3.64	1.82
3.22	65.19	1.50	0.199	0.0452	4.40	2.20
4.07	71.18	2.00	0.217	0.0454	4.79	2.39
5.78	76.90	3.00	0.235	0.0459	5.13	2.56
7.49	79.92	4.00	0.245	0.0464	5.28	2.64
9.20	81.75	5.00	0.250	0.0469	5.34	2.67
10.91	83.30	6.00	0.255	0.0474	5.39	2.69
12.62	84.32	7.00	0.258	0.0479	5.40	2.70
14.33	85.17	8.00	0.261	0.0484	5.39	2.70
16.04	85.65	9.00	0.262	0.0489	5.37	2.68
17.76	85.88	10.00	0.263	0.0495	5.32	2.66
19.47	85.80	11.00	0.263	0.0500	5.26	2.63
21.18	85.76	12.00	0.263	0.0506	5.20	2.60
22.89	85.30	13.00	0.261	0.0512	5.11	2.55
24.60	83.90	14.00	0.257	0.0518	4.97	2.48
26.31	83.06	15.00	0.254	0.0524	4.86	2.43
28.02	82.36	16.00	0.252	0.0530	4.76	2.38
29.73	81.15	17.00	0.248	0.0536	4.63	2.32
31.44	80.25	18.00	0.246	0.0543	4.53	2.26
33.15	79.56	19.00	0.244	0.0550	4.43	2.22
34.86	79.20	20.00	0.242	0.0556	4.36	2.18

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	04/28/99
SAMPLE:	U-2B	TESTED BY:	ACS
DEPTH:	7.4 ft	CHECKED:	JWM
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.550 ft	WATER CONTENT:	65.4 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	45.1 pcf
AREA:	0.0443 ft^2	INITIAL VOID RATIO:	2.76

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.41 ksf
COMPRESSIVE STRENGTH:	4.81 ksf
FAILURE STRAIN:	13.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	04/28/99
SAMPLE:	U-2B	TESTED BY:	ACS
DEPTH:	7.4 ft	CHECKED:	JWM
DESCRIPTION:	Clayey SILT		

HEIGHT:	0.550 ft	WATER CONTENT:	65.4 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	45.1 pcf
AREA:	0.0443 ft^2	INITIAL VOID RATIO:	2.76

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.41 ksf
COMPRESSIVE STRENGTH:	4.81 ksf
FAILURE STRAIN:	13.0 %

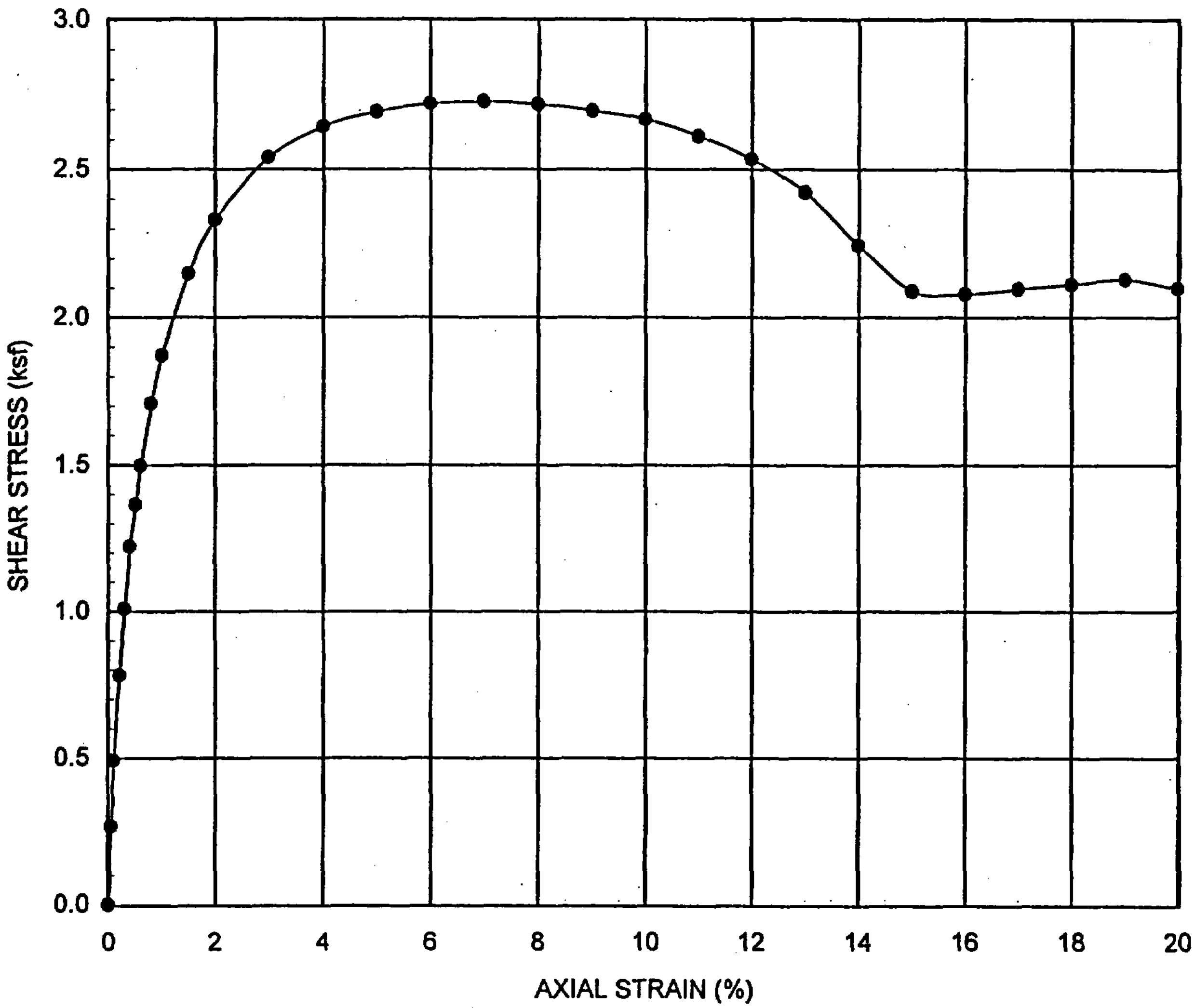
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.65	1.40	0.00	0.000	0.0443	0.00	0.00
0.73	6.04	0.05	0.014	0.0444	0.33	0.16
0.82	11.52	0.10	0.031	0.0444	0.71	0.35
0.98	16.95	0.20	0.048	0.0444	1.09	0.54
1.15	21.68	0.30	0.063	0.0445	1.42	0.71
1.32	25.53	0.40	0.075	0.0445	1.69	0.84
1.49	28.42	0.50	0.084	0.0446	1.89	0.94
1.65	31.54	0.60	0.094	0.0446	2.10	1.05
1.99	36.19	0.80	0.108	0.0447	2.42	1.21
2.32	39.90	1.00	0.120	0.0448	2.67	1.34
3.16	45.30	1.50	0.137	0.0450	3.03	1.52
4.00	51.83	2.00	0.157	0.0452	3.47	1.73
5.67	58.60	3.00	0.178	0.0457	3.89	1.95
7.35	63.25	4.00	0.192	0.0462	4.16	2.08
9.02	67.18	5.00	0.205	0.0467	4.38	2.19
10.70	69.64	6.00	0.212	0.0472	4.50	2.25
12.37	72.00	7.00	0.220	0.0477	4.61	2.30
14.05	74.00	8.00	0.226	0.0482	4.68	2.34
15.72	75.80	9.00	0.231	0.0487	4.75	2.37
17.40	77.02	10.00	0.235	0.0493	4.77	2.39
19.07	78.25	11.00	0.239	0.0498	4.80	2.40
20.75	79.30	12.00	0.242	0.0504	4.81	2.40
22.42	80.23	13.00	0.245	0.0510	4.81	2.41
24.10	80.97	14.00	0.247	0.0516	4.80	2.40
25.77	81.57	15.00	0.249	0.0522	4.78	2.39
27.45	82.18	16.00	0.251	0.0528	4.76	2.38
29.12	82.35	17.00	0.252	0.0534	4.71	2.36
30.80	81.50	18.00	0.249	0.0541	4.61	2.30
32.47	79.50	19.00	0.243	0.0547	4.44	2.22
34.15	78.40	20.00	0.239	0.0554	4.32	2.16

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	04/28/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	10.2 ft	CHECKED:	
DESCRIPTION:	CLAY		
HEIGHT:	0.553 ft	WATER CONTENT:	52.2 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT:	56.7 pcf
AREA:	0.0443 ft^2	INITIAL VOID RATIO:	1.98

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.73 ksf
COMPRESSIVE STRENGTH:	5.45 ksf
FAILURE STRAIN:	7.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	04/28/99
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	10.2 ft	CHECKED:	
DESCRIPTION:	CLAY		
HEIGHT:	0.553 ft	WATER CONTENT:	52.2 %
DIAMETER:	0.238 ft	INITIAL DRY UNIT WEIGHT	56.7 pcf
AREA:	0.0443 ft^2	INITIAL VOID RATIO:	1.98

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH: 2.73 ksf
COMPRESSIVE STRENGTH: 5.45 ksf
FAILURE STRAIN: 7.0 %

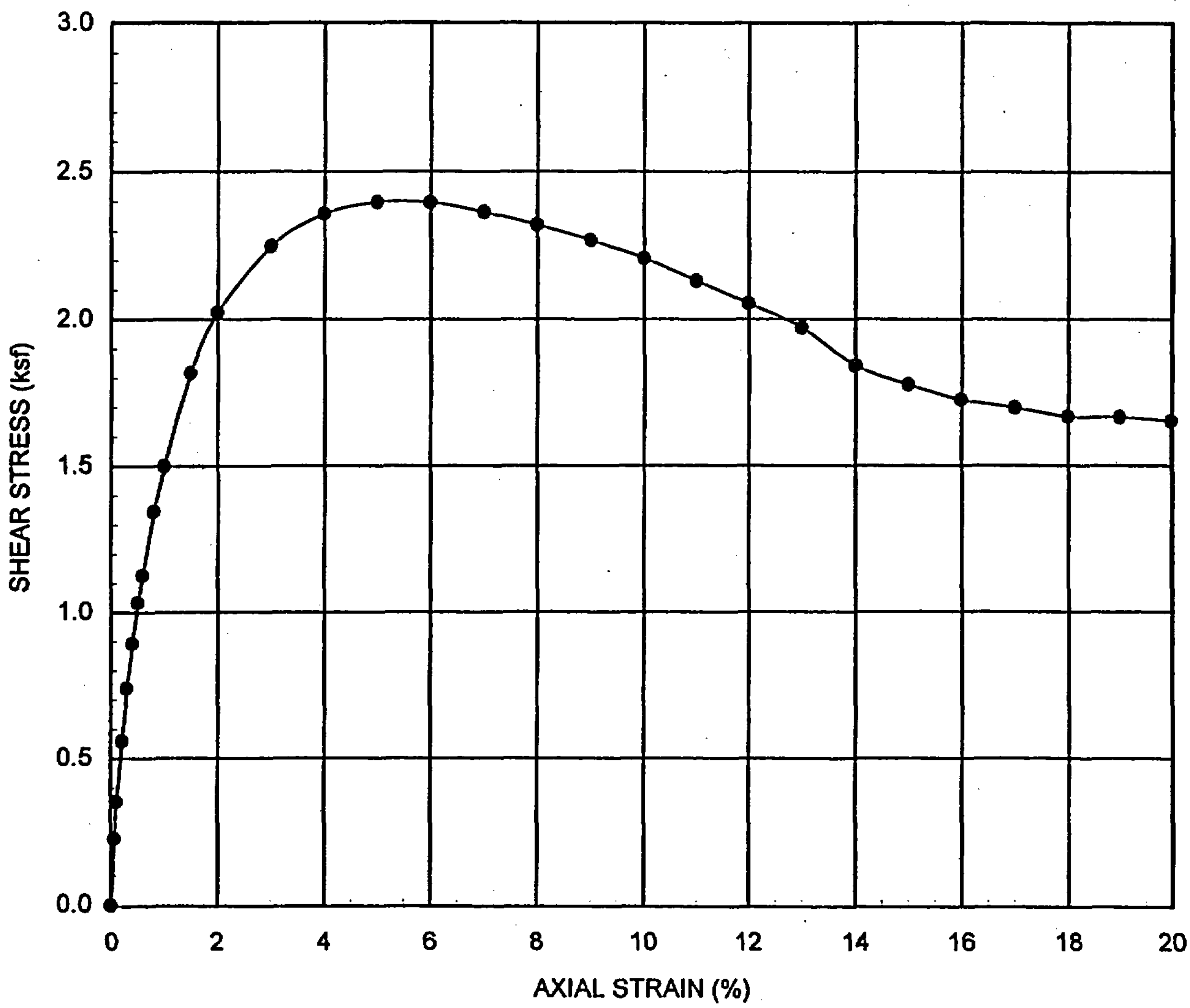
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.53	1.40	0.00	0.000	0.0443	0.00	0.00
0.61	9.00	0.05	0.024	0.0443	0.53	0.27
0.70	15.44	0.10	0.044	0.0444	0.98	0.49
0.87	23.73	0.20	0.069	0.0444	1.56	0.78
1.04	30.25	0.30	0.090	0.0444	2.02	1.01
1.20	36.40	0.40	0.109	0.0445	2.45	1.22
1.37	40.50	0.50	0.122	0.0445	2.73	1.37
1.54	44.32	0.60	0.133	0.0446	2.99	1.50
1.88	50.50	0.80	0.153	0.0447	3.42	1.71
2.22	55.30	1.00	0.168	0.0448	3.74	1.87
3.06	63.58	1.50	0.193	0.0450	4.30	2.15
3.90	69.20	2.00	0.211	0.0452	4.66	2.33
5.59	76.00	3.00	0.232	0.0457	5.08	2.54
7.28	79.86	4.00	0.244	0.0462	5.29	2.64
8.97	82.21	5.00	0.251	0.0466	5.39	2.69
10.65	83.87	6.00	0.256	0.0471	5.44	2.72
12.34	84.96	7.00	0.260	0.0476	5.45	2.73
14.03	85.60	8.00	0.262	0.0482	5.44	2.72
15.71	85.86	9.00	0.263	0.0487	5.39	2.70
17.40	85.88	10.00	0.263	0.0492	5.34	2.67
19.09	85.05	11.00	0.260	0.0498	5.22	2.61
20.77	83.46	12.00	0.255	0.0504	5.07	2.53
22.46	80.78	13.00	0.247	0.0509	4.85	2.42
24.15	75.80	14.00	0.231	0.0515	4.49	2.25
25.84	71.45	15.00	0.218	0.0521	4.18	2.09
27.52	71.94	16.00	0.219	0.0528	4.16	2.08
29.21	73.36	17.00	0.224	0.0534	4.19	2.10
30.90	74.80	18.00	0.228	0.0540	4.22	2.11
32.58	76.29	19.00	0.233	0.0547	4.26	2.13
34.27	76.14	20.00	0.232	0.0554	4.20	2.10

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	04/30/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	8.1 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.554 ft	WATER CONTENT:	54.6 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT:	58.2 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	1.92

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.40 ksf
COMPRESSIVE STRENGTH:	4.79 ksf
FAILURE STRAIN:	5.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	04/30/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	8.1 ft	CHECKED:	JWM
DESCRIPTION:	CLAY		
HEIGHT:	0.554 ft	WATER CONTENT:	54.6 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT.	58.2 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	1.92

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.40 ksf
COMPRESSIVE STRENGTH:	4.79 ksf
FAILURE STRAIN:	5.0 %

DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.87	1.30	0.00	0.000	0.0442	0.00	0.00
0.95	7.74	0.05	0.020	0.0442	0.45	0.23
1.04	11.33	0.10	0.031	0.0442	0.71	0.35
1.21	17.22	0.20	0.050	0.0443	1.12	0.56
1.38	22.38	0.30	0.066	0.0443	1.48	0.74
1.55	26.78	0.40	0.079	0.0444	1.79	0.89
1.71	30.80	0.50	0.092	0.0444	2.07	1.03
1.88	33.51	0.60	0.100	0.0445	2.25	1.13
2.22	39.82	0.80	0.120	0.0445	2.69	1.34
2.56	44.35	1.00	0.134	0.0446	3.00	1.50
3.40	53.71	1.50	0.163	0.0449	3.63	1.82
4.25	59.97	2.00	0.182	0.0451	4.05	2.02
5.93	67.17	3.00	0.205	0.0456	4.50	2.25
7.62	71.10	4.00	0.217	0.0460	4.72	2.36
9.31	73.00	5.00	0.223	0.0465	4.79	2.40
11.00	73.73	6.00	0.225	0.0470	4.79	2.40
12.69	73.55	7.00	0.225	0.0475	4.73	2.36
14.37	72.97	8.00	0.223	0.0480	4.64	2.32
16.06	72.15	9.00	0.220	0.0486	4.54	2.27
17.75	70.97	10.00	0.217	0.0491	4.41	2.21
19.44	69.35	11.00	0.212	0.0497	4.26	2.13
21.13	67.64	12.00	0.206	0.0502	4.11	2.05
22.81	65.68	13.00	0.200	0.0508	3.94	1.97
24.50	62.18	14.00	0.189	0.0514	3.68	1.84
26.19	60.70	15.00	0.185	0.0520	3.55	1.78
27.88	59.66	16.00	0.181	0.0526	3.45	1.72
29.57	59.44	17.00	0.181	0.0532	3.40	1.70
31.25	59.08	18.00	0.180	0.0539	3.33	1.67
32.94	59.71	19.00	0.182	0.0546	3.33	1.66
34.63	60.00	20.00	0.183	0.0552	3.30	1.65

APPENDIX C

Cyclic Triaxial Test Results and Data

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

SAMPLE INFORMATION:

BORING:	CTB-4	DATE:	05/13/99
SAMPLE:	U-13D	TESTED BY:	ACS
DEPTH:	24.9 ft	CHECKED:	TVC

	Before Consolidation	After Consolidation
HEIGHT:	0.544 ft	0.541 ft
DIAMETER:	0.238 ft	0.237 ft
AREA:	0.0447 ft^2	0.0442 ft^2
WATER CONTENT:	37.4 %	37.4 %
DRY UNIT WEIGHT:	73.8 pcf	75.0 pcf
VOID RATIO:	1.30	1.26

CYCLIC LOAD DATA:

CONSOLIDATION PRESSURE:	2.0 ksf
STRESS RATIO:	0.48
CYCLIC STRESS:	1.9 ksf
CYCLIC LOAD:	86 lb

TEST RESULTS:

NUMBER OF CYCLES:	513
DOUBLE AMPLITUDE DISPL:	0.08 inches
DOUBLE AMPLITUDE STRAIN:	1.23 %

NOTE: During the eleventh cycle, the cyclic controller applied a tensile load greater than the confing load. This lifted the top cap off the specimen. This caused a increase in height of the specimen of approximately 0.04 inches.

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

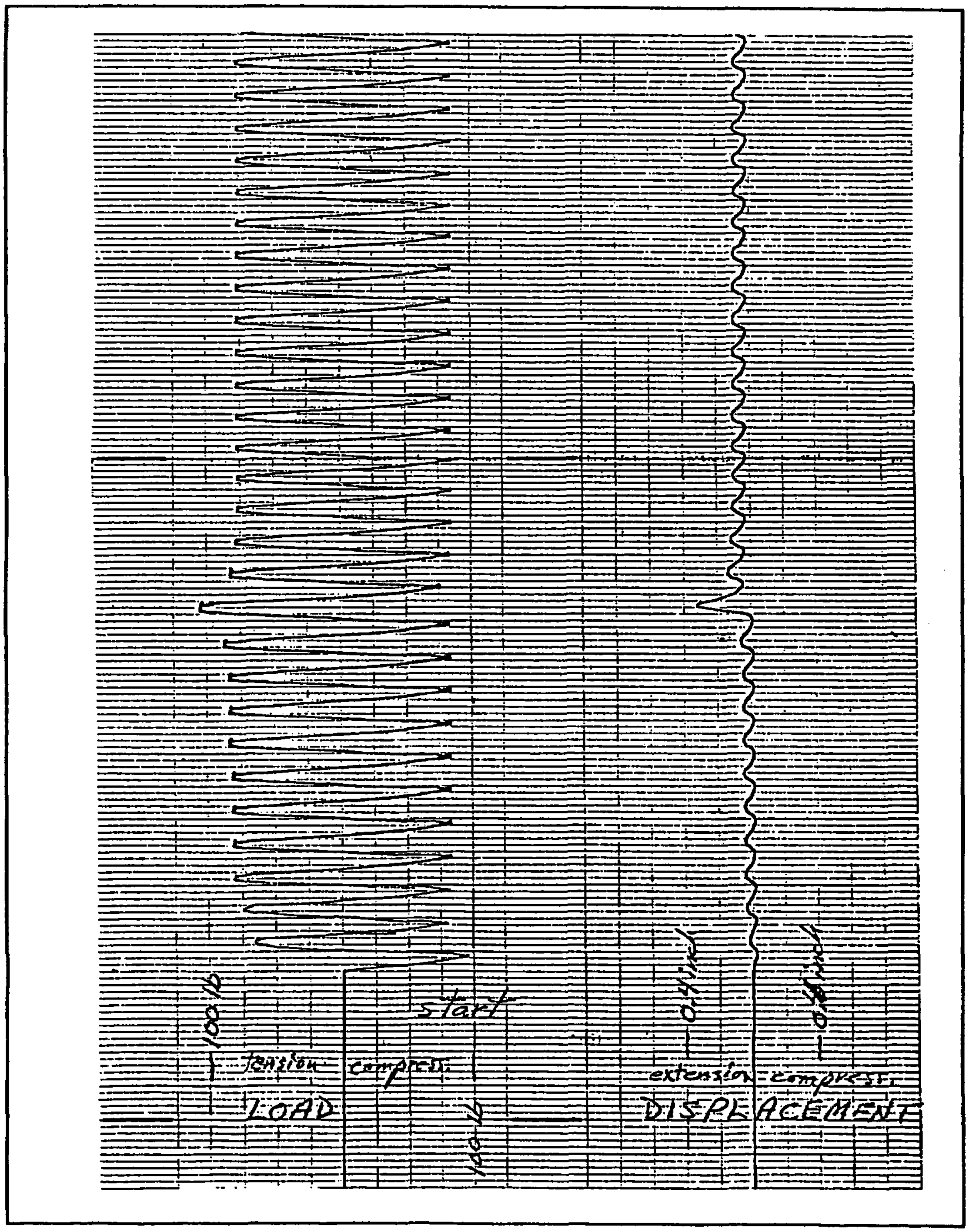
JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING: CTB-4
SAMPLE: U-13D
DEPTH: 24.9 ft

DATE: 05/13/99
TESTED BY: ACS

STRIP CHART OUTPUT AT START OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

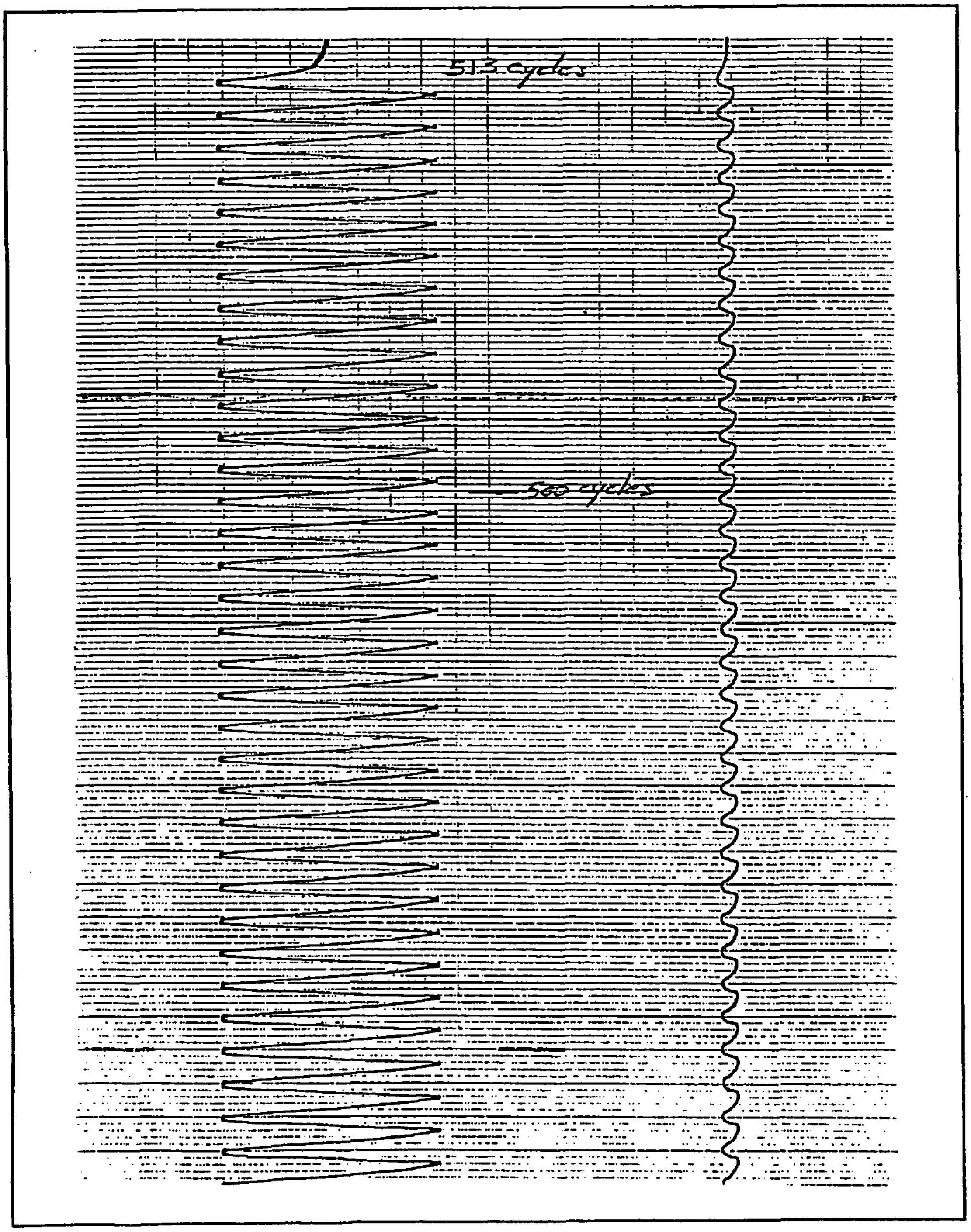
JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING: CTB-4
SAMPLE: U-13D
DEPTH: 24.9 ft

DATE: 05/13/99
TESTED BY: ACS

STRIP CHART OUTPUT AT END OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

SAMPLE INFORMATION:

BORING:	CTB-5	DATE:	05/12/99
SAMPLE:	U-12B	TESTED BY:	ACS
DEPTH:	23.0 ft	CHECKED:	TYC

	Before Consolidation	After Consolidation
HEIGHT:	0.557 ft	0.555 ft
DIAMETER:	0.237 ft	0.236 ft
AREA:	0.0441 ft^2	0.0438 ft^2
WATER CONTENT:	42.3 %	42.3 %
DRY UNIT WEIGHT:	65.8 pcf	66.5 pcf
VOID RATIO:	1.58	1.55

CYCLIC LOAD DATA:

CONSOLIDATION PRESSURE:	2.0 ksf
STRESS RATIO:	0.48
CYCLIC STRESS:	1.9 ksf
CYCLIC LOAD:	85 lb

TEST RESULTS:

NUMBER OF CYCLES:	507
DOUBLE AMPLITUDE DISPL:	0.04 inches
DOUBLE AMPLITUDE STRAIN:	0.60 %

STONE & WEBSTER ENGINEERING CORPORATION

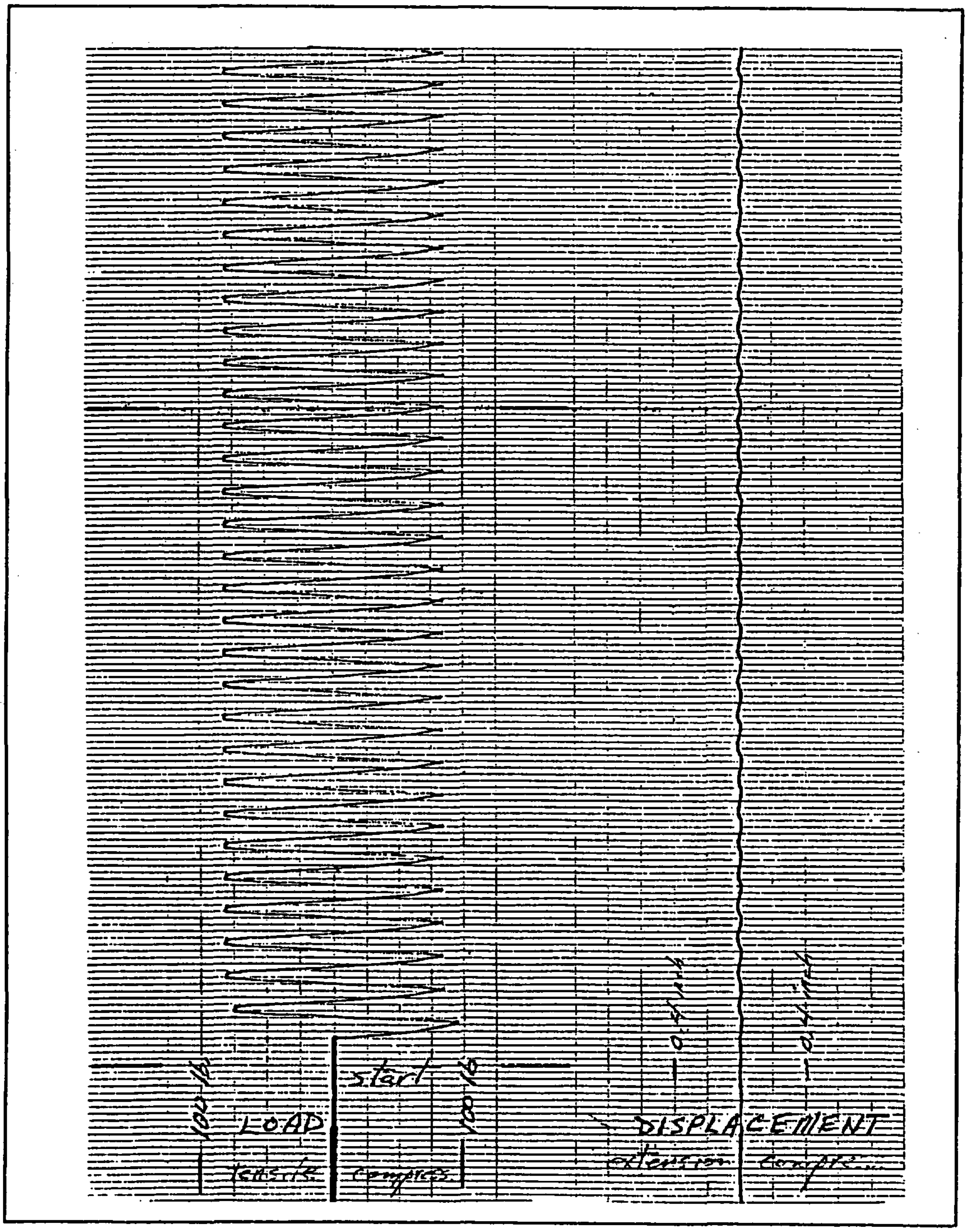
Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-5	DATE:	05/12/99
SAMPLE:	U-12B	TESTED BY:	ACS
DEPTH:	23.0 ft		

STRIP CHART OUTPUT AT START OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

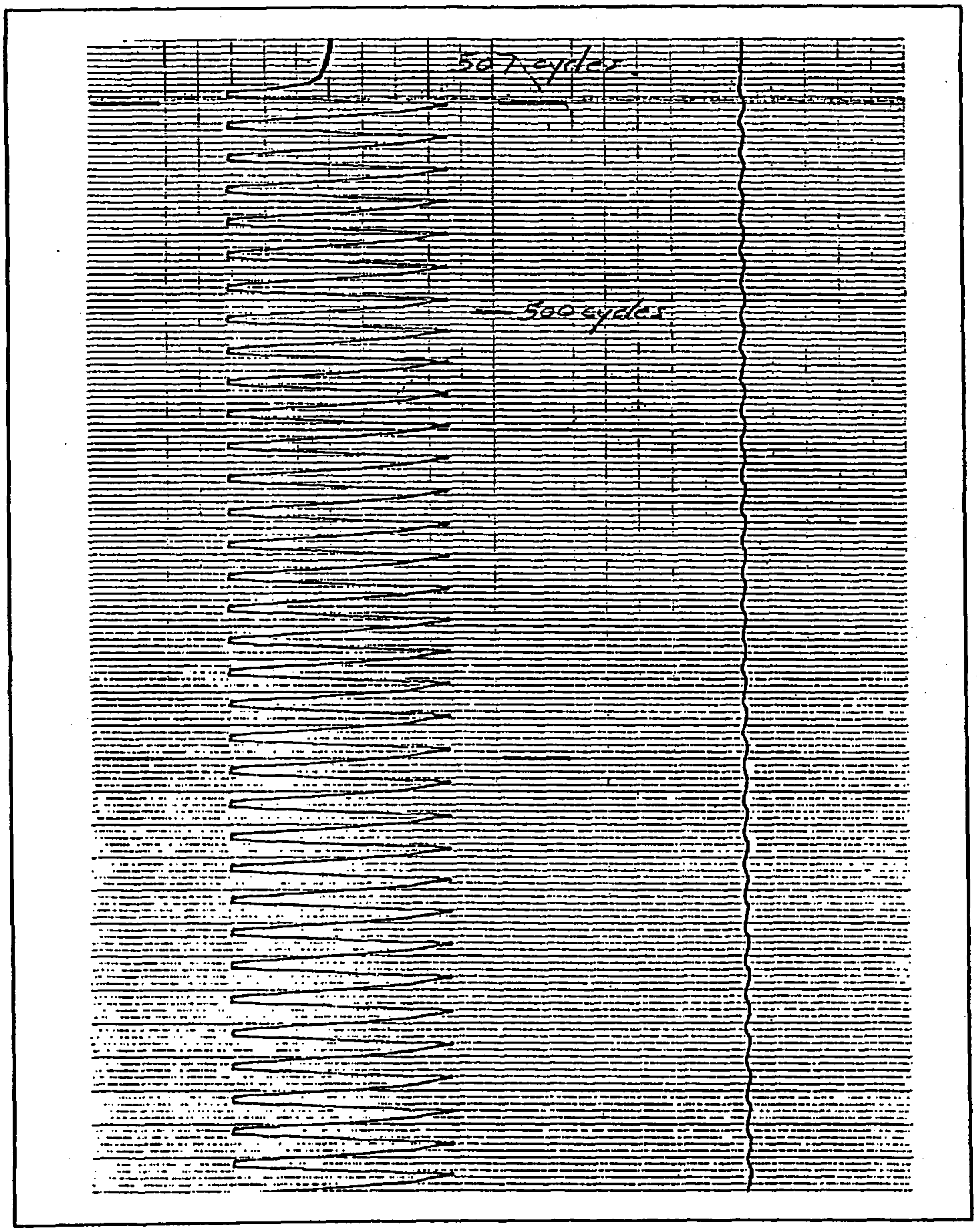
Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05998.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-5	DATE:	05/12/99
SAMPLE:	U-12B	TESTED BY:	ACS
DEPTH:	23.0 ft		

STRIP CHART OUTPUT AT END OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	06/16/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	8.0 ft	CHECKED:	TWC

	Before Consolidation	After Consolidation
HEIGHT:	0.493 ft	0.491 ft
DIAMETER:	0.237 ft	0.236 ft
AREA:	0.0441 ft^2	0.0438 ft^2
WATER CONTENT:	52.6 %	52.6 %
DRY UNIT WEIGHT:	56.5 pcf	57.2 pcf
VOID RATIO:	2.01	1.97

CYCLIC LOAD DATA:

CONSOLIDATION PRESSURE:	2.0 ksf
STRESS RATIO:	0.48
CYCLIC STRESS:	1.9 ksf
CYCLIC LOAD:	84 lb

TEST RESULTS:

NUMBER OF CYCLES:	515
DOUBLE AMPLITUDE DISPL:	0.04 inches
DOUBLE AMPLITUDE STRAIN:	0.68 %

STONE & WEBSTER ENGINEERING CORPORATION

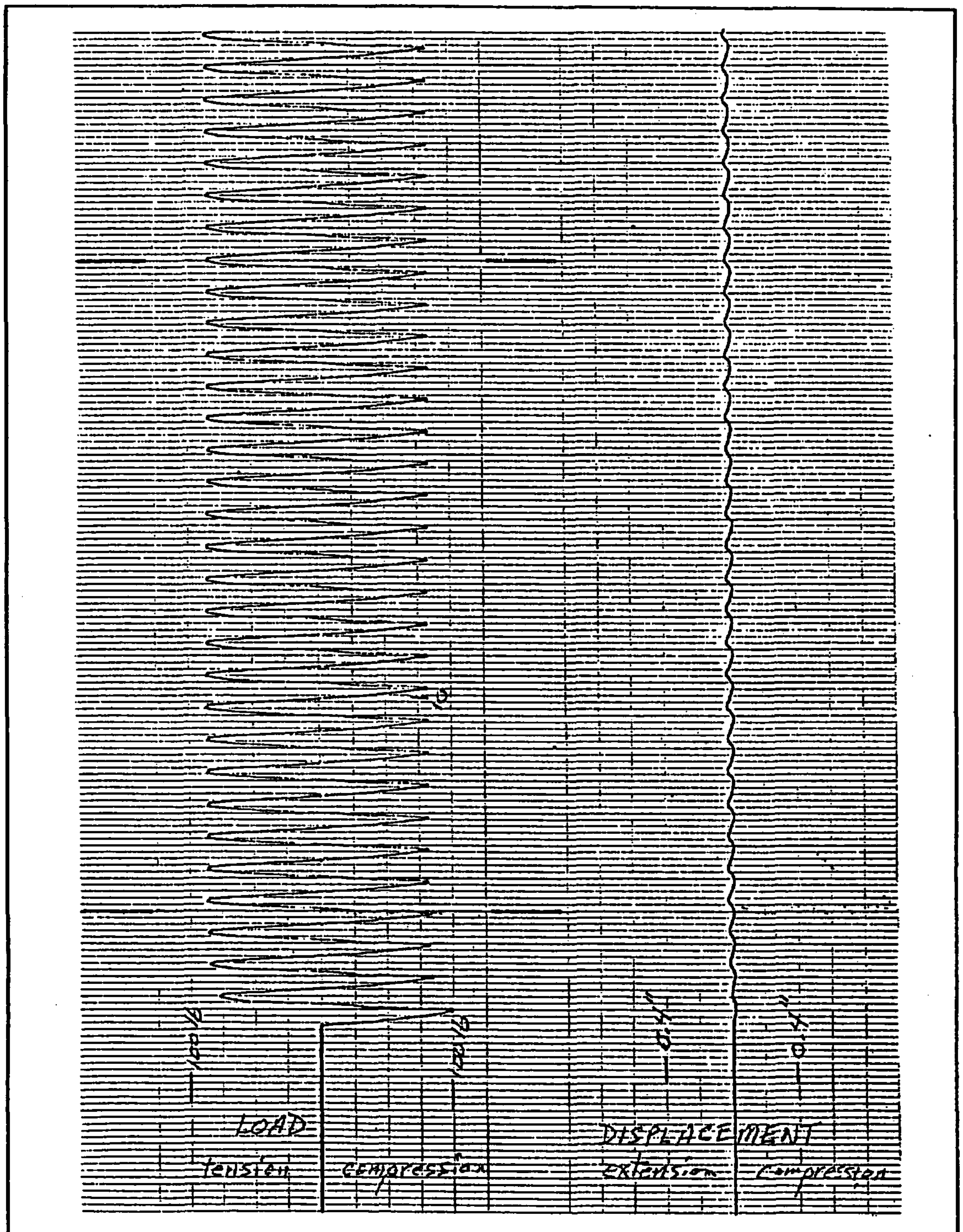
Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-N	DATE:	06/16/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	8.0 ft		

STRIP CHART OUTPUT AT START OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

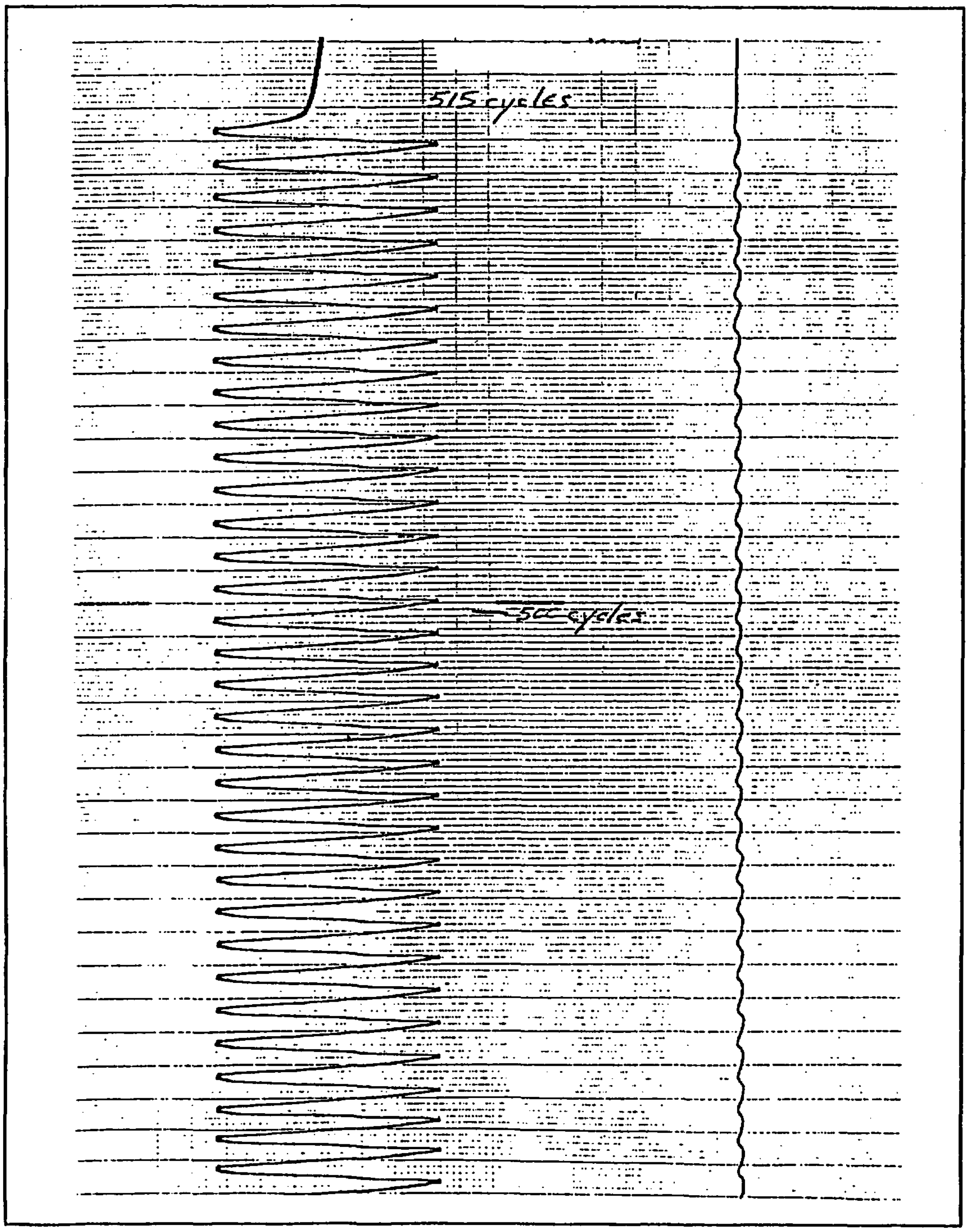
Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-N	DATE:	06/16/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	8.0 ft		

STRIP CHART OUTPUT AT END OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	05/11/99
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	9.6 ft	CHECKED:	TGC

	Before Consolidation	After Consolidation
HEIGHT:	0.558 ft	0.556 ft
DIAMETER:	0.238 ft	0.237 ft
AREA:	0.0443 ft^2	0.0440 ft^2
WATER CONTENT:	47.1 %	47.1 %
DRY UNIT WEIGHT:	58.5 pcf	59.1 pcf
VOID RATIO:	1.90	1.87

CYCLIC LOAD DATA:

CONSOLIDATION PRESSURE:	2.0 ksf
STRESS RATIO:	0.48
CYCLIC STRESS:	1.9 ksf
CYCLIC LOAD:	85 lb

TEST RESULTS:

NUMBER OF CYCLES:	514
DOUBLE AMPLITUDE DISPL:	0.02 inches
DOUBLE AMPLITUDE STRAIN:	0.30 %

STONE & WEBSTER ENGINEERING CORPORATION

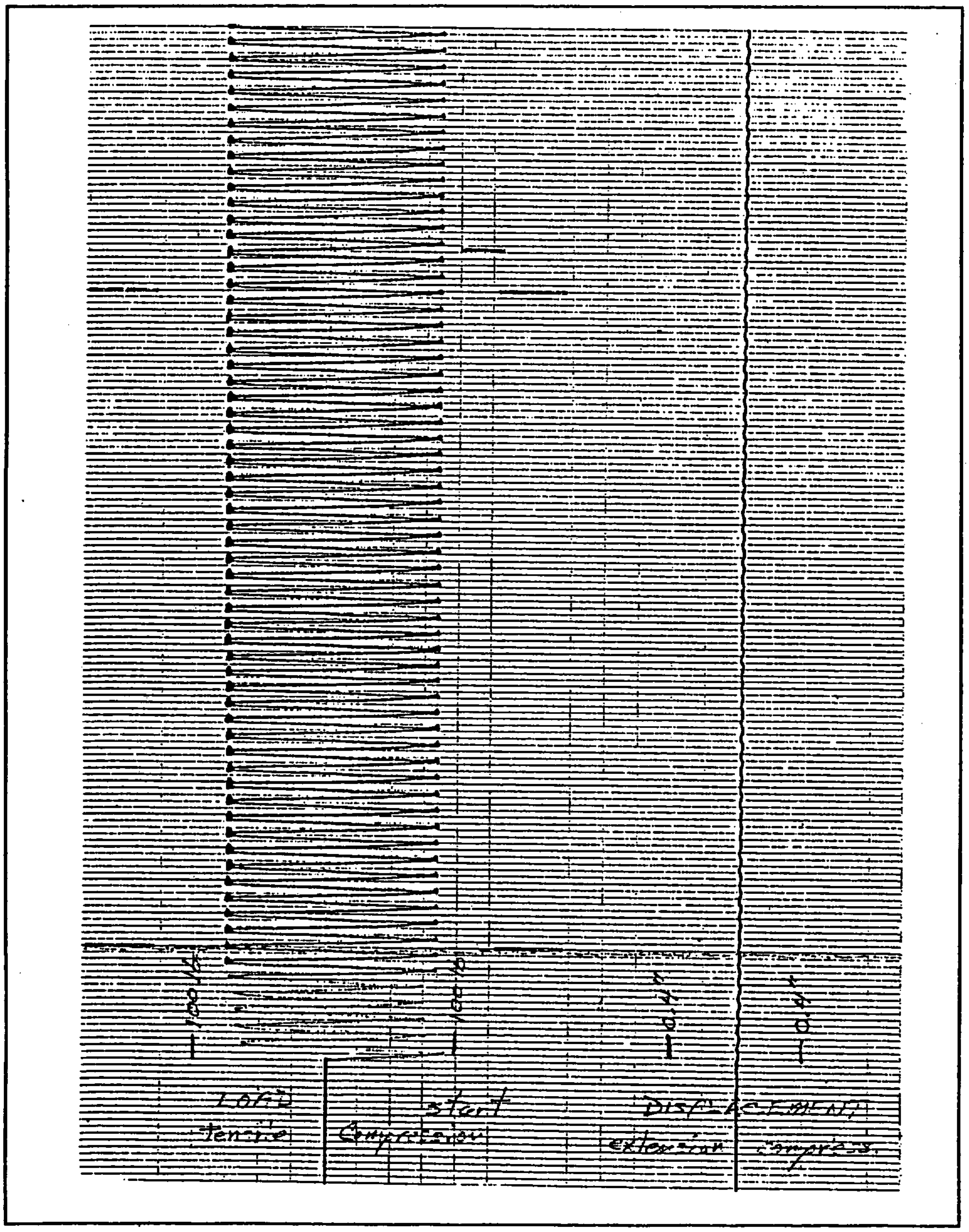
Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-N	DATE:	05/11/99
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	9.6 ft		

STRIP CHART OUTPUT AT START OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

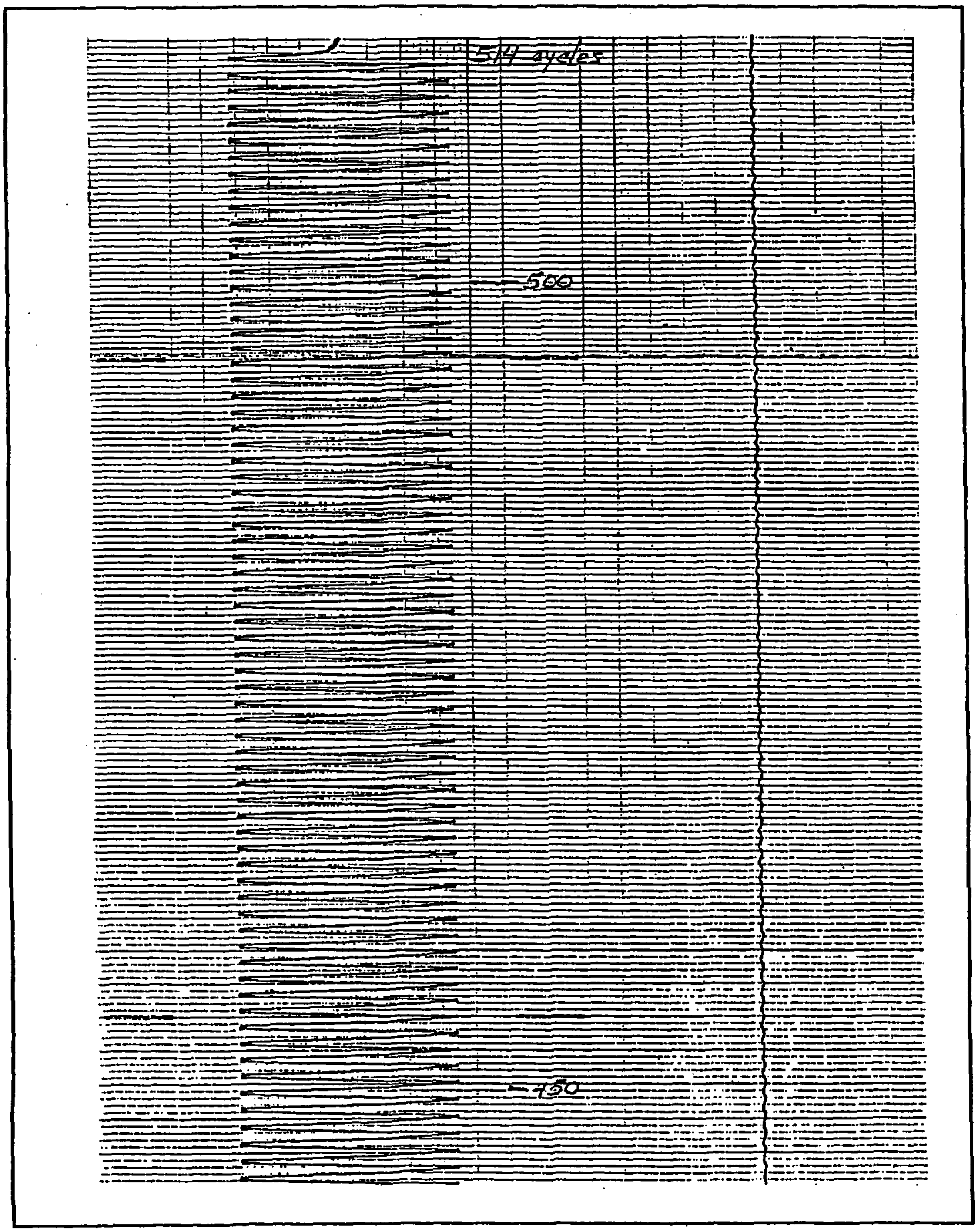
JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING: CTB-N
SAMPLE: U-3C
DEPTH: 9.6 ft

DATE: 05/11/99
TESTED BY: ACS

STRIP CHART OUTPUT AT END OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	05/04/99
SAMPLE:	U-1D	TESTED BY:	ACS
DEPTH:	6.3 ft	CHECKED:	TUC

	Before Consolidation	After Consolidation
HEIGHT:	0.556 ft	0.553 ft
DIAMETER:	0.236 ft	0.235 ft
AREA:	0.0438 ft^2	0.0432 ft^2
WATER CONTENT:	60.7 %	60.7 %
DRY UNIT WEIGHT:	52.8 pcf	53.7 pcf
VOID RATIO:	2.22	2.16

CYCLIC LOAD DATA:

CONSOLIDATION PRESSURE:	2.0 ksf
STRESS RATIO:	0.48
CYCLIC STRESS:	1.9 ksf
CYCLIC LOAD:	83 lb

TEST RESULTS:

NUMBER OF CYCLES:	518
DOUBLE AMPLITUDE DISPL:	0.06 inches
DOUBLE AMPLITUDE STRAIN:	0.90 %

STONE & WEBSTER ENGINEERING CORPORATION

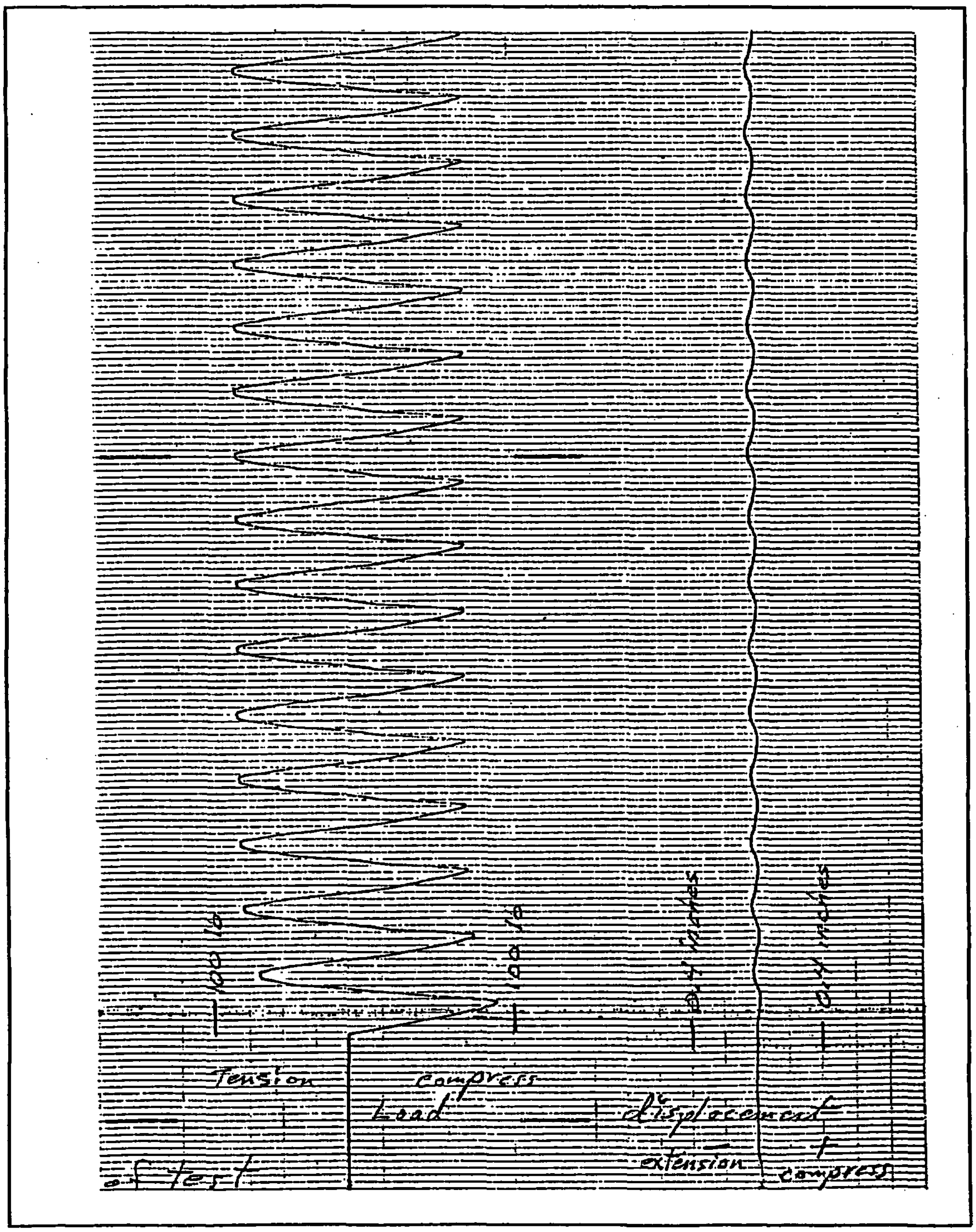
Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING:	CTB-S	DATE:	05/04/99
SAMPLE:	U-1D	TESTED BY:	ACS
DEPTH:	6.3 ft		

STRIP CHART OUTPUT AT START OF TEST



STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

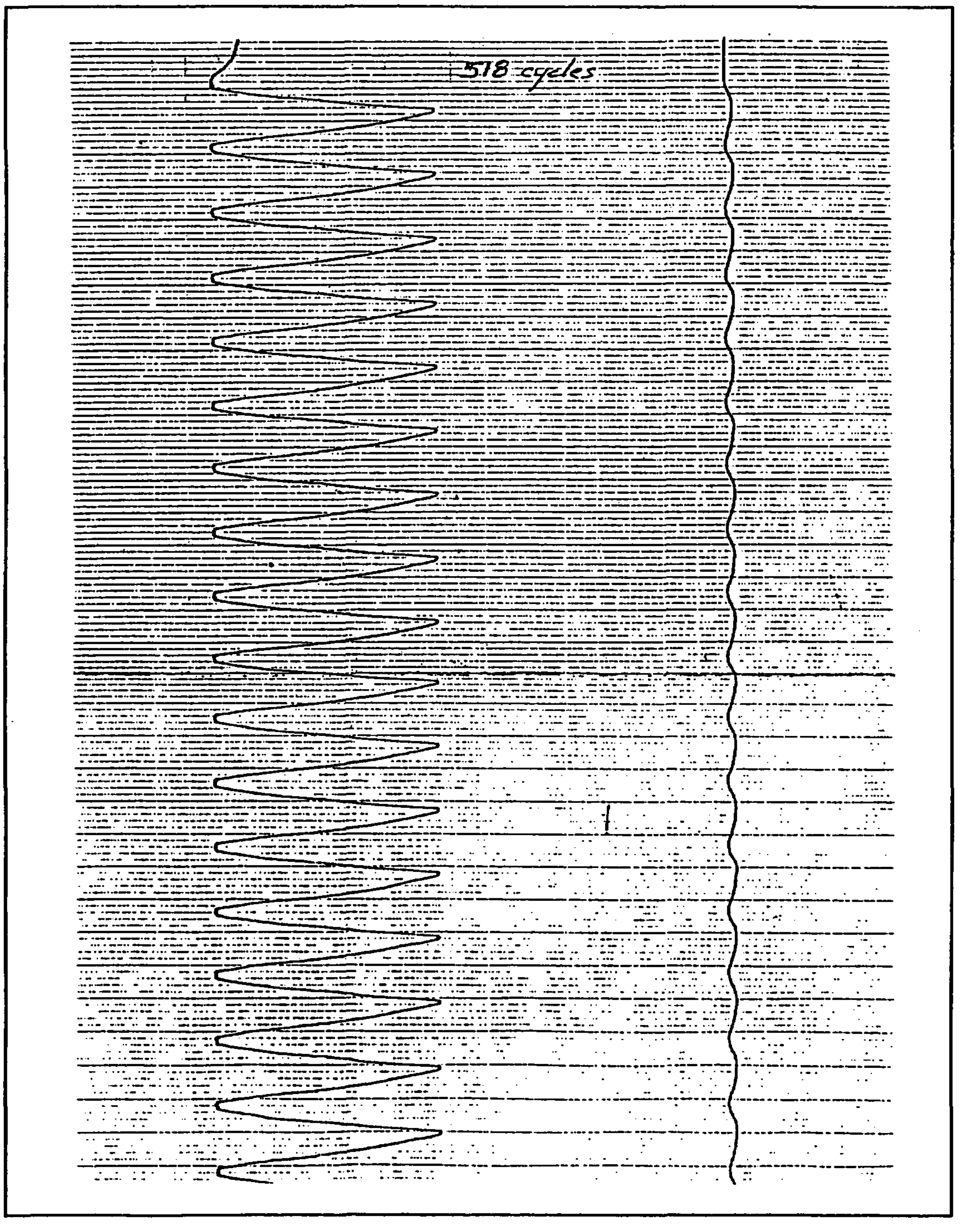
JO: 05996.02
June 1999

CYCLIC TRIAXIAL TEST

BORING: CTB-S
SAMPLE: U-1D
DEPTH: 6.3 ft

DATE: 05/04/99
TESTED BY: ACS

STRIP CHART OUTPUT AT END OF TEST



APPENDIX D

Resonant - Column Test Plots and Data

Stone & Webster Engineering Corp.

Resonant Column Test - Specimen Properties

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02

Boring: CTB-1
Sample: U-3C
Depth: 7.9 ft

Date: 5/19/99
Tested By: ACS
Checked By: TUC

confining pressure:	5.0 psi	17.0 psi	29.0 psi
total weight:	867.34 gm	867.34 gm	867.34 gm
dry weight:	575.92 gm	575.92 gm	575.92 gm
height:	15.14 cm	15.07 cm	14.99 cm
diameter:	7.259 cm	7.238 cm	7.217 cm
volume:	626.6 cc	619.9 cc	613.2 cc
dry density:	57.4 pcf	58.0 pcf	58.6 pcf
void ratio:	1.96	1.93	1.90

J_o: 30.38 gm-cm-sec^2

specific gravity: 2.72 (est.)

tors. acc.: 2500 pk-mv/pk-g

strain ampl. factor:	0.070711	0.070857	0.071005
J:	5.825518	5.79186	5.758301
J/J_o:	0.19176	0.19065	0.18954
ψ_s:	0.4244	0.42324	0.42208
$\psi_s TAN(\psi_s)$:	0.19177	0.19065	0.18954
C_2:	70.9	71.4	71.8

Effective Confining Pressure (psi)	Coil Voltage (mv_{rms})	Acceler. Voltage (mv_{rms})	Damping Calibration Factor
5.0	370	109	0.069297
17.0	145	42.6	0.069132
29.0	150	44.0	0.069046
		average	0.069158

Stone & Webster Engineering Corp.

Resonant Column Test - Vibration Data

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02

Boring: CTB-1
Sample: U-3C
Depth: 7.9 ft

Date: 5/19/99
Tested By: ACS
Checked By: TLC

Effective Confining Pressure (psi)	Coil Voltage (mv_{rms})	Acceler. Voltage (mv_{rms})	Resonant Frequency (Hz)	Notes
5.0	6.9	54.7	64.1	
	12.7	96.9	63.9	
	21.8	154	63.5	
	35.4	223	62.9	
	73.0	380	61.9	
	135	595	60.4	
	194	735	59.2	
	352	1036	56.9	
17.0	6.50	62.2	76.8	
	13.2	124	76.7	
	19.9	180	76.6	
	40.0	335	76.2	
	73.1	550	75.5	
	136	870	74.5	
	248	1260	72.9	
	483	1870	70.2	
	911	2650	66.6	
	1770	3730	61.4	
29.0	6.6	53.3	80.2	
	11.2	90.8	80.2	
	20.3	161.3	80.1	
	37.6	280	79.9	
	74.7	515	79.4	
	143	867	78.5	
	277	1380	77.1	
	540	2094	74.7	
	976	2920	71.8	
	1960	4240	67.4	

Stone & Webster Engineering Corp.

Resonant Column Test
Modulus and Damping Ratio Computations

Private Fuel Storage, LLC — JO: 05996.02
PFSF, Skull Valley, UT

Boring: CTB-1 — Date: 5/19/99
Sample: U-3C — Tested By: ACS
Depth: 7.9 ft — Checked By: TEC

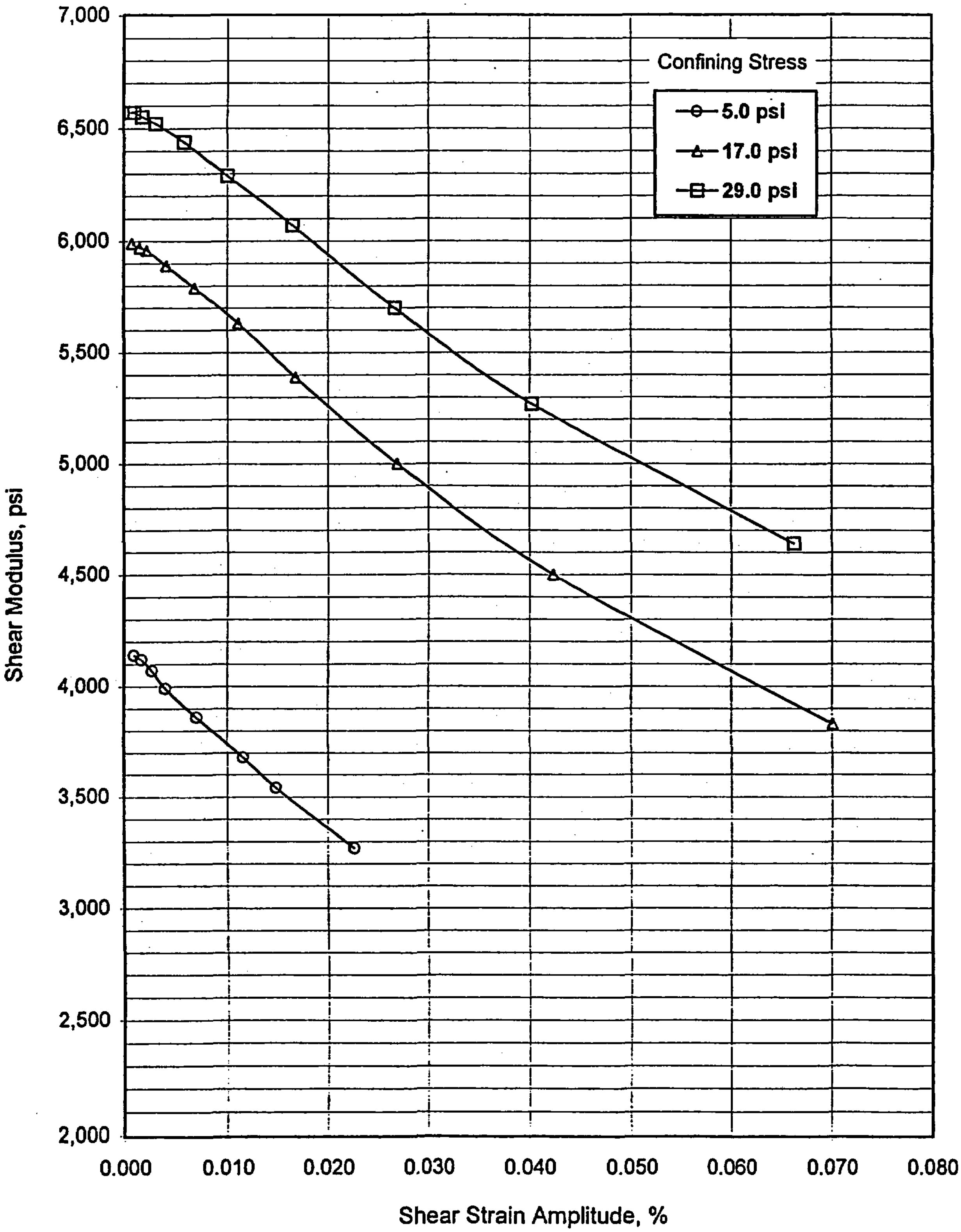
Effective Confining Pressure (psi)	G_o (psi)	Shear Strain Amplitude (%)	Shear Modulus (psi)	G/G_o	Damping Ratio (%)
5.0	4,160	0.00094	4,140	0.995	0.87
		0.00168	4,120	0.990	0.91
		0.00270	4,070	0.978	0.98
		0.00399	3,990	0.959	1.10
		0.00701	3,860	0.928	1.33
		0.01153	3,680	0.885	1.57
		0.01483	3,540	0.851	1.83
		0.02263	3,270	0.786	2.35
17.0	6,000	0.00075	5,990	0.998	0.72
		0.00149	5,970	0.995	0.74
		0.00217	5,960	0.993	0.76
		0.00409	5,890	0.982	0.83
		0.00684	5,790	0.965	0.92
		0.01111	5,630	0.938	1.08
		0.01680	5,390	0.898	1.36
		0.02689	5,000	0.833	1.79
		0.04233	4,500	0.750	2.38
		0.07011	3,830	0.638	3.28
29.0	6,600	0.00059	6,570	0.995	0.86
		0.00100	6,570	0.995	0.85
		0.00179	6,550	0.992	0.87
		0.00311	6,520	0.988	0.93
		0.00580	6,440	0.976	1.00
		0.00999	6,290	0.953	1.14
		0.01648	6,070	0.920	1.39
		0.02665	5,700	0.864	1.78
		0.04022	5,270	0.798	2.31
		0.06627	4,640	0.703	3.20

Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

Boring: CTB-1, Sample: U-3C, Depth: 7.9 ft

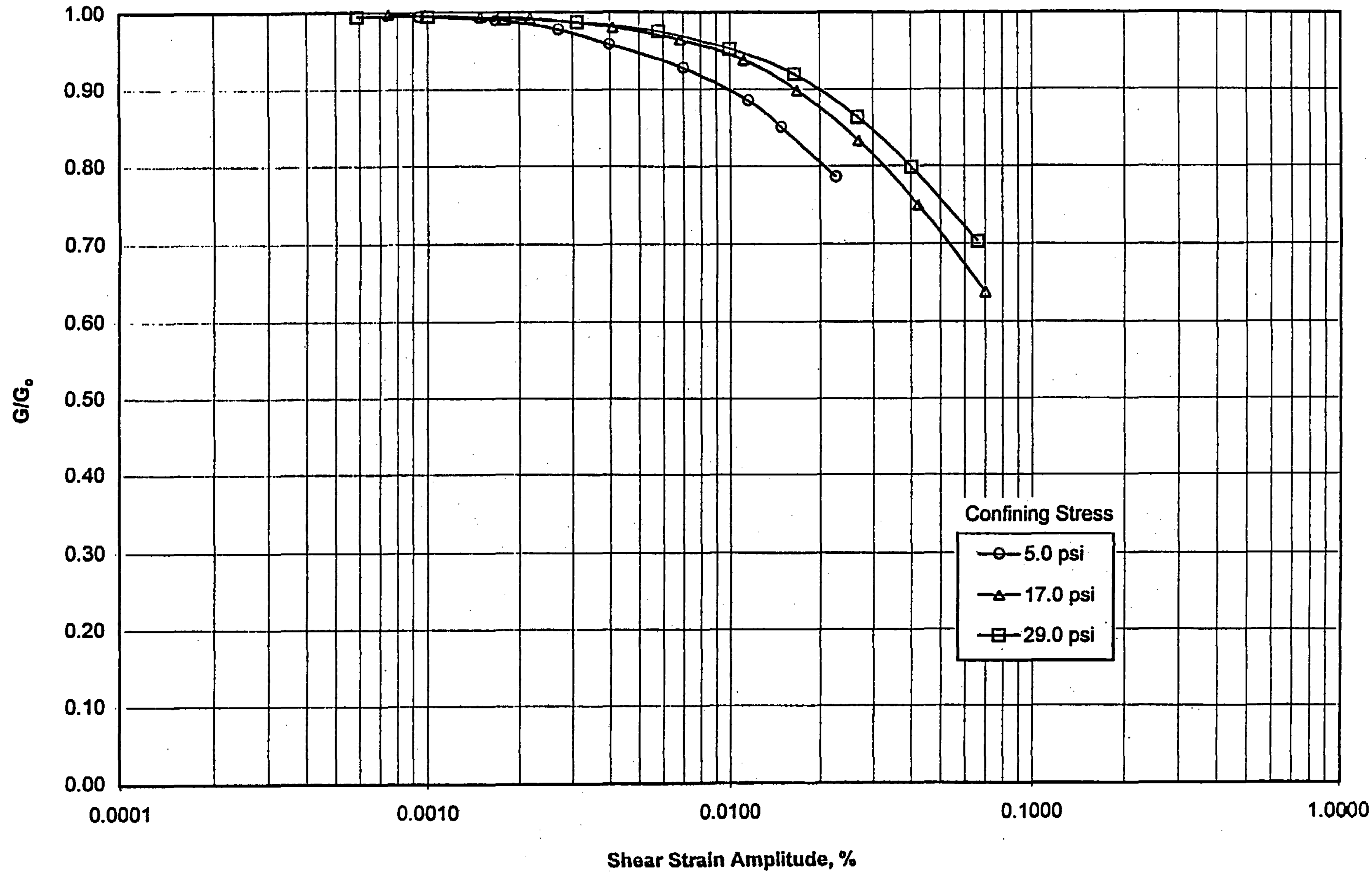


Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

Boring: CTB-1, Sample: U-3C, Depth: 7.9 ft

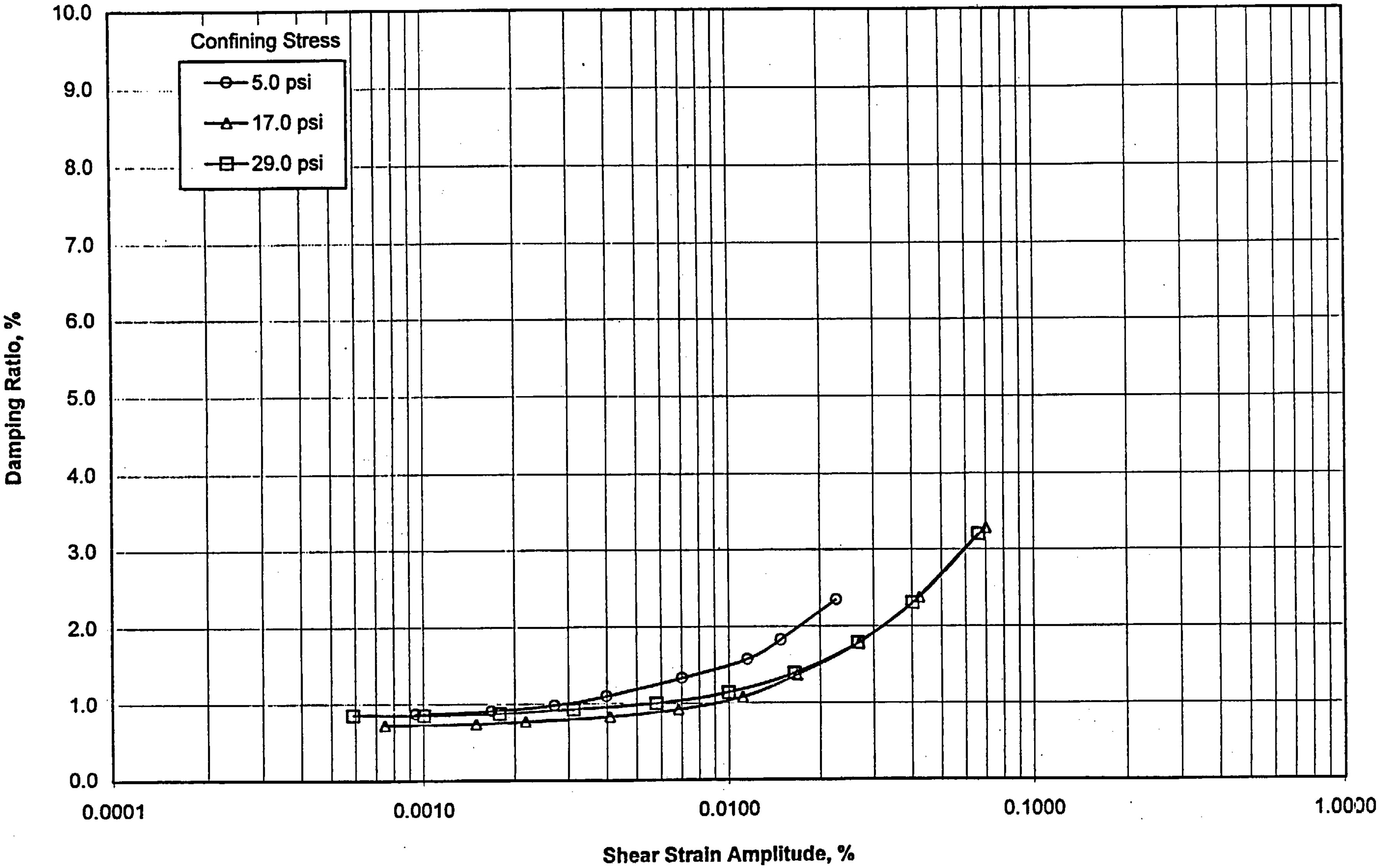


Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

Boring: CTB-1, Sample: U-3C, Depth: 7.9 ft

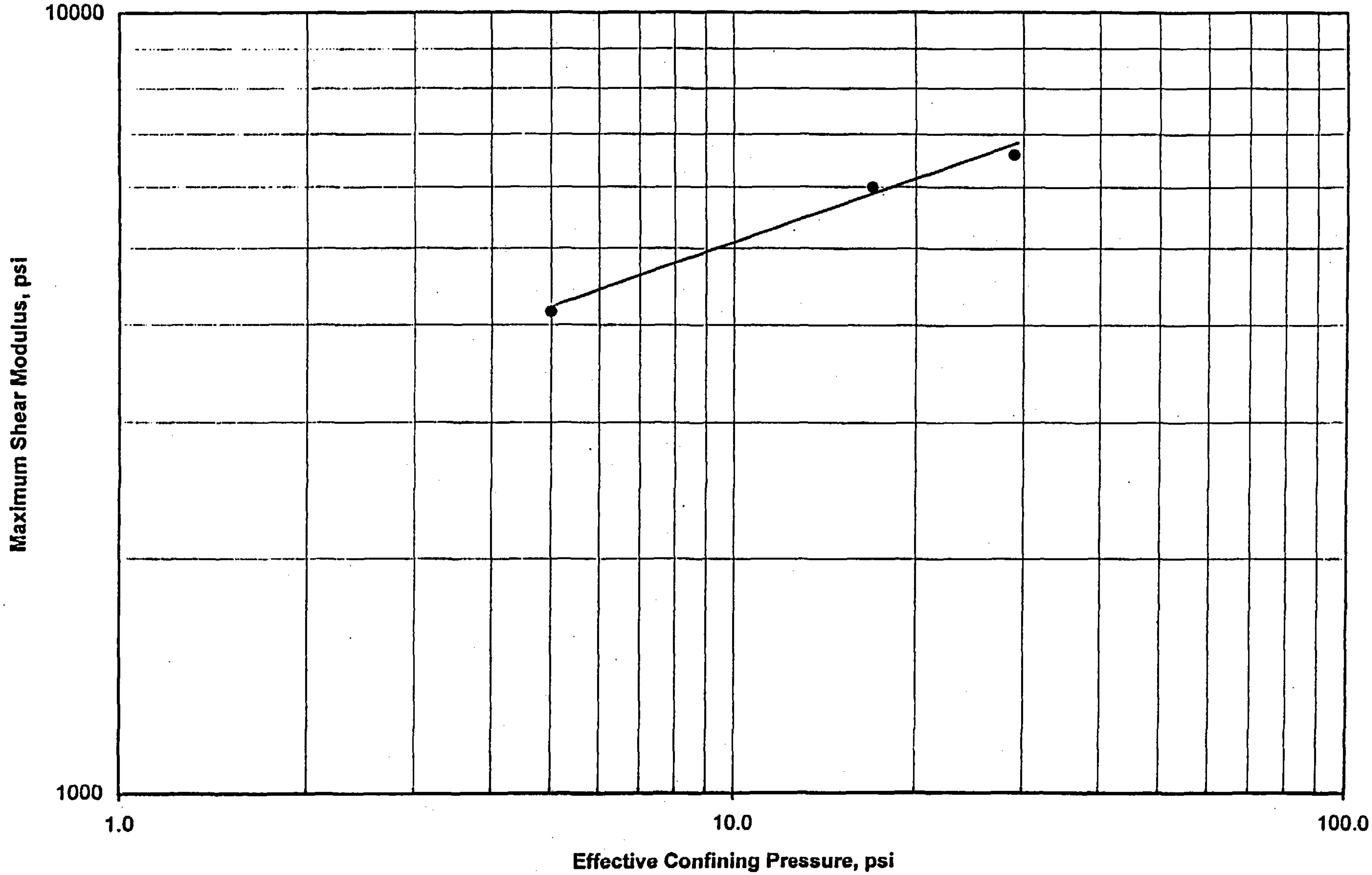


Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO 05996.02
May 1999

Boring: CTB-1, Sample: U-3C, Depth: 7.9 ft



Stone & Webster Engineering Corp.

Resonant Column Test - Specimen Properties

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02

Boring: CTB-1
Sample: U-7C
Depth: 20.8 ft

Date: 5/24/99
Tested By: ACS
Checked By: TLC

confining pressure:	12.0 psi	24.0 psi	36.0 psi
total weight:	826.08 gm	826.08 gm	826.08 gm
dry weight:	543.83 gm	543.83 gm	543.83 gm
height:	14.96 cm	14.91 cm	14.81 cm
diameter:	7.236 cm	7.220 cm	7.214 cm
volume:	615.2 cc	610.4 cc	605.3 cc
dry density:	55.2 pcf	55.6 pcf	56.1 pcf
void ratio:	2.08	2.05	2.03

J_o: 30.38 gm-cm-sec^2 specific gravity: 2.72 (est.)

tors. acc.: 2500 pk-mv/pk-g

strain ampl. factor:	0.071335	0.071416	0.071838
J:	5.513289	5.488935	5.479815
J/J_o:	0.18148	0.18068	0.18038
ψ_s:	0.41354	0.41268	0.41236
$\psi_s TAN(\psi_s)$:	0.18148	0.18068	0.18038
C_2:	70.7	71.1	70.9

Effective Confining Pressure (psi)	Coil Voltage (mv_{rms})	Acceler. Voltage (mv_{rms})	Damping Calibration Factor
12.0	140	42.5	0.0716
24.0	141	41.3	0.0691
36.0	149	43.0	0.0681
		average	0.0696

Stone & Webster Engineering Corp.

Resonant Column Test - Vibration Data

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02

Boring: CTB-1
Sample: U-7C
Depth: 20.8 ft

Date: 5/24/99
Tested By: ACS
Checked By: TLC

Effective Confining Pressure (psi)	Coil Voltage (mv_{rms})	Acceler. Voltage (mv_{rms})	Resonant Frequency (Hz)	Notes
12.0	8.2	81.5	69.1	
	16.7	157.5	68.9	
	33.9	285	68.4	
	67.6	476	67.4	
	133.0	740	66.0	
	269	1105	64.2	
	340	1261	63.2	
24.0	8.4	73.1	73.7	
	17.0	145	73.6	
	34.2	272	73.2	
	68.1	483	72.5	
	134	792	71.4	
	262	1235	70.0	
	518	1828	67.2	
	983	2590	64.2	
36.0	10.1	83.0	79.0	
	20.4	165	78.9	
	40.5	315	78.7	
	71.2	511	78.2	
	141.5	862	77.1	
	279	1375	75.6	
	564	2068	72.7	
	990	2852	70.0	
	1479	3580	67.6	
	1972	4180	65.3	
	2940	5160	62.4	
	4420	6520	58.1	
	4840	6830	56.8	

Stone & Webster Engineering Corp.

Resonant Column Test
Modulus and Damping Ratio Computations

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02

Boring: CTB-1
Sample: U-7C
Depth: 20.8 ft

Date: 5/24/99
Tested By: ACS
Checked By: TYC

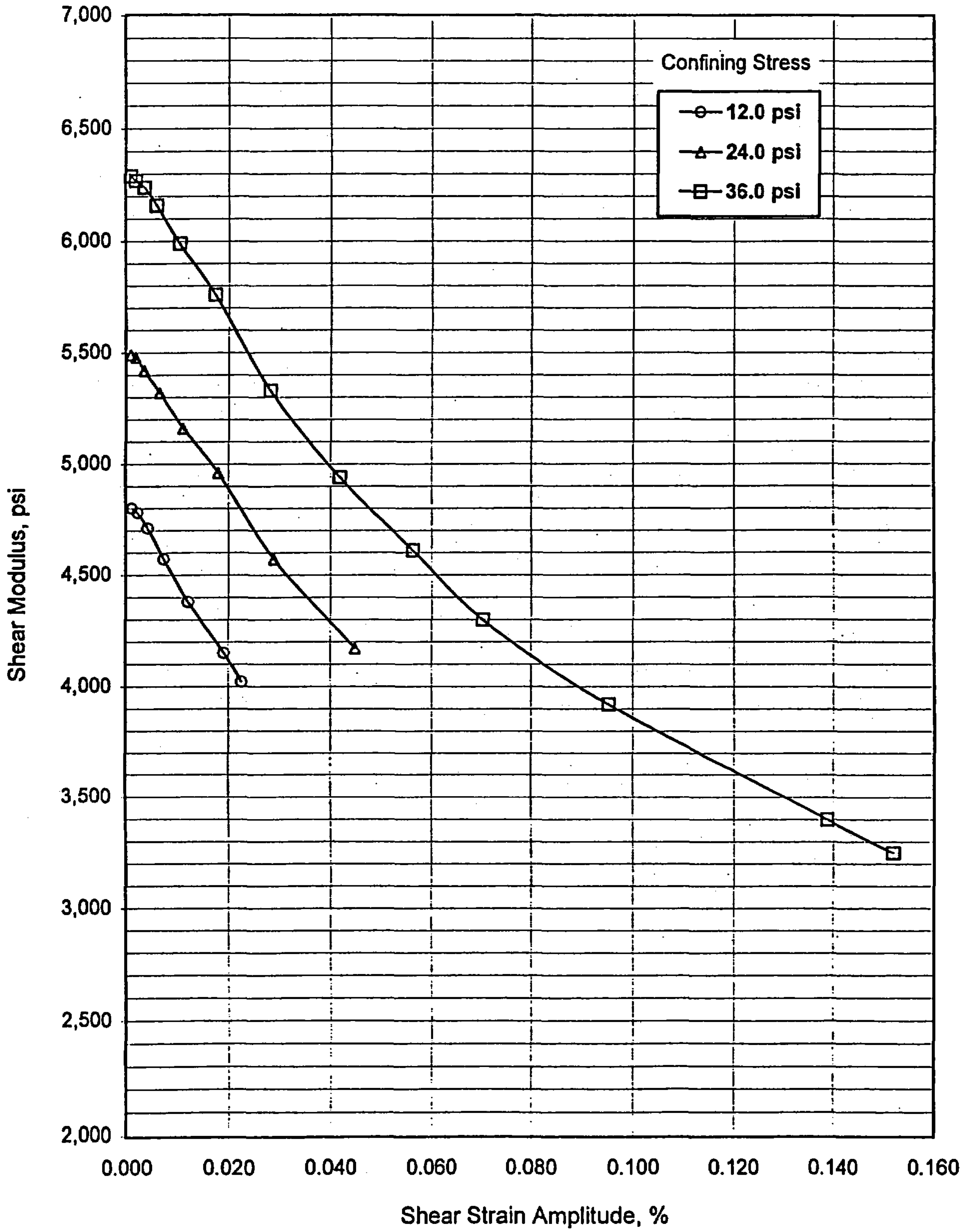
Effective Confining Pressure (psi)	G_o (psi)	Shear Strain Amplitude (%)	Shear Modulus (psi)	G/G_o	Damping Ratio (%)
12.0	4,820	0.00122	4,800	0.996	0.70
		0.00237	4,780	0.992	0.74
		0.00435	4,710	0.977	0.83
		0.00747	4,570	0.948	0.99
		0.01212	4,380	0.909	1.25
		0.01912	4,150	0.861	1.69
		0.02252	4,020	0.834	1.88
24.0	5,510	0.00096	5,490	0.996	0.80
		0.00191	5,480	0.995	0.82
		0.00363	5,420	0.984	0.88
		0.00656	5,320	0.966	0.98
		0.01109	5,160	0.936	1.18
		0.01800	4,960	0.900	1.48
		0.02891	4,570	0.829	1.97
		0.04488	4,170	0.757	2.64
36.0	6,300	0.00096	6,290	0.998	0.85
		0.00190	6,270	0.995	0.86
		0.00365	6,240	0.990	0.90
		0.00600	6,160	0.978	0.97
		0.01042	5,990	0.951	1.14
		0.01728	5,760	0.914	1.41
		0.02811	5,330	0.846	1.90
		0.04181	4,940	0.784	2.42
		0.05628	4,610	0.732	2.88
		0.07042	4,300	0.683	3.28
		0.09520	3,920	0.622	3.97
		0.13876	3,400	0.540	4.72
		0.15208	3,250	0.516	4.93

Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

Boring: CTB-1, Sample: U-7C, Depth: 20.8 ft

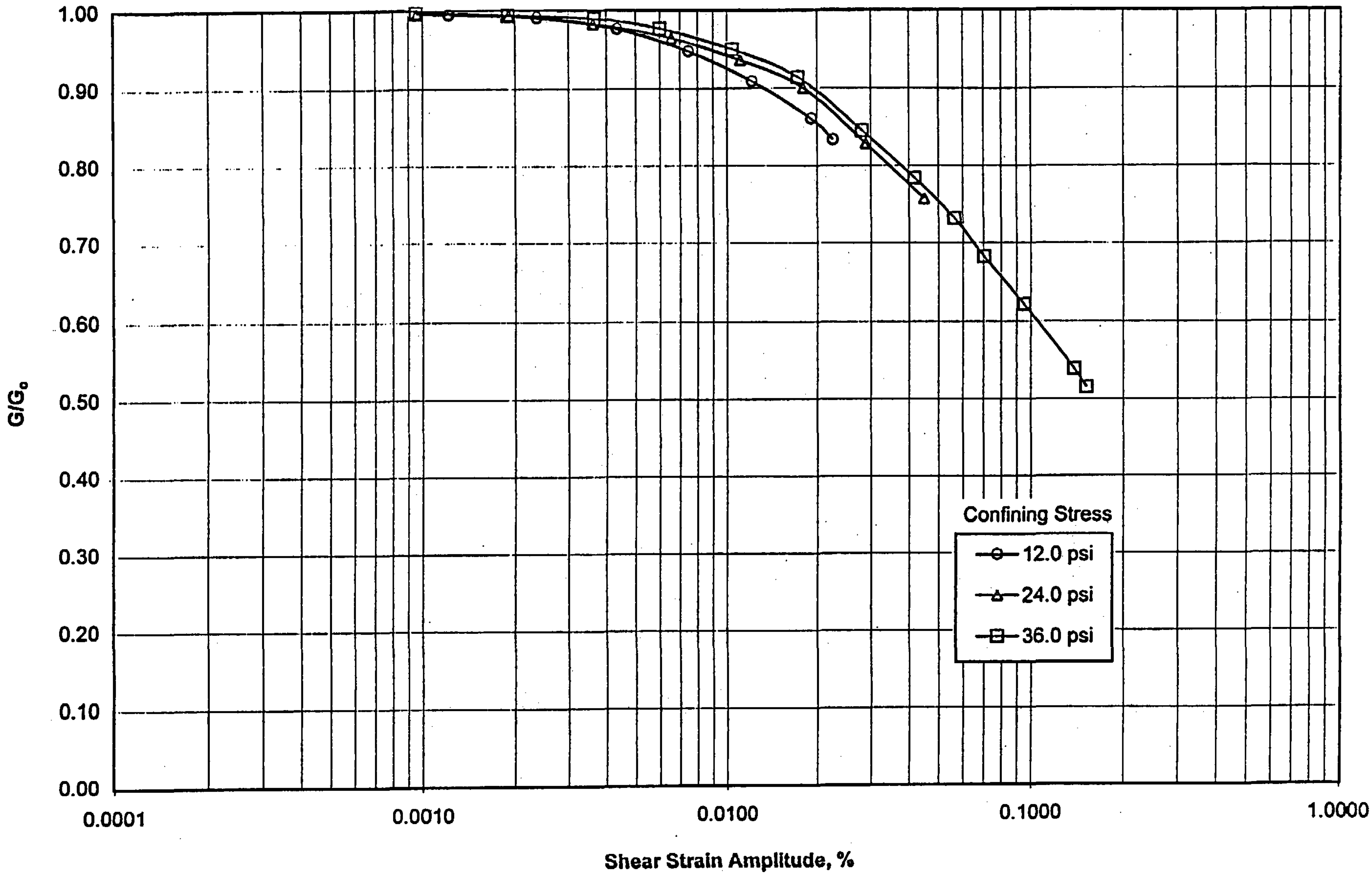


Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

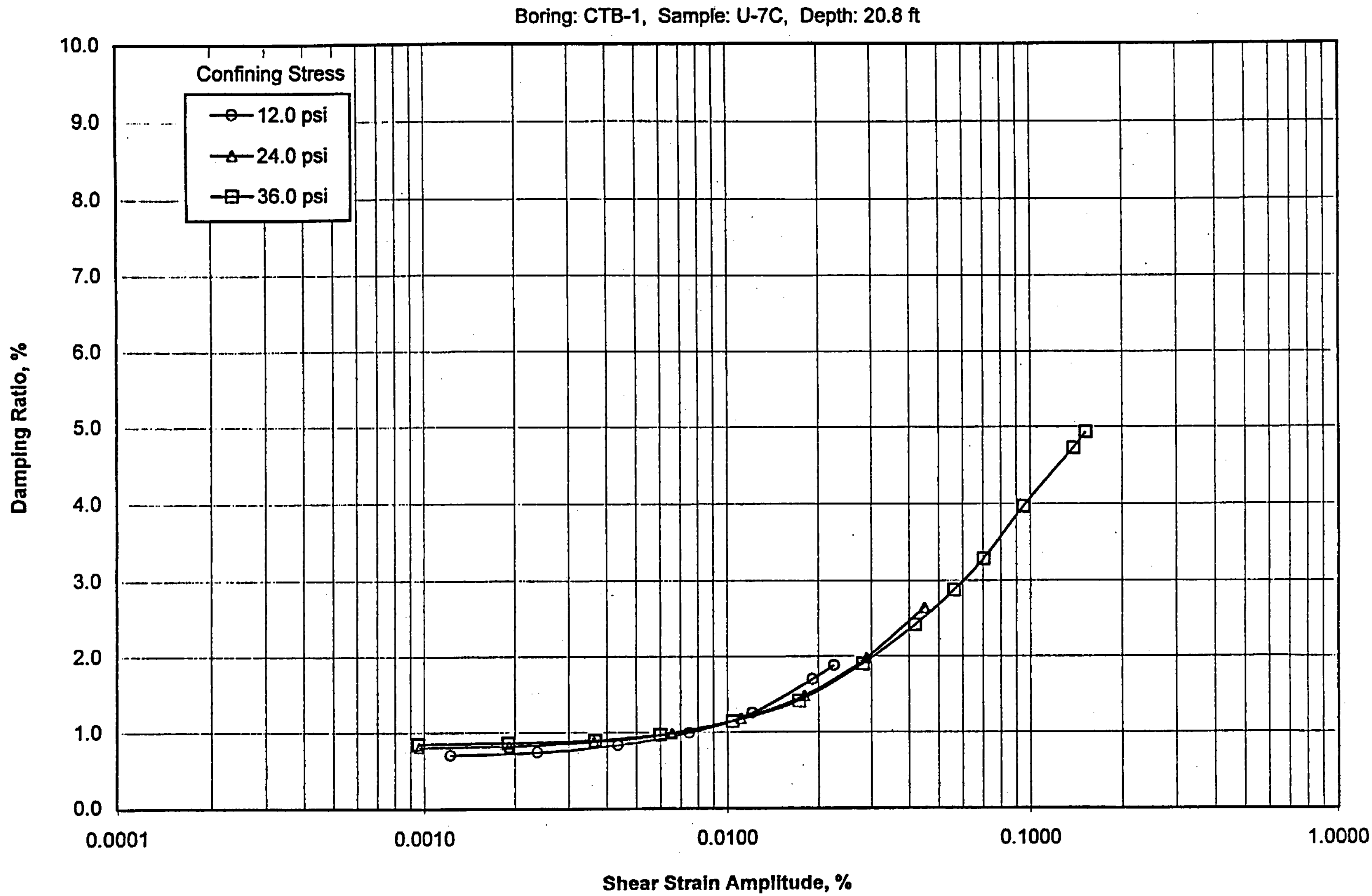
Boring: CTB-1, Sample: U-7C, Depth: 20.8 ft



Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO: 05996.02
May 1999

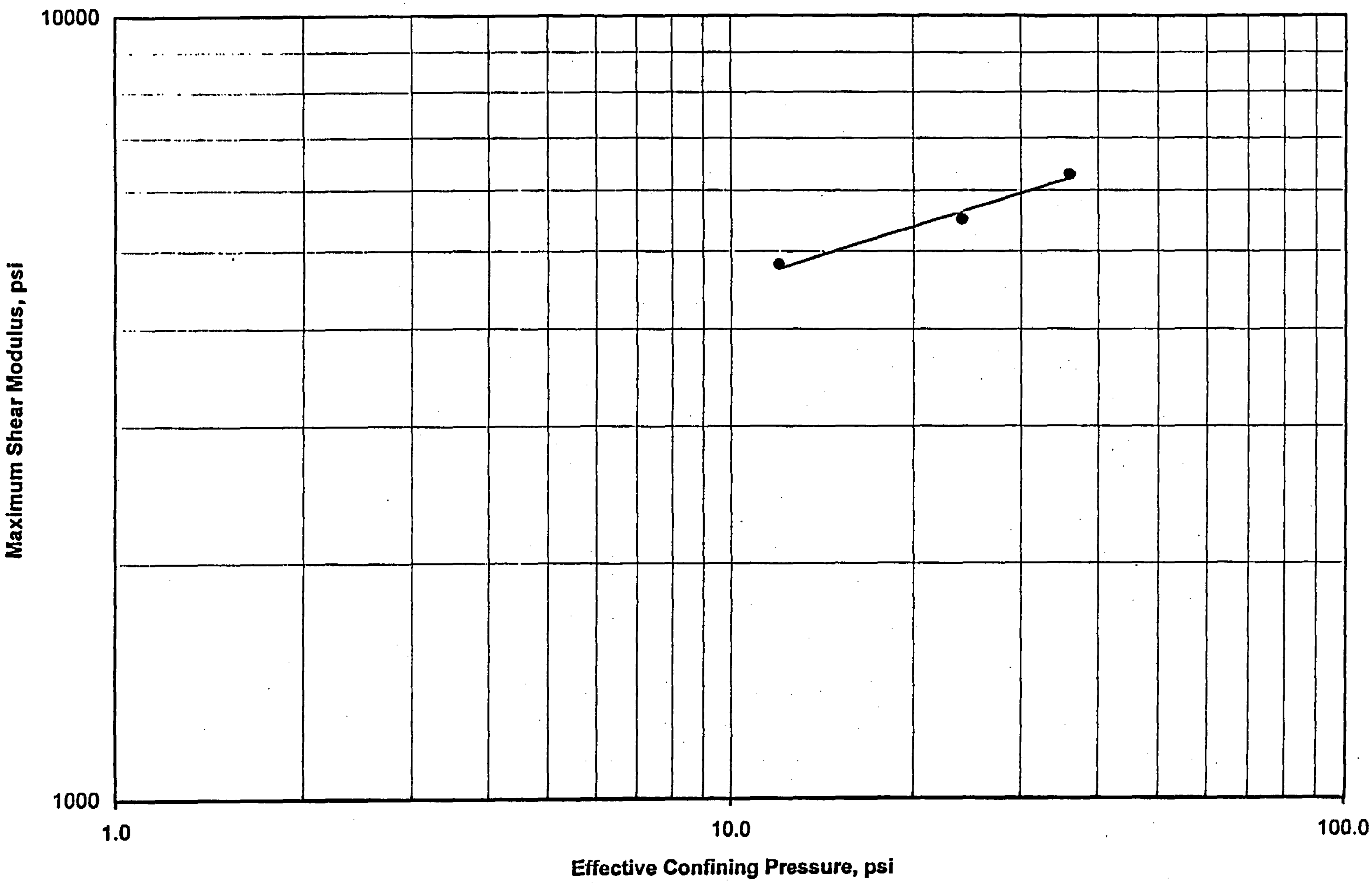


Stone & Webster Engineering Corporation

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO 05996.02
May 1999

Boring: CTB-1, Sample: U-7C, Depth: 20.8 ft



PROJECT DOCUMENT INDEPENDENT REVIEW CHECKLIST
USING ASME N45.2 OR NQA-1 FORMAT

Project No. 05996.02
Job Book File Location Q2.9

Type of Document

- Report: Supplemental Geotechnical Laboratory Testing – June 1999
- Design Criteria ______ Rev. No. ______
- Project Specification ______ Rev. No. ______
- Project Diagram ______ Rev. No. ______
- License Document ______ Rev. No. ______
- List Drawing/Diagrams and Revision Number

This sheet may be used for more than 1 diagram (list or reference all diagrams below)

	Yes	No	N/A
Were the inputs correctly selected and incorporated into the design?			✓
Are assumptions necessary to perform the design activity adequately described and reasonable?			✓
Where necessary, are the assumptions identified for subsequent reverifications when the detailed design activities are completed?			✓
Are the appropriate quality and quality assurance requirements specified?	✓		
Are the applicable codes, standards, and regulatory requirements, including applicable issues and addenda, properly identified and are their requirements for design met?	✓		
Have applicable construction and operating experience been considered?			✓
Have the design interface requirements been satisfied?			✓
Was an appropriate design method used?			✓
Is the output reasonable compared to inputs?	✓		

JO 05996.02
REPORT: SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING - JUNE 1999

	Yes	No	N/A
Are the specified parts, equipment, and processes suitable for the required applications?	✓		
Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?			✓
Have adequate maintenance features and requirements been specified?			✓
Are accessibility and other design provisions adequate for performance of needed maintenance and repair?			✓
Has adequate accessibility been provided to perform the inservice inspection expected to be required during the plant life?			✓
Has the design properly considered radiation exposure to the public and plant personnel?			✓
Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?			✓
Have adequate preoperational and subsequent periodic test requirements been appropriately specified?			✓
Are adequate handling, storage, cleaning, and shipping requirements specified?			✓
Are adequate identification requirements specified?	✓		
Are requirements for record preparation, review, approval, retention, etc., adequately specified?			✓
Is the design output reasonable compared to the design inputs?			✓
Are the necessary design input and verification requirements for interfacing organizations specified in the design documents or in supporting procedures as instructions?			✓

[This checklist meets the requirements for both N45.2 and NQA-1 for design verification requirements]

Thomas Y. Chang	Thomas Y. Chang	8-9-99
Printed Name	Signature	Date

APPENDIX 2A

ATTACHMENT 3

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
MAY 1998

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
MAY 1998

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
May 1998

Private Fuel Storage Facility
Skull Valley
Private Fuel Storage, LLC

Prepared by: Alan C Smith 5/12/98
Date

Reviewed by: Thomas Y. Chang 5/13/98
Date

Independent Review by: Thomas Y. Chang 5/13/98
Date

Approved by: Paul J. Trudeau 5/13/98
Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

DISCUSSION OF TESTING PROGRAM AND RESULTS

Additional geotechnical laboratory testing was performed in April and May of 1998 in preparation of the response to Question 2-8 of NRC Request for Additional Information Number 1. The focus of this question was the void ratio and the relative density of the nonplastic silts in the upper, ~30-ft thick layer of silt, silty clay, and clayey silt. The purpose of this addendum is to document these test results and incorporate them into Attachment 2 of Appendix 2A of the SAR.

A review of the borings indicated that nonplastic silts were observed in the split-spoon samples obtained above and below Sample U2 in Boring A-2. Therefore, this Shelby tube was opened to see if it contained nonplastic silts that could be tested to determine the void ratio. The dimensions and weight of the tube sections were carefully measured and the water contents of the samples were determined, permitting determination of the void ratio of these samples. Prior to extruding Sample U2C, we used the vibratory table, normally used for determining the maximum density of soils (ASTM D-4253), to vibrate the sample in an attempt to dynamically compact the sample in the tube section. However, no dynamically induced settlement occurred.

Upon extruding the samples, we found that this tube contained highly plastic clayey silt, as indicated by the Atterberg limits test results shown on the table below. Torvane tests performed on these samples demonstrated that the undrained shear strength ranged from 0.65 to 1.8 tons/ft^2, with an average value of 1.25 tons/ft^2, and the void ratio averaged 2.1. The locations and the results of these tests are presented on the attached undisturbed tube log. These results are consistent with the test results reported in the SAR for the clayey silt.

Additional Atterberg limits tests were performed on split-spoon samples obtained in Borings A-2, B-3, C-4, and D-4. These results, shown in the table below, confirmed that Samples S3 in Borings A-2 and C-4, and Sample S3A in Boring D-4 were essentially nonplastic. However, these Atterberg limits indicate that Samples S1 in Borings A-2 and B-3 and Sample S2 in Boring D-4, which were described as nonplastic in the logs included in Appendix 2A of the original SAR submittal, are actually slightly or moderately plastic.

Examination of these soils under a microscope indicates the presence of numerous tiny shells (Ostracodes). Considerable void space was present under some of these shells, and it is believed that these voids are contributing to the high, in situ void ratio measured for the clayey silt. A considerable amount of calcium carbonate is present in these soils, as evidenced by a vigorous reaction upon application of hydrochloric acid to these soils. Therefore, these soils are believed to be cemented, the result of carbonate cement bonding of the silt and clay-size particles, imparting cohesion to these soils.

The tests were performed in accordance with the following American Society for Testing and Materials standards.

D-2216 1992 Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures

D-4318 1995A Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

TABLE 1

Atterberg Limits Testing Performed in April-May 1998

Boring	Sample	Depth Feet	Water Content %	LL %	PL %	PI %	Plastic
A-2	S1	1.0	15.6	28.9	23.3	5.6	Slightly
A-2	U2C	5.9	52.8	70.2	42.9	27.3	Highly
A-2	U2E	7.0	45.4	61.8	36.7	25.1	Highly
A-2	S3	11.0	18.4	27.0	24.5	2.5	NP
B-3	S1	1.0	8.9	26.6	19.7	6.9	Slightly
C-4	S3	11.0	18.2	26.5	26.0	0.5	NP
D-4	S2	6.0	38.0	49.3	27.7	21.6	Moderately
D-4	S3A	10.2	16.8	24.7	23.3	1.4	NP

THE ATTACHED CHECKLIST FOR
QA 1 INDEPENDANT REVIEW
PROVIDES DOCUMENTATION
THAT THE SUPPLEMENTAL TEST
AND TEST REPORT WAS REVIEWED
PER PROCEDURES. IF THE
SUPPLEMENTAL INFO IS ADDED
TO THE GEOTECH. REPORT, THE
CHECKLIST SHALL BE FILED IN
JOB BK Q 2.9.

Private Fuel Storage Facility 5/13

PROJECT DOCUMENT INDEPENDENT REVIEW CHECKLIST USING ASME N45.2 OR NQA-1 FORMAT

Project No. 0599602
Job Book File Location Q2.9

Type of Document SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING

- Design Criteria ______ Rev. No. ______
- Project Specification ______ Rev. No. ______
- Project Diagram ______ Rev. No. ______

- List Diagrams and Revision Number

This sheet may be used for more than 1 diagram (list or reference all diagrams below)

	Yes	No	N/A
Were the inputs correctly selected and incorporated into the design?			✓
Are assumptions necessary to perform the design activity adequately described and reasonable?			✓
Where necessary, are the assumptions identified for subsequent reverifications when the detailed design activities are completed?			✓
Are the appropriate quality and quality assurance requirements specified?	✓		
Are the applicable codes, standards, and regulatory requirements, including applicable issues and addenda, properly identified and are their requirements for design met?	✓		
Have applicable construction and operating experience been considered?			✓
Have the design interface requirements been satisfied?			✓
Was an appropriate design method used?			✓
Is the output reasonable compared to inputs?	✓		

	Yes	No	N/A
Are the specified parts, equipment, and processes suitable for the required applications?	✓		
Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?			✓
Have adequate maintenance features and requirements been specified?			✓
Are accessibility and other design provisions adequate for performance of needed maintenance and repair?			✓
Has adequate accessibility been provided to perform the inservice inspection expected to be required during the plant life?			✓
Has the design properly considered radiation exposure to the public and plant personnel?			✓
Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?			✓
Have adequate preoperational and subsequent periodic test requirements been appropriately specified?			✓
Are adequate handling, storage, cleaning, and shipping requirements specified?			✓
Are adequate identification requirements specified?	✓		
Are requirements for record preparation, review, approval, retention, etc., adequately specified?			✓
Is the design output reasonable compared to the design inputs?			✓
Are the necessary design input and verification requirements for interfacing organizations specified in the design documents or in supporting procedures as instructions?			✓

[This checklist meets the requirements for both N45.2 and NQA-1 for design verification requirements]

Thomas Y. Chang
Printed Name

Thomas Y. Chang
Signature

5/13/98
Date

APPENDIX 2A

ATTACHMENT 4

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
NOVEMBER 1998

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
NOVEMBER 1998

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
NOVEMBER 1998

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

Prepared by: Paul J. Trudeau 8/20/99
Date

Reviewed by: Thomas Y. Chang 8/20/99
Date

Independently Reviewed by: Thomas Y. Chang 8/20/99
Date

Approved by: J. Cooper 8/20/99
Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

DISCUSSION OF TESTING PROGRAM AND RESULTS

Two soil borings were drilled in the vicinity of the proposed location of the Canister Transfer Building at the Private Fuel Storage Facility in October 1998 for the purpose of retrieving thin-walled tube samples of the in situ soils near the founding level of the proposed location of the Canister Transfer Building for laboratory testing.

Undisturbed samples were obtained using 3 in. diameter thin-walled tubes (Shelby tube) that were 30-in. long. Thin-walled tube sampling methods were performed in accordance with the methods outlined in ASTM D1587. All sampling tubes were new, hardened extruded steel, and were coated by spraying with silicone prior to sampling. These samples were carefully packed and shipped, via air-freight to minimize disturbance, to Stone & Webster's Geotechnical Laboratory in Boston, MA for testing.

As indicated on the boring logs, continuous samples were obtained in each of these borings over the depth interval from 5 ft to 11 ft, pushing these tubes 24 inches for each sample. Recovery in these tubes was 23 to 24 inches for all samples, and the condition of the tube tip was good for all tubes.

Laboratory tests were performed on these soils primarily to determine design parameters applicable for the in situ soils for use in assessing their ability to resist sliding of the Canister Transfer Building due to estimated loads from the Design Earthquake. Attachment A presents the general aspects of this testing program, which was prepared in accordance with the requirements of Stone & Webster Geotechnical Technical Procedure GTP 3.1. As indicated on the Utilization of Sample form (Sheet 3 of Attachment A), these tests included consolidated undrained triaxial compression tests, visual classifications, selected index property tests, and Atterberg Limits tests. The purpose of this document is to provide a record of these tests and test results.

These tests were performed in accordance with the following American Society for Testing and Materials standards:

D2216	1992	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock
D2487	1993	Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)
D2488	1993	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
D4318A	1995	Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

D4767 1995 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

Attachment B presents the results of the consolidated undrained triaxial compression tests. These tests were performed in accordance with the requirements of ASTM D4767, except the specimens were not saturated prior to consolidation or testing because these soils are not expected to be saturated during the life of the facility. These tests were performed on samples obtained at approximately 5.5 ft below ground surface, which is the approximate depth of the bottom of the foundation for the Canister Transfer Building. The specimens were subjected to confining pressures of 1.7 ksf to emulate the anticipated loading condition beneath the foundation of this building prior to the earthquake. They were sheared undrained, without pore pressure measurements, at a strain rate of 0.7 percent per minute, until the axial strain reached 20 percent. The results indicate that Sample U-1B in Boring CTB-N, which was a moderately plastic silty clay, had an undrained shear strength of 3.00 ksf, and Sample U-1B in Boring CTB-S, which was a highly plastic silt, had an undrained shear strength of 2.05 ksf.

Atterberg limits tests were performed in accordance with the requirements of ASTM D4318A using the one-point liquid limit method. The specimen used for the plastic limit determination was taken from the liquid limit specimen. The results of these tests are summarized in Table 1 below and indicate that the soils near the founding level of the Canister Transfer Building are moderately to highly plastic silty clay and highly plastic silt.

TABLE 1

Atterberg Limits Testing Performed in November 1998

Boring	Sample	Depth Feet	Water Content %	LL %	PL %	PI %	Plasticity
CTB-N	U-1A	5.05	30.6	38.4	23.1	15.3	Mod'ly
CTB-N	U-1B	5.70	30.1	41.3	22.5	18.8	Mod'ly
CTB-N	U-1D	6.70	46.6	50.8	23.1	27.7	Mod'ly
CTB-N	U-1E	6.95	67.7				Highly
CTB-N	U-2A	7.10	69.0	74.2	45.4	28.8	Highly
CTB-S	U-1A	5.05	85.5				Highly
CTB-S	U-1B	5.80	73.6	66.2	40.9	25.3	Highly
CTB-S	U-1E	6.95	56.4				Highly

APPENDIX A

Laboratory Testing Program

LABORATORY TESTING PROGRAM
GENERAL INFORMATION

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: PRIVATE FUEL STORAGE, LLC	J.O. NUMBER: 05996.02	REVISION	1	2	3	4	PREPARED BY: PJ Trudeau	DATE PREPARED: 10/19/98
SITE: PFSF SKULL VALLEY, UT		DATE					APPROVED BY: J.L. Rosenblad	RECEIVED BY: AC SMITH
		REVISED BY						

FEATURE OR PHASE OF PROJECT: CANISTER TRANSFER BUILDING	NEAREST CITY OR TOWN: TOOELE, UT	NEAREST AIRPORT HANDLING FREIGHT: SALT LAKE CITY, UT

PROJECT MANAGER: JL DONNELL	LEAD GEOTECHNICAL ENGINEER: PJ TRUDEAU	SUPERVISOR OF FIELD WORK: RP GILLESPIE	SUPERVISOR OF OFFICE WORK: JL ROSENBLAD	CONTACT FOR LABORATORY WORK: PJ TRUDEAU
DATE PROGRAM ASSIGNED: 10/16/98	DATE TESTING TO START: 10/17/98	DATE BORING LOG DATA NEEDED:	DATE ALL TEST DATA AVAILABLE:	DATE ALL FINAL PLOTS FORWARDED:

TYPES OF STRUCTURES INVOLVED:
MAT-SUPPORTED CANISTER TRANSFER BUILDING

PARTICULAR CRITICAL FEATURES OF STRUCTURES:
SLIDING STABILITY DURING DESIGN EARTHQUAKE ($a_h = 0.67\,g$)

TYPES OF BEHAVIOR TO BE ANALYZED:
COHESION AVAILABLE TO RESIST SLIDING

TYPICAL ANTICIPATED LOADING STRESSES:
STATIC $\Delta q \sim 1.7$ KSF . STATIC + DESIGN EARTHQUAKE $\Delta q \sim 3.6$ KSF

PRIMARY OBJECTIVES OF LABORATORY TESTS:
DETERMINE PLASTICITY & COHESION AVAILABLE TO RESIST SLIDING DUE TO DESIGN EARTHQUAKE

GENERAL MAGNITUDE OF TESTING PROGRAM:
SMALL

NUMBER OF DISTURBED SAMPLES: 0
NUMBER OF UNDISTURBED SAMPLES: 6 TUBES

TYPE OF REPORT TO CONTAIN RESULTS:
SAFETY ANALYSIS REPORT (SAR)

UNITS FOR REPORTING STRESSES: KSF

ADDITIONAL:
TESTING & REPORTING MUST MEET QA CAT I REQUIREMENTS.

LABORATORY TESTING PROGRAM
SPECIFIC TEST REQUIREMENTS

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	SHEET 2 / DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02	DATE					Paul J. Dunlean	10/19/98
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: J.L. Rosenblad	RECEIVED BY: AC SMITH

NOTE	COL. NO.	TYPE OF TEST	REQUIREMENTS OF TYPE OF TEST AND PROCEDURES TO BE USED (SIZE OF SPECIMENS, RATES OF STRAIN, CONFINING PRESSURES, CYCLES OF RECOMPRESSION ETC.)
A	ALL	ALL TESTS	SELECTION OF NECESSARY AND APPROPRIATE DETERMINATIONS AND/OR TEST PROCEDURES TO BE MADE BY LABORATORY PERSONNEL.
B	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION ONLY; NO INDEX PROPERTIES TO BE DETERMINED.
C	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION TO BE SUPPLEMENTED BY DETERMINATIONS OF SPECIFIC INDEX DETERMINATIONS INDICATED IN SUBSEQUENT COLUMNS.
D	3	ATTERBERG LIMITS	FOR CLASSIFICATION PURPOSES ONLY; ONE-POINT LIQUID LIMIT DETERMINATION ACCEPTABLE.
E	5	GRADATION ANALYSES	SIEVE ANALYSIS ONLY; NO HYDROMETER ANALYSIS REGARDLESS OF PERCENT FINES.
F	8	CU	CONSOLIDATION PRESSURE = 1.72 KSF, SHEAR UNDRAINED

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02	DATE					Paul J. Trudeau	10/19/98
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: J.L. Rosenblad	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15 VOID RATIO; 16; 17

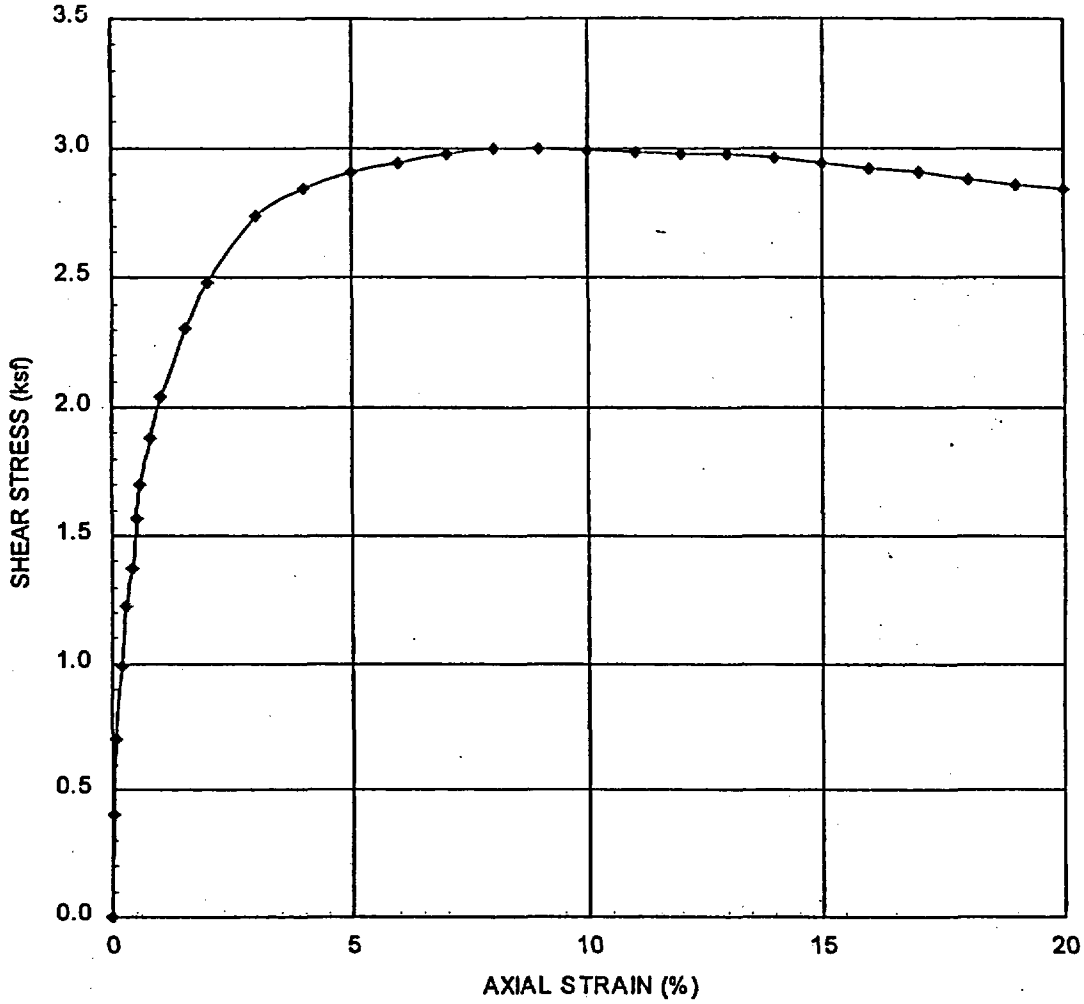
BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D						F									
CTB-N	U	1	5-7	—	SEE		BELOW —														
↓	U	1A	5	X	X	X															
	U	1E	7	X	X																
	U	1B	~5.5	X	X	X				X		X						X			IF NO APPARENT COHESION, PERFORM 7, 8, & 15 ON U1D
	U	1D	~6.5	X	X	X															INSTEAD OF U1B.
CTB-S	U	1	5-7	—	SEE		BELOW —														
↓	U	1A	5	X	X																
	U	1E	7	X	X																
	U	1B	~5.5	X	X	/				X		X						X			IF NO APPARENT COHESION, PERFORM 7, 8, & 15 ON U1D
	U	1D	~6.5	~~/~~	~~/~~	~~/~~	← CANCELLED					P.J.T.									INSTEAD OF U1B.
CTB-N	U	2	7-9	X	X	X															} SELECT SPECIMEN
CTB-S	U	2	7-9	~~/~~	~~/~~	~~/~~	← CANCELLED					P.J.T.									NEAR TOP OF TUBE

TEST TO BE PERFORMED (/) TEST IN PROGRESS TEST COMPLETED & CHECKED (X) TEST CANCELLED

APPENDIX B

Consolidated Undrained Triaxial Tests

Plots and Data



SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	10/26/98
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.4 ft	CHECKED:	PJT
DESCRIPTION:	Silty CLAY		

SPECIMEN INFORMATION:

HEIGHT:	0.547 ft	WATER CONTENT:	30.1 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	77.3 pcf
AREA:	0.0443 ft^2	VOID RATIO:	1.20

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	3.00 ksf
COMPRESSIVE STRENGTH:	6.00 ksf
FAILURE STRAIN:	8.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY, UTAH
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATED UNDRAINED COMPRESSION TEST BORING CTB-N, SAMPLE U-1B	JO 05996.02 November 1998

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-N	DATE:	10/26/98
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.4 ft	CHECKED:	PJT
DESCRIPTION:	Silty CLAY		

SPECIMEN INFORMATION:

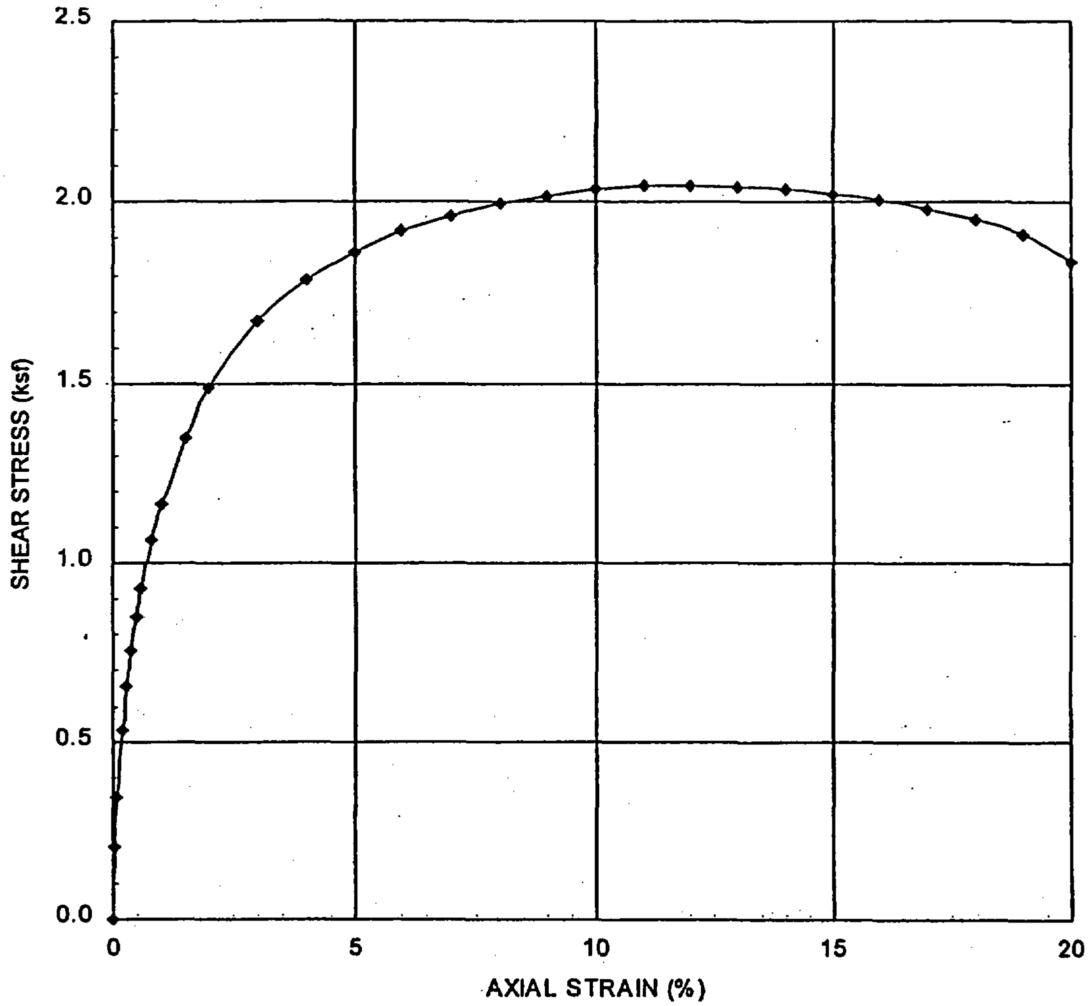
HEIGHT:	0.547 ft	WATER CONTENT:	30.1 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	77.3 pcf
AREA:	0.0443 ft^2	VOID RATIO:	1.20

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH: 3.00 ksf
COMPRESSIVE STRENGTH: 6.00 ksf
FAILURE STRAIN: 8.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.42	32.79	0.00	0.000	0.0443	0.00	0.00
0.50	38.94	0.05	0.036	0.0443	0.81	0.40
0.59	43.50	0.10	0.062	0.0443	1.41	0.70
0.75	47.84	0.20	0.088	0.0444	1.98	0.99
0.92	51.55	0.30	0.109	0.0444	2.46	1.23
1.09	53.80	0.40	0.122	0.0445	2.75	1.38
1.25	56.80	0.50	0.140	0.0445	3.14	1.57
1.42	58.78	0.60	0.151	0.0446	3.40	1.70
1.75	61.70	0.80	0.168	0.0447	3.77	1.89
2.09	64.16	1.00	0.183	0.0447	4.08	2.04
2.92	68.46	1.50	0.208	0.0450	4.62	2.31
3.75	71.36	2.00	0.225	0.0452	4.97	2.49
5.42	75.70	3.00	0.250	0.0457	5.47	2.74
7.09	77.85	4.00	0.263	0.0461	5.69	2.84
8.76	79.28	5.00	0.271	0.0466	5.81	2.90
10.42	80.39	6.00	0.277	0.0471	5.88	2.94
12.09	81.48	7.00	0.284	0.0476	5.95	2.98
13.76	82.38	8.00	0.289	0.0482	6.00	3.00
15.42	82.90	9.00	0.292	0.0487	6.00	3.00
17.09	83.32	10.00	0.294	0.0492	5.98	2.99
18.76	83.76	11.00	0.297	0.0498	5.97	2.98
20.42	84.26	12.00	0.300	0.0503	5.96	2.98
22.09	84.85	13.00	0.303	0.0509	5.96	2.98
23.76	85.18	14.00	0.305	0.0515	5.93	2.96
25.43	85.45	15.00	0.307	0.0521	5.89	2.94
27.09	85.70	16.00	0.308	0.0527	5.84	2.92
28.76	86.00	17.00	0.310	0.0534	5.81	2.90
30.43	86.26	18.00	0.312	0.0540	5.77	2.88
32.09	86.50	19.00	0.313	0.0547	5.72	2.86
33.76	86.83	20.00	0.315	0.0554	5.69	2.84



SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	10/26/98
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.5 ft	CHECKED:	PJT
DESCRIPTION:	SILT		

SPECIMEN INFORMATION:

HEIGHT:	0.543 ft	WATER CONTENT:	73.6 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	44.9 pcf
AREA:	0.0445 ft^2	VOID RATIO:	2.78

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.05 ksf
COMPRESSIVE STRENGTH:	4.09 ksf
FAILURE STRAIN:	12.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY, UTAH
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATED UNDRAINED COMPRESSION TEST BORING CTB-S, SAMPLE U-1B	JO 05996.02 November 1998

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	10/26/98
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.5 ft	CHECKED:	PJT
DESCRIPTION:	SILT		

SPECIMEN INFORMATION:

HEIGHT:	0.543 ft	WATER CONTENT:	73.6 %
DIAMETER:	0.238 ft	DRY UNIT WEIGHT:	44.9 pcf
AREA:	0.0445 ft^2	VOID RATIO:	2.78

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.7 %/min
CELL PRESSURE:	1.7 ksf		

UNDRAINED SHEAR STRENGTH:	2.05 ksf
COMPRESSIVE STRENGTH:	4.09 ksf
FAILURE STRAIN:	12.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.16	32.92	0.00	0.000	0.0445	0.00	0.00
1.24	36.04	0.05	0.018	0.0446	0.41	0.20
1.33	38.16	0.10	0.031	0.0446	0.68	0.34
1.49	41.05	0.20	0.047	0.0446	1.06	0.53
1.66	42.99	0.30	0.059	0.0447	1.31	0.66
1.82	44.58	0.40	0.068	0.0447	1.52	0.76
1.99	46.02	0.50	0.076	0.0448	1.70	0.85
2.15	47.27	0.60	0.084	0.0448	1.87	0.93
2.48	49.32	0.80	0.096	0.0449	2.13	1.06
2.82	50.92	1.00	0.105	0.0450	2.33	1.17
3.64	53.90	1.50	0.122	0.0452	2.70	1.35
4.47	56.15	2.00	0.135	0.0455	2.98	1.49
6.13	59.27	3.00	0.154	0.0459	3.34	1.67
7.78	61.38	4.00	0.166	0.0464	3.57	1.79
9.44	62.92	5.00	0.175	0.0469	3.73	1.86
11.09	64.18	6.00	0.182	0.0474	3.84	1.92
12.75	65.21	7.00	0.188	0.0479	3.93	1.96
14.40	66.10	8.00	0.193	0.0484	3.99	2.00
16.06	66.82	9.00	0.198	0.0490	4.03	2.02
17.71	67.50	10.00	0.201	0.0495	4.07	2.04
19.37	68.05	11.00	0.205	0.0501	4.09	2.04
21.02	68.47	12.00	0.207	0.0506	4.09	2.05
22.68	68.81	13.00	0.209	0.0512	4.08	2.04
24.33	69.10	14.00	0.211	0.0518	4.07	2.03
25.99	69.32	15.00	0.212	0.0524	4.05	2.02
27.64	69.42	16.00	0.213	0.0530	4.01	2.00
29.30	69.48	17.00	0.213	0.0537	3.97	1.98
30.95	69.36	18.00	0.212	0.0543	3.91	1.95
32.61	69.05	19.00	0.210	0.0550	3.83	1.91
34.26	68.00	20.00	0.204	0.0557	3.67	1.84

PROJECT DOCUMENT INDEPENDENT REVIEW CHECKLIST
USING ASME N45.2 OR NQA-1 FORMAT

Project No. 05996.02
Job Book File Location Q2.9

Type of Document
o REPORT: SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING - NOVEMBER 1998

- Design Criteria _______ Rev. No. _______
- Project Specification _______ Rev. No. _______
- Project Diagram _______ Rev. No. _______
- License Document _______ Rev. No. _______
- List Drawing/Diagrams and Revision Number

This sheet may be used for more than 1 diagram (list or reference all diagrams below)

	Yes	No	N/A
Were the inputs correctly selected and incorporated into the design?			✓
Are assumptions necessary to perform the design activity adequately described and reasonable?			✓
Where necessary, are the assumptions identified for subsequent reverifications when the detailed design activities are completed?			✓
Are the appropriate quality and quality assurance requirements specified?	✓		
Are the applicable codes, standards, and regulatory requirements, including applicable issues and addenda, properly identified and are their requirements for design met?	✓		
Have applicable construction and operating experience been considered?			✓
Have the design interface requirements been satisfied?			✓
Was an appropriate design method used?			✓
Is the output reasonable compared to inputs?	✓		

JO 05996.02
REPORT: SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING – NOVEMBER 1998

	Yes	No	N/A
Are the specified parts, equipment, and processes suitable for the required applications?	✓		
Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?			✓
Have adequate maintenance features and requirements been specified?			✓
Are accessibility and other design provisions adequate for performance of needed maintenance and repair?			✓
Has adequate accessibility been provided to perform the inservice inspection expected to be required during the plant life?			✓
Has the design properly considered radiation exposure to the public and plant personnel?			✓
Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?			✓
Have adequate preoperational and subsequent periodic test requirements been appropriately specified?			✓
Are adequate handling, storage, cleaning, and shipping requirements specified?			✓
Are adequate identification requirements specified?	✓		
Are requirements for record preparation, review, approval, retention, etc., adequately specified?			✓
Is the design output reasonable compared to the design inputs?			✓
Are the necessary design input and verification requirements for interfacing organizations specified in the design documents or in supporting procedures as instructions?			✓

[This checklist meets the requirements for both N45.2 and NQA-1 for design verification requirements]

Thomas Y. Chang	Thomas Y. Chang	8/20/99
Printed Name	Signature	Date

APPENDIX 2A

ATTACHMENT 5

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
MARCH 1999

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
MARCH 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
MARCH 1999

Private Fuel Storage Facility
Skull Valley
Private Fuel Storage, LLC

Prepared by:	Alan C Smith	4-27-99 Date
Reviewed by:	Thomas Y. Chang	5-3-99 Date
Independent Review by:	Thomas Y. Chang	5-3-99 Date
Approved by:	J L Cooper	5-3-99 Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

INTRODUCTION

Additional geotechnical laboratory testing was performed in March of 1999 in preparation of the response to the NRC request (NRC/PFS TELECONFERENCE of 3/16/99) that PFS (Private Fuel Storage, LLC) provide profiles of Atterberg limits and shear strength across the pad emplacement area. These tests were performed on undisturbed tube samples and Standard Penetration Test (SPT) jar samples obtained from the borings that were drilled in the pad emplacement area in October of 1996. The logs of these borings are included in Appendix 2A of the Safety Analysis Report (SAR) of the Private Fuel Storage Facility, Docket No. 72-22. The purpose of this addendum is to document these test results and incorporate them into Attachment 2 of Appendix 2A of the SAR.

The tests performed included water content, Atterberg limits, and consolidated-undrained triaxial compression tests. A total of 63 natural water content tests, 63 Atterberg limits tests, and 3 consolidated–undrained triaxial compression tests were performed on soil samples that were obtained from borings drilled in the pad emplacement area.

The tests were conducted in accordance with the following American Society for Testing and Materials standards:

D-2216	1992	Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D-2850	1995	Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression
D-4318	1995A	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

DISCUSSION OF TESTING PROGRAM AND RESULTS

Table 1 presents results of testing for water content and Atterberg limits in the form of a matrix of these data vs depth, along with data for all samples that have been tested in the borings drilled in the proposed emplacement area. The z_{avg} column shows the midpoint depth for the sample (e.g.; z_{avg} = 3 ft for a split-spoon sample that was driven from 2 ft to 4 ft). For the undisturbed

tube samples, the z_{avg} value is the midpoint of the section of the undisturbed tube sample that was tested.

In this matrix, the data are arranged to correspond with the locations of the borings in the field, with north situated at the top of the sheet. For example, Boring A-1, which was drilled in the northwest corner of the proposed emplacement area, is located in the upper left corner of the matrix. Borings B-1, C-1, and D-1 were drilled in locations east of A-1, and these are arranged to the right of A-1 in the matrix. Boring A-2 was drilled at a location south of A-1, and it is arranged in the matrix in the row just below A-1. The remaining borings are located similarly with respect to their locations in the plan view. This results, therefore, in Borings A-4 through D-4 being located at the bottom of the matrix, with A-4 on the left (i.e., the southwestern corner of the proposed emplacement area) and D-4 on the far right (i.e., the southeastern corner of the proposed emplacement area).

Since the samples were taken in October 1996, the water contents determined for the SPT samples, which were stored in glass jars, are questionable. The lids of the jar samples were sealed in wax when received in the laboratory in November of 1996, but some of them were opened and examined since then. Most of the jar samples tested for water content in March 1999, however, appeared to have been unopened.

Two of the samples, Boring D-1, Sample S-5, and Boring D-4, Sample S-6, had been opened and tested for water content in January 1997. The water contents measured in January 1997 were 20.7% and 18.0%. As measured during this program in March 1999, the water contents of these samples had dropped to 18.0% and 13.4%. This drop in water content for these samples is likely due to exposure to air during a check of the soil description. The water contents reported in Table 1 for these samples are those from the January 1997 tests.

The consolidated-undrained triaxial compression tests were conducted on undisturbed tube samples from Borings B-1, B-3, and C-2. These samples were setup in a triaxial cell, consolidated to a confining pressure equal to the expected overburden pressure at the center of the fully loaded pad, and sheared undrained at a strain rate of 0.8% per minute. The procedures specified in ASTM D-2850 were followed in performing these tests, except, prior to loading, the samples were allowed to consolidate for at least one hour. Previous testing on the soils from this site, presented in Appendix 2A of the SAR, indicates that these soils are partially saturated; therefore, consolidation due to the imposed confining pressure was not expected to take long. The one-hour wait appeared to be adequate, since most of the change in vertical deformation took place in the first 10 minutes of consolidation. Subsequently, the drain line was closed and the sample was loaded to an axial strain of 20%. The results of the triaxial compression tests are presented in the form of tables and plots of the axial strain vs shear stress on the pages following Table 1. As shown, the undrained shear strength exceeded 3 ksf for all of these samples.

LABORATORY TESTING PROGRAM
GENERAL INFORMATION

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE	3/25/99				Paul J. Trudeau	3/17/99
SITE PFSF SKULL VALLEY, UT		REVISED BY	P.J.T.				APPROVED BY JLCOOPER P.J.T.	RECEIVED BY ACS

FEATURE OR PHASE OF PROJECT	NEAREST CITY OR TOWN	NEAREST AIRPORT HANDLING FREIGHT
STORAGE FACILITY		

PROJECT MANAGER	LEAD GEOTECHNICAL ENGINEER	SUPERVISOR OF FIELD WORK	SUPERVISOR OF OFFICE WORK	CONTACT FOR LABORATORY WORK
JLDONNELL	PJTRUDEAU		GRDOTON	PJTRUDEAU
DATE PROGRAM ASSIGNED 3/17/99	DATE TESTING TO START 3/17/99	DATE BORING LOG DATA NEEDED	DATE ALL TEST DATA AVAILABLE	DATE ALL FINAL PLOTS FORWARDED

TYPES OF STRUCTURES INVOLVED:
STORAGE PADS (CONCRETE MATS 30' x 64' x 3')

PARTICULAR CRITICAL FEATURES OF STRUCTURES:
SLIDING STABILITY DURING DESIGN EARTHQUAKE

TYPES OF BEHAVIOR TO BE ANALYZED:
COHESION AVAILABLE TO RESIST SLIDING

TYPICAL ANTICIPATED LOADING STRESSES:
~ 2 KSF AT BASE OF PAD

PRIMARY OBJECTIVES OF LABORATORY TESTS:
DETERMINE ATTERBERG LIMITS VS DEPTH ACROSS THE SITE & COHESION AVAILABLE TO RESIST SLIDING

GENERAL MAGNITUDE OF TESTING PROGRAM:	NUMBER OF DISTURBED SAMPLES:
MINOR	NUMBER OF UNDISTURBED SAMPLES: 4 TO BE TESTED
TYPE OF REPORT TO CONTAIN RESULTS: SAR & RESPONSE TO NRC RAI No. 2	UNITS FOR REPORTING STRESSES: KSF

ADDITIONAL:

RESULTS REQUIRED BY 3/30/99 FOR INCORPORATION IN RESPONSE TO NRC THAT IS DUE 3/31/91

TESTING & REPORTING MUST MEET QA CAT I REQUIREMENTS

GTP 3.1-1
Attachment 1

LABORATORY TESTING PROGRAM
SPECIFIC TEST REQUIREMENTS

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
2

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE	3/25/99				Paul J. Trudeau	3/17/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY	P.J.T.				APPROVED BY: J L COOPER P.J.T.	RECEIVED BY: ACS

NOTE	COL. NO.	TYPE OF TEST	REQUIREMENTS OF TYPE OF TEST AND PROCEDURES TO BE USED (SIZE OF SPECIMENS, RATES OF STRAIN, CONFINING PRESSURES, CYCLES OF RECOMPRESSION ETC.)
A	ALL	ALL TESTS	SELECTION OF NECESSARY AND APPROPRIATE DETERMINATIONS AND/OR TEST PROCEDURES TO BE MADE BY LABORATORY PERSONNEL.
B	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION ONLY; NO INDEX PROPERTIES TO BE DETERMINED.
C	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION TO BE SUPPLEMENTED BY DETERMINATIONS OF SPECIFIC INDEX DETERMINATIONS INDICATED IN SUBSEQUENT COLUMNS.
D	3	ATTERBERG LIMITS	FOR CLASSIFICATION PURPOSES ONLY; ONE-POINT LIQUID LIMIT DETERMINATION ACCEPTABLE.
E	5	GRADATION ANALYSES	SIEVE ANALYSIS ONLY; NO HYDROMETER ANALYSIS REGARDLESS OF PERCENT FINES.
F	9	CU	CONSOLIDATION PRESSURE = 2.13 KSF (= σ_{vo} UNDER CENTER OF PAD
			WITH FULLY LOADED CASKS - P 28 CALC 05996.01-G(B)-03-1)

1

GTP 3.1-1
Attachment 2

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Dudeau	3/17/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCOOPER P.J.T.	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	AVG DEPTH (FT.)	1 TYPE OF TEST	2 CLASSIFICATION	3 WATER CONTENT	4 ATTERBERG LIMITS	5 PERCENT FINES	6 GRADATION ANALYSES	7 SPECIFIC GRAVITY	8 UNIT WEIGHT	9 UNDRAINED COMPRESSION	10 CONSOLIDATED TRIAXIAL	11 DRAINED DIRECT SHEAR	12 CONSOLIDATION	13 COMPACTION	14 RELATIVE DENSITY	15 PERMEABILITY	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
A-1	S	1	0.8																		
A-1	S	2	5.8																		
A-1	S	3	10.8																		
A-1	S	4	15.8																		
A-1	S	5	20.8																		
B-2	S	1	0.8																		
B-2	S	2	5.8																		
B-2	U	1A	8+																		
B-2	S	3	10.8																		
B-2	S	4	15.8																		Nonplastic, unable to roll thread at 15 blows
B-2	S	5	20.8																		
B-2	S	6	25.8																		

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
4

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Trudeau	3/17/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCOOPER RJD.	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	AVG DEPTH (FT.)	1 TYPE OF TEST	2 CLASSIFICATION	3 WATER CONTENT	4 ATTERBERG LIMITS	5 PERCENT FINES	6 GRADATION ANALYSES	7 SPECIFIC GRAVITY	8 UNIT WEIGHT	9 UNDRAINED COMPRESSION	10 CONSOLIDATED TRIAXIAL	11 DRAINED DIRECT SHEAR	12 CONSOLIDATION	13 COMPACTION	14 RELATIVE DENSITY	15 PERMEABILITY	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
C-3	S	1	0.8																		
C-3	S	2	5.8																		
C-3	S	3	10.8																		
C-3	S	4	15.8																		
C-3	S	5	20.8																		
C-3	S	6	25.8																		
D-4	S	1	0.8																		
D-4	S	4 A/B	15.8																		S-4A nonplastic
D-4	S	5	20.8																		
D-4	S	6	25.8																		
B-3	S	2	15.8																		
B-3	S	3	20.8																		

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
5

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Trudeau	3/17/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCooper RJD.	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15; 16; 17

BORING NO.	SAMPLE TYPE	SAMPLE NO.	AVG DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
B-3	S	4	25.8																		
B-3	S	5	30.8																		
C-2	S	2	15.8																		
C-2	S	3	20.8																		
C-2	S	4	25.8																		
D-3	S	2	5.8																		
D-3	S	3	10.8																		Nonplastic, unable to roll thread @ 15 blows
D-3	S	4	15.8																		
D-3	S	5	20.8																		
C-4	S	2A/B	5.8																		
C-4	S	4	15.8																		
C-4	S	5	20.8																		

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET: 6

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Trudeau	3/17/99
SITE PFSF SKULL VALLEY, UT		REVISED BY					**APPROVED BY** JLCOOPER PJT.	**RECEIVED BY** ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST

BORING NO.	SAMPLE TYPE	SAMPLE NO.	AVG DEPTH (FT.)	1 CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
C-4	S	6	25.8		X	X															
B-4	S	2	5.8		X	X															
B-4	S	4	15.8		X	X															
B-4	S	5	20.8		X	X															
B-4	S	6	25.8		X	X															
A-4	S	2	5.8		X	X															
A-4	S	3	10.8		X	X															Nonplastic
A-4	S	4	15.8		X	X															
A-4	S	5	20.8		X	X															
A-4	S	6	25.8		X	X															
A-3	S	2	5.8		X	X															
A-3	S	3	10.8		X	X															

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 7

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. [signature]	3/17/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCOOPER BJD.	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	AVG DEPTH (FT.)	1 TYPE OF TEST	2 CLASSIFICATION	3 WATER CONTENT	4 ATTERBERG LIMITS	5 PERCENT FINES	6 GRADATION ANALYSES	7 SPECIFIC GRAVITY	8 UNIT WEIGHT	9 UNDRAINED COMPRESSION	10 CONSOLIDATED TRIAXIAL	11 DRAINED DIRECT SHEAR	12 CONSOLIDATION	13 COMPACTION	14 RELATIVE DENSITY	15 PERMEABILITY	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
A-3	S	4	15.8		X	X															
A-2	S	4	15.8		X	X															
A-2	S	5	20.8		X	X															
A-2	S	6	25.8		X	X															
D-2	S	2	5.8		X	X															
D-2	S	3	10.8		X	X															
D-2	S	4	15.8		X	X															
D-2	S	5	20.8		X	X															
D-2	S	6	25.8		X	X															
B-1	S	3	10.8		X	X															
B-1	S	4	15.8		X	X															
B-1	S	5	20.8		X	X															

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: PRIVATE FUEL STORAGE, LLC	J.O. NUMBER: 05996.02 345W	REVISION	1	2	3	4	PREPARED BY: Paul J. Trudeau	DATE PREPARED: 3/17/99
SITE: PFSF SKULL VALLEY, UT		DATE	3/25/99				APPROVED BY: JLCooper PJT.	RECEIVED BY: ACS
		REVISED BY	PJT.					SHEET 8

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL (F)	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15 VOID RATIO	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
C-1	S	2	5.8		X	X															
C-1	S	4	15.8		X	X															
C-1	S	5	20.8		X	X															
D-1	S	2	5.8		X	X															
D-1	S	3	10.8		X	X															
D-1	S	4	15.8		X	X															
D-1	S	5	20.8		X	X															
B-1	U	2	6	X	X	X				X		X						X			
A-2	U	2	6	~~/~~	~~/~~	~~/~~				~~/~~		~~/~~						~~/~~			Used tube for previous tests May 1998
C-2	U	1	6	X	X	X				X		X						X			
B-3	U	1	6	X	X	X				X		X						X			
					/	/															

1

TEST TO BE PERFORMED TEST IN PROGRESS TEST COMPLETED & CHECKED TEST CANCELLED

GTP 3.1-1
Attachment 3

Boring No.	Sample	z_{avg}	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Liquidity Index
A-1	S 1	0.8					
A-1	S 2	5.8	34.7	54.8	30.9	23.9	0.16
A-1	S 3	10.8	19.8	28.8	25.8	3.0	-2.00
A-1	S 4	15.8	22.3	30.2	27.6	2.6	-2.04
A-1	S 5	20.8	55.4	58.6	43.0	15.6	0.79
A-1	S 6	25.8					
A-2	S 1	0.8	15.6	28.9	23.3	5.6	-1.38
A-2	U 2C	6.2	52.8	70.2	42.9	27.3	0.36
A-2	U 2E	7.0	45.4	61.8	36.7	25.1	0.35
A-2	S 3	10.8	18.4	27.0	24.5	2.5	-2.44
A-2	S 4	15.8	29.7	36.5	26.5	10.0	0.32
A-2	S 5	20.8	28.2	38.0	26.8	11.2	0.13
A-2	S 6	25.8	27.9	41.4	30.4	11.0	-0.23
A-3	S 1	0.8					
A-3	S 2	5.8	36.0	49.8	23.3	26.5	0.48
A-3	S 3	10.8	43.3	60.1	35.1	25.0	0.33
A-3	S 4	15.8	25.9	35.8	27.7	8.1	-0.22
A-3	S 5	20.8					
A-3	S 6	25.8					
A-4	S 1	0.8					
A-4	S 2	5.8	44.2	69.0	42.4	26.6	6.8
A-4	S 3	10.8	10.8		Nonplastic		
A-4	S 4	15.8	19.3	29.9	22.4	7.5	-41.3
A-4	S 5	20.8	37.8	56.5	41.6	14.9	-25.5
A-4	S 6	25.8	15.2	29.1	19.8	9.3	-49.5

Boring No.	Sample	z_{avg}	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Liquidity Index
B-1	S 1	0.8					
B-1	U 2D	6.7	45.2	59.8	34.7	25.1	0.42
B-1	S 3	10.8	23.0	39.4	29.0	10.4	-0.58
B-1	S 4	15.8	23.0	35.2	25.9	9.3	-0.31
B-1	S 5	20.8	45.9	50.3	35.8	14.5	0.70
B-1	S 6	25.8					
B-2	S 1	0.8					
B-2	S 2	5.8	32.0	47.4	25.6	21.8	0.29
B-2	U 1	9.0					
B-2	S 3	10.8	18.9	29.8	25.8	4.0	-1.73
B-2	S 4	15.8	12.6		Nonplastic		
B-2	S 5	20.8	43.9	55.1	46.2	8.9	-0.26
B-2	S 6	25.8	20.1	31.8	20.0	11.8	0.01
B-3	S 1	0.8	8.9	26.6	19.7	6.9	-1.57
B-3	U 1B	5.5	33.5	52.4	25.2	27.2	0.31
B-3	U 2	11.0					
B-3	S 2	15.8					
B-3	S 3	20.8	44.6	54.3	41.6	12.7	0.24
B-3	S 4	25.8					
B-3	S 5	30.8					
B-4	S 1	0.8					
B-4	S 2	5.8	48.4	56.5	27.8	28.7	0.72
B-4	U 3D	10.7	42.6	42.5	24.7	17.8	1.01
B-4	S 4	15.8	19.9	30.7	24.6	6.1	-0.77
B-4	S 5	20.8	24.2	35.4	29.9	5.5	-1.04
B-4	S 6	25.8	24.5	32.6	24.3	8.3	0.02

Boring No.	Sample	z_{avg}	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Liquidity Index
C-1	S 1	0.8					
C-1	S 2	5.8	53.0	67.4	39.3	28.1	0.49
C-1	U 3B	10.9	30.3	33.0	28.1	4.9	0.45
C-1	U 3C	11.1	38.9	47.8	34.6	13.2	0.33
C-1	U 3D	11.5	46.7	61.1	44.1	17.0	0.15
C-1	S 4	15.8	27.4	34.2	24.4	9.8	0.31
C-1	S 5	20.8	42.7	49.7	38.7	11.0	0.36
C-1	S 6	25.8					
C-2	S 1	0.8					
C-2	U 1D	6.5	50.5	70.3	41.3	29.0	0.32
C-2	U 2C	11.0	27.6	34.6	26.9	7.7	0.09
C-2	U 2E	11.8	39.7	41.2	28.5	12.7	0.88
C-2	S 2	15.8	30.3	40.0	24.4	15.6	0.38
C-2	S 3	20.8	41.8	48.8	37.2	11.6	0.40
C-2	S 4	25.8					
C-2	S 5	30.8					
C-3	S 1	0.8					
C-3	S 2	5.8	26.8	43.1	22.4	20.7	0.21
C-3	S 3	10.8	32.6	48.8	29.4	19.4	0.16
C-3	S 4	15.8	27.9	32.9	23.1	9.8	0.49
C-3	S 5	20.8	39.5	50.8	35.8	15.0	0.25
C-3	S 6	25.8	18.1	26.2	19.5	6.7	-0.21
C-4	S 1	0.8					
C-4	S 2A	5.2	28.6	46.1	22.9	23.2	0.25
C-4	S 2B	6.0	50.6	69.5	44.2	25.3	0.25
C-4	S 3	10.8	18.2	26.5	26.0	0.5	-15.60
C-4	S 4	15.8	26.5	36.6	26.9	9.7	-0.04
C-4	S 5	20.8	40.7	52.5	41.5	11.0	-0.07
C-4	S 6	25.8	18.7	29.2	20.1	9.1	-0.15

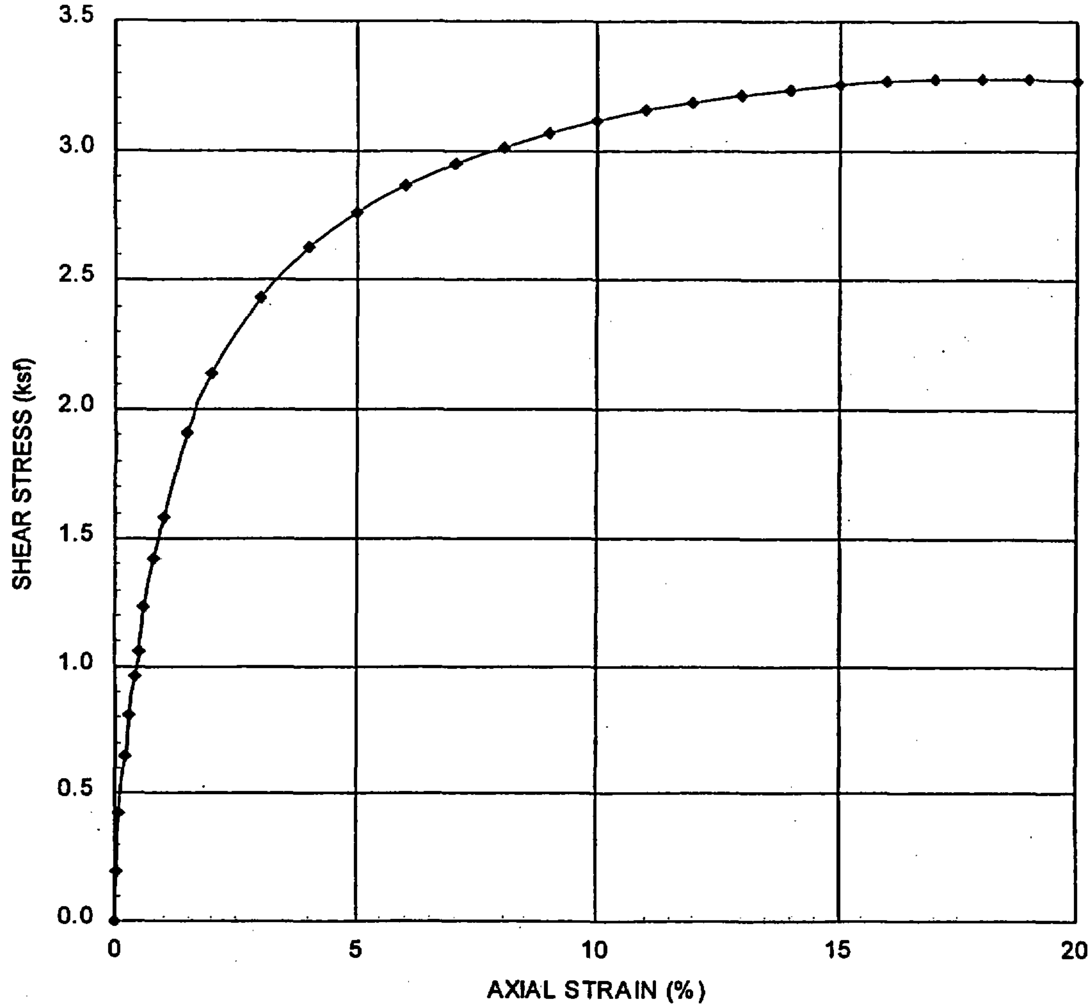
Boring No.	Sample	z_{avg}	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Liquidity Index
D-1	S 1	0.8					
D-1	S 2	5.8	36.3	54.6	29.4	25.2	0.27
D-1	S 3	10.8	28.6	40.5	25.2	15.3	0.22
D-1	S 4	15.8	32.2	47.3	33.1	14.2	-0.06
D-1	S 5	20.8	20.7	30.0	19.5	10.5	0.11
D-2	S 1	0.8					
D-2	S 2	5.8	36.9	46.4	31.1	15.3	0.38
D-2	S 3	10.8	34.2	54.0	28.6	25.4	0.22
D-2	S 4	15.8	22.6	44.3	29.9	14.4	-0.51
D-2	S 5	20.8	12.2	37.7	31.6	6.1	-3.18
D-2	S 6	25.8	13.9	31.4	19.5	11.9	-0.47
D-3	S 1	0.8					
D-3	S 2	5.8	23.5	43.4	27.3	16.1	-0.24
D-3	S 3	10.8	25.0		Nonplastic		
D-3	S 4	15.8	36.8	40.6	28.0	12.6	0.70
D-3	S 5	20.8	42.0	47.7	34.2	13.5	0.58
D-3	S 6	25.8					
D-4	S 1	0.8					
D-4	S 2	5.8	38.0	49.3	27.7	21.6	0.48
D-4	S 3A	10.3	16.8	24.7	23.3	1.4	-4.64
D-4	S 4A	15.4	8.3		Nonplastic		
D-4	S 4B	16.2	32.8	42.8	25.7	17.1	0.42
D-4	S 5	20.8	43.4	56.8	41.2	15.6	0.14
D-4	S 6	25.8	18.0	27.0	21.6	5.4	-0.67

Note: The natural water contents on split spoon samples shown above, other than D-1/S-5 and D-4/S-6, were tested in April 1998 and March 1999. Although the jar lids were sealed, some of them had been opened for visual classification at various times. Therefore, these values may not reflect actual conditions at the time of the sampling in October 1996.

TABLE 1
MATRIX OF ATTERBERG LIMITS IN PROPOSED EMPLACEMENT AREA
PFSF SAFETY ANALYSIS REPORT

APPENDIX A

Triaxial Test Plots and Data



SAMPLE INFORMATION:

BORING:	B-1	DATE:	03/29/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	6.5 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

HEIGHT:	0.539 ft	WATER CONTENT:	45.2 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	52.8 pcf
AREA:	0.0441 ft^2	VOID RATIO:	2.22

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.1 ksf		

UNDRAINED SHEAR STRENGTH:	3.26 ksf
COMPRESSIVE STRENGTH:	6.51 ksf
FAILURE STRAIN:	15.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY, UTAH
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATED UNDRAINED COMPRESSION TEST BORING B-1, SAMPLE U-2D	JO 05996.02 March 1999

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-1	DATE:	03/29/99
SAMPLE:	U-2D	TESTED BY:	ACS
DEPTH:	6.5 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

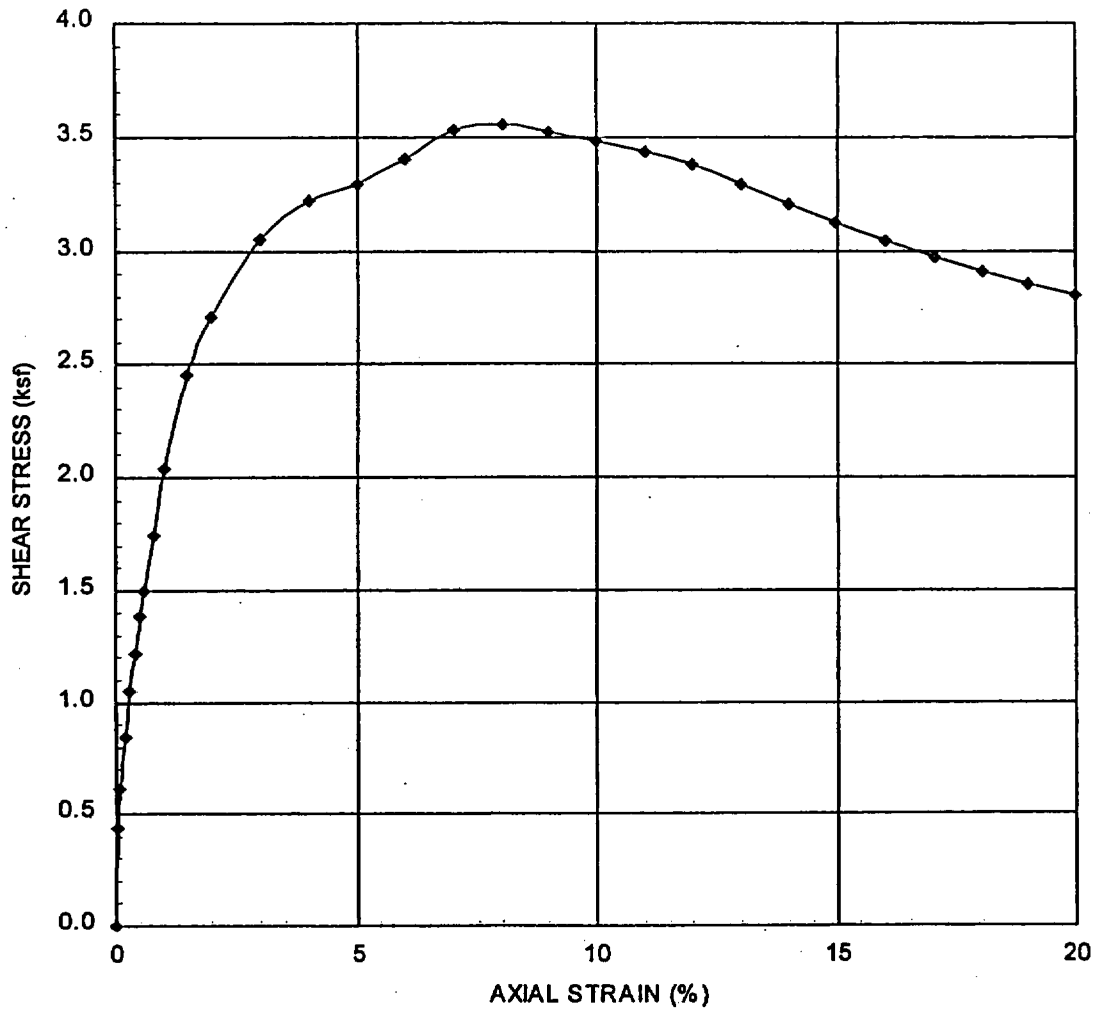
HEIGHT:	0.539 ft	WATER CONTENT:	45.2 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	52.8 pcf
AREA:	0.0441 ft^2	VOID RATIO:	2.22

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.13 ksf		

UNDRAINED SHEAR STRENGTH:	3.26 ksf
COMPRESSIVE STRENGTH:	6.51 ksf
FAILURE STRAIN:	15.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.80	1.84	0.00	0.000	0.0441	0.00	0.00
0.88	7.32	0.05	0.017	0.0441	0.39	0.19
0.96	13.90	0.10	0.038	0.0441	0.85	0.42
1.13	20.33	0.20	0.058	0.0442	1.30	0.65
1.29	24.90	0.30	0.072	0.0442	1.62	0.81
1.46	29.32	0.40	0.085	0.0443	1.93	0.96
1.62	32.00	0.50	0.094	0.0443	2.12	1.06
1.79	36.96	0.60	0.109	0.0444	2.46	1.23
2.11	42.44	0.80	0.126	0.0445	2.84	1.42
2.44	47.09	1.00	0.141	0.0446	3.16	1.58
3.26	56.80	1.50	0.171	0.0448	3.82	1.91
4.09	63.76	2.00	0.193	0.0450	4.28	2.14
5.73	73.08	3.00	0.222	0.0455	4.87	2.44
7.37	79.50	4.00	0.242	0.0459	5.26	2.63
9.02	84.30	5.00	0.256	0.0464	5.52	2.76
10.66	88.30	6.00	0.269	0.0469	5.73	2.87
12.30	91.73	7.00	0.280	0.0474	5.89	2.95
13.94	94.79	8.00	0.289	0.0479	6.03	3.01
15.59	97.52	9.00	0.298	0.0485	6.14	3.07
17.23	100.00	10.00	0.305	0.0490	6.23	3.11
18.87	102.42	11.00	0.313	0.0496	6.31	3.16
20.52	104.60	12.00	0.320	0.0501	6.38	3.19
22.16	106.70	13.00	0.326	0.0507	6.43	3.22
23.80	108.60	14.00	0.332	0.0513	6.47	3.24
25.45	110.45	15.00	0.338	0.0519	6.51	3.25
27.09	112.25	16.00	0.343	0.0525	6.54	3.27
28.73	113.80	17.00	0.348	0.0531	6.55	3.28
30.37	115.20	18.00	0.353	0.0538	6.55	3.28
32.02	116.50	19.00	0.357	0.0545	6.55	3.27
33.66	117.71	20.00	0.360	0.0551	6.54	3.27



SAMPLE INFORMATION:

BORING:	B-3	DATE:	03/29/99
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.2 ft	CHECKED:	TYC
DESCRIPTION:	Silty CLAY		

SPECIMEN INFORMATION:

HEIGHT:	0.515 ft	WATER CONTENT:	33.5 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	67.9 pcf
AREA:	0.0440 ft^2	VOID RATIO:	1.50

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.1 ksf		

UNDRAINED SHEAR STRENGTH:	3.55 ksf
COMPRESSIVE STRENGTH:	7.10 ksf
FAILURE STRAIN:	8.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY, UTAH
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATED UNDRAINED COMPRESSION TEST BORING B-3, SAMPLE U-1B	JO 05996.02 March 1999

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-3	DATE:	03/29/99
SAMPLE:	U-1B	TESTED BY:	ACS
DEPTH:	5.2 ft	CHECKED:	TYC
DESCRIPTION:	Silty CLAY		

SPECIMEN INFORMATION:

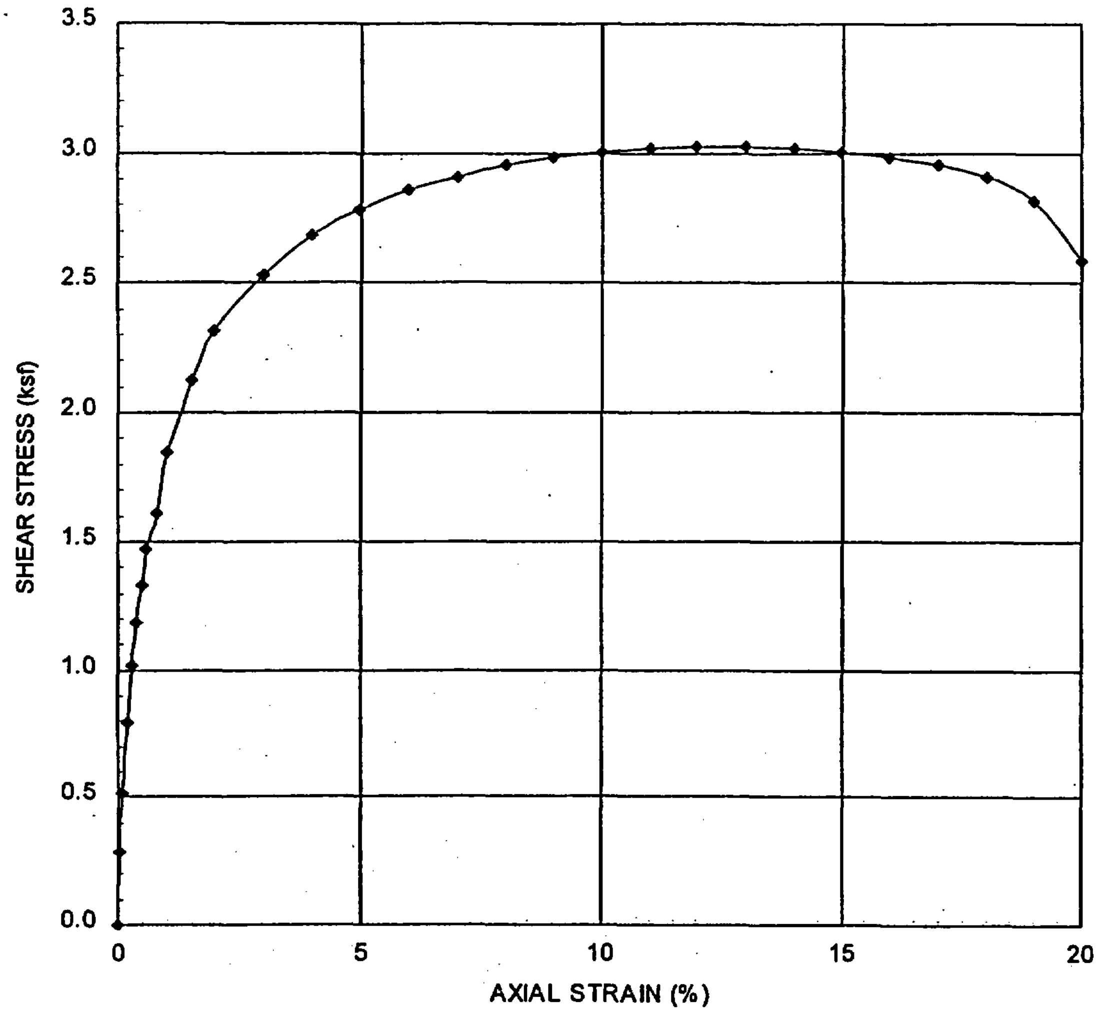
HEIGHT:	0.515 ft	WATER CONTENT:	33.5 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	67.9 pcf
AREA:	0.0440 ft^2	VOID RATIO:	1.50

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.13 ksf		

UNDRAINED SHEAR STRENGTH:	3.55 ksf
COMPRESSIVE STRENGTH:	7.10 ksf
FAILURE STRAIN:	8.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.76	2.10	0.00	0.000	0.0440	0.00	0.00
0.84	14.50	0.05	0.039	0.0440	0.88	0.44
0.92	19.45	0.10	0.054	0.0440	1.23	0.61
1.07	26.10	0.20	0.075	0.0441	1.69	0.85
1.23	31.85	0.30	0.093	0.0441	2.10	1.05
1.39	36.62	0.40	0.107	0.0442	2.43	1.22
1.54	41.41	0.50	0.122	0.0442	2.77	1.38
1.70	44.66	0.60	0.132	0.0442	2.99	1.50
2.02	51.80	0.80	0.155	0.0443	3.49	1.74
2.33	60.30	1.00	0.181	0.0444	4.07	2.04
3.11	72.60	1.50	0.219	0.0447	4.91	2.46
3.90	80.38	2.00	0.243	0.0449	5.42	2.71
5.47	91.17	3.00	0.277	0.0453	6.11	3.05
7.04	97.00	4.00	0.295	0.0458	6.44	3.22
8.61	100.00	5.00	0.304	0.0463	6.58	3.29
10.18	104.40	6.00	0.318	0.0468	6.80	3.40
11.75	109.54	7.00	0.334	0.0473	7.06	3.53
13.32	111.25	8.00	0.339	0.0478	7.10	3.55
14.88	111.52	9.00	0.340	0.0483	7.04	3.52
16.45	111.64	10.00	0.341	0.0489	6.97	3.49
18.02	111.15	11.00	0.339	0.0494	6.86	3.43
19.59	110.80	12.00	0.338	0.0500	6.76	3.38
21.16	109.00	13.00	0.332	0.0506	6.58	3.29
22.73	107.35	14.00	0.327	0.0511	6.40	3.20
24.30	105.95	15.00	0.323	0.0517	6.24	3.12
25.87	104.50	16.00	0.318	0.0524	6.08	3.04
27.44	103.30	17.00	0.315	0.0530	5.94	2.97
29.01	102.45	18.00	0.312	0.0536	5.82	2.91
30.58	101.80	19.00	0.310	0.0543	5.71	2.86
32.15	101.24	20.00	0.308	0.0550	5.61	2.80



SAMPLE INFORMATION:

BORING:	C-2	DATE:	03/29/99
SAMPLE:	U-1D	TESTED BY:	ACS
DEPTH:	6.3 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

HEIGHT:	0.543 ft	WATER CONTENT:	50.5 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	49.5 pcf
AREA:	0.0441 ft^2	VOID RATIO:	2.43

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.1 ksf		

UNDRAINED SHEAR STRENGTH:	3.03 ksf
COMPRESSIVE STRENGTH:	6.06 ksf
FAILURE STRAIN:	12.0 %

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY, UTAH
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP. BOSTON, MASSACHUSETTS	CONSOLIDATED UNDRAINED COMPRESSION TEST BORING C-2, SAMPLE U-1D	JO 05996.02 March 1999

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	03/29/99
SAMPLE:	U-1D	TESTED BY:	ACS
DEPTH:	6.3 ft	CHECKED:	TJC
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

HEIGHT:	0.543 ft	WATER CONTENT:	50.5 %
DIAMETER:	0.237 ft	DRY UNIT WEIGHT:	49.5 pcf
AREA:	0.0441 ft^2	VOID RATIO:	2.43

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	2.13 ksf		

UNDRAINED SHEAR STRENGTH: 3.03 ksf
COMPRESSIVE STRENGTH: 6.06 ksf
FAILURE STRAIN: 12.0 %

DIAL READING mm	FORCE GAGE mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
1.04	2.00	0.00	0.000	0.0441	0.00	0.00
1.12	10.15	0.05	0.025	0.0441	0.57	0.29
1.21	16.62	0.10	0.045	0.0442	1.03	0.51
1.37	24.68	0.20	0.071	0.0442	1.60	0.80
1.54	30.93	0.30	0.090	0.0442	2.03	1.02
1.70	35.68	0.40	0.105	0.0443	2.36	1.18
1.87	40.00	0.50	0.118	0.0443	2.67	1.33
2.03	43.90	0.60	0.130	0.0444	2.94	1.47
2.36	48.08	0.80	0.143	0.0445	3.22	1.61
2.69	54.85	1.00	0.164	0.0446	3.69	1.84
3.52	63.30	1.50	0.191	0.0448	4.26	2.13
4.35	68.96	2.00	0.208	0.0450	4.63	2.31
6.00	76.10	3.00	0.230	0.0455	5.07	2.53
7.66	81.30	4.00	0.247	0.0459	5.37	2.68
9.31	85.00	5.00	0.258	0.0464	5.56	2.78
10.96	88.30	6.00	0.268	0.0469	5.72	2.86
12.62	90.75	7.00	0.276	0.0474	5.82	2.91
14.27	93.11	8.00	0.283	0.0479	5.91	2.96
15.93	95.08	9.00	0.289	0.0485	5.97	2.99
17.58	96.81	10.00	0.295	0.0490	6.02	3.01
19.23	98.20	11.00	0.299	0.0496	6.04	3.02
20.89	99.64	12.00	0.304	0.0501	6.06	3.03
22.54	100.60	13.00	0.307	0.0507	6.05	3.02
24.20	101.60	14.00	0.310	0.0513	6.04	3.02
25.85	102.35	15.00	0.312	0.0519	6.01	3.01
27.50	102.87	16.00	0.314	0.0525	5.97	2.99
29.16	103.00	17.00	0.314	0.0531	5.91	2.96
30.81	102.60	18.00	0.313	0.0538	5.82	2.91
32.47	100.65	19.00	0.307	0.0545	5.63	2.82
34.12	93.60	20.00	0.285	0.0551	5.17	2.58

PROJECT DOCUMENT INDEPENDENT REVIEW CHECKLIST
USING ASME N45.2 OR NQA-1 FORMAT

Project No. 05996.02
Job Book File Location Q2.9

Type of Document
- Report: Supplemental Geotechnical Laboratory Testing - March 1999
- Design Criteria ______ Rev. No. ______
- Project Specification ______ Rev. No. ______
- Project Diagram ______ Rev. No. ______
- License Document ______ Rev. No. ______
- List Drawing/Diagrams and Revision Number

This sheet may be used for more than 1 diagram (list or reference all diagrams below)

	Yes	No	N/A
Were the inputs correctly selected and incorporated into the design?			✓
Are assumptions necessary to perform the design activity adequately described and reasonable?			✓
Where necessary, are the assumptions identified for subsequent reverifications when the detailed design activities are completed?			✓
Are the appropriate quality and quality assurance requirements specified?	✓		
Are the applicable codes, standards, and regulatory requirements, including applicable issues and addenda, properly identified and are their requirements for design met?	✓		
Have applicable construction and operating experience been considered?			✓
Have the design interface requirements been satisfied?			✓
Was an appropriate design method used?			✓
Is the output reasonable compared to inputs?	✓		

JO 05996.02
REPORT: SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING - MARCH 1999

	Yes	No	N/A
Are the specified parts, equipment, and processes suitable for the required applications?	✓		
Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?			✓
Have adequate maintenance features and requirements been specified?			✓
Are accessibility and other design provisions adequate for performance of needed maintenance and repair?			✓
Has adequate accessibility been provided to perform the inservice inspection expected to be required during the plant life?			✓
Has the design properly considered radiation exposure to the public and plant personnel?			✓
Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?			✓
Have adequate preoperational and subsequent periodic test requirements been appropriately specified?			✓
Are adequate handling, storage, cleaning, and shipping requirements specified?			✓
Are adequate identification requirements specified?	✓		
Are requirements for record preparation, review, approval, retention, etc., adequately specified?			✓
Is the design output reasonable compared to the design inputs?			✓
Are the necessary design input and verification requirements for interfacing organizations specified in the design documents or in supporting procedures as instructions?			✓

[This checklist meets the requirements for both N45.2 and NQA-1 for design verification requirements]

Thomas Y. Chang	Thomas Y. Chang	5-3-99
Printed Name	Signature	Date

APPENDIX 2A

ATTACHMENT 6

SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING

JUNE 1999

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
JUNE 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
JUNE 1999

Private Fuel Storage Facility
Skull Valley
Private Fuel Storage, LLC

Prepared by: Alan C Smith 7-22-99
Date

Reviewed by: Thomas Y. Chang 8-6-99
Date

Independent Review by: Thomas Y. Chang 8-6-99
Date

Approved by: J L Cooper 8-9-99
Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

INTRODUCTION

The Geotechnical Laboratory received 21 boxes of split spoon jar samples and 18 thin-walled tube samples from the Skull Valley site in December 1998 and 4 boxes of split spoon samples in January 1999. Six thin-walled tube samples from Borings CTB-N and CTB-S were delivered in October 1998. A testing program was developed identifying types of tests to be performed and which samples to test. The primary objective of the laboratory testing program was to develop the static and dynamic properties of the soils underlying the Canister Transfer Building. Testing began on March 8, 1999 and ended on June 22, 1999.

The tests performed were classification, water content, Atterberg limits, gradation analysis, specific gravity, density, consolidation, consolidated–undrained triaxial compression, cyclic triaxial compression, and resonant–column. They were conducted in accordance with the following American Society for Testing and Materials standards.

C-136	1996	Test Method for Sieve Analysis of Fine and Coarse Aggregates
D-422	1990	Test Method for Particle-Size Analysis of Soils
D-854	1992	Test Method for Specific Gravity of Soils
D-1140	1992	Test Method for Amount of Material in Soils Finer Than the No. 200 Sieve
D-2216	1992	Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D-2435	1990	Test Method for One-Dimensional Consolidation Properties of Soils
D-4015	1992	Test Methods for Modulus and Damping of Soils by the Resonant–Column Method
D-4318	1995a	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D-4767	1995	Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils
D-5311	1992	Test Method for Load Controlled Cyclic Triaxial Strength of Soil

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

TEST RESULTS

A total of 92 classification, 146 water content, 35 Atterberg limits, 4 percent fines, 18 gradation analysis, 36 density, 6 specific gravity, 5 consolidation, 10 consolidated–undrained triaxial compression, 5 cyclic triaxial compression, and 2 resonant–column tests were performed on the soil samples.

The results of testing for classification, water content, Atterberg limits, percent fines, specific gravity, wet density, dry density, void ratio, consolidation, and consolidated–undrained triaxial compression are presented in Table 1. Void ratio was determined from the dry density and the percent fines determined from the gradation analysis are shown. This table also contains data from tests conducted in October 1998 on soil from Borings CTB-N and CTB-S.

The gradation analysis results are shown in Figures 1 through 18. Hydrometer analyses and sieve analyses were performed on the samples shown in Figures 5, 6, 8, and 18. Only sieve analyses were performed on the rest of the samples shown in these figures.

The consolidation tests were conducted on thin-walled (Shelby) tube samples obtained from Borings CTB-4, CTB-5, CTB-N, and CTB-S. The stress vs strain plots, strain vs time plots, and data from these tests are presented in Appendix A. The samples from Borings CTB-N and CTB-S were inundated with water after primary consolidation had occurred during the pressure increment of 2 kips per square foot (ksf). This loading is slightly higher than the static load at the base of the Canister Transfer Building. The samples did not collapse after being inundated. The other samples were not inundated. Normally the applied loads are doubled for each increment of loading, but the load increments were reduced when the applied pressure neared the existing overburden pressure. This was done to obtain more data for the applied pressure vs strain plot to help define the maximum past pressure.

The consolidated–undrained triaxial compression tests were conducted on thin-walled tube samples from Borings CTB-1, CTB-4, CTB-5, CTB-6, CTB-N and CTB-S. The samples were setup in a triaxial cell and consolidated using a confining pressure (1.7 ksf) equivalent to the final effective stresses expected underneath the Canister Transfer Building. They were sheared undrained after consolidating for at least one hour. Since the in situ soils are not expected to be saturated throughout the life of the facility, these samples were not saturated prior to shearing. The axial strain vs shear stress plots and data from the triaxial compression tests are presented in Appendix B.

Cyclic triaxial compression tests were performed on thin-walled tube samples from Borings CTB-4, CTB-5, CTB-N, and CTB-S to demonstrate that these soils, even though they have high void ratios, will not collapse due to shaking caused by the Design Earthquake. The void ratios of the soils tested, listed in order of increasing depth below existing grade, were 2.20 at a depth of 6.3 ft, 2.01 at 8 ft, 1.90 at 9.6 ft, 1.58 at 23 ft, and 1.30 at 24.9 ft. The samples were consolidated under a confining pressure of 2.0 ksf for at least 2 hours, but they were not saturated prior to testing, since the in situ soils are not expected to be saturated throughout the life of the facility.

The tests were run undrained using a sinusoidal cyclic stress of 1.9 ksf applied at a frequency of 1 hertz for 500 cycles. The applied axial load and displacement were recorded using a strip-chart recorder. The test results and sample data, presented in Appendix C, indicate that the double-amplitude strain test results range from 0.3% to 1.2%. As indicated by the measured double-amplitude strains, none of these samples experienced strains that would be characterized as collapse due to shaking at cyclic stress levels in excess of those that might be caused due to the design earthquake.

Two thin-walled tube samples from Boring CTB-1 were used to conduct the resonant–column test. The shear modulus vs shear strain amplitude plots, G/G_o vs shear strain amplitude plots, damping ratio vs shear strain amplitude plots, maximum shear modulus vs effective confining pressure plots, and data are presented in Appendix D. The each sample was tested unsaturated at three confining pressures. First at the existing overburden pressure, then at the expected overburden pressure from the Canister Transfer building, and last at the expected overburden pressure plus the increase in overburden pressure due to the building.

LABORATORY TESTING PROGRAM
GENERAL INFORMATION

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 1 OF 22

CLIENT PRIVATE FUEL STORAGE, LLC	J.O. NUMBER 0599602 345H	REVISION	1	2	3	4	PREPARED BY PJ TRUDEAU / TY CHANG	DATE PREPARED 3/8/99
SITE PFSF SKULL VALLEY, UT		DATE					APPROVED BY FOR JLC Paul J. Trudeau	RECEIVED BY AC Smith
		REVISED BY						
FEATURE OR PHASE OF PROJECT CANISTER TRANSFER BUILDING		NEAREST CITY OR TOWN TOOELE, UT					NEAREST AIRPORT HANDLING FREIGHT SALT LAKE CITY, UT	

PROJECT MANAGER JL DONNELL	LEAD GEOTECHNICAL ENGINEER PJ TRUDEAU	SUPERVISOR OF FIELD WORK RP GILLESPIE	SUPERVISOR OF OFFICE WORK GR DOTON	CONTACT FOR LABORATORY WORK PJ TRUDEAU
DATE PROGRAM ASSIGNED 3/9/99	DATE TESTING TO START 3/9/99	DATE BORING LOG DATA NEEDED	DATE ALL TEST DATA AVAILABLE	DATE ALL FINAL PLOTS FORWARDED

TYPES OF STRUCTURES INVOLVED:
MAT-SUPPORTED CANISTER TRANSFER BUILDING

PARTICULAR CRITICAL FEATURES OF STRUCTURES:
SLIDING STABILITY DURING DESIGN EARTHQUAKE ($a_h = 0.67g$ & $a_v = 0.69g$)

TYPES OF BEHAVIOR TO BE ANALYZED:

TYPICAL ANTICIPATED LOADING STRESSES:
STATIC $\Delta q \sim 1.7$ KSF   STATIC + DESIGN EARTHQUAKE $\Delta q \sim 2.6$ KSF

PRIMARY OBJECTIVES OF LABORATORY TESTS:
DEVELOP STATIC & DYNAMIC PROPERTIES OF THE SOILS UNDERLYING THE CANISTER TRANSFER BUILDING.

GENERAL MAGNITUDE OF TESTING PROGRAM:
MODERATE

NUMBER OF DISTURBED SAMPLES: 156
NUMBER OF UNDISTURBED SAMPLES: 18

TYPE OF REPORT TO CONTAIN RESULTS:
GEOTECHNICAL DATA REPORT 05996.02-G(B)-2 REV 5 2 & SAR APP 2A

UNITS FOR REPORTING STRESSES: KSF

ADDITIONAL:
CANISTER TRANSFER BUILDING IS QA CATEGORY I (IMPORTANT TO SAFETY). ALL LABORATORY EQUIPMENT & CALIBRATION & TESTING PROCEDURES SHALL BE CARRIED OUT UNDER A QA PROGRAM THAT MEETS THE APPLICABLE REQUIREMENTS OF 10 CFR 50, APPENDIX B, AND 10 CFR 72.

GTP 3.1-1
Attachment 1

LABORATORY TESTING PROGRAM
SPECIFIC TEST REQUIREMENTS

STONE & WEBSTER
ENGINEERING CORPORATION

								SHEET 2
CLIENT Private Fuel Storage, LLC	J.O. NUMBER 05996.02 345H	REVISION	1	2	3	4	PREPARED BY TY Chang	DATE PREPARED 3/6/99
SITE PFSF Skull Valley, UT		DATE					APPROVED BY [signature] 4/8/99	RECEIVED BY AC Smith
		REVISED BY						

NOTE	COL. NO.	TYPE OF TEST	REQUIREMENTS OF TYPE OF TEST AND PROCEDURES TO BE USED (SIZE OF SPECIMENS, RATES OF STRAIN, CONFINING PRESSURES, CYCLES OF RECOMPRESSION ETC.)
A	ALL	ALL TESTS	SELECTION OF NECESSARY AND APPROPRIATE DETERMINATIONS AND/OR TEST PROCEDURES TO BE MADE BY LABORATORY PERSONNEL.
B	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION ONLY; NO INDEX PROPERTIES TO BE DETERMINED.
C	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION TO BE SUPPLEMENTED BY DETERMINATIONS OF SPECIFIC INDEX DETERMINATIONS INDICATED IN SUBSEQUENT COLUMNS.
D	3	ATTERBERG LIMITS	FOR CLASSIFICATION PURPOSES ONLY; ONE-POINT LIQUID LIMIT DETERMINATION ACCEPTABLE.
E	5	GRADATION ANALYSES	SIEVE ANALYSIS ONLY; NO HYDROMETER ANALYSIS REGARDLESS OF PERCENT FINES.
F	9	CU	Consolidation pressure = 1.7 KSF, Shear Undrained
G	15	CYCLIC TRIAXIAL	Consolidation pressure = 2.0 KSF, SR = 0.48 $\Delta\sigma_{dc}$ = 1.92 KSF 500 cycles, or 20% DA, 10% SA strain
H	16	RESONANT COLUMN	Confining pressures σ_c = Overburden pressure to Overburden pressure + 1.7 KSF

GTP 3.1-1
Attachment 2

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED	SHEET
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99	3
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: BJD. 4/8/99	RECEIVED BY: AC Smith	

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15 CYCLIC TRIAXIAL	16 RESONANT COLUMN	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D						F						G			
CTB-N	U	2		X	X	X				X		X		X				X			11. Inundate Sample at 1 TSF after Primary Consolidation.
	U	3		X	X	X			X	X		X						X			replaced CTB-4/U-1

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

LABORATORY TESTING PROGRAM

UTILIZATION OF SAMPLE

STONE & WEBSTER ENGINEERING CORPORATION

SHEET	4

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					T Y Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: PJ.U 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION (B)	2 WATER CONTENT	3 ATTERBERG LIMITS (D)	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL (F)	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15 CYCLIC TRIAXIAL (G)	16	17 Hydrometer	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CJB-5	U	2		X	X	X				X		X									
	U	3		X	X	X			X	X				X						X	11 Inundate Sample at 1 T.S.F after Primary Consolidation.
	U	1		X	X													X			1st Cyclic triaxial SR = 0.48 σ_{o} = 1.92 ksf. σ_{c} = 2 ksf

☐ TEST TO BE PERFORMED ☐ TEST IN PROGRESS ☒ TEST COMPLETED & CHECKED ☐ TEST CANCELLED

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
5

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: P&D. 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION (B)	2 WATER CONTENT	3 ATTERBERG LIMITS (D)	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL (F)	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15 CYCLIC TRIAXIAL	16 RESONANT COLUMN (H)	17 HYDROMETER	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CTB-1	S	1		X	X																
	S	2 A B		X	X	X															
	U	3		X	X	X				X		X							X		
	S	4 A B		X	X	X															
	U	5		X	X					X											
	S	6		X	X			X													
	U	7		X	X	X				X		X							X		
	S	8		X	X																
	S	9		X	X			X													
	S	10		X	X																

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET: 6

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: P.J.V. 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 TYPE OF TEST / CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D															
CTB-2	S	1		X	X																
	S	2 A B		X	X	X															
	S	3		X	X																
	S	4		X	X	X															
	S	5		X	X																
	S	6		X	X	X															
	S	7		X	X																
	S	8		X	X																
	S	9 A B		X	X																
	S	10		X	X			X													

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: Private Fuel Storage, LLC	J.O. NUMBER: 05996.02 345H	REVISION	1	2	3	4	PREPARED BY: T Y Chang		DATE PREPARED: 3/6/99	SHEET: 7
SITE: PFSF Skull Valley, UT		DATE					APPROVED BY: P.J.L.	4/8/99	RECEIVED BY: AC Smith	
		REVISED BY								

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15; 16; 17

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D															
CTB-3	S	1		X	X																
	S	2		X	X	X															
	S	3		X	X																
	S	4																			No Recovery
	S	5		X	X																
	S	6		X	X																
	S	7		X	X																
	S	8		X	X	X															
	S	9		X	X			X													
	S	10		X	X																

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
8

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE	4/22/99				TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY	RJD				APPROVED BY: RJD 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION (B); 2 WATER CONTENT; 3 ATTERBERG LIMITS (D); 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL (F); 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15 CYCLIC TRIAXIAL (G); 16 Resonant Column; 17 HYDROMETER

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 (B)	2	3 (D)	4	5	6	7	8	9 (F)	10	11	12	13	14	15 (G)	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CTB-4	S	2 T/B		X	X																
	S	3		X	X	X															use CTB-N/U-2 after discussion with PJT/TYC 9/15/9
	U	1(4)		X	X	X			X	X								~~clay~~		X	THIRD CYC TRIAX: SR=0.4 & σ_{dc} = 1.6 KSF &
	U	2(5)		X	X	X				X		X		X							σ_c = 2 KSF IF DA STRAIN EXCEEDS 5% AT 15 CYC
	S	6		X	X	X															IN PREVIOUS TESTS
	U	7		X	X					X										X	
	S	8 T/B		X	X			X													
	U	9		X	X	~~/~~			X	X				~~/~~							no suitable sample for consolidation, nonplast. (1)
	S	10		X	X	X															
	U	11		X	X	X				X		X								X	
	S	12		X	X	X															
	U	13		X	X	X			X	X								X			

TEST TO BE PERFORMED   TEST IN PROGRESS   TEST COMPLETED & CHECKED   TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
9

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: RZ.L. 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15 CYCLIC TRIAXIAL; 16 RESONANT COLUMN; 17 HYDROMETER

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D															
CTB-4	S	14		X	X	X															
	U	15		X	X					X											
	S	16		X	X			X													
	S	17		X	X			X													

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
10

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE	4/22/99				T Y. Chang	3/6/99
SITE PFSF Skull Valley, UT		REVISED BY	P.J.D.				APPROVED BY P.J.D. 4/8/99	RECEIVED BY A C Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15 CYCLIC TRIAXIAL; 16 RESONANT COLUMN; 17 HYDROMETER

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D						F						G			
CTB-5	S	2		X	X																
	S	3		X	X	X															
	S	4 A/B		X	X																
	S	5 A/B		X	X	X															
	U	6		X	X	≠	nonplastic			X				≠				sand ≠			2nd triaxial SR = 0.29 τ_{dc} = 1.16 ksf, τ_{dc} = 2 ksf IF DA STRAIN EXCEEDS 5% AT 15 CYCLES IN PREVIOUS TEST. ①
	S	7		X	X			X													
	U	8		X	X	≠				X		≠	nonplastic silty sand								
	S	9		X	X			X													
	U	10		X	X					X		X									
	S	11		X	X	X															
	U	12		X	X	X				X				X				X			
	S	13 A/B		X	X	X															

TEST TO BE PERFORMED / TEST IN PROGRESS / TEST COMPLETED & CHECKED / TEST CANCELLED

GTP 3.1-1
Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET: 11

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: B.F.D. 4/8/99	RECEIVED BY: A C Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15; 16; 17

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D						F									
CB-5	U	14		X	X	X				X		X		X							
	S	15A, 15B		X	X																
	S	16A, 16B		X	X																
	S	17		X	X			X													

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

UTILIZATION OF SAMPLE

ENGINEERING CORPORATION

SHEET	12
CLIENT	Private Fuel Storage, LLC
J.O. NUMBER	05996.02 345H
SITE	PFSF Skull Valley, UT
PREPARED BY	TY Chang
DATE PREPARED	3/6/99
APPROVED BY	PJT. 4/8/99
RECEIVED BY	AC Smith

REVISION	1	2	3	4
DATE				
REVISED BY				

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION; 2 WATER CONTENT; 3 ATTERBERG LIMITS; 4 PERCENT FINES; 5 GRADATION ANALYSES; 6 SPECIFIC GRAVITY; 7 UNIT WEIGHT; 8 UNDRAINED COMPRESSION; 9 CONSOLIDATED TRIAXIAL; 10 DRAINED DIRECT SHEAR; 11 CONSOLIDATION; 12 COMPACTION; 13 RELATIVE DENSITY; 14 PERMEABILITY; 15; 16; 17

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 (B)	2	3 (D)	4	5	6	7	8	9 (F)	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CTB-6	S	1		X	X																
	S	2		X	X	X															
	U	3		X	X					X		X									
	S	4 A/B		X	X	X															
	S	5 A/B		X	X																
	S	6		X	X																
	S	7 A/B		X	X	X															
	S	8 A/B		X	X																

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1 Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: Private Fuel Storage, LLC	J.O. NUMBER: 05996.02 345H	PREPARED BY: TY Chang	DATE PREPARED: 3/6/99
SITE: PFSF Skull Valley, UT		APPROVED BY: P.J.T. 4/8/99	RECEIVED BY: AC Smith
SHEET: 13			

REVISION	1	2	3	4
DATE				
REVISED BY				

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 Classification; 2 Water Content; 3 Atterberg Limits; 4 Percent Fines; 5 Gradation Analyses; 6 Specific Gravity; 7 Unit Weight; 8 Undrained Compression; 9 Consolidated Triaxial; 10 Drained Direct Shear; 11 Consolidation; 12 Compaction; 13 Relative Density; 14 Permeability; 15; 16; 17

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D															
GTB-7	S	1		X	X																
	S	2		X	X	X															
	U	3		X	X			X	X	X											
	S	4 A/B		X	\																
	S	5 A/B		X	X																
	S	6		X	X	X															
	S	7		X	X																
	S	8		X	X																

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

LABORATORY TESTING PRO...

UTILIZATION OF SAMPLE

ENGINEERING CORPORATION

SHEET 14

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: BJD 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: 1 CLASSIFICATION, 2 WATER CONTENT, 3 ATTERBERG LIMITS, 4 PERCENT FINES, 5 GRADATION ANALYSES, 6 SPECIFIC GRAVITY, 7 UNIT WEIGHT, 8 UNDRAINED COMPRESSION, 9 CONSOLIDATED TRIAXIAL, 10 DRAINED DIRECT SHEAR, 11 CONSOLIDATION, 12 COMPACTION, 13 RELATIVE DENSITY, 14 PERMEABILITY

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
				B		D															
CTB-8	S	1		X	X																
	S	2		X	X	X															
	S	3		X	X																
	S	4		X	X			X													
	S	5		X	X																
	S	6		X	X			X													
	S	7		X	X																
	S	8		X	X	X															
	S	9		X	X			X													

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

GTP 3.1-1 Attachment 3

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 15

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 345H	DATE					TY Chang	3/6/99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: PJD 4/8/99	RECEIVED BY: AC Smith

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15 CYCLIC TRIAXIAL	16 RESONANT COLUMN	17 HYDROMETER	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
Total Number of Tests				92	92	38	-	14	6	21	-	~~11~~	-	~~6~~	-	-	-	~~6~~	2	4	
												10		5				5			

TEST TO BE PERFORMED

TEST IN PROGRESS

TEST COMPLETED & CHECKED

TEST CANCELLED

GTP 3.1-1
Attachment 3

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

NOTED MAR 1999

5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 16
05996.02	G(B)		345H	

OBJECTIVE: DETERMINE CONSOLIDATION PRESSURES & CYCLIC STRESSES TO APPLY IN PERFORMING THE CYCLIC TRIAXIAL TESTS.

PURPOSE OF TESTS IS TO DEVELOP EXPERIMENTAL DATA FOR RESPONDING TO NRC RAI No. 1, SAR QUESTION 2-8, RE POTENTIAL FOR SETTLEMENT DUE TO DYNAMIC COMPACTION OF FOUNDATION SOIL CONSIDERING THE HIGH IN SITU VOID RATIO OF ABOUT 2. THIS HIGH VOID RATIO IS APPLICABLE FOR THE CLAYEY SILT. THE PLAN IS TO PERFORM CYCLIC TRIAXIAL TESTS ON SAMPLES OF THE CLAYEY SILT & MEASURE THE VERTICAL STRAIN AS A FUNCTION OF NUMBER OF CYCLES OF LOADING. SINCE COLLAPSE OF THESE SOILS WOULD BE SIGNIFICANT ONLY FOR THOSE SOILS UNDERNEATH THE FOUNDATIONS, PERFORM TESTS AT CONSOLIDATION PRESSURES THAT EMULATE CONDITIONS UNDER THE CANISTER TRANSFER BUILDING AND UNDER THE STORAGE PADS.

NOTE: CALC 05996.01-G(B)-3, REV 2, FIG 1 INDICATES

q AT BASE OF STORAGE PAD = 1.935 KSF

NOTED MAR 9 1999

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

A 5010.65

2 CALCULATION IDENTIFICATION NUMBER				PAGE 17
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO.	OPTIONAL TASK CODE 345H	

CALC 05996.01-SC-5 INDICATES F_{VS} = 75000 K (VERT STATIC LOAD) FOR CANISTER TRANSFER BUILDING, SUPPORTED BY MAT ~ 147' x 275'.

$$\therefore \; q_{STATIC} \simeq \frac{75{,}000 \text{ K}}{147' \times 275'} = 1.86 \text{ KSF}$$

NOTE CANISTER TRANSFER BUILDING IS FOUNDED AT $Z = 5'$ & STORAGE PADS AT $Z = 3'$.

∴ THESE LOADINGS ARE VERY SIMILAR.

TRIAXIAL TEST PERFORMED ON CTB-S U-1 INDICATED $e = 2.78$. THERE MAY BE SUFFICIENT SAMPLE LEFT FOR 1' TRIAXIAL SPECIMEN. SINCE THIS TUBE HAD HIGH VOID RATIO SILT, TRY PERFORMING CYCLIC TRIAXIAL TEST ON THIS SPECIMEN.

$Z \simeq 6.5'$ FOR THIS SPECIMEN & $\gamma_m = 78$ PCF.

CONSOLIDATE SPECIMEN TO EMULATE LOADING UNDER CANISTER TRANSFER BUILDING; i.e.

$\Delta q_{STATIC} = 1.86$ KSF

$+ \; \sigma_V = (6.5' - 5')(0.08 \text{ KCF}) = 0.12$

$\Rightarrow \; \sigma_c \simeq 2$ KSF

NOTED MAR 9 1999

▲ 5010.65

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

3 CALCULATION IDENTIFICATION NUMBER				PAGE 18
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO.	OPTIONAL TASK CODE 345 H	

EQ 1

$$\tau_{AVG} = 0.65 \cdot a_{max} \cdot \frac{\sigma_v}{g} \cdot r_d$$

SEED, IDRISS, & ARANGO (1983)

WHERE: $a_{MAX} = 0.67g$ FOR PFSF DESIGN EARTHQUAKE

$\sigma_v = \Delta g + \Delta\sigma_v \doteq 2$ KSF

r_d VARIES FROM 1.0 AT $Z=0$ TO 0.9 AT $Z=30'$.

ASSUME $r_d = 1$.

FIG 4 OF SEED & IDRISS (1971)

$$\therefore \tau_{AVG} = 0.65 \times \frac{0.67g}{g} \times 2 \text{ KSF} \cdot 1.0 = 0.87 \text{ KSF}$$

SEED (IN "LIQUEFACTION PROBLEMS IN GEOTECHNICAL ENGINEERING", ASCE ANNUAL CONVENTION, PHILADELPHIA, PA, 1976) INDICATES

$$\left(\frac{\tau_h}{\sigma_v'}\right)_{field} = 0.9\left(\frac{\tau_h}{\sigma_v'}\right)_{l-SS} = c_r\left(\frac{\sigma_{dc}}{2\sigma_{3c}'}\right)_{l\text{-TRIAXIAL}}$$

HERE, $\tau_h = \tau_{AVG}$ CALC'D ABOVE.

GROUNDWATER IS ~ 125' DEEP (BORING CTB-5(OW)); $\therefore \sigma_{3c}' = \sigma_{3c}$

ASSUME $K_o = 1.0$ for OCR ≥ 4.0 P.I ≈ 20% material at site

$c_r \sim 0.9$ FOR $K_o = 1.0$ (SEED, 1976)

$$\therefore \left(\frac{\sigma_{dc}}{2\sigma_{3c}'}\right)_{l\text{-triaxial}} = \left(\frac{\tau_{AVG}}{\sigma_v'}\right)_{field}\frac{1}{c_r} = \frac{1}{0.9}\left(\frac{0.87 \text{ KSF}}{2 \text{ KSF}}\right) = 0.48$$

for $\sigma_{3c}' = 2$ KSF $\therefore \sigma_{dc} = 2 \times 2 \text{ KSF} \times 0.48 = 1.92$ KSF

NOTED MAR 1 0 1999

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 19
05996.02	G(B)		345H	

COMPARE THIS VALUE OF σ_{dc} WITH $\Delta\sigma_v$ DUE TO EARTHQUAKE. FOR CANISTER TRANSFER BUILDING, CALC 05996.02-SC-5 INDICATES VERTICAL LOADING DUE TO DESIGN EARTHQUAKE,

F_{VD}, = 37,282 K.

$$\therefore \Delta\sigma_{V_{EQ}} = \frac{37{,}282\ K}{147' \times 275'} = 0.92\ KSF$$

$\therefore$ σ_{dc} > $\Delta\sigma_v$ DUE TO EARTHQUAKE.

$\therefore$ HORIZ SHEAR COMPONENT CONTROLS AND σ_{dc} DETERMINED ABOVE WILL PRODUCE LARGER SHEAR STRESSES THAN THOSE APPLICABLE FOR THE VERTICAL COMPONENT OF THE EARTHQUAKE.

NOTED MAR 10 1999 P. L.

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 20
05996.02	G(B)		345H	

CYC TRIAX

RECOMMEND RUNNING 1ST CYCLIC TRIAXIAL TEST WITH σ_c = 2 KSF TO EMULATE STATIC LOADING (~ 2 KSF FOR CANISTER TRANSFER BUILDING ON MAT & FOR FULLY LOADED STORAGE PADS) AND TEST SPECIMEN FROM CTB-S, U-1, AT Z ~ 6.5'; & APPLY CYCLIC VERTICAL LOAD TO OBTAIN SR = 0.48.

$\therefore$ σ_{dc} = 1.92 KSF. USE THIS SPECIMEN BECAUSE IT IS LIKELY TO BE SILT WITH A HIGH VOID RATIO, BASED ON RESULTS OF TESTING CTB-S U-1B (e = 2.78)

NOTE: BASED ON GEOMATRIX (1997) (APPENDIX 2D OF SAR) SECT 4.2.3 INDICATES "... THE STANSBURY FAULT IS THE CONTROLLING GROUND MOTION SOURCE." AND IN SECT 4.1.3, INDICATES: "THE MAXIMUM MAGNITUDE DISTRIBUTION FOR THE STANSBURY FAULT RANGES FROM MOMENT MAGNITUDE M 6.5 TO 7.5, WITH A MEAN VALUE OF 7.0." TABLE 4-2 "REPRESENTATIVE NUMBER OF CYCLES AND CORRESPONDING CORRECTION FACTORS" OF HOUSNER ET AL (1985) INDICATES

EARTHQUAKE MAGNITUDE M	NO. OF REPRESENTATIVE CYCLES AT 0.65 τ_{MAX}
7.5	15
6.75	10

NOTED MAR 1 0 1999 P. J. Trikian

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

6 CALCULATION IDENTIFICATION NUMBER				PAGE 21
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO.	OPTIONAL TASK CODE 345 H	

∴ FOR PFSF DESIGN EARTHQUAKE WITH M = 7.0, THE NUMBER OF CYCLES IS EXPECTED TO BE BETWEEN 10 & 15.

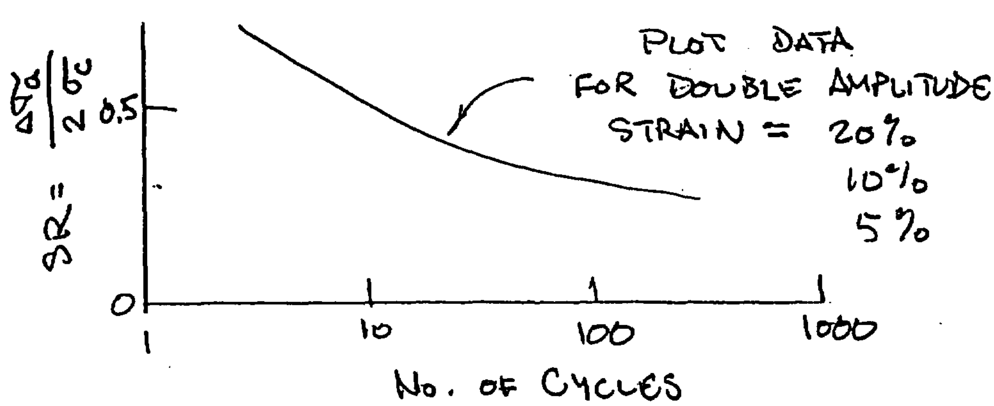
RUN 1ST TEST AT SR = 0.48. AS INDICATED IN NUREG-0031, SECT 4.6, CYCLE LOAD UNTIL EITHER:

1) CYCLIC DOUBLE AMPLITUDE STRAIN ≥ 20%
2) SINGLE AMPLITUDE STRAIN ≥ 10% (EXTENSION OR COMPRESSION)
3) 500 LOAD CYCLES
4) LOAD WAVE FORM DETERIORATES BEYOND ACCEPTABLE VALUES

RUN 2nd TEST ON U-6 OF CTB-5, z ~ 11', WHICH IS VERY CLOSE TO CTB-S, SAMPLE U-6, & WHICH RECOVERED 19" OF CLAYEY SILT AT TOP OF TUBE & SILTY SAND AT BOTTOM OF TUBE. USE SR = 0.29*, WHICH IS APPLICABLE FOR a_{max} = 0.4g, AS SHOWN ON NEXT PAGE, TO OBTAIN DATA FOR ESTABLISHING CURVE. HOPEFULLY WE CAN GET TEST #3 FROM THIS SAME TUBE & TEST IT AT SR = 0.4*.

---

* RUN AT THESE REDUCED SR VALUES ONLY IF DOUBLE AMPLITUDE STRAIN IN PREVIOUS TEST EXCEEDS 5% AT 15 CYCLES



STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

7 CALCULATION IDENTIFICATION NUMBER				PAGE 22
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO.	OPTIONAL TASK CODE 345H	

For $a_{max} = 0.4g$

$$\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = 0.65 \times 0.4 \times r_d$$

Assume $r_d = 1$

$$\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = 0.65 \times 0.4 = 0.26$$

$$\therefore \left(\frac{\sigma_{dc}}{2\sigma_{3c}'}\right)_{\text{l-triaxial}} = \frac{1}{C_r}\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = \frac{1}{0.9}(0.26) = 0.29$$

For $\sigma_{3c}' = 2$ K.S.F $\quad \sigma_{dc} = 2 \times 2 \times 0.29 = 1.16$ K.S.F

For $a_{max} = 0.55g$

$$\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = 0.65 \times 0.55 \times r_d$$

Assume $r_d = 1$

$$\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = 0.65 \times 0.55 = 0.36$$

$$\therefore \left(\frac{\sigma_{dc}}{2\sigma_{3c}'}\right)_{\text{l-triaxial}} = \frac{1}{C_r}\left(\frac{\tau_{avg}}{\sigma_v'}\right)_{\text{l-field}} = \frac{1}{0.9}(0.36) = 0.40$$

For $\sigma_{3c}' = 2$ K.S.F $\quad \sigma_{dc} = 2 \times 2 \times 0.4 = 1.60$ K.S.F

TABLE 1
Laboratory Test Results - 1998 CTB Borings

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits			USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test					CU Triaxial Test		
					LL	PL	PI							σ_{mpp} (ksf)	CR	RR	C_c	C_r	σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-1	S-1	1.0	4471.4	25.3				ML													
CTB-1	S-2 (top)	5.1	4467.3	30.1	40.1	22.3	17.8	CL													
CTB-1	S-2 (bot)	6.1	4466.3	65.6				MH													
CTB-1	U-3C	8.1	4464.3	50.6	56.0	28.9	27.1	CH			86.4	57.4	1.96								
CTB-1	U-3D	8.7	4463.7	47.9				CH			91.9	62.1	1.73						1.7	2.84	5.0
CTB-1	U-3E	9.1	4463.3	48.8				CH													
CTB-1	S-4 (top)	9.5	4462.9	37.4	41.2	23.2	18.0	CL													
CTB-1	S-4 (bot)	10.5	4461.9	27.3				SM													
CTB-1	U-5D	11.6	4460.8	31.6				SC			111.2	84.5	0.987								
CTB-1	U-5E	11.8	4460.6	28.6				SC													
CTB-1	S-6	16.0	4456.4	10.7				ML	56.8												
CTB-1	U-7C	21.1	4451.3	51.9	56.5	42.4	14.1	MH			83.8	55.2	2.08								
CTB-1	U-7D	21.7	4450.7	45.1				MH			91.2	62.9	1.70						1.7	2.73	5.0
CTB-1	U-7E	22.1	4450.3	43.0				MH													
CTB-1	S-8	26.0	4446.4	20.9				ML													
CTB-1	S-9	31.0	4441.4	4.0				SM	14.6												
CTB-1	S-10	35.7	4436.7	1.5				SM													
CTB-2	S-1	1.0	4473.0	24.6				ML													
CTB-2	S-2 (top)	5.3	4468.7	28.7				ML													
CTB-2	S-2 (bot)	6.3	4467.7	29.4	40.8	21.1	19.7	CL													
CTB-2	S-3	8.0	4466.0	60.1				MH													
CTB-2	S-4	10.0	4464.0	45.8	56.2	29.9	26.3	MH													
CTB-2	S-5	12.0	4462.0	26.0				ML													
CTB-2	S-6	16.0	4458.0	27.8	34.3	21.9	12.4	CL													
CTB-2	S-7	21.0	4453.0	28.6				ML													
CTB-2	S-8	26.0	4448.0	30.0				ML													

TABLE 1

Laboratory Test Results - 1998 CTB Borings

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits			USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test					CU Triaxial Test		
					LL	PL	PI							σ_{mpp} (ksf)	CR	RR	C_c	C_r	σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-2	S-9 (top)	30.1	4443.9	26.8				ML													
CTB-2	S-9 (bot)	31.1	4442.9	2.6				SP													
CTB-2	S-10	35.7	4438.3	1.7				SP	7.9												
CTB-3	S-1	1.0	4471.9	18.7				ML													
CTB-3	S-2	6.0	4466.9	55.2	58.7	32.3	26.4	MH													
CTB-3	S-3	8.0	4464.9	53.7				MH													
CTB-3	S-5	12.0	4460.9	39.5				ML													
CTB-3	S-6 (top)	15.4	4457.5	14.6				SM													
CTB-3	S-6 (bot)	16.4	4456.5	24.0				ML													
CTB-3	S-7 (bot)	21.2	4451.7	53.1				MH													
CTB-3	S-8	26.0	4446.9	28.3	32.0	22.1	9.9	CL													
CTB-3	S-9	31.0	4441.9	2.8				SP	8.4												
CTB-3	S-10	35.4	4437.5	1.4				SP													
CTB-4	S-2 (top)	2.2	4472.8	22.6				ML													
CTB-4	S-2 (bot)	3.2	4471.8	41.1				ML													
CTB-4	S-3	5.0	4470.0	27.9	39.9	22.4	17.5	CL													
CTB-4	U-1A	6.0	4469.0	28.9				CL													
CTB-4	U-1C	7.0	4468.0	34.5				CL	97.6		95.7	71.2	1.38								
CTB-4	U-1D	7.5	4467.5	60.3	67.9	39.3	28.6	MH		2.73	74.9	46.7	2.65								
CTB-4	U-1E	7.9	4467.1	64.2				MH													
CTB-4	U-2D	9.5	4465.5	45.2				CH			87.7	60.4	1.81						1.7	3.11	6.0
CTB-4	U-2E	9.9	4465.1	48.9	58.1	28.6	29.5	CH			94.1	63.2	1.69	12.6	0.35	0.02	0.93	0.05			
CTB-4	U-2F	10.1	4464.9	53.0				CH													
CTB-4	S-6	11.0	4464.0	28.5	34.3	24.8	9.5	ML													
CTB-4	U-7D	13.0	4462.0	22.6				ML	69.2		101.3	82.7	1.03								
CTB-4	U-7E	13.2	4461.8	10.2				SP													

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

Stone & Webster Engineering Corporation

JO: 05996.02
June 1999

TABLE 1

Laboratory Test Results - 1998 CTB Borings

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits			USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test					CU Triaxial Test		
					LL	PL	PI							σ_{mpp} (ksf)	CR	RR	C_c	C_r	σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-4	S-8 (top)	14.3	4460.7	20.4				ML													
CTB-4	S-8 (bot)	15.4	4459.6	5.4				SM	37.5												
CTB-4	U-9A	16.0	4459.0	4.6				ML													
CTB-4	U-9D	16.7	4458.3	4.5				SM		2.69											
CTB-4	U-9E	16.9	4458.1	5.2				SM	16.7		98.4	93.5	0.796								
CTB-4	U-9F	17.1	4457.9	9.7				SM	34.2		101.0	92.1	0.823								
CTB-4	U-9H	17.5	4457.5	6.6				SM													
CTB-4	S-10	19.0	4456.0	32.7	41.4	24.1	17.3	CL													
CTB-4	U-11D	21.2	4453.8	31.5	37.2	33.5	3.7	ML	97.2		89.8	68.4	1.48						1.7	3.15	8.0
CTB-4	U-11E	21.6	4453.4	25.0				ML													
CTB-4	S-12	23.0	4452.0	52.0	57.8	48.1	9.7	MH													
CTB-4	U-13D	25.2	4449.8	37.4	43.2	26.7	16.5	ML		2.72	101.4	73.8	1.30								
CTB-4	U-13E	25.5	4449.5	40.3				ML													
CTB-4	S-14	27.0	4448.0	14.8	28.3	18.5	9.8	CL													
CTB-4	U-15C	28.0	4447.0	18.3				ML			115.5	97.6	0.721								
CTB-4	U-15D	29.2	4445.8	14.4				ML													
CTB-4	U-15E	29.3	4445.7	7.7				SM													
CTB-4	S-16	31.0	4444.0	3.9				SM	32.4												
CTB-4	S-17	35.3	4439.7	2.1				SP	7.6												
CTB-5	S-2	3.0	4471.8	32.7				ML													
CTB-5	S-3	5.0	4469.8	72.6	75.3	43.5	31.8	MH													
CTB-5	S-4 (bot)	7.2	4467.6	51.2				CH													
CTB-5	S-5	9.0	4465.8	48.8	51.5	27.3	24.2	CH													
CTB-5	U-6A	10.0	4464.8	31.7				ML													
CTB-5	U-6C	10.8	4464.0	12.7				ML			101.8	90.3	0.860								
CTB-5	U-6D	11.1	4463.7	18.6				ML			111.3	93.8	0.790								

TABLE 1
Laboratory Test Results - 1998 CTB Borings

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits LL	PL	PI	USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test σ_{mpp} (ksf)	CR	RR	C_c	C_r	CU Triaxial Test σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-5	U-6E	11.3	4463.5	20.0				ML	79.8		118.0	98.3	0.708								
CTB-5	U-6F	11.5	4463.3	16.4				ML													
CTB-5	S-7	13.0	4461.8	4.1				SM	21.6												
CTB-5	U-8A	14.0	4460.8	3.7				SM													
CTB-5	U-8D	15.4	4459.4	3.4				SM			105.8	102.4	0.640								
CTB-5	U-8E	15.6	4459.2	6.5				SM													
CTB-5	S-9	17.0	4457.8	12.2				ML	63.3												
CTB-5	U-10D	19.4	4455.4	27.7				ML			94.5	74.0	1.29						1.7	2.93	8.0
CTB-5	U-10E	19.8	4455.0	33.3				ML													
CTB-5	S-11	21.0	4453.8	47.6	51.5	47.2	4.3	MH													
CTB-5	U-12B	23.2	4451.6	42.3				ML			93.6	65.8	1.58								
CTB-5	U-12C	23.6	4451.2	52.4	51.5	32.8	18.7	MH			96.4	63.3	1.68	12.3	0.33	0.014	0.89	0.04			
CTB-5	U-12D	23.9	4450.9	45.1				MH			93.7	64.6	1.63								
CTB-5	U-12E	24.1	4450.7	50.8				MH													
CTB-5	S-13	25.0	4449.8	33.6	39.8	24.2	15.6	CL													
CTB-5	U-14D	27.0	4447.8	30.5				CL			113.9	87.2	0.947						1.7	1.66	12.0
CTB-5	U-14E	27.4	4447.4	26.2	30.0	19.5	10.5	CL			114.7	90.9	0.868	25.5	0.13	0.014	0.25	0.03			
CTB-5	U-14F	27.6	4447.2	27.1				CL													
CTB-5	S-15 (top)	28.2	4446.6	17.6				CL													
CTB-5	S-15 (bot)	29.2	4445.6	9.0				ML													
CTB-5	S-16 (top)	30.2	4444.6	3.3				SP													
CTB-5	S-16 (mid	30.7	4444.1	26.5				CL													
CTB-5	S-16 (bot)	31.4	4443.4	3.3				SP													
CTB-5	S-17	35.6	4439.2	1.5				SP	5.6												
CTB-6	S-1	1.0	4475.2	20.3				ML													
CTB-6	S-2	6.0	4470.2	31.0	42.9	21.5	21.4	CL													

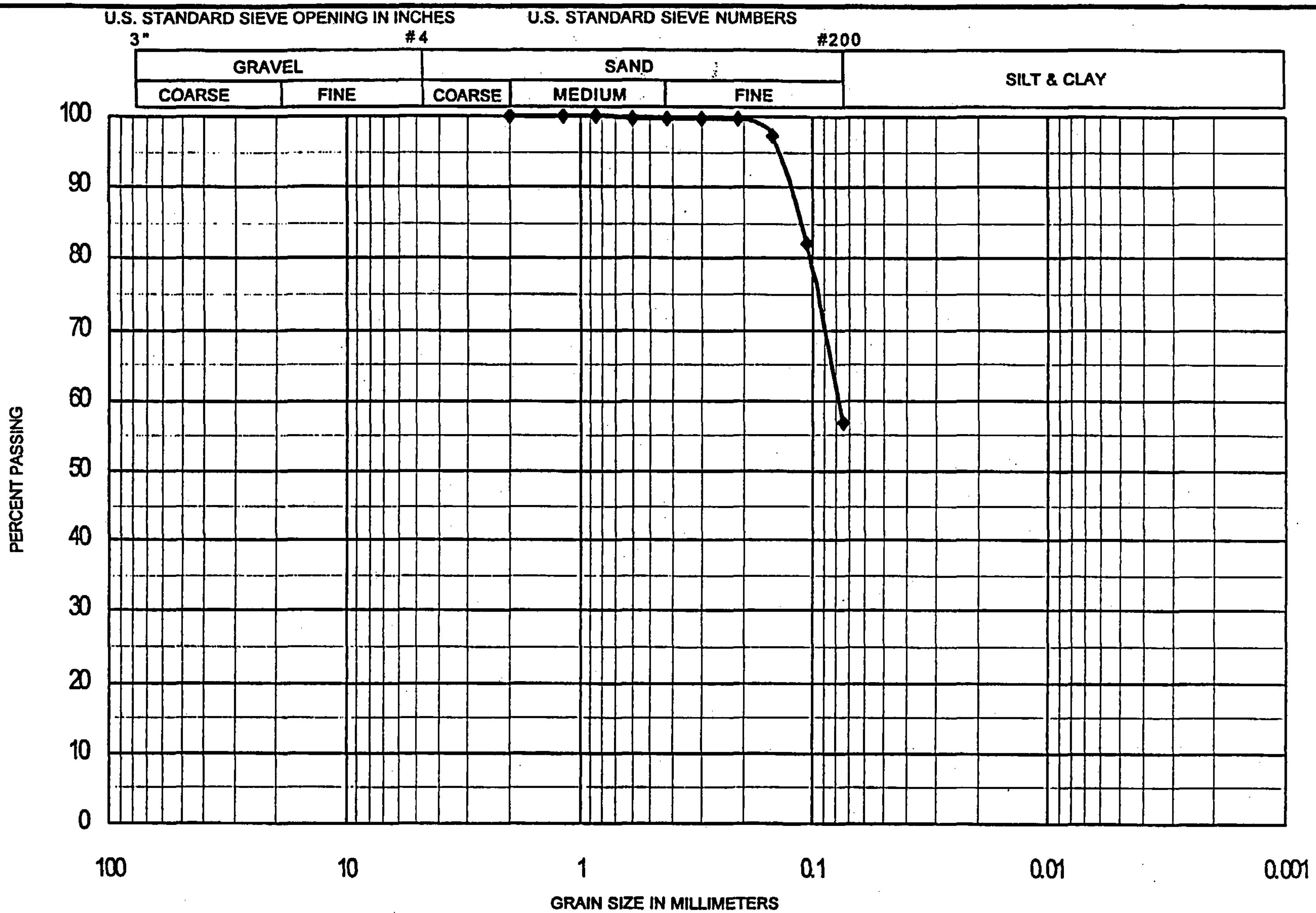
TABLE 1

Laboratory Test Results - 1998 CTB Borings

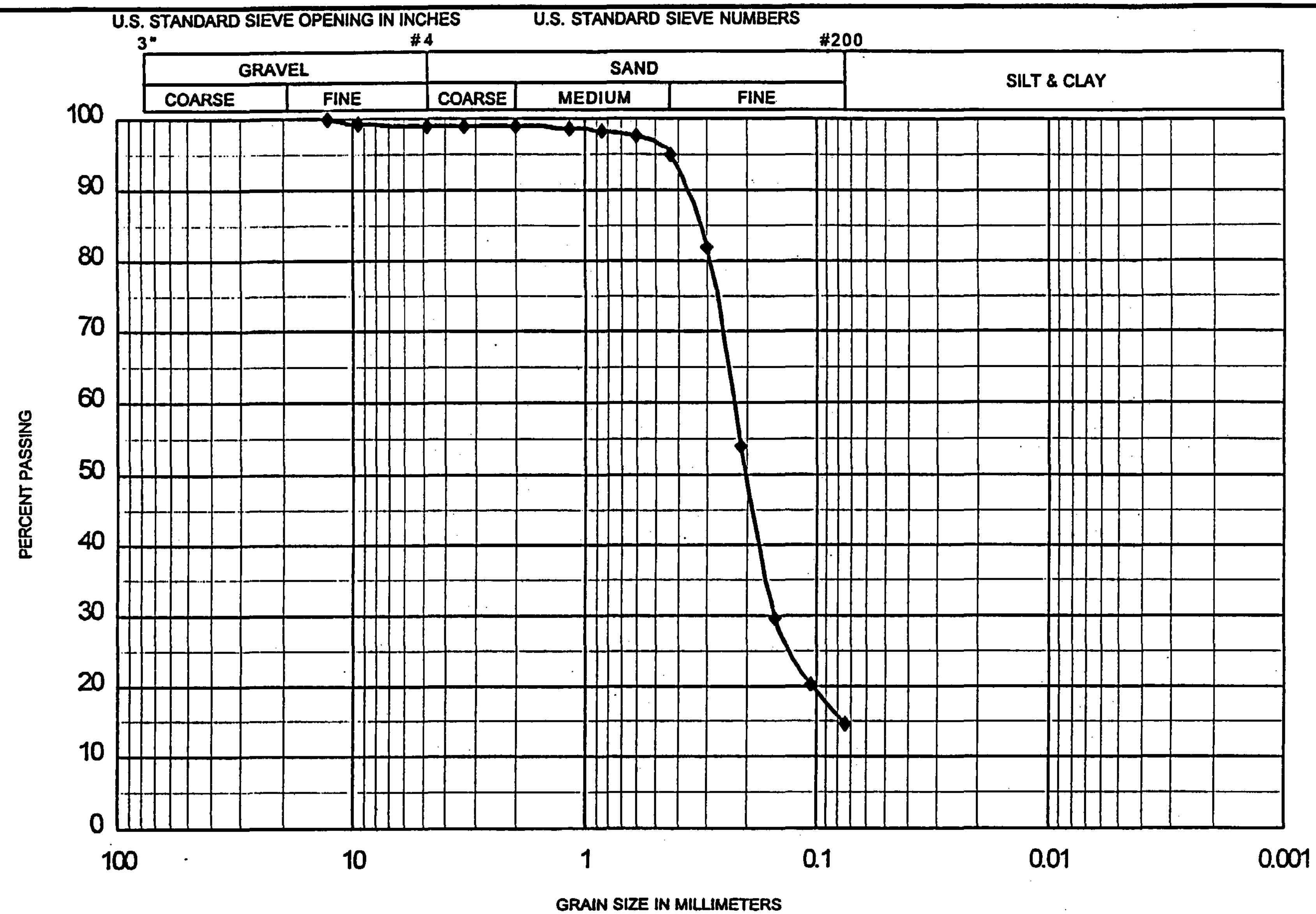
Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits LL	PL	PI	USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test σ_{mpp} (ksf)	CR	RR	C_c	C_r	CU Triaxial Test σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-6	U-3D	8.3	4467.9	52.7				CH			85.7	56.2	2.02						1.7	2.70	7.0
CTB-6	U-3E	8.7	4467.5	55.5				CH													
CTB-6	S-4 (top)	10.5	4465.7	52.9	56.9	27.9	29.0	CH													
CTB-6	S-4 (bot)	11.5	4464.7	42.1				ML													
CTB-6	S-5 (top)	15.2	4461.0	10.2				ML													
CTB-6	S-5 (bot)	16.2	4460.0	5.6				SM													
CTB-6	S-6	21.0	4455.2	30.7				ML													
CTB-6	S-7	26.0	4450.2	37.8	41.5	33.9	7.6	ML													
CTB-6	S-8 (top)	30.2	4446.0	6.5				ML													
CTB-6	S-8 (bot)	31.2	4445.0	3.1				SP													
CTB-7	S-1	1.0	4472.1	21.1				CL													
CTB-7	S-2	6.0	4467.1	52.8	58.1	29.9	28.2	CH													
CTB-7	U-3D	8.3	4464.8	2.7				SP	8.7	2.69	102.3	99.6	0.686								
CTB-7	U-3E	8.5	4464.6	2.6				SP													
CTB-7	S-4	11.0	4462.1	6.4				SM													
CTB-7	S-5 (top)	15.2	4457.9	7.4				ML													
CTB-7	S-5 (bot)	16.2	4456.9	33.6				ML													
CTB-7	S-6	21.0	4452.1	46.9	51.6	33.5	18.1	MH													
CTB-7	S-7	26.0	4447.1	20.9				ML													
CTB-7	S-8	31.0	4442.1	2.0				SP													
CTB-8	S-1 (bot)	1.1	4472.8	31.8				CL													
CTB-8	S-2	6.0	4467.9	53.3	55.3	28.5	26.8	CH													
CTB-8	S-3	8.0	4465.9	24.1				ML													
CTB-8	S-4	10.0	4463.9	3.6				SM	14.8												
CTB-8	S-5	12.0	4461.9	3.0				SM													
CTB-8	S-6	16.0	4457.9	5.5				SM	34.8												

TABLE 1
Laboratory Test Results - 1998 CTB Borings

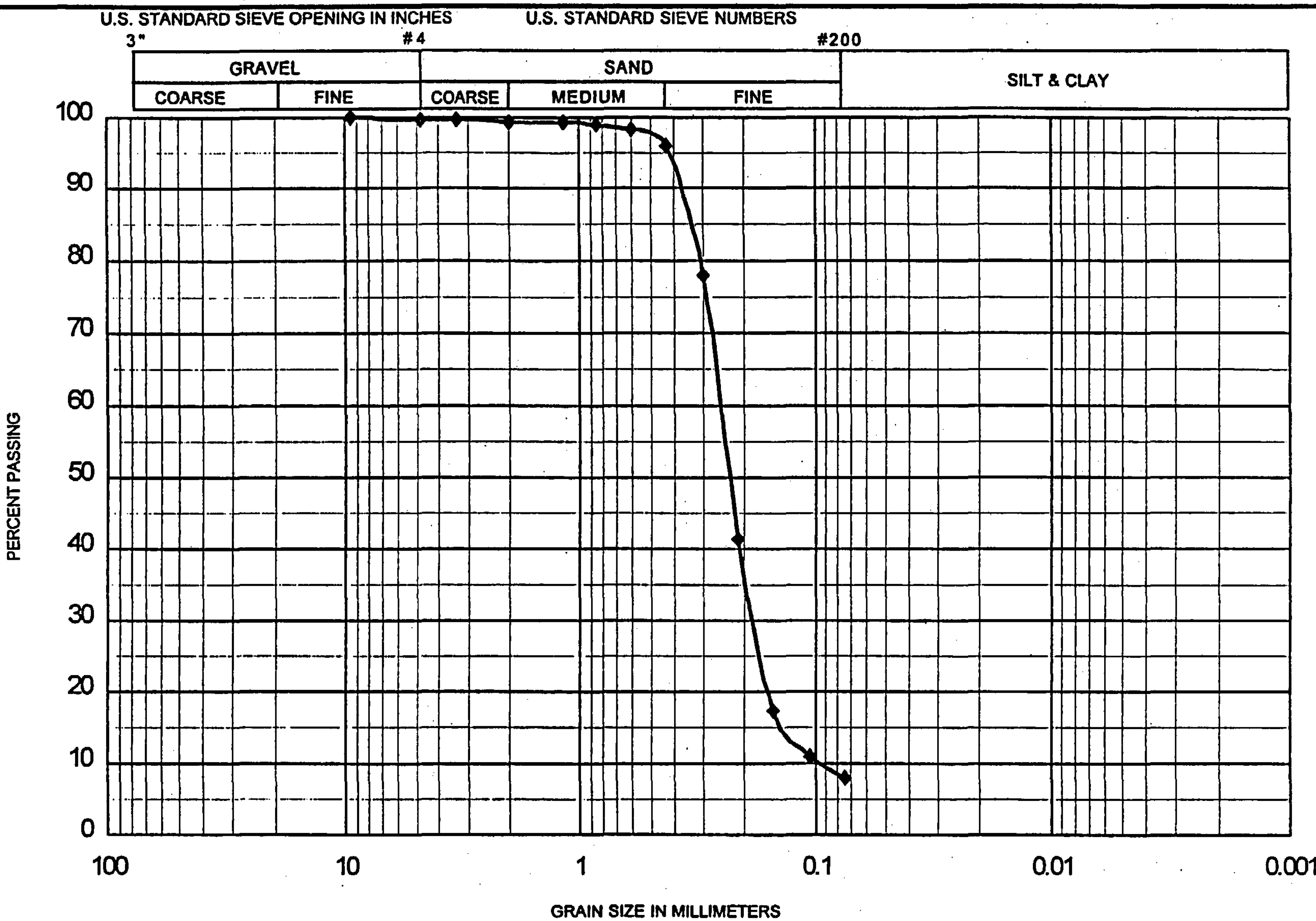
Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits LL	PL	PI	USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test σ_{mpp} (ksf)	CR	RR	C_c	C_r	CU Triaxial Test σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-8	S-7 (bot)	21.1	4452.8	57.0				MH													
CTB-8	S-8	26.0	4447.9	26.7	30.5	18.3	12.2	CL													
CTB-8	S-9	30.7	4443.2	2.2				SM	11.3												
CTB-N	U-1A	5.1	4469.0	30.6	38.4	23.1	15.3	CL													
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL			100.6	77.3	1.20						1.7	3.00	8.0
CTB-N	U-1D	6.7	4467.4	46.6	50.8	23.1	27.7	CH													
CTB-N	U-1E	6.9	4467.2	67.7				MH													
CTB-N	U-2A	7.1	4467.0	69.0	74.2	45.4	28.8	MH													
CTB-N	U-2B	7.7	4466.4	65.4				MH			74.6	45.1	2.76						1.7	2.41	13.0
CTB-N	U-2C	8.3	4465.8	52.6				MH			86.3	56.5	2.01								
CTB-N	U-2D	8.7	4465.4	63.0	60.6	36.8	23.8	MH			78.8	48.4	2.51	6.1	0.37	0.02	1.31	0.07			
CTB-N	U-2E	8.8	4465.3	52.1				MH													
CTB-N	U-3A	9.0	4465.1	53.7				CH													
CTB-N	U-3C	9.9	4464.2	47.1				CH			86.1	58.5	1.90								
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH		2.71	86.3	56.7	1.98						1.7	2.73	7.0
CTB-N	U-3E	10.9	4463.2	53.1				CH													
CTB-S	U-1A	5.1	4469.4	85.5				MH													
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	MH			78.0	44.9	2.78						1.7	2.05	12.0
CTB-S	U-1D	6.6	4467.9	60.7				MH			84.8	52.8	2.22								
CTB-S	U-1E	6.9	4467.6	56.4				MH													
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	CH			90.0	58.2	1.92						1.7	2.40	5.0
CTB-S	U-2E	8.8	4465.7	56.7				CH													
CTB-S	U-3C	10.1	4464.4	72.2	66.0	37.8	28.2	MH	99.2	2.72	89.5	51.9	2.27	8.4	0.36	0.02	1.17	0.07			
CTB-S	U-3D	10.4	4464.1	10.0				SM	18.9		84.7	77.0	1.18								
CTB-S	U-3F	10.9	4463.6	31.2				ML													



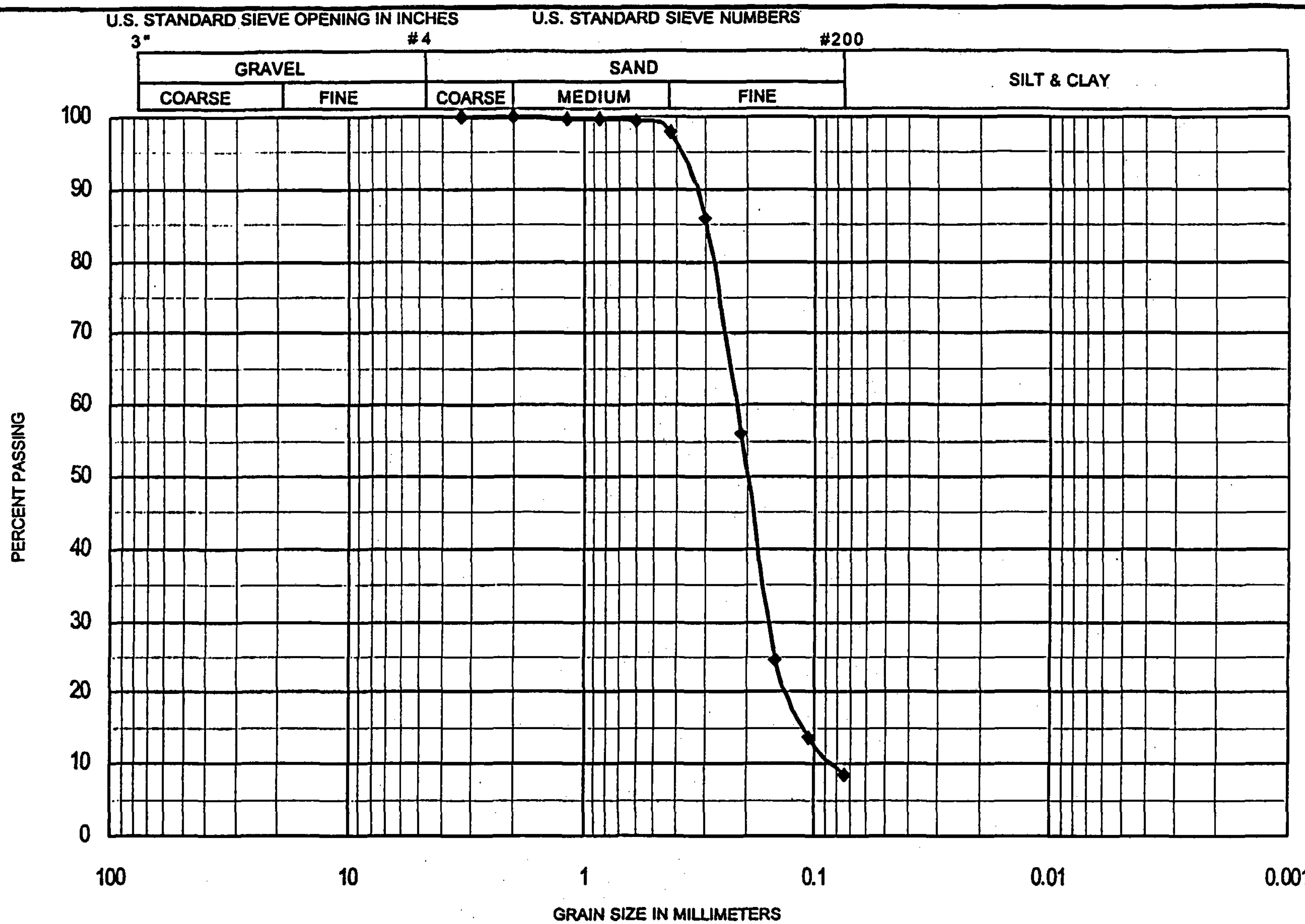
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-1, SAMPLE S-6, DEPTH 15.0-17.0 ft DESCRIPTION: Sandy SILT (ML)	JUNE 1999 Figure 1



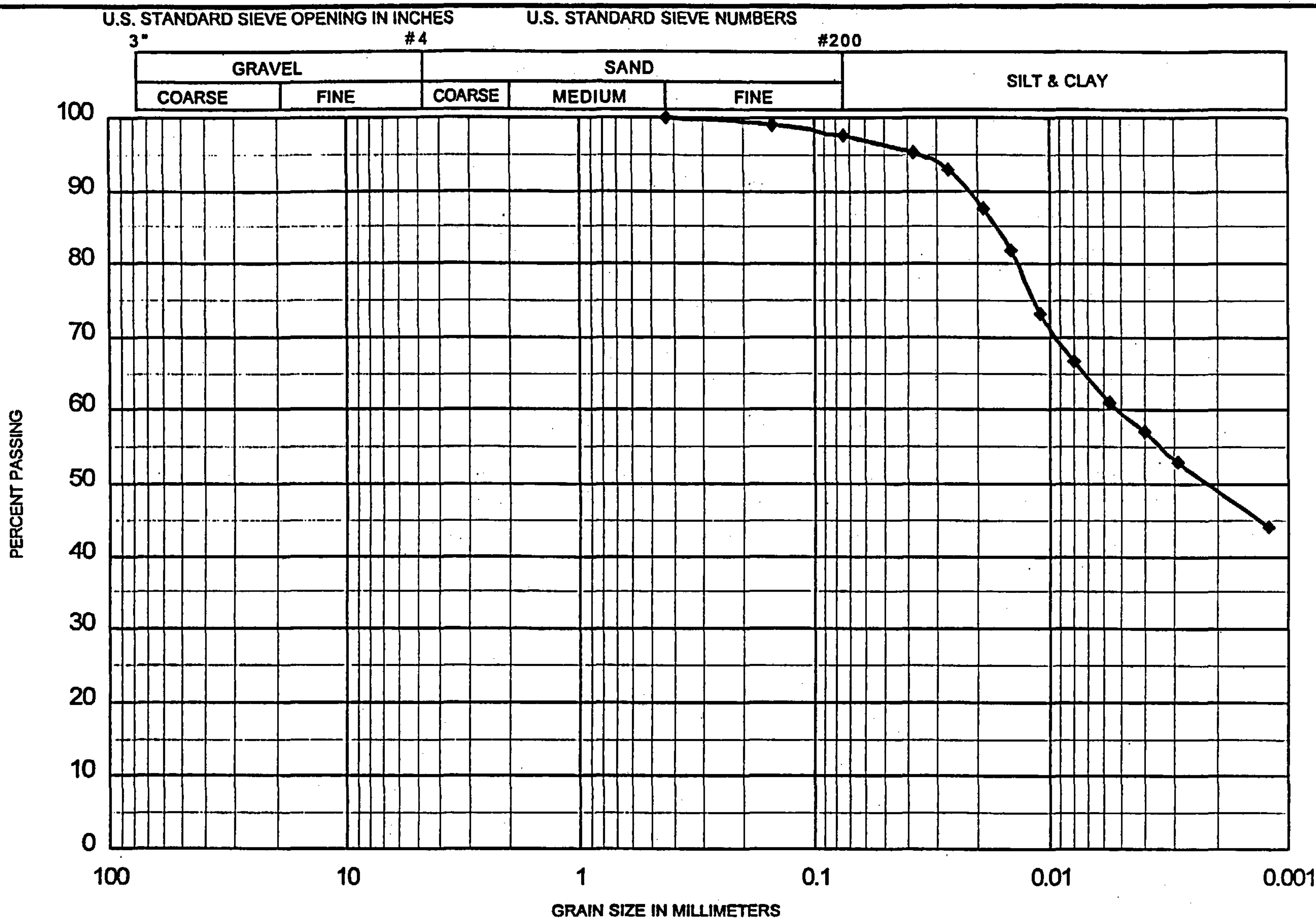
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-1, SAMPLE S-9, DEPTH 30.0-32.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 2



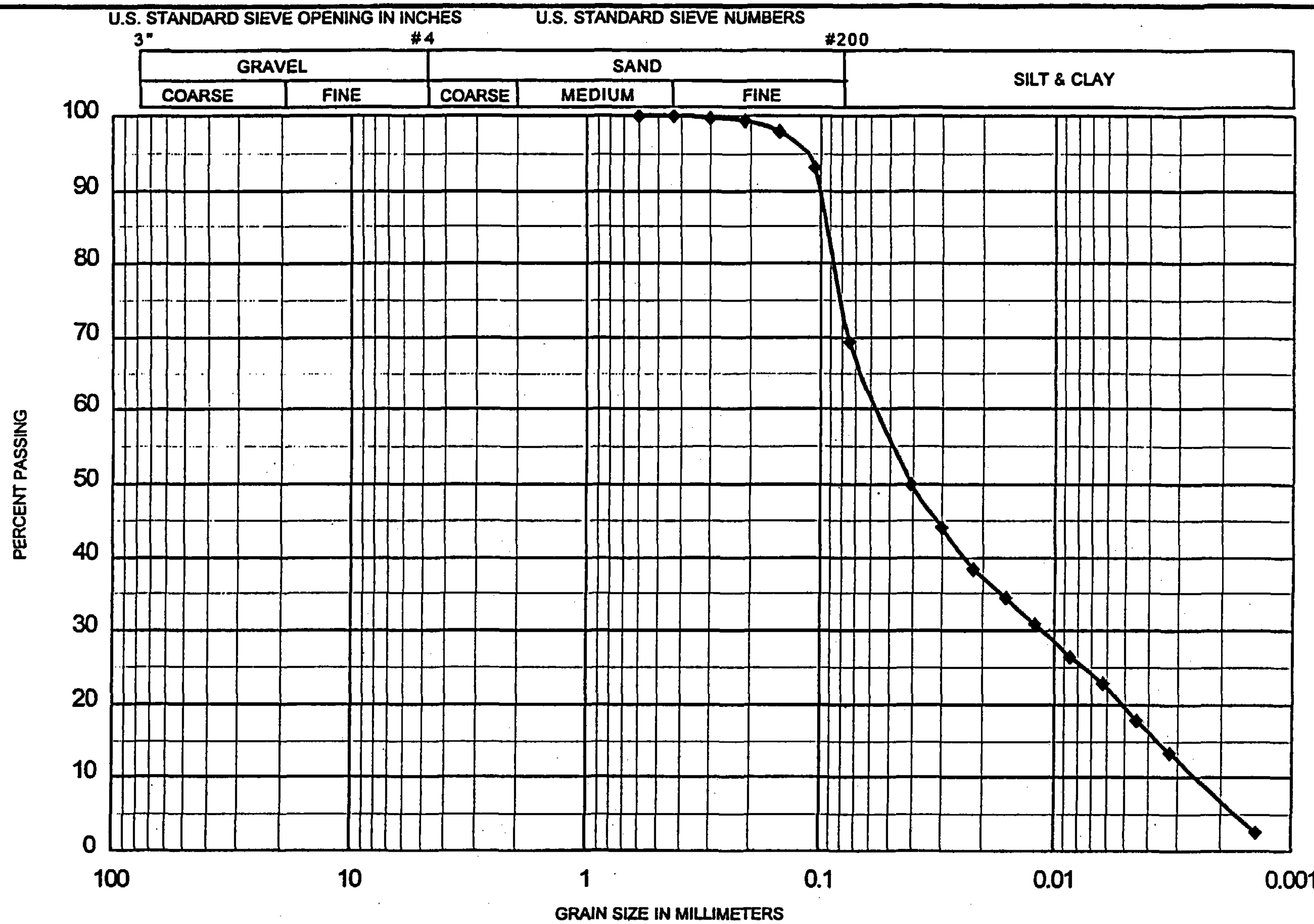
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-2, SAMPLE S-10, DEPTH 35.0-36.5 ft DESCRIPTION: SAND (SP)	JUNE 1999 Figure 3



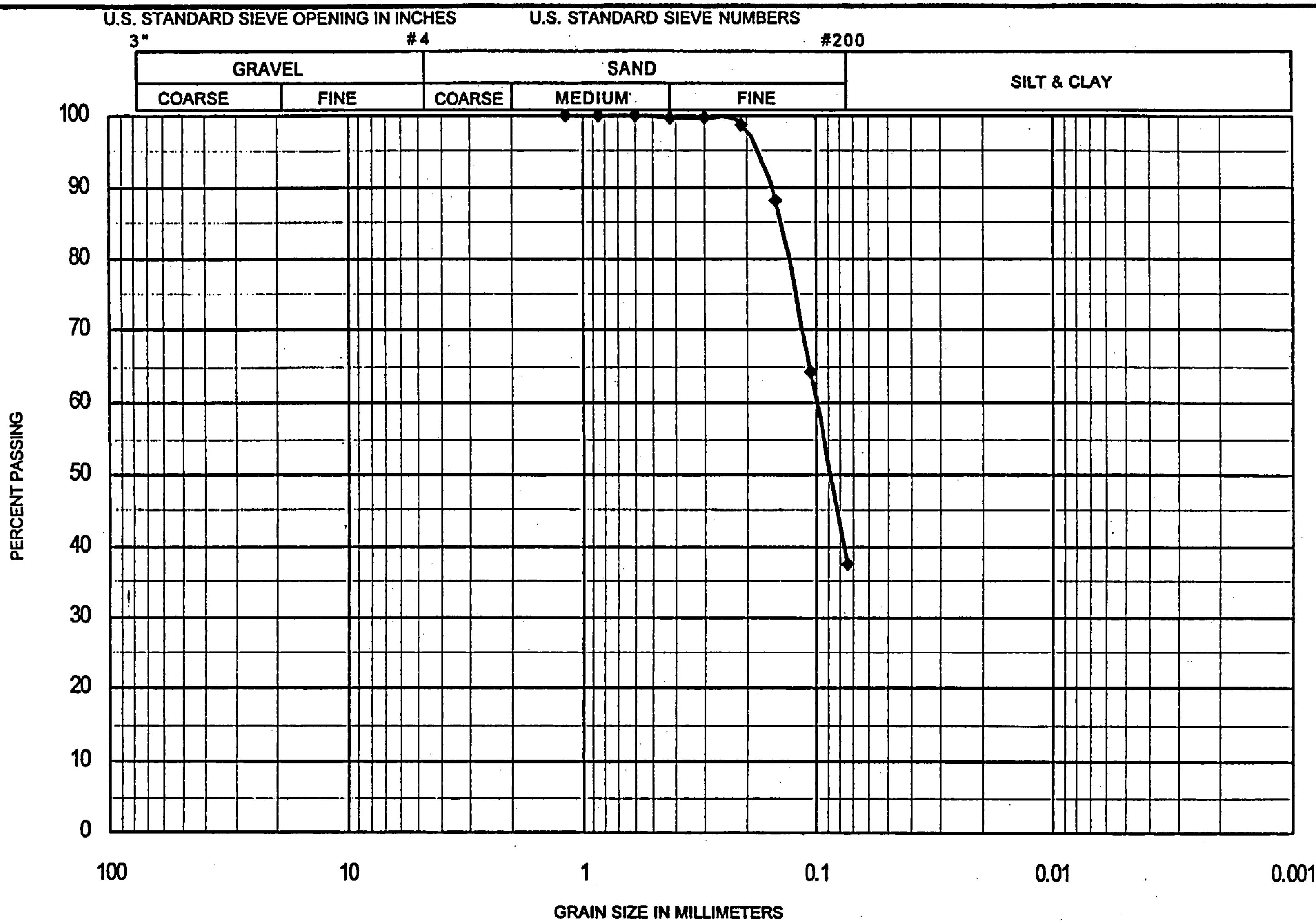
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-3, SAMPLE S-9, DEPTH 30.0-32.0 ft DESCRIPTION: SAND (SP)	JUNE 1999 Figure 4



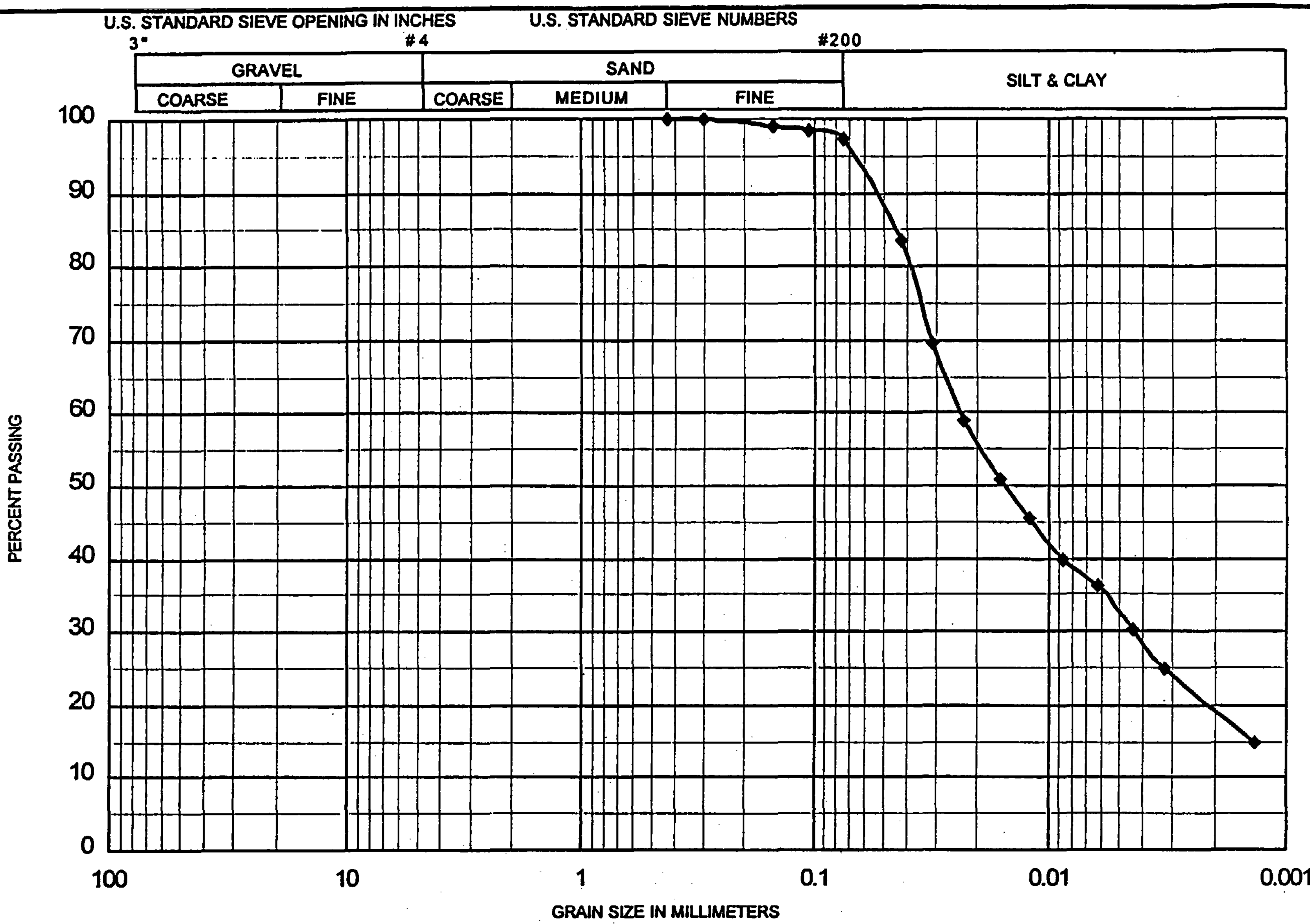
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE U-1C, DEPTH 6.7-7.2 ft DESCRIPTION: Silty CLAY (CL)	JUNE 1999 Figure 5



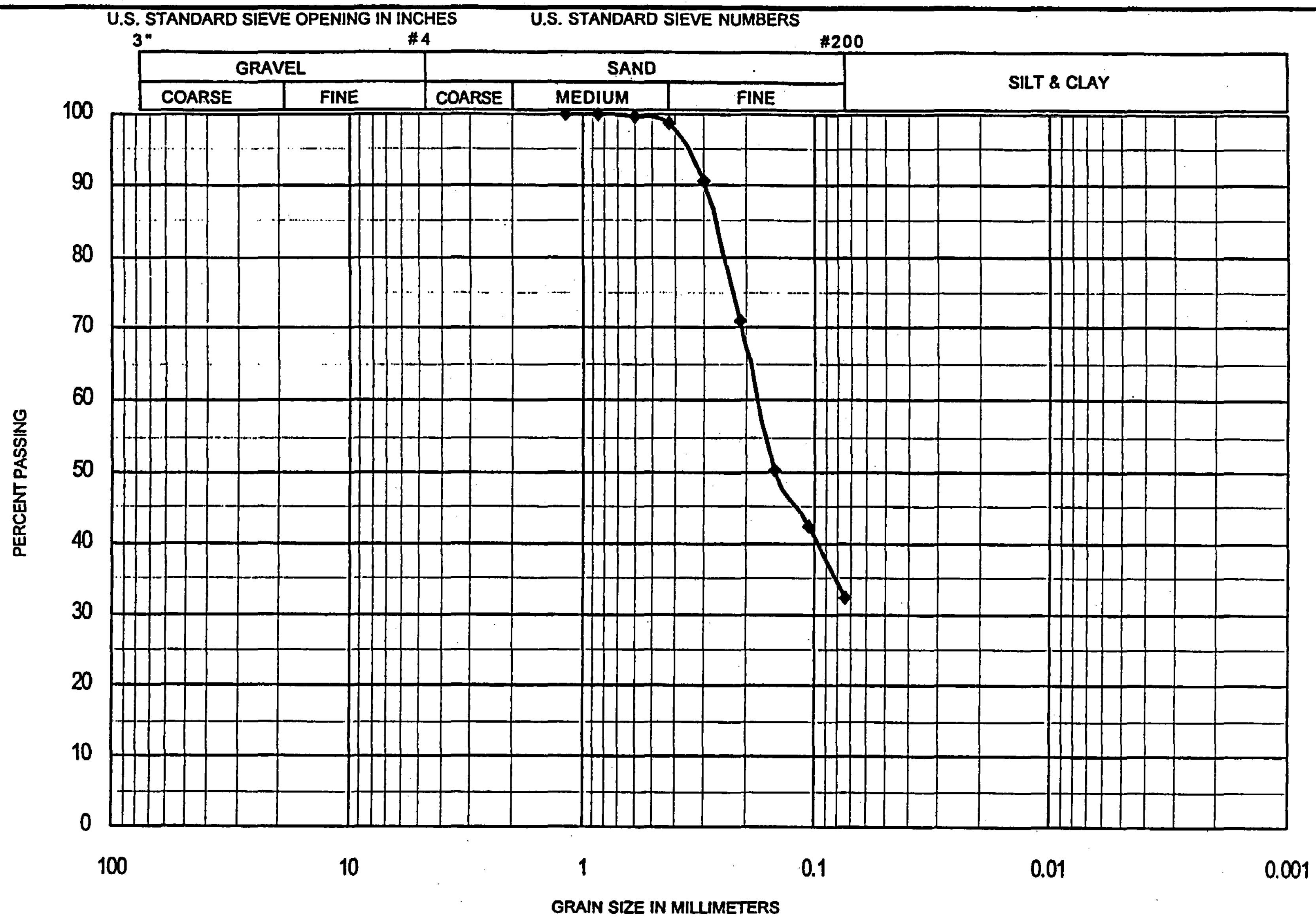
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE U-7D, DEPTH 12.9-13.1 ft DESCRIPTION: Sandy SILT (ML)	JUNE 1999 Figure 6



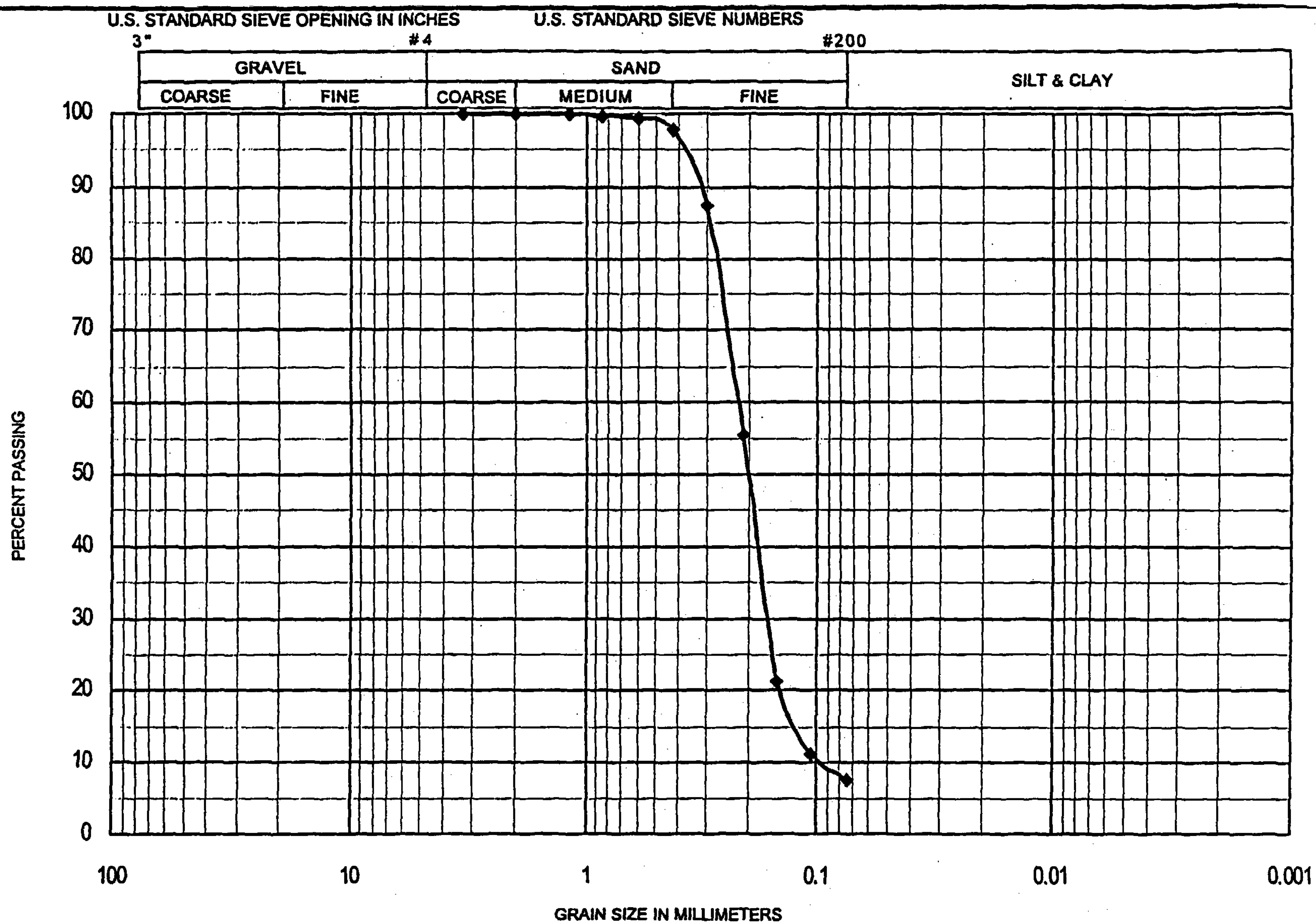
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE S-8, DEPTH 14.7-16.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 7



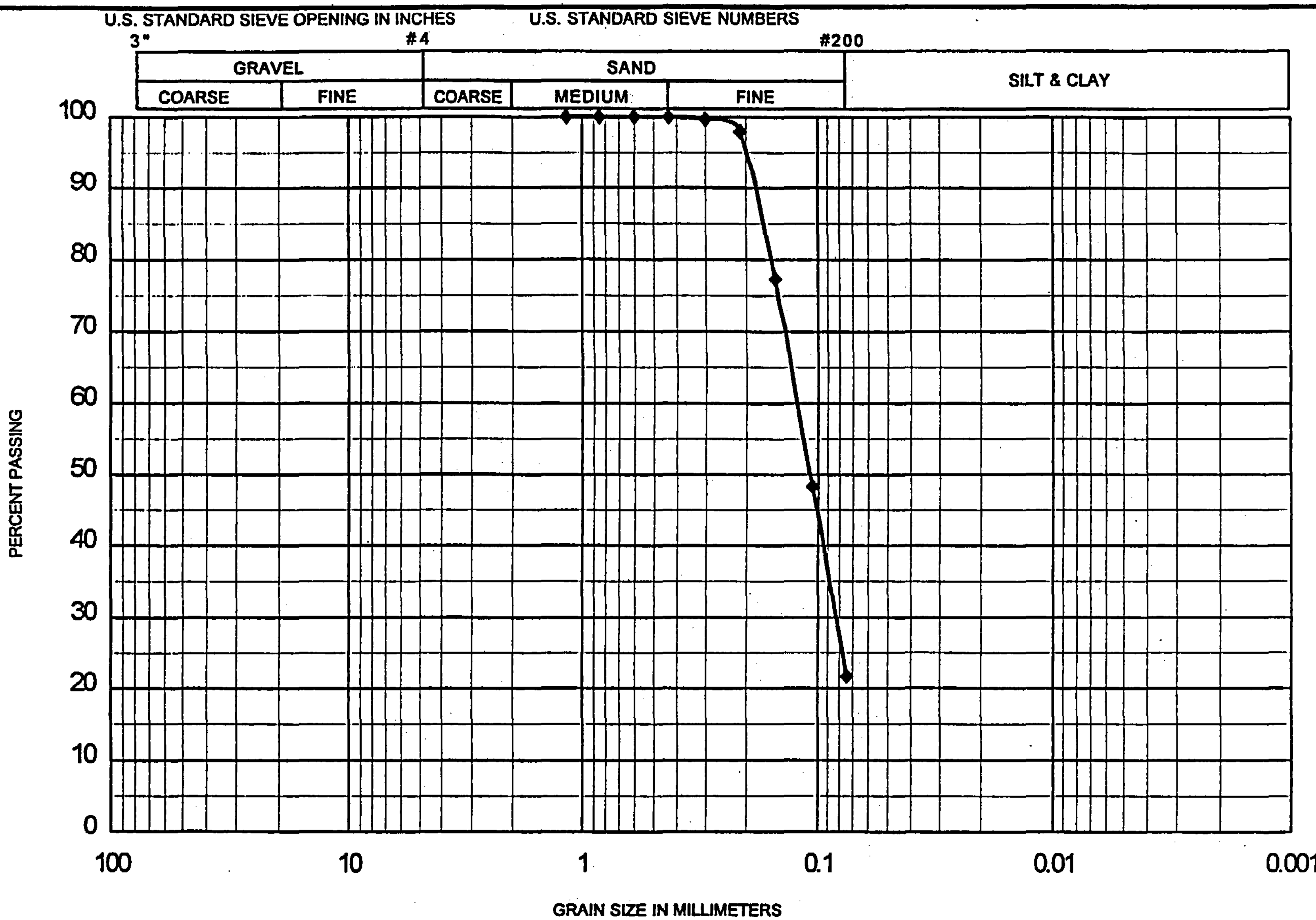
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE U-11D, DEPTH 20.9-21.5 ft DESCRIPTION: SILT (ML)	JUNE 1999 Figure 8



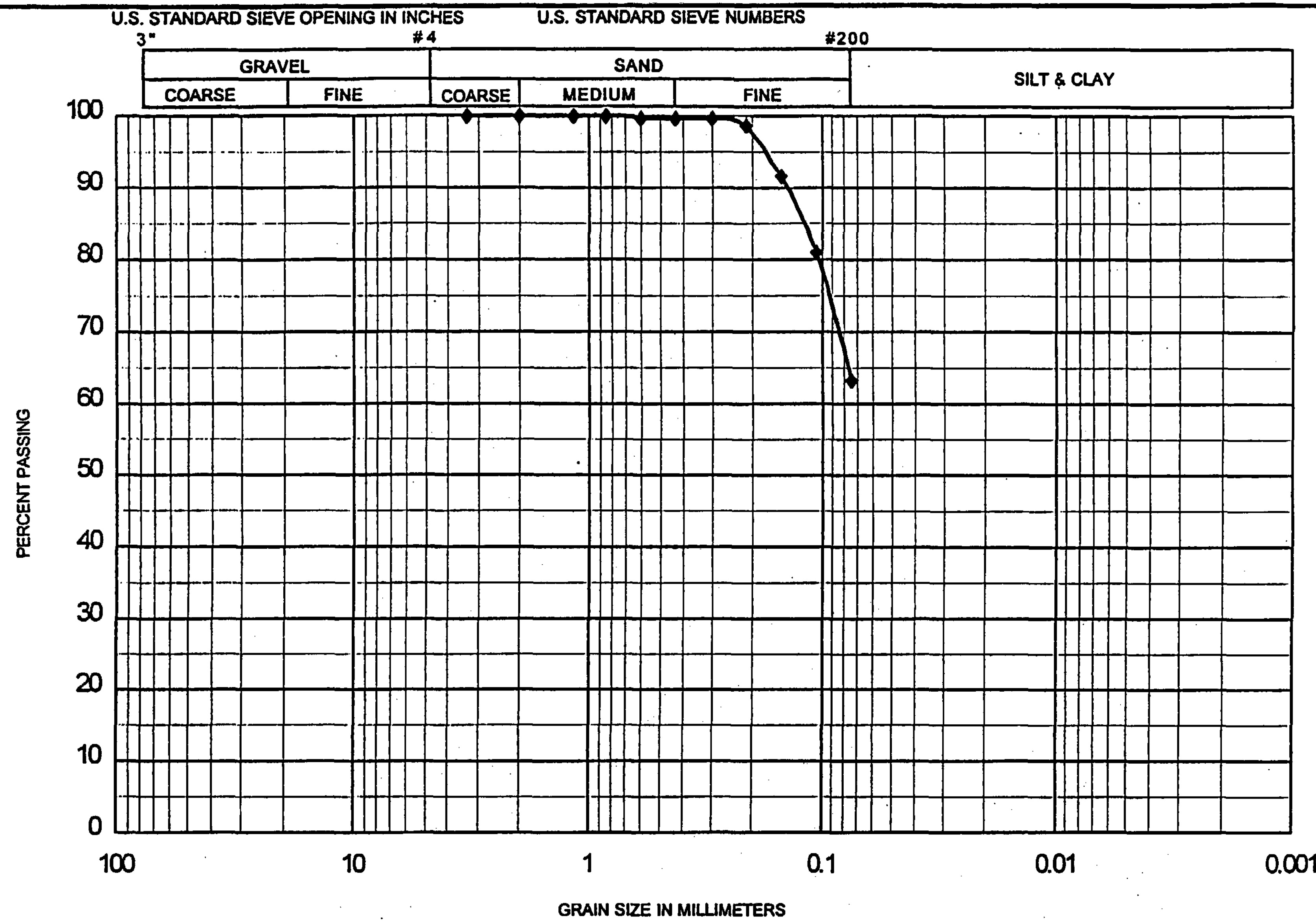
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE S-16, DEPTH 30.0-32.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 9



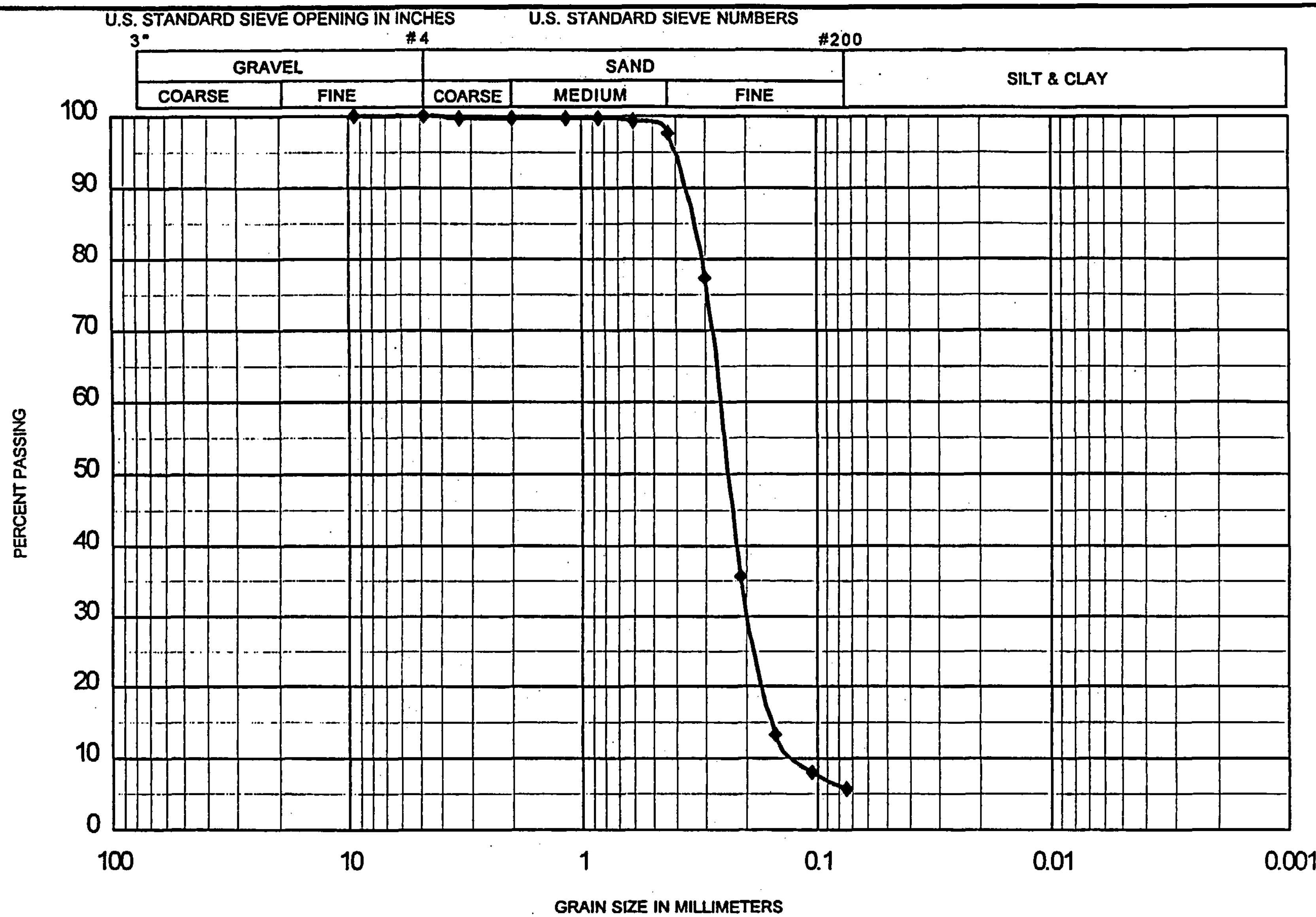
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-4, SAMPLE S-17, DEPTH 35.0-36.0 ft DESCRIPTION: SAND (SP)	JUNE 1999 Figure 10



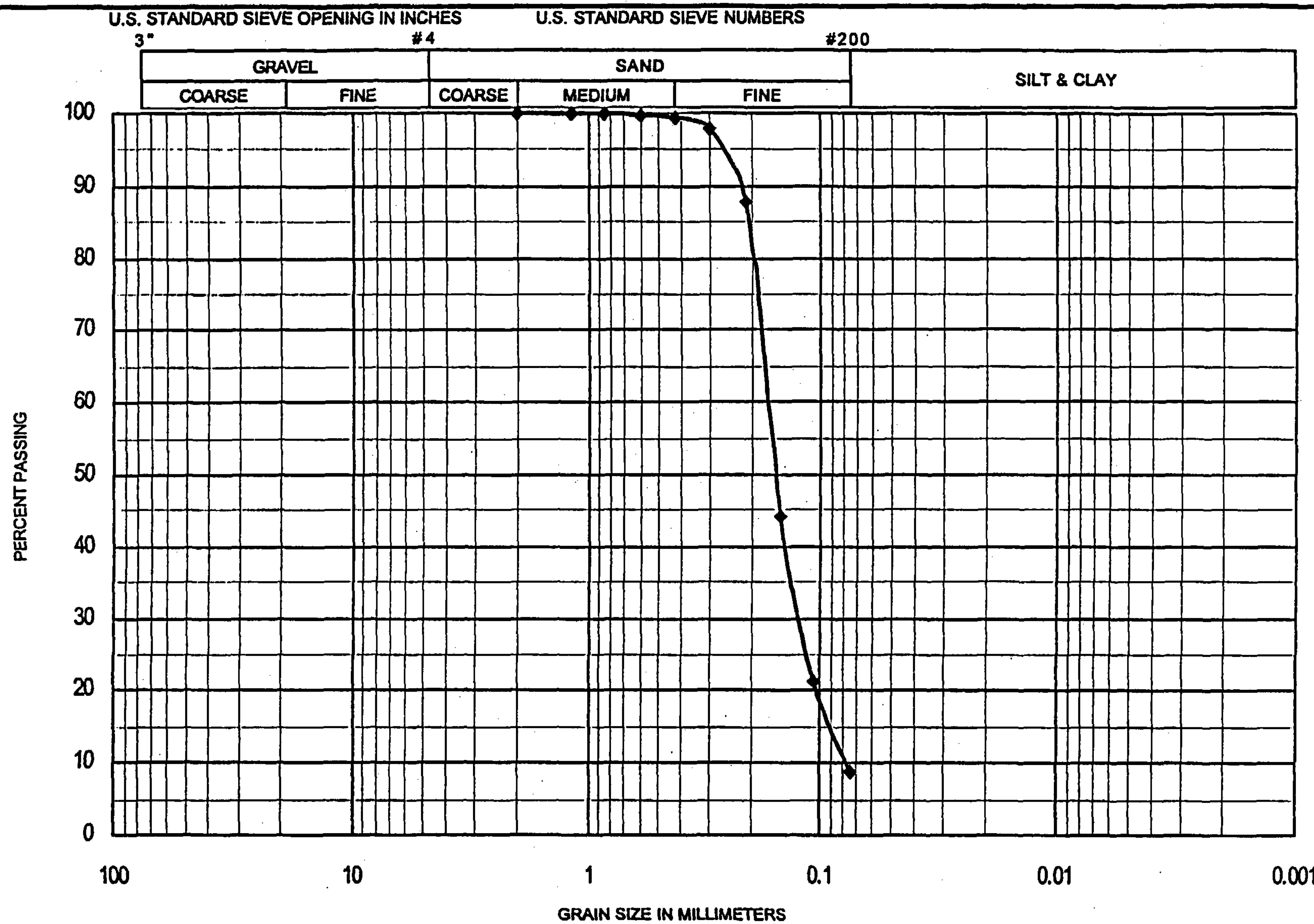
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-5, SAMPLE S-7, DEPTH 12.0-14.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 11



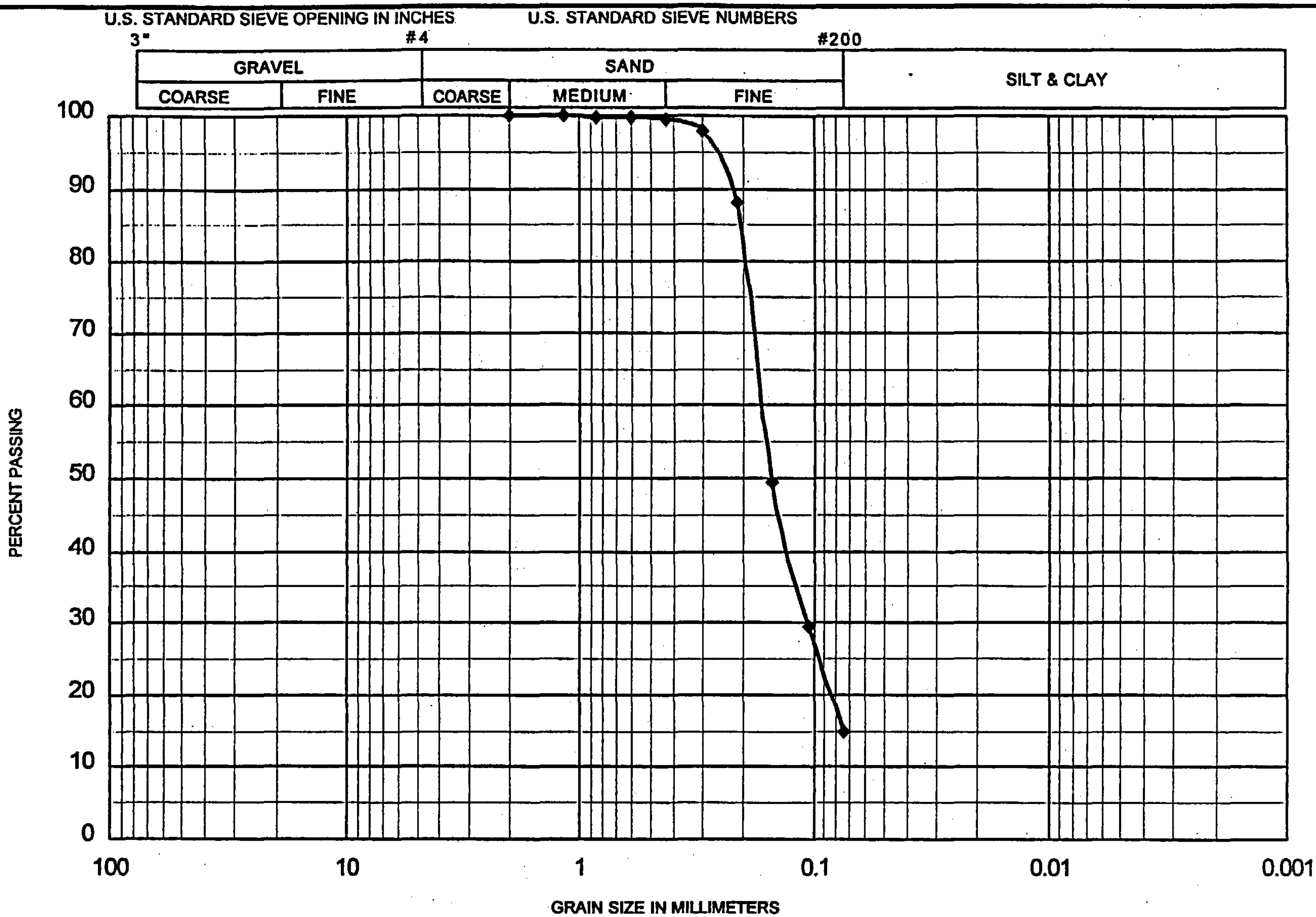
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-5, SAMPLE S-9, DEPTH 16.0-16.5 ft DESCRIPTION: Sandy SILT (ML)	JUNE 1999 Figure 12



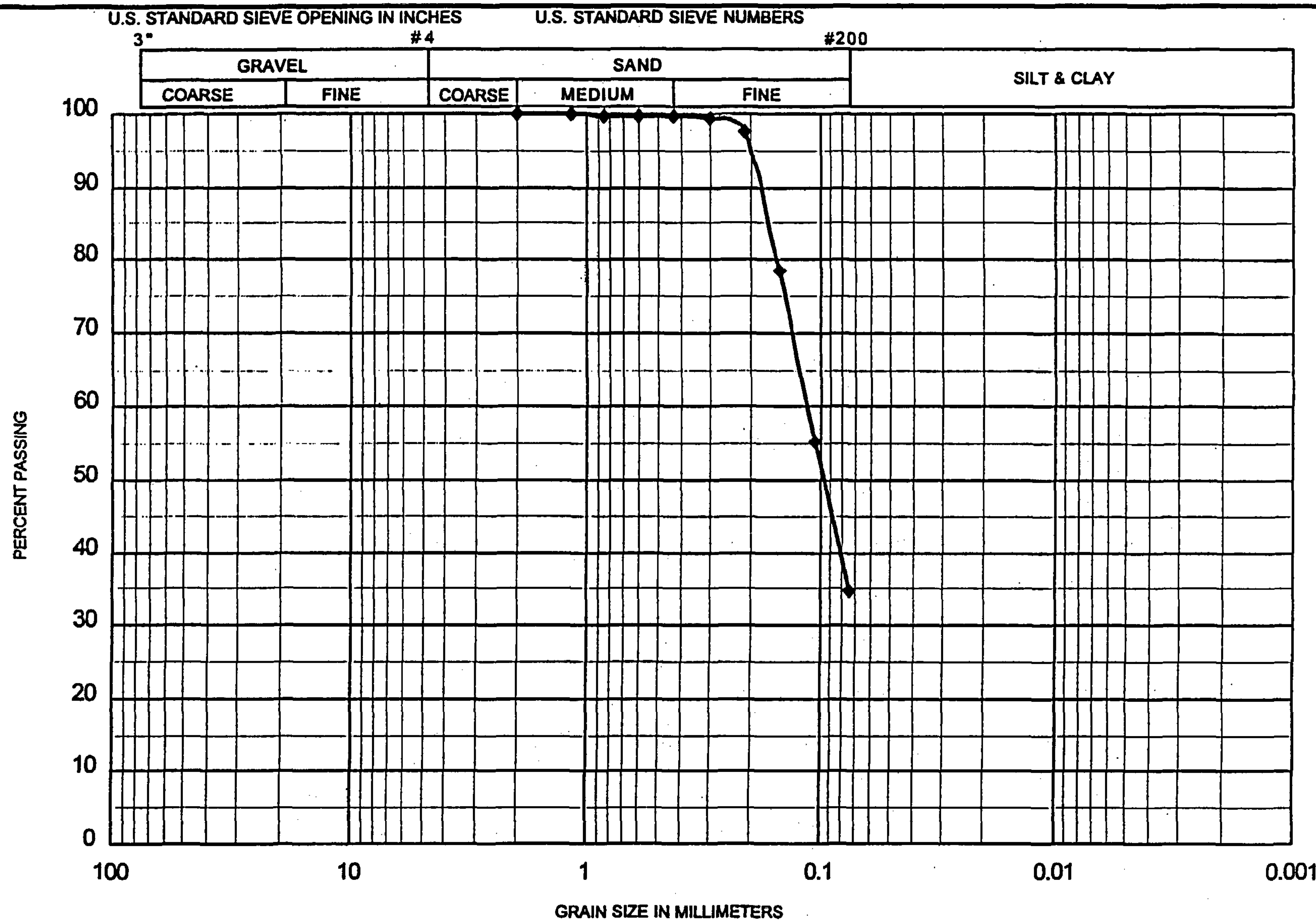
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-5, SAMPLE S-17, DEPTH 35.0-36.5 ft DESCRIPTION: SAND (SP)	JUNE 1999 Figure 13



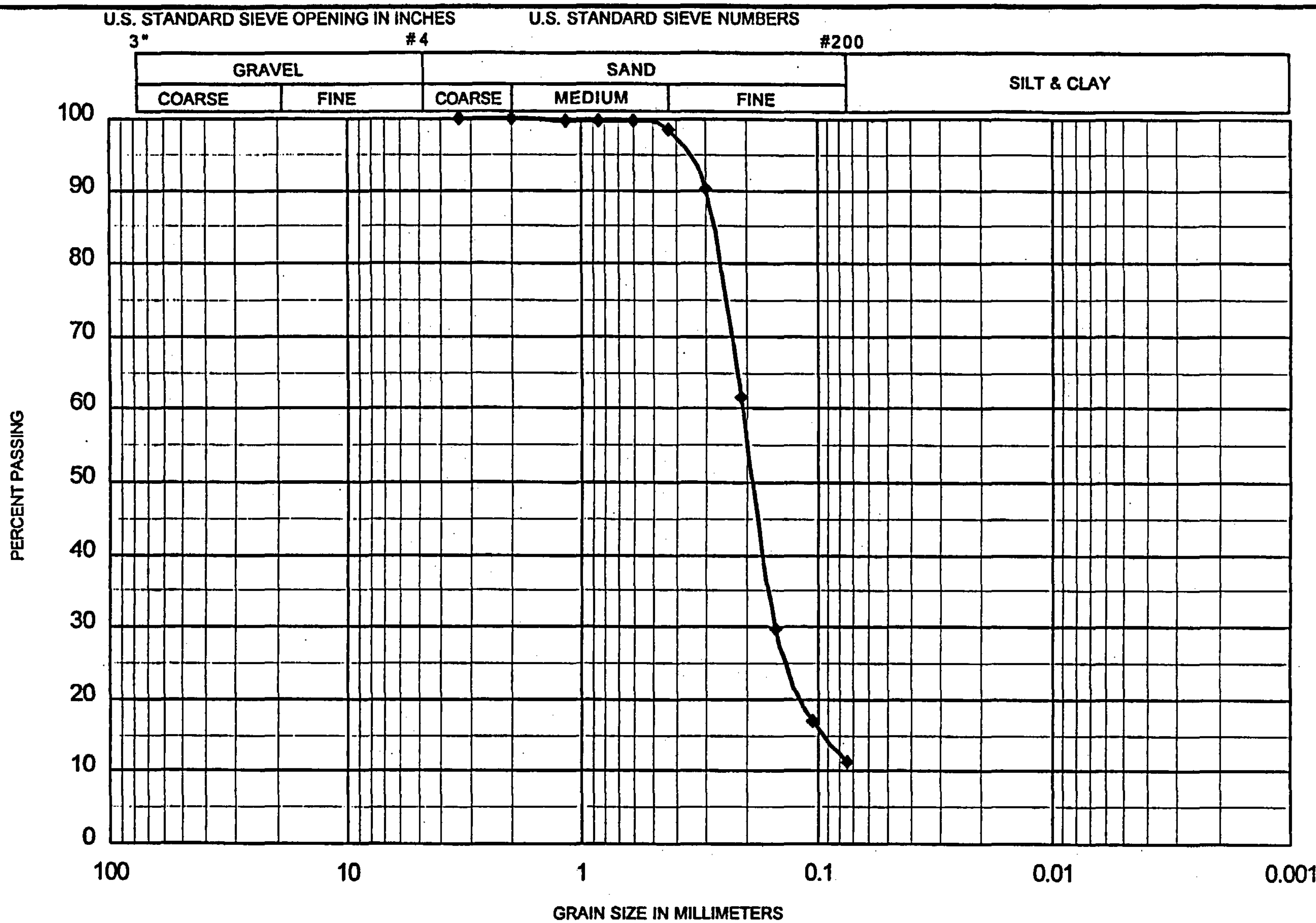
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-7, SAMPLE U-3D, DEPTH 8.2-8.4 ft DESCRIPTION: SAND (SP)	JUNE 1999 Figure 14



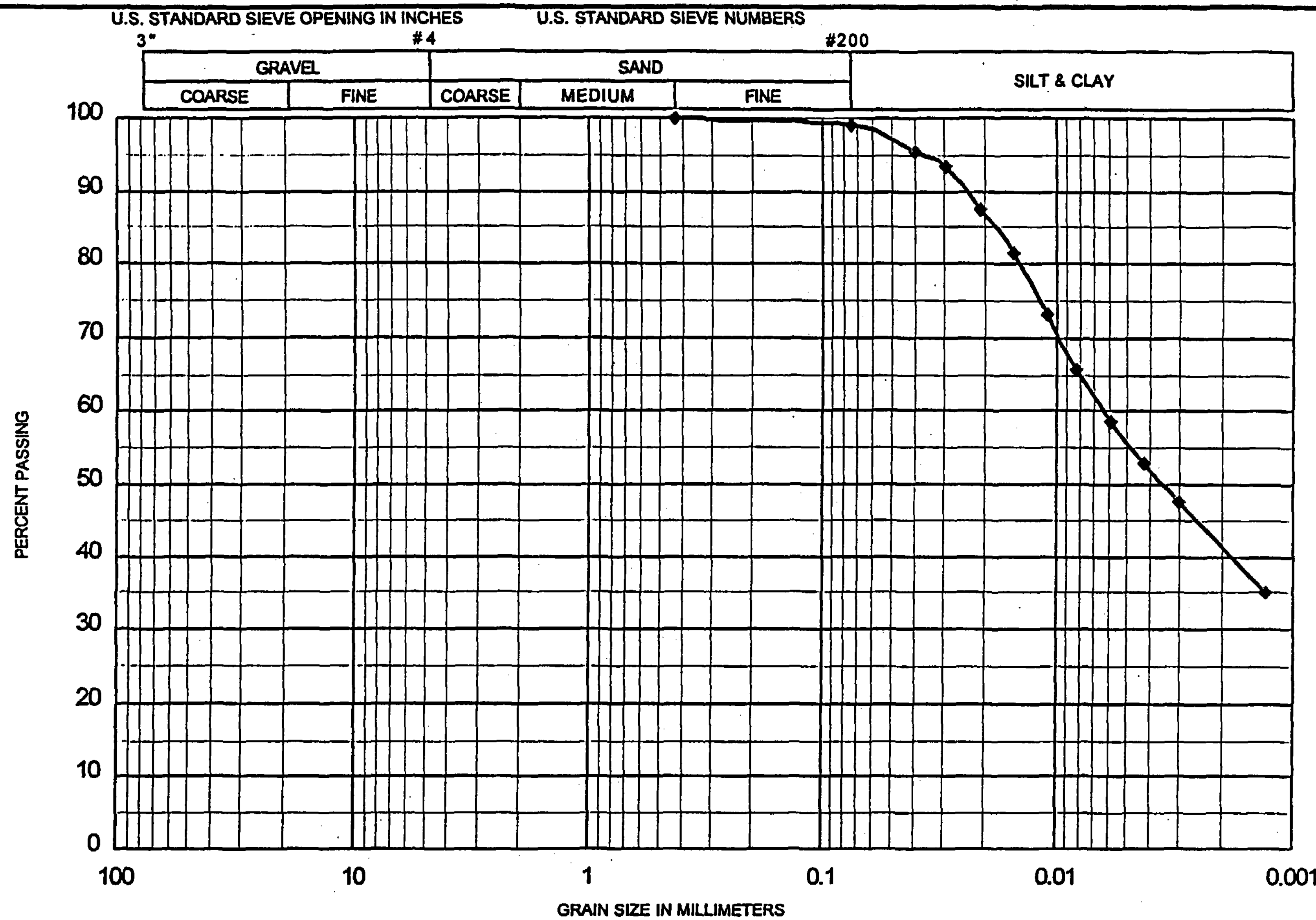
STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-8, SAMPLE S-4, DEPTH 9.0-11.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 15



STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-8, SAMPLE S-6, DEPTH 15.0-17.0 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 16



STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-8, SAMPLE S-9, DEPTH 30.0-31.5 ft DESCRIPTION: Silty SAND (SM)	JUNE 1999 Figure 17



STONE & WEBSTER ENGINEERING INC. BOSTON, MASSACHUSETTS	PRIVATE FUEL STORAGE, LLC PFSF, SKULL VALLEY, UT JO 05996.02	GRAIN SIZE DISTRIBUTION BORING CTB-S, SAMPLE U-3C, DEPTH 10.0-10.2 ft DESCRIPTION: Clayey SILT (MH)	JUNE 1999 Figure 18

APPENDIX 2A

ATTACHMENT 7

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
AUGUST 1999

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
AUGUST 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING AUGUST 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

Prepared by:	Alan C Smith	8/18/99 Date
Reviewed by:	Thomas Y. Chang	8/19/99 Date
Independently Reviewed by:	Thomas Y. Chang	8/19/99 Date
Approved by:	J L Cooper	8/19/99 Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

INTRODUCTION

The primary objective of the laboratory testing program was to determine the cohesion and friction angle of the soils at the base of the mat foundations for use in the sliding stability analyses. The direct shear test specimens were obtained from preserved tube sections of thin-walled tube samples obtained from prior testing programs. Testing began on August 9, 1999 and ended on August 18, 1999.

The tests performed included classification, water content, Atterberg limits, and direct shear. They were conducted in accordance with the following American Society for Testing and Materials standards.

D-2216	1992	Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D-3080	1998	Test Methods for Direct Shear Test of Soils Under Consolidated Drained Conditions
D-4318	1995a	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

TEST RESULTS

A total of 10 direct shear tests were performed on a thin-walled tube sample from Borings C-2, CTB-6, and CTB-S. The test results are shown in Table 1. A plot of the normal stress vs peak shear stress for each tube sample is shown after Table 1. The plots of horizontal displacement vs shear stress and data for each test are in Appendix A.

The samples were trimmed into a nominal 2.5-inch diameter ring and placed in the direct shear apparatus. The samples were not inundated because the soils at the site are not expected to be saturated during the life of the facility. A normal load was applied and the deformation measured. Primary consolidation occurred prior to 1 minute. After at minimum of 90 minutes, the sample was sheared at a displacement rate of 18 mm/hr. The test continued until the shear load peaked and remained constant or started decreasing after 0.25 inches of displacement. The sample was removed and oven dried.

The trimmings from each test were retained and used to prepare an Atterberg limits test sample. The trimmings from the same tube were combined for one Atterberg limits test. The results are shown in Table 1.

LABORATORY TESTING PROGRAM
GENERAL INFORMATION

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 1 OF 3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Trudeau	8-6-99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCOOPER PJT	RECEIVED BY: ACS
FEATURE OR PHASE OF PROJECT: CANISTER TRANSFER BUILDING		NEAREST CITY OR TOWN					NEAREST AIRPORT HANDLING FREIGHT	

PROJECT MANAGER	LEAD GEOTECHNICAL ENGINEER	SUPERVISOR OF FIELD WORK	SUPERVISOR OF OFFICE WORK	CONTACT FOR LABORATORY WORK
JL DONNELL	PJ TRUDEAU	RP GILLESPIE	GR DOTON	PJT / TY CHANG
DATE PROGRAM ASSIGNED 8-9-99	DATE TESTING TO START 8-9-99	DATE BORING LOG DATA NEEDED	DATE ALL TEST DATA AVAILABLE	DATE ALL FINAL PLOTS FORWARDED

TYPES OF STRUCTURES INVOLVED:
MAT-SUPPORTED CANISTER TRANSFER BUILDING

PARTICULAR CRITICAL FEATURES OF STRUCTURES:
SLIDING STABILITY DURING DESIGN EARTHQUAKE $a_h = 0.528\,g$ $a_v = 0.533\,g$ 2,000-YR EQK

TYPES OF BEHAVIOR TO BE ANALYZED:
DIRECT SHEAR

TYPICAL ANTICIPATED LOADING STRESSES:
STATIC $\Delta q \sim 1.9$ KSF $\Delta\sigma_v$ DUE TO EARTHQUAKE ~ 0.8 KSF

PRIMARY OBJECTIVES OF LABORATORY TESTS:
DETERMINE c & ϕ OF SOILS AT BASE OF MAT FOR USE IN SLIDING STABILITY ANALYSIS.

GENERAL MAGNITUDE OF TESTING PROGRAM:
SMALL

NUMBER OF DISTURBED SAMPLES:

NUMBER OF UNDISTURBED SAMPLES:

TYPE OF REPORT TO CONTAIN RESULTS:
SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING - AUGUST 1999

UNITS FOR REPORTING STRESSES: KSF

ADDITIONAL:
CANISTER TRANSFER BUILDING IS QA CATEGORY I (IMPORTANT TO SAFETY). ALL LABORATORY EQUIPMENT & TESTING PROCEDURES SHALL BE CARRIED OUT UNDER A QA PROGRAM THAT MEETS THE APPLICABLE REQUIREMENTS OF 10 CFR 50, APPENDIX B, AND 10 CFR 72.

Attachment 1

LABORATORY TESTING PROGRAM
SPECIFIC TEST REQUIREMENTS

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: PRIVATE FUEL STORAGE, LLC	J.O. NUMBER: 05996.02 345W	REVISION	1	2	3	4	PREPARED BY: Paul J. Trudeau	SHEET 2 / DATE PREPARED: 8-6-99
SITE: PFSF SKULL VALLEY, UT		DATE					APPROVED BY: J.L.COOPER PJT.	RECEIVED BY: ACS
		REVISED BY						

NOTE	COL. NO.	TYPE OF TEST	REQUIREMENTS OF TYPE OF TEST AND PROCEDURES TO BE USED (SIZE OF SPECIMENS, RATES OF STRAIN, CONFINING PRESSURES, CYCLES OF RECOMPRESSION ETC.)
A	ALL	ALL TESTS	SELECTION OF NECESSARY AND APPROPRIATE DETERMINATIONS AND/OR TEST PROCEDURES TO BE MADE BY LABORATORY PERSONNEL.
B	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION ONLY; NO INDEX PROPERTIES TO BE DETERMINED.
C	1	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION TO BE SUPPLEMENTED BY DETERMINATIONS OF SPECIFIC INDEX DETERMINATIONS INDICATED IN SUBSEQUENT COLUMNS.
D	3	ATTERBERG LIMITS	FOR CLASSIFICATION PURPOSES ONLY; ONE-POINT LIQUID LIMIT DETERMINATION ACCEPTABLE.
E	5	GRADATION ANALYSES	SIEVE ANALYSIS ONLY; NO HYDROMETER ANALYSIS REGARDLESS OF PERCENT FINES.
F	10	DIRECT SHEAR	σ_c = 1 KSF, 2 KSF, & 3 KSF FOR EACH TEST LOCATION TO PERMIT
			DETERMINATION OF c & ϕ AVAILABLE TO RESIST SLIDING

Attachment 2

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET	3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
PRIVATE FUEL STORAGE, LLC	05996.02 345W	DATE					Paul J. Trudeau	8-6-99
SITE: PFSF SKULL VALLEY, UT		REVISED BY					APPROVED BY: JLCOOPER RJT.	RECEIVED BY: ACS

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

TYPE OF TEST: CLASSIFICATION, WATER CONTENT, ATTERBERG LIMITS, PERCENT FINES, GRADATION ANALYSES, SPECIFIC GRAVITY, UNIT WEIGHT, UNDRAINED COMPRESSION, CONSOLIDATED TRIAXIAL, DRAINED DIRECT SHEAR, CONSOLIDATION, COMPACTION, RELATIVE DENSITY, PERMEABILITY

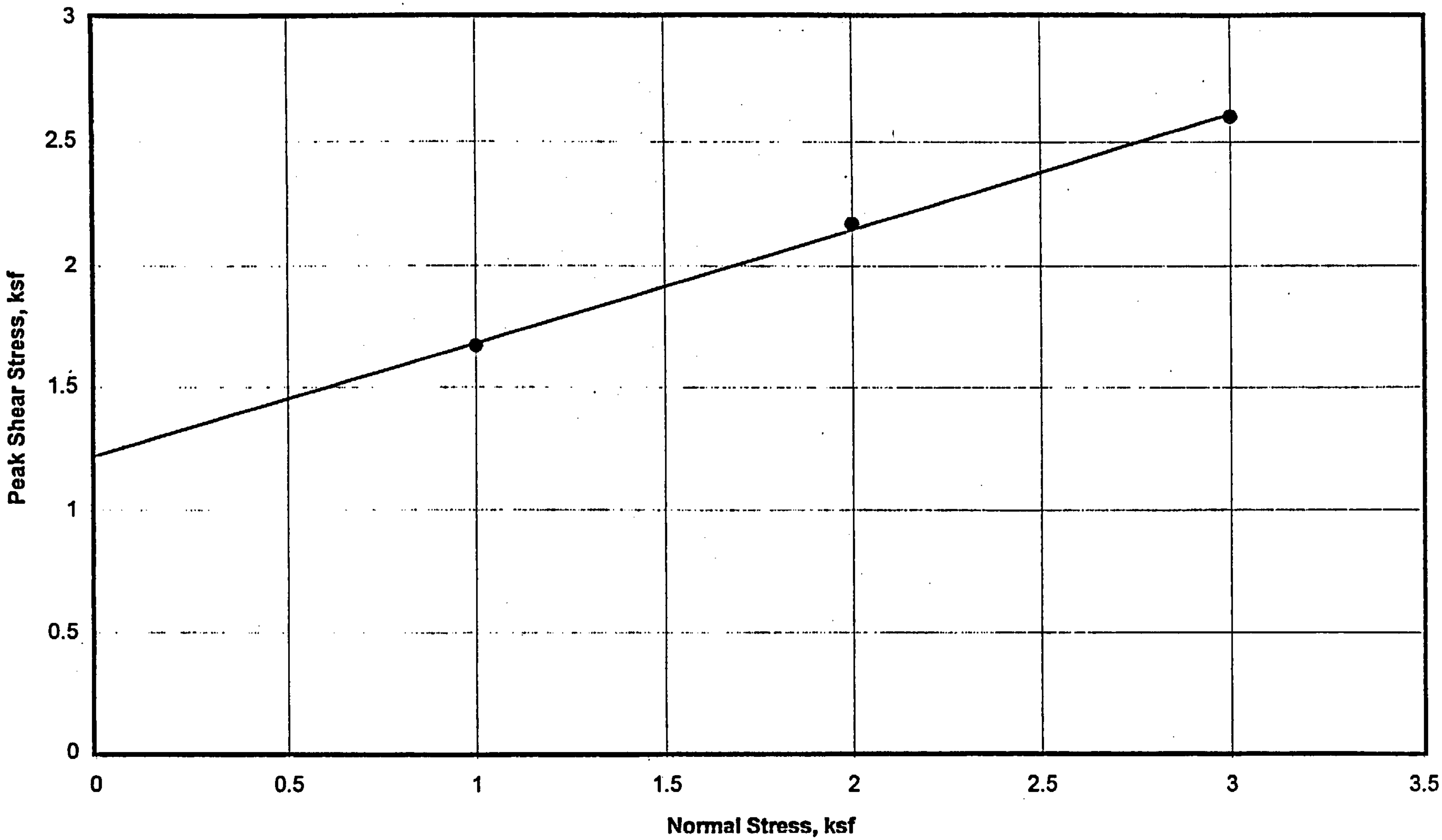
BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1	2	3	4	5	6	7	8	9	10 (F)	11	12	13	14	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CTB-5	U	1AA	5.1-5.5	X	X	X				X			X								3 TESTS @ VARIOUS σc TO DETERMINE c & φ
CTB-6	U	3	7-8	X	X	X				X			X								" "
C-2	U	1	5-6.3	X	X	X				X			X								

TEST TO BE PERFORMED — TEST IN PROGRESS — TEST COMPLETED & CHECKED — TEST CANCELLED

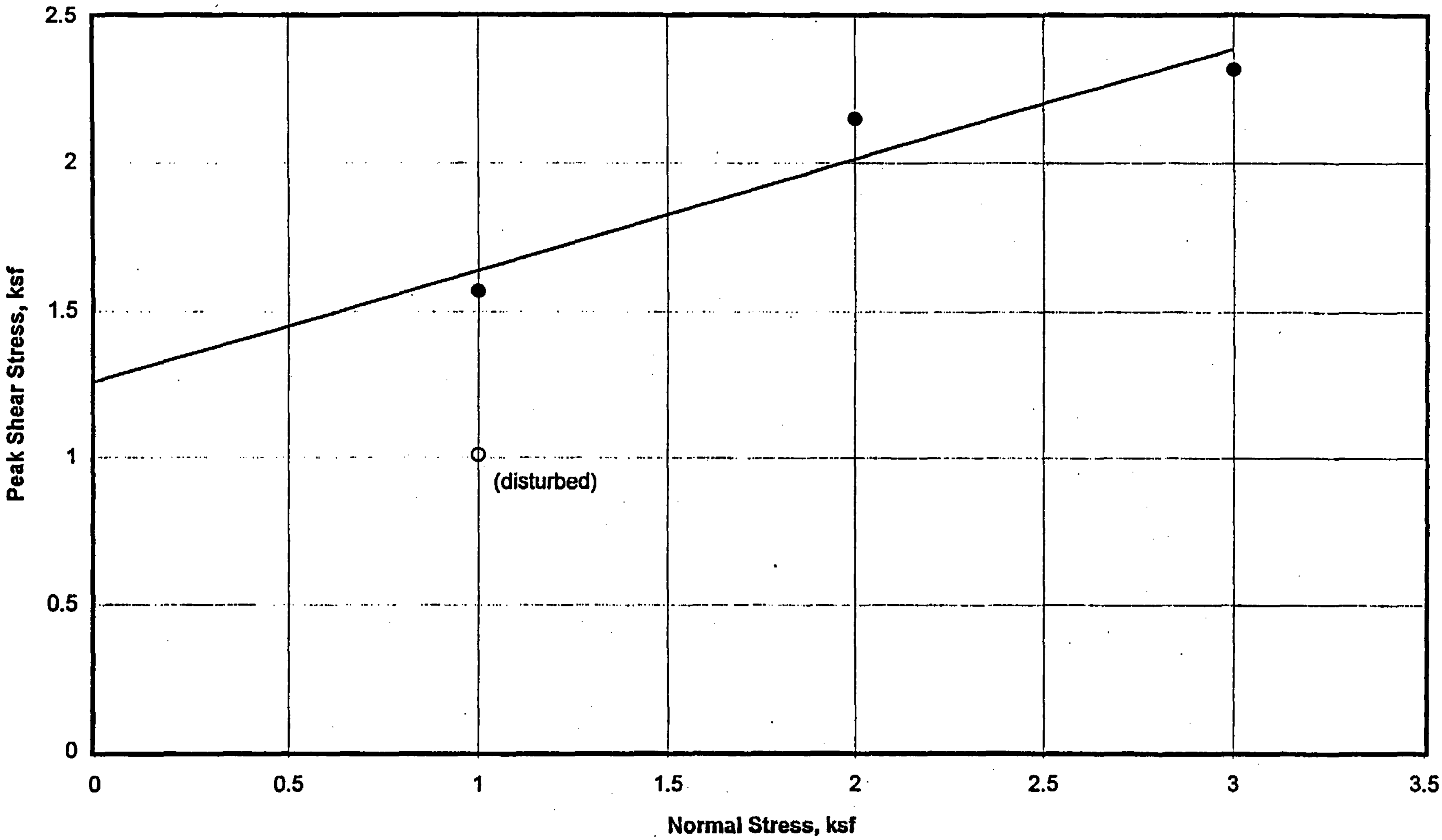
TABLE 1
Direct Shear Test Results

Boring	Sample	Depth (ft)	Elevation (ft)	Atterberg Limits			USC Code	Water Content (%)	Initial			After Consolid.		Normal Stress (ksf)	Peak Shear (ksf)	Cohesion (ksf)	Tan ϕ	ϕ (deg)
				LL	PL	PI			γ_m (pcf)	γ_d (pcf)	Void Ratio	γ_d (pcf)	Void Ratio					
C-2	U-1C1	5.7	4458.5					55.7	69.4	44.50	2.81	45.1	2.76	3.0	2.60			
C-2	U-1C2	5.9	4458.3	76.9	39.1	37.8	MH	58.2	63.7	40.20	3.22	40.5	3.19	2.0	2.17	1.22	0.465	24.9
C-2	U-1C3	6.0	4458.2					52.7	75.1	49.2	2.45	49.3	2.44	1.0	1.67			
CTB-6	U-3B1	7.2	4469.0					61.7	74.7	46.2	2.68	46.5	2.65	1.0	1.01			
CTB-6	U-3B3	7.5	4468.7	65.3	32.5	32.8	MH	61.3	81.9	50.7	2.35	51.2	2.32	2.0	2.15	1.26	0.375	20.6
CTB-6	U-3B4	7.7	4468.5					60.3	80.5	50.2	2.38	50.9	2.34	3.0	2.32			
CTB-6	U-3C	7.8	4468.4					56.6	88.5	56.4	2.01	56.7	2.00	1.0	1.57			
CTB-S	U-1AA3	5.1	4469.4					80.9	75.7	41.8	3.06	42.6	2.98	3.0	2.24			
CTB-S	U-1AA2	5.3	4469.2	82.7	44.8	37.9	MH	84.6	73.1	39.6	3.29	39.9	3.25	2.0	1.75	1.00	0.397	21.6
CTB-S	U-1AA1	5.4	4469.1					86.8	70.9	37.9	3.48	38.1	3.45	1.0	1.42			

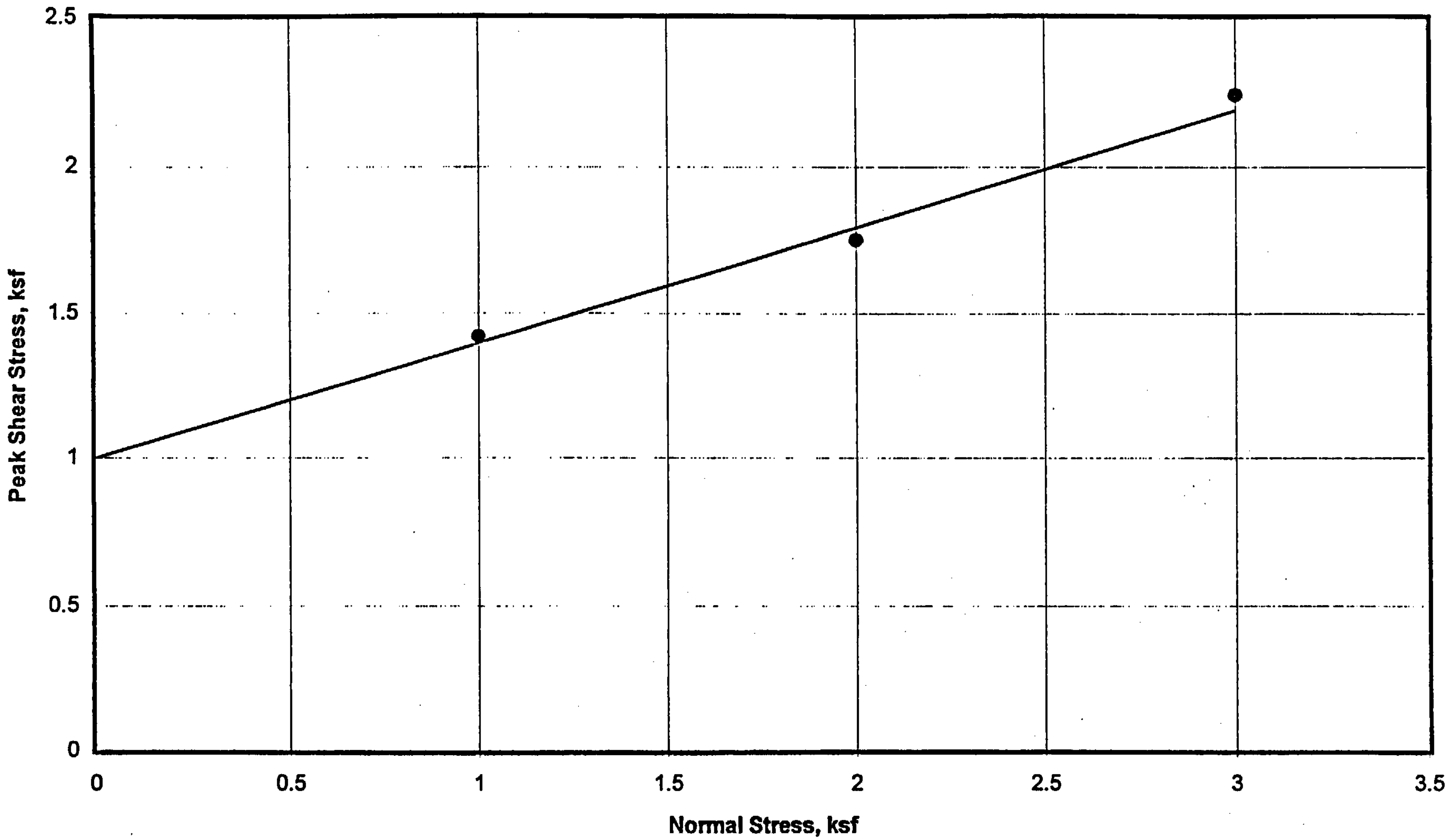
DIRECT SHEAR TEST
Boring C-2, Sample U-1C



DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C



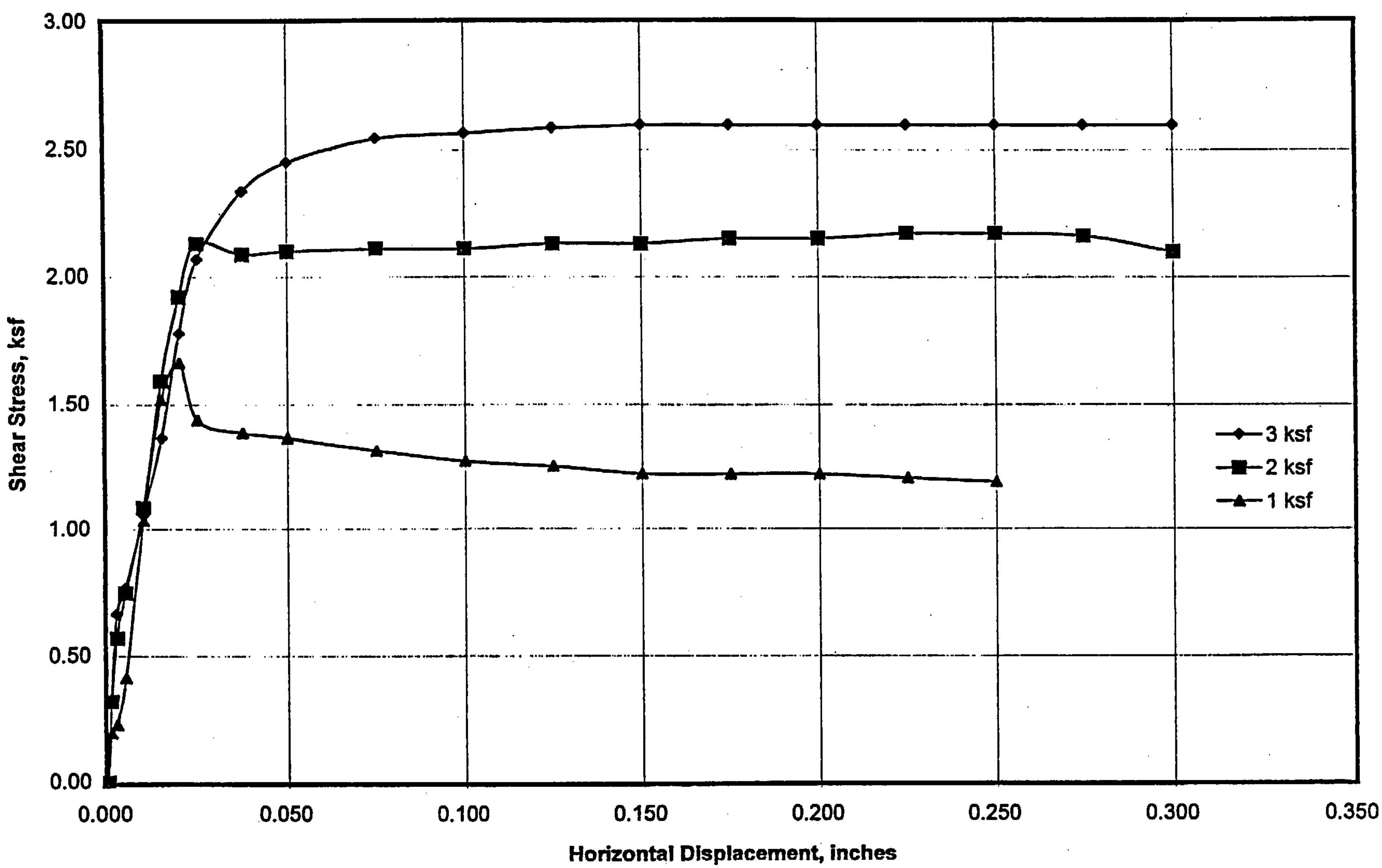
DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA



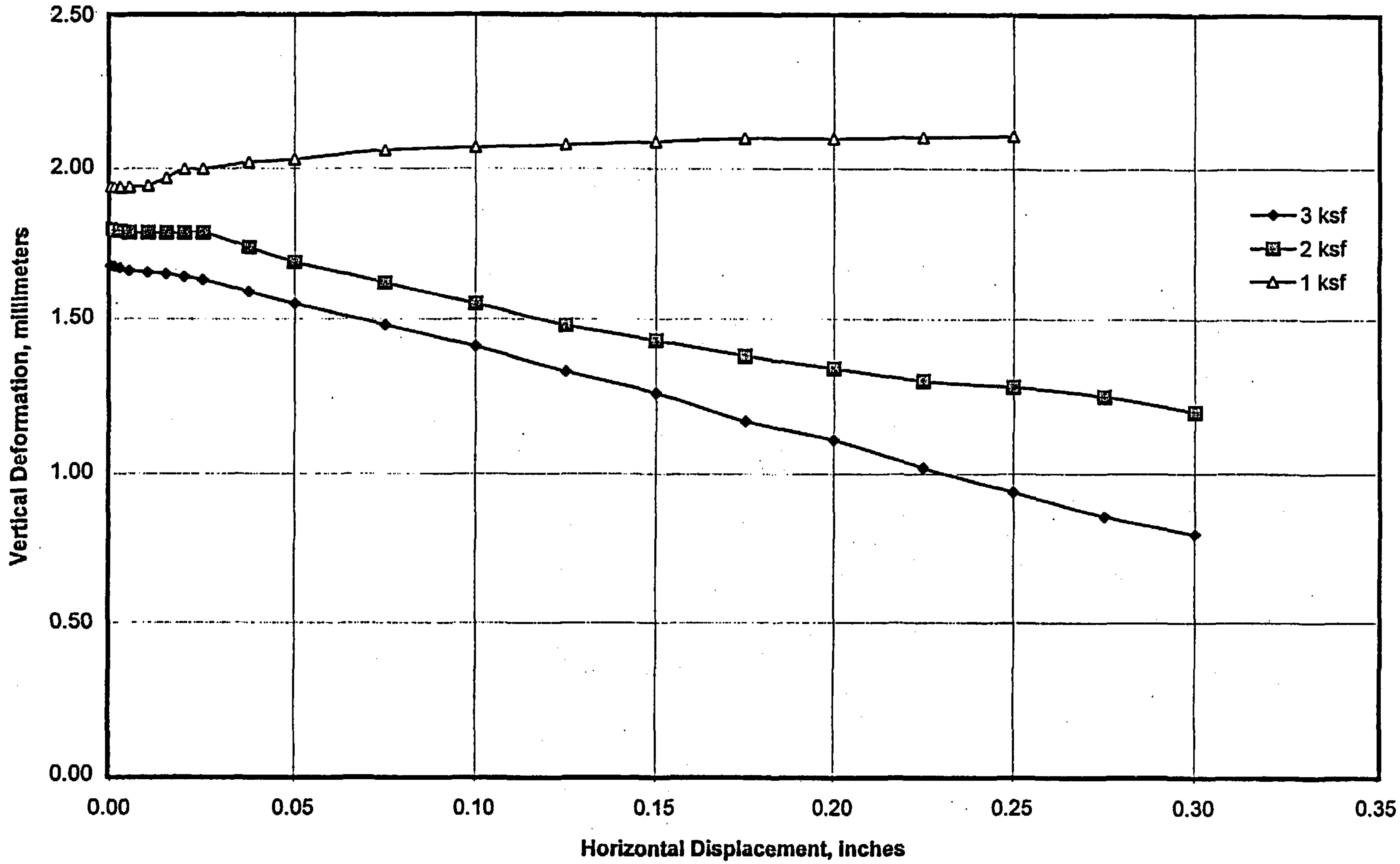
APPENDIX A

Direct Shear Test Plots and Data

DIRECT SHEAR TEST
Boring C-2, Sample U-1C



DIRECT SHEAR TEST
Boring C-2, Sample U-1C



DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	08/17/99
SAMPLE:	U-1C3	TESTED BY:	ACS
DEPTH:	6.0 ft	CHECKED:	TLC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	52.7 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	62.79 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	75.1 pcf	
DRY UNIT WEIGHT:	49.2 pcf	49.3 pcf
VOID RATIO:	2.45	2.44

TEST DATA:

NORMAL STRESS:	1.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	1.67 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.94	0	0.0	0.00	0.00
0.55	0.001	1.94	19	6.7	0.20	0.20
0.76	0.0025	1.94	22	7.7	0.23	0.23
1.18	0.005	1.94	40	14.1	0.41	0.41
2.33	0.010	1.945	100	35.2	1.03	1.03
3.19	0.015	1.97	147	51.7	1.52	1.52
3.58	0.020	2.00	161	56.7	1.67	1.67
3.96	0.025	2.00	139	48.9	1.44	1.44
4.98	0.0375	2.02	134	47.2	1.39	1.39
6.00	0.050	2.03	132	46.5	1.37	1.37
8.06	0.075	2.06	127	44.7	1.31	1.31
10.14	0.100	2.07	123	43.3	1.27	1.27
12.20	0.125	2.08	121	42.6	1.25	1.25
14.20	0.150	2.09	118	41.5	1.22	1.22
16.20	0.175	2.10	118	41.5	1.22	1.22
18.20	0.200	2.10	118	41.5	1.22	1.22
20.20	0.225	2.105	116.5	41.0	1.20	1.20
22.20	0.250	2.11	115	40.5	1.19	1.19

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING: C-2
DATE: 08/17/99
SAMPLE: U-1C2
TESTED BY: ACS
DEPTH: 5.9 ft
CHECKED: TYC
DESCRIPTION: Clayey SILT (MH)

HEIGHT: 0.99 inches
DIAMETER: 2.50 inches
AREA: 4.90 sq. in.

WATER CONTENT: 58.2 %
DRY WEIGHT SOIL: 51.31 g
SPECIFIC GRAVITY: 2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	63.7 pcf	
DRY UNIT WEIGHT:	40.2 pcf	40.5 pcf
VOID RATIO:	3.22	3.19

TEST DATA:

NORMAL STRESS: 2.0 ksf
STRAIN RATE: 0.012 in/min
PEAK SHEAR STRESS: 2.17 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.80	0	0.0	0.00	0.00
0.66	0.001	1.795	31	10.9	0.32	0.16
1.05	0.0025	1.795	55	19.4	0.57	0.28
1.44	0.005	1.79	72	25.3	0.74	0.37
2.16	0.010	1.79	105	37.0	1.09	0.54
3.06	0.015	1.79	154	54.2	1.59	0.80
3.78	0.020	1.79	186	65.5	1.92	0.96
4.44	0.025	1.79	206	72.5	2.13	1.07
5.46	0.0375	1.74	202	71.1	2.09	1.04
6.50	0.050	1.69	203	71.5	2.10	1.05
8.58	0.075	1.62	204	71.8	2.11	1.05
10.68	0.100	1.55	204	71.8	2.11	1.05
12.82	0.125	1.48	206	72.5	2.13	1.07
14.88	0.150	1.43	206	72.5	2.13	1.07
16.96	0.175	1.38	208	73.2	2.15	1.08
19.10	0.200	1.34	208	73.2	2.15	1.08
21.24	0.225	1.30	210	73.9	2.17	1.09
23.30	0.250	1.28	210	73.9	2.17	1.09
25.36	0.275	1.25	209	73.6	2.16	1.08
27.42	0.300	1.20	203	71.5	2.10	1.05

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	C-2	DATE:	08/17/99
SAMPLE:	U-1C1	TESTED BY:	ACS
DEPTH:	5.7 ft	CHECKED:	TKC
DESCRIPTION:	Clayey SILT (MH)		

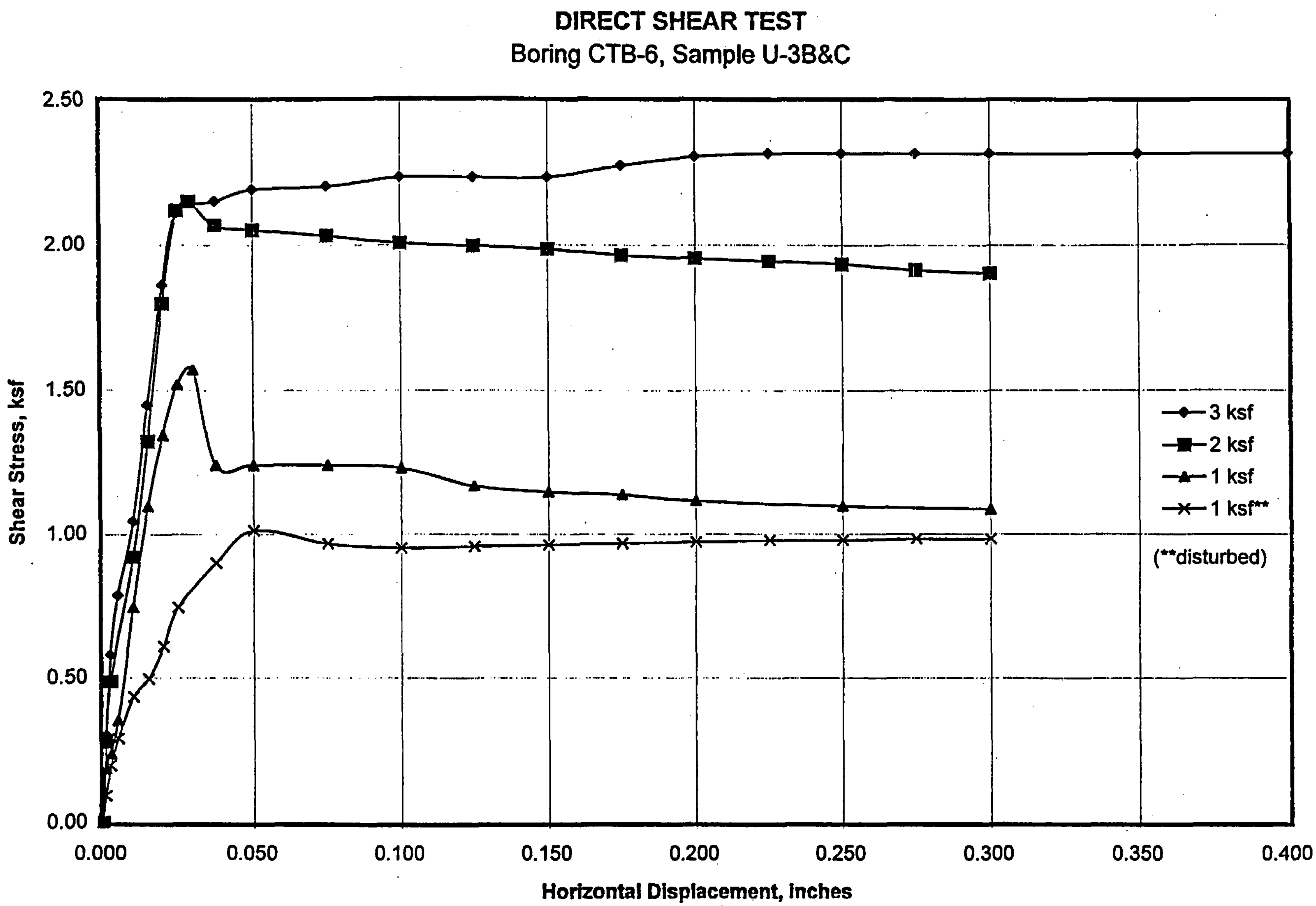
HEIGHT:	0.99 inches	WATER CONTENT:	55.7 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	56.85 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

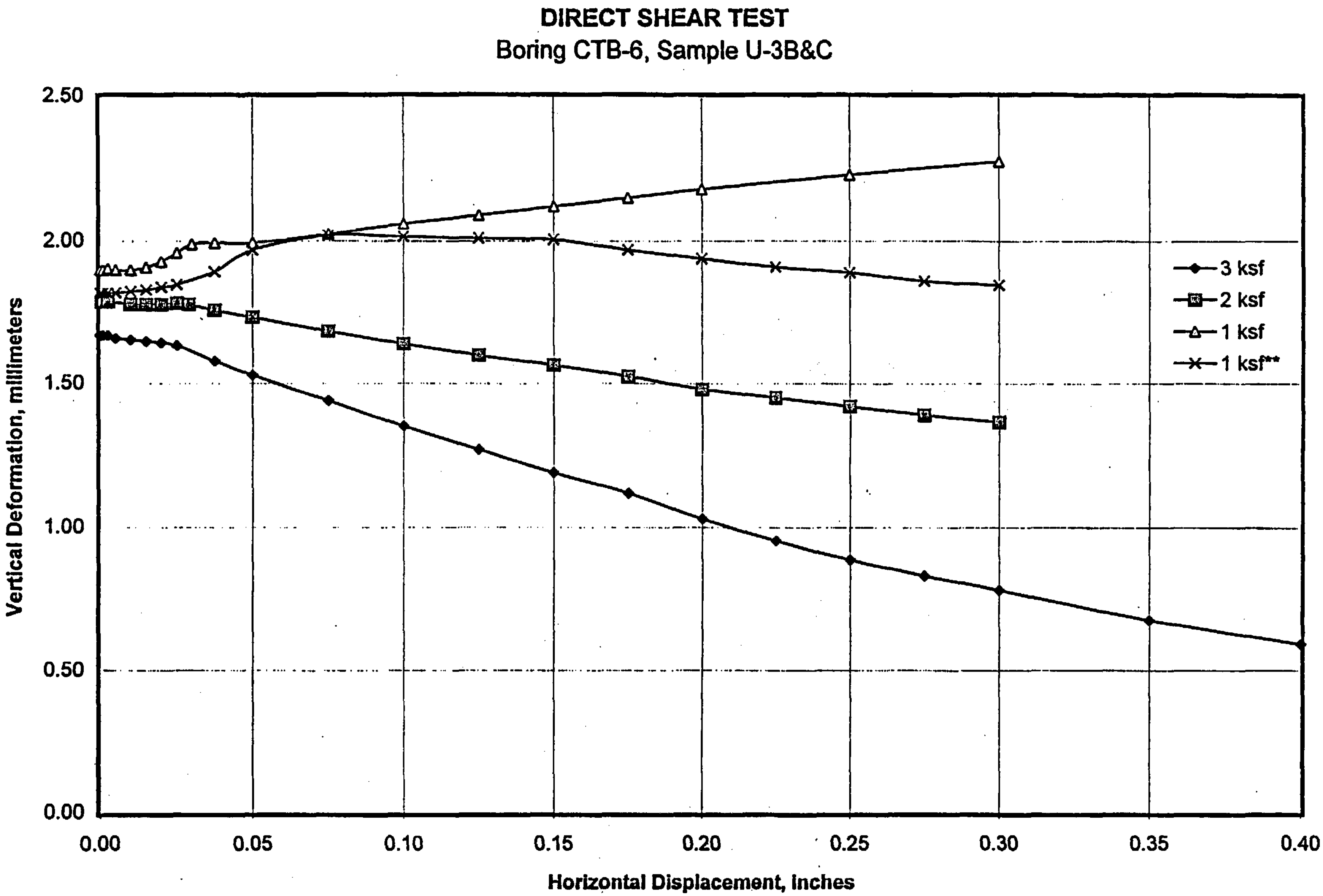
	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	69.4 pcf	
DRY UNIT WEIGHT:	44.5 pcf	45.1 pcf
VOID RATIO:	2.81	2.76

TEST DATA:

NORMAL STRESS:	3.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	2.60 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.675	0	0.0	0.00	0.00
0.69	0.001	1.675	32	11.3	0.33	0.11
1.20	0.0025	1.67	64	22.5	0.66	0.22
1.53	0.005	1.66	74	26.0	0.77	0.26
2.27	0.010	1.655	102	35.9	1.05	0.35
2.95	0.015	1.65	132	46.5	1.37	0.46
3.75	0.020	1.64	172	60.5	1.78	0.59
4.41	0.025	1.63	200	70.4	2.07	0.69
5.81	0.0375	1.59	226	79.6	2.34	0.78
6.95	0.050	1.55	237	83.4	2.45	0.82
9.15	0.075	1.48	246	86.6	2.54	0.85
11.30	0.100	1.41	248	87.3	2.56	0.85
13.44	0.125	1.33	250	88.0	2.59	0.86
15.50	0.150	1.26	251	88.4	2.60	0.87
17.62	0.175	1.17	251	88.4	2.60	0.87
19.74	0.200	1.11	251	88.4	2.60	0.87
21.87	0.225	1.02	251	88.4	2.60	0.87
23.98	0.250	0.94	251	88.4	2.60	0.87
26.04	0.275	0.86	251	88.4	2.60	0.87
28.12	0.300	0.80	251	88.4	2.60	0.87


DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C
Shear Stress, ksf
2.50
2.00
1.50
1.00
0.50
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.400
Horizontal Displacement, inches
3 ksf
2 ksf
1 ksf
1 ksf**
(**disturbed)



DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C
Vertical Deformation, millimeters
Horizontal Displacement, inches
2.50
2.00
1.50
1.00
0.50
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
3 ksf
2 ksf
1 ksf
1 ksf**

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	08/12/99
SAMPLE:	U-3B1	TESTED BY:	ACS
DEPTH:	7.2 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	61.7 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	58.95 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	74.7 pcf	
DRY UNIT WEIGHT:	46.2 pcf	46.5 pcf
VOID RATIO:	2.68	2.65

TEST DATA:

NORMAL STRESS:	1.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	1.01 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.82	0	0.0	0.00	0.00
0.29	0.001	1.82	9	3.2	0.09	0.09
	0.0025	1.82	19	6.7	0.20	0.20
0.89	0.005	1.82	28	9.9	0.29	0.29
1.50	0.010	1.825	42	14.8	0.43	0.43
1.98	0.015	1.83	48	16.9	0.50	0.50
2.50	0.020	1.84	59	20.8	0.61	0.61
3.06	0.0250	1.85	72	25.3	0.74	0.74
4.48	0.0375	1.895	87	30.6	0.90	0.90
5.67	0.050	1.97	98	34.5	1.01	1.01
7.77	0.075	2.02	93.5	32.9	0.97	0.97
9.90	0.100	2.015	92	32.4	0.95	0.95
12.04	0.125	2.01	92.5	32.6	0.96	0.96
14.12	0.150	2.005	93	32.7	0.96	0.96
16.24	0.175	1.97	93.5	32.9	0.97	0.97
18.32	0.200	1.94	94	33.1	0.97	0.97
20.44	0.225	1.91	94.5	33.3	0.98	0.98
22.50	0.250	1.89	94.5	33.3	0.98	0.98
24.62	0.275	1.86	95	33.4	0.98	0.98
26.69	0.300	1.845	95	33.4	0.98	0.98

NOTE: Soil sample taken from near the top of the tube was disturbed.

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	08/12/99
SAMPLE:	U-3B3	TESTED BY:	ACS
DEPTH:	7.5 ft	CHECKED:	TLC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	81.3 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	64.79 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	81.9 pcf	
DRY UNIT WEIGHT:	50.7 pcf	51.2 pcf
VOID RATIO:	2.35	2.32

TEST DATA:

NORMAL STRESS:	2.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	2.15 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.79	0	0.0	0.00	0.00
0.56	0.001	1.795	27	9.5	0.28	0.14
0.87	0.0025	1.79	47	16.5	0.49	0.24
1.97	0.010	1.78	89	31.3	0.92	0.46
2.78	0.015	1.78	128	45.1	1.32	0.66
3.61	0.020	1.78	174	61.2	1.80	0.90
4.36	0.025	1.785	205	72.2	2.12	1.06
	0.029	1.78	208	73.2	2.15	1.08
5.38	0.0375	1.76	200	70.4	2.07	1.03
6.41	0.050	1.735	198.5	69.9	2.05	1.03
8.49	0.075	1.685	196.5	69.2	2.03	1.02
10.58	0.100	1.64	194	68.3	2.01	1.00
12.70	0.125	1.60	193	67.9	2.00	1.00
14.72	0.150	1.565	192	67.6	1.99	0.99
16.82	0.175	1.525	190	66.9	1.97	0.98
18.91	0.200	1.48	189	66.5	1.95	0.98
21.00	0.225	1.45	188	66.2	1.94	0.97
23.07	0.250	1.42	187	65.8	1.93	0.97
25.15	0.275	1.39	185	65.1	1.91	0.96
27.24	0.300	1.365	184	64.8	1.90	0.95

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	08/13/99
SAMPLE:	U-3B4	TESTED BY:	ACS
DEPTH:	7.7 ft	CHECKED:	TLC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	60.3 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	64.1 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	80.5 pcf	
DRY UNIT WEIGHT:	50.2 pcf	50.9 pcf
VOID RATIO:	2.38	2.34

TEST DATA:

NORMAL STRESS:	3.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	2.32 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.67	0	0.0	0.00	0.00
0.70	0.001	1.67	29	10.2	0.30	0.10
1.18	0.0025	1.67	56	19.7	0.58	0.19
1.60	0.005	1.66	76	26.8	0.79	0.26
2.30	0.010	1.655	101	35.6	1.04	0.35
3.09	0.015	1.65	140	49.3	1.45	0.48
3.90	0.020	1.645	180	63.4	1.86	0.62
4.63	0.025	1.635	204	71.8	2.11	0.70
5.80	0.0375	1.58	208	73.2	2.15	0.72
6.88	0.050	1.53	212	74.6	2.19	0.73
9.02	0.075	1.44	213	75.0	2.20	0.73
11.10	0.100	1.35	216	76.0	2.23	0.74
13.30	0.125	1.27	216	76.0	2.23	0.74
15.36	0.150	1.19	216	76.0	2.23	0.74
17.49	0.175	1.12	220	77.4	2.28	0.76
19.60	0.200	1.03	223	78.5	2.31	0.77
21.70	0.225	0.955	224	78.8	2.32	0.77
23.79	0.250	0.89	224	78.8	2.32	0.77
25.88	0.275	0.835	224	78.8	2.32	0.77
27.98	0.300	0.785	224	78.8	2.32	0.77
32.25	0.350	0.68	224	78.8	2.32	0.77
36.46	0.400	0.595	224	78.8	2.32	0.77

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-6	DATE:	08/13/99
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	7.8 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT (MH)		

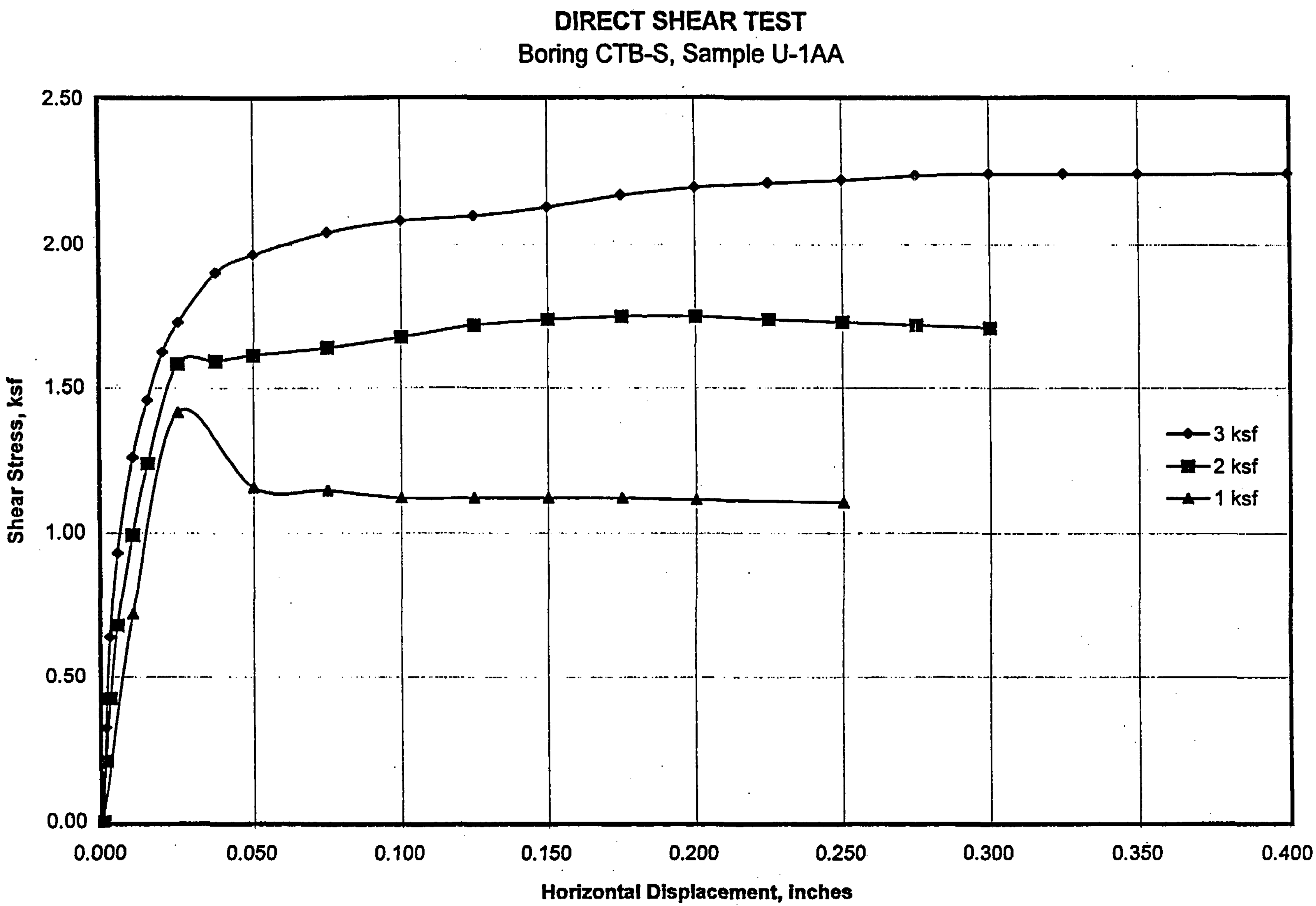
HEIGHT:	0.99 inches	WATER CONTENT:	56.6 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	72.05 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	88.5 pcf	
DRY UNIT WEIGHT:	56.4 pcf	56.7 pcf
VOID RATIO:	2.01	2.00

TEST DATA:

NORMAL STRESS:	1.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	1.57 ksf

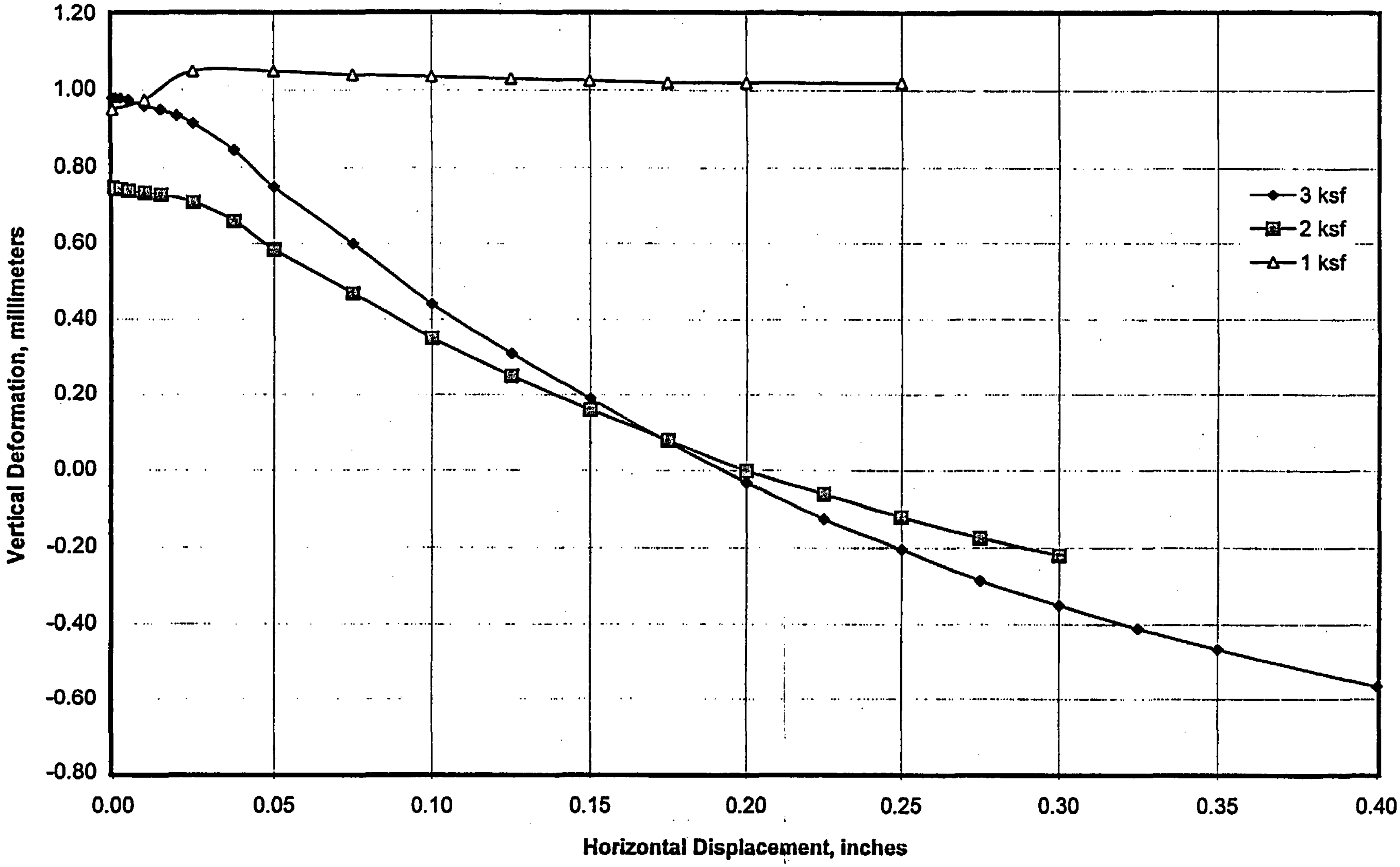
ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.90	0	0.0	0.00	0.00
0.50	0.001	1.90	18	6.3	0.19	0.19
0.72	0.0025	1.905	23	8.1	0.24	0.24
0.92	0.005	1.90	34	12.0	0.35	0.35
1.86	0.010	1.90	72	25.3	0.74	0.74
2.60	0.015	1.91	106	37.3	1.10	1.10
3.22	0.020	1.93	130	45.8	1.34	1.34
3.82	0.0250	1.96	147	51.7	1.52	1.52
	0.030	1.99	152	53.5	1.57	1.57
4.70	0.0375	1.995	120	42.2	1.24	1.24
5.74	0.050	1.995	120	42.2	1.24	1.24
7.84	0.075	2.025	120	42.2	1.24	1.24
9.92	0.100	2.06	119	41.9	1.23	1.23
11.96	0.125	2.09	113	39.8	1.17	1.17
14.00	0.150	2.12	111	39.1	1.15	1.15
16.09	0.175	2.15	110	38.7	1.14	1.14
18.18	0.200	2.18	108	38.0	1.12	1.12
22.30	0.250	2.23	106	37.3	1.10	1.10
26.50	0.300	2.275	105	37.0	1.09	1.09



DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA
2.50
2.00
1.50
1.00
0.50
0.00
Shear Stress, ksf
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.400
Horizontal Displacement, inches
3 ksf
2 ksf
1 ksf

DIRECT SHEAR TEST

Boring CTB-S, Sample U-1AA



DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	08/10/99
SAMPLE:	U-1AA1	TESTED BY:	ACS
DEPTH:	5.4 ft	CHECKED:	THC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	86.8 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	48.40 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	70.9 pcf	
DRY UNIT WEIGHT:	37.9 pcf	38.1 pcf
VOID RATIO:	3.48	3.45

TEST DATA:

NORMAL STRESS:	1.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	1.42 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	0.950	0.0	0.0	0.00	0.00
1.74	0.010	0.975	70.0	24.6	0.72	0.72
3.90	0.0250	1.050	137.0	48.2	1.42	1.42
6.05	0.050	1.050	112.0	39.4	1.16	1.16
8.28	0.075	1.040	111.0	39.1	1.15	1.15
10.52	0.100	1.035	108.5	38.2	1.12	1.12
12.77	0.125	1.030	108.5	38.2	1.12	1.12
15.00	0.1500	1.025	108.5	38.2	1.12	1.12
17.20	0.175	1.020	108.5	38.2	1.12	1.12
19.36	0.200	1.020	108.0	38.0	1.12	1.12
23.60	0.250	1.020	107.0	37.7	1.11	1.11

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	08/11/99
SAMPLE:	U-1AA2	TESTED BY:	ACS
DEPTH:	5.3 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	84.6 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	50.50 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	73.1 pcf	
DRY UNIT WEIGHT:	39.6 pcf	39.9 pcf
VOID RATIO:	3.29	3.25

TEST DATA:

NORMAL STRESS:	2.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	1.75 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	0.750	0.0	0.0	0.00	0.00
1.00	0.001	0.745	20.0	7.0	0.21	0.10
1.52	0.0025	0.745	41.0	14.4	0.42	0.21
2.06	0.005	0.740	66.0	23.2	0.68	0.34
2.87	0.010	0.735	96.0	33.8	0.99	0.50
3.57	0.015	0.730	120.0	42.2	1.24	0.62
4.85	0.025	0.710	153.0	53.9	1.58	0.79
	0.0375	0.660	154.0	54.2	1.59	0.80
7.19	0.050	0.585	156.0	54.9	1.61	0.81
9.50	0.075	0.470	158.5	55.8	1.64	0.82
11.83	0.100	0.350	162.0	57.0	1.68	0.84
14.10	0.125	0.250	166.0	58.4	1.72	0.86
16.30	0.150	0.160	168.0	59.1	1.74	0.87
18.56	0.175	0.080	169.0	59.5	1.75	0.87
20.80	0.200	0.000	169.0	59.5	1.75	0.87
23.00	0.225	-0.060	168.0	59.1	1.74	0.87
25.20	0.250	-0.120	167.0	58.8	1.73	0.86
27.35	0.275	-0.175	166.0	58.4	1.72	0.86
29.50	0.300	-0.220	165.0	58.1	1.71	0.85

DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	08/11/99
SAMPLE:	U-1AA3	TESTED BY:	ACS
DEPTH:	5.1 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT (MH)		

HEIGHT:	0.99 inches	WATER CONTENT:	80.9 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	53.35 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	75.7 pcf	
DRY UNIT WEIGHT:	41.8 pcf	42.6 pcf
VOID RATIO:	3.06	2.98

TEST DATA:

NORMAL STRESS:	3.0 ksf
STRAIN RATE:	0.012 in/min
PEAK SHEAR STRESS:	2.24 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	0.980	0.0	0.0	0.00	0.00
0.56	0.001	0.980	31.0	10.9	0.32	0.11
1.03	0.0025	0.980	62.0	21.8	0.64	0.21
1.50	0.005	0.975	90.0	31.7	0.93	0.31
2.21	0.010	0.960	122.0	42.9	1.26	0.42
2.74	0.015	0.950	141.0	49.6	1.46	0.49
3.24	0.020	0.935	157.0	55.3	1.62	0.54
3.72	0.025	0.915	167.0	58.8	1.73	0.58
4.87	0.0375	0.845	184.0	64.8	1.90	0.63
5.88	0.050	0.750	190.0	66.9	1.97	0.66
7.82	0.075	0.600	197.5	69.5	2.04	0.68
9.80	0.100	0.440	201.5	70.9	2.08	0.69
11.78	0.125	0.310	203.0	71.5	2.10	0.70
13.76	0.150	0.190	206.0	72.5	2.13	0.71
15.75	0.175	0.075	210.0	73.9	2.17	0.72
17.80	0.200	-0.030	212.5	74.8	2.20	0.73
19.90	0.225	-0.125	214.0	75.3	2.21	0.74
22.05	0.250	-0.205	215.0	75.7	2.22	0.74
24.15	0.275	-0.285	216.5	76.2	2.24	0.75
26.30	0.300	-0.350	217.0	76.4	2.24	0.75
28.45	0.325	-0.410	217.0	76.4	2.24	0.75
30.55	0.350	-0.465	217.0	76.4	2.24	0.75
34.80	0.400	-0.565	217.0	76.4	2.24	0.75

PROJECT DOCUMENT INDEPENDENT REVIEW CHECKLIST USING ASME N45.2 OR NQA-1 FORMAT

Project No. 05996.02
Job Book File Location Q2.9

Type of Document

- Report: Supplemental Geotechnical Laboratory Testing - August 1999
- Design Criteria ______ Rev. No. ______
- Project Specification ______ Rev. No. ______
- Project Diagram ______ Rev. No. ______
- License Document ______ Rev. No. ______
- List Drawing/Diagrams and Revision Number

This sheet may be used for more than 1 diagram (list or reference all diagrams below)

	Yes	No	N/A
Were the inputs correctly selected and incorporated into the design?			✓
Are assumptions necessary to perform the design activity adequately described and reasonable?			✓
Where necessary, are the assumptions identified for subsequent reverifications when the detailed design activities are completed?			✓
Are the appropriate quality and quality assurance requirements specified?	✓		
Are the applicable codes, standards, and regulatory requirements, including applicable issues and addenda, properly identified and are their requirements for design met?	✓		
Have applicable construction and operating experience been considered?			✓
Have the design interface requirements been satisfied?			✓
Was an appropriate design method used?			✓
Is the output reasonable compared to inputs?	✓		

Private Fuel Storage Facility

JO 05996.02
Report: Supplemental Geotechnical Laboratory Testing - August 1999

	Yes	No	N/A
Are the specified parts, equipment, and processes suitable for the required applications?	✓		
Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?			✓
Have adequate maintenance features and requirements been specified?			✓
Are accessibility and other design provisions adequate for performance of needed maintenance and repair?			✓
Has adequate accessibility been provided to perform the inservice inspection expected to be required during the plant life?			✓
Has the design properly considered radiation exposure to the public and plant personnel?			✓
Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?			✓
Have adequate preoperational and subsequent periodic test requirements been appropriately specified?			✓
Are adequate handling, storage, cleaning, and shipping requirements specified?			✓
Are adequate identification requirements specified?	✓		
Are requirements for record preparation, review, approval, retention, etc., adequately specified?			✓
Is the design output reasonable compared to the design inputs?			✓
Are the necessary design input and verification requirements for interfacing organizations specified in the design documents or in supporting procedures as instructions?			✓

[This checklist meets the requirements for both N45.2 and NQA-1 for design verification requirements]

Thomas Y. Chang	Thomas Y. Chang	8-19-99
Printed Name	Signature	Date

APPENDIX 2A

ATTACHMENT 8

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
NOVEMBER 1999

SWEC Project No. 05996.02

SUPPLEMENTAL GEOTECHNICAL LABORATORY TESTING NOVEMBER 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

STONE & WEBSTER

SWEC Project No. 05996.02

SUPPLEMENTAL
GEOTECHNICAL LABORATORY TESTING
NOVEMBER 1999

Prepared for:
Private Fuel Storage, LLC
Private Fuel Storage Facility
Skull Valley, Utah

Prepared by:	Alan C. Smith	12/10/99 Date
Reviewed by:	Thomas Y. Chang	12/10/99 Date
Independently Reviewed by:	Thomas Y. Chang	12/10/99 Date
Approved by:	J L Cooper	12/10/99 Date

QUALITY ASSURANCE CATEGORY I
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

INTRODUCTION

The primary objective of the laboratory testing program was to determine the apparent cohesion and friction angle (total strength parameters) of the soils at the base of the mat foundations for use in the sliding stability analyses and bearing capacity. The direct shear and the consolidated-undrained compression test specimens were obtained from preserved tube sections of thin-walled tube samples obtained from prior testing programs. Testing began on November 8, 1999 and ended on November 12, 1999.

The tests performed included classification, water content, Atterberg limits, direct shear, and consolidated-undrained triaxial compression. They were conducted in accordance with the following American Society for Testing and Materials standards.

D-2216	1992	Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
D-3080	1998	Test Methods for Direct Shear Test of Soils Under Consolidated Drained Conditions
D-4318	1995a	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D-4767	1995	Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils

All laboratory equipment and materials used to conduct this testing program were calibrated and maintained in accordance with the requirements of the Stone & Webster Standard Nuclear Quality Assurance Program.

TEST RESULTS

A total of one additional direct shear test was performed on thin-walled tube sample U-1 from Boring CTB-S and two additional consolidated-undrained triaxial compression tests were performed on thin-walled tube sample U-2 from Boring B-1. The test results are shown in Table 1, along with the results from previous direct shear tests (from Attachment 7) and consolidated-undrained triaxial tests (Attachment 5). A plot of the normal stress vs peak shear stress for each

tube sample is shown after Table 1. The plot of horizontal displacement vs shear stress and data for the direct shear test are presented in Appendix A. The axial strain vs shear stress plots and data from the triaxial compression tests are presented in Appendix B.

For direct shear tests, the samples were trimmed into a nominal 2.5-inch diameter ring and placed in the direct shear apparatus. The samples were not inundated because the soils at the site are not expected to be saturated during the life of the facility. A normal load was applied and the deformation measured. Primary consolidation occurred prior to 1 minute. After at minimum of 90 minutes, the sample was sheared at a displacement rate of 18 mm/hr. The test continued until the shear load peaked and remained constant or started decreasing after 0.25 inches of displacement. The sample was removed and oven dried.

For consolidated-undrained triaxial compression tests, the samples were setup in a triaxial cell and consolidated using a confining pressure of 0 and 1 ksf. They were sheared undrained after consolidating for at least 1 hour. Since the *in situ* soils are not expected to be saturated throughout the life of the proposed facility, these samples were not saturated prior to shearing.

The trimmings from each test were retained and used to prepare an Atterberg limits test sample. For the previous direct shear tests, the trimmings from the same tube were combined for one Atterberg limits test. The results are shown in Table 1.

LABORATORY TESTING PROGRAM
GENERAL INFORMATION

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET 1 OF 3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 WP 345W	DATE					Alan C Smith	11/8/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: JL Cooper	RECEIVED BY: ACS
FEATURE OR PHASE OF PROJECT: Mat Foundations		NEAREST CITY OR TOWN					NEAREST AIRPORT HANDLING FREIGHT	

PROJECT MANAGER	LEAD GEOTECHNICAL ENGINEER	SUPERVISOR OF FIELD WORK	SUPERVISOR OF OFFICE WORK	CONTACT FOR LABORATORY WORK
J.L. Donnell	PJ Trudeau	RP Gillespie	GR Doton	PJT/TY Chang
DATE PROGRAM ASSIGNED 11/8/99	DATE TESTING TO START 11/8/99	DATE BORING LOG DATA NEEDED	DATE ALL TEST DATA AVAILABLE	DATE ALL FINAL PLOTS FORWARDED

TYPES OF STRUCTURES INVOLVED:
Mat Supported Storage Pads and Canister Transfer Building

PARTICULAR CRITICAL FEATURES OF STRUCTURES:
Soils at Base of Mat Foundations

TYPES OF BEHAVIOR TO BE ANALYZED:
Sliding Stability and Bearing Capacity

TYPICAL ANTICIPATED LOADING STRESSES:
~2 ksf at base of pads

PRIMARY OBJECTIVES OF LABORATORY TESTS:
Determine c & ϕ at Base of Mat in soils supporting mat

GENERAL MAGNITUDE OF TESTING PROGRAM:
Small

NUMBER OF DISTURBED SAMPLES:

NUMBER OF UNDISTURBED SAMPLES:

TYPE OF REPORT TO CONTAIN RESULTS:
Supplemental Geotechnical Laboratory Testing November 1999

UNITS FOR REPORTING STRESSES:
ksf

ADDITIONAL:
All Laboratory Equipment and Testing Procedures shall be carried out under a QA Category I program that meets the applicable requirements of 10CFR50, Appendix B, and 10CFR72.

LABORATORY TESTING PROGRAM
SPECIFIC TEST REQUIREMENTS

STONE & WEBSTER
ENGINEERING CORPORATION

CLIENT: Private Fuel Storage, LLC	J.O. NUMBER: 05996.02 WP 3456	REVISION	1	2	3	4	PREPARED BY: A C Smith	DATE PREPARED: 11/8/99
SITE: PFSF Skull Valley, UT		DATE					APPROVED BY: JL Cooper	RECEIVED BY: ACS
		REVISED BY						

NOTE	COL. NO.	TYPE OF TEST	REQUIREMENTS OF TYPE OF TEST AND PROCEDURES TO BE USED (SIZE OF SPECIMENS, RATES OF STRAIN, CONFINING PRESSURES, CYCLES OF RECOMPRESSION ETC.)
A	ALL	ALL TESTS	SELECTION OF NECESSARY AND APPROPRIATE DETERMINATIONS AND/OR TEST PROCEDURES TO BE MADE BY LABORATORY PERSONNEL.
B	I	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION ONLY; NO INDEX PROPERTIES TO BE DETERMINED.
C	I	CLASSIFICATION	VISUAL-MANUAL CLASSIFICATION TO BE SUPPLEMENTED BY DETERMINATIONS OF SPECIFIC INDEX DETERMINATIONS INDICATED IN SUBSEQUENT COLUMNS.
D	3	ATTERBERG LIMITS	FOR CLASSIFICATION PURPOSES ONLY; ONE-POINT LIQUID LIMIT DETERMINATION ACCEPTABLE.
E	5	GRADATION ANALYSES	SIEVE ANALYSIS ONLY; NO HYDROMETER ANALYSIS REGARDLESS OF PERCENT FINES.
F	10	DIRECT SHEAR	$\sigma_N = 0.25$ ksf
G	9	CU	Consolidation Pressure = 0 and 1.0 ksf

LABORATORY TESTING PROGRAM
UTILIZATION OF SAMPLE

STONE & WEBSTER
ENGINEERING CORPORATION

SHEET
3 of 3

CLIENT	J.O. NUMBER	REVISION	1	2	3	4	PREPARED BY	DATE PREPARED
Private Fuel Storage, LLC	05996.02 WP 3454	DATE					AC Smith	11/8/99
SITE: PFSF Skull Valley, UT		REVISED BY					APPROVED BY: JLCooper	RECEIVED BY: AC S

LETTERS BELOW SPECIFIC TYPES OF TESTS CORRESPOND TO SIMILARLY LETTERED NOTES GIVEN ON SHEET 2

BORING NO.	SAMPLE TYPE	SAMPLE NO.	DEPTH (FT.)	1 CLASSIFICATION	2 WATER CONTENT	3 ATTERBERG LIMITS	4 PERCENT FINES	5 GRADATION ANALYSES	6 SPECIFIC GRAVITY	7 UNIT WEIGHT	8 UNDRAINED COMPRESSION	9 CONSOLIDATED TRIAXIAL	10 DRAINED DIRECT SHEAR	11 CONSOLIDATION	12 COMPACTION	13 RELATIVE DENSITY	14 PERMEABILITY	15	16	17	REMARKS OR REQUIREMENTS FOR TESTING OF SPECIFIC SAMPLES (REFER TO COL. NO. FOR TYPE OF TEST)
CTB-5	U	1				X							X								σ_N = 0.25 ksf
B-2	U	2				X						X									σ_c = 0+1 ksf

/ TEST TO BE PERFORMED   ⊻ TEST IN PROGRESS   X TEST COMPLETED & CHECKED   TEST CANCELLED

TABLE 1

Direct Shear Test Results

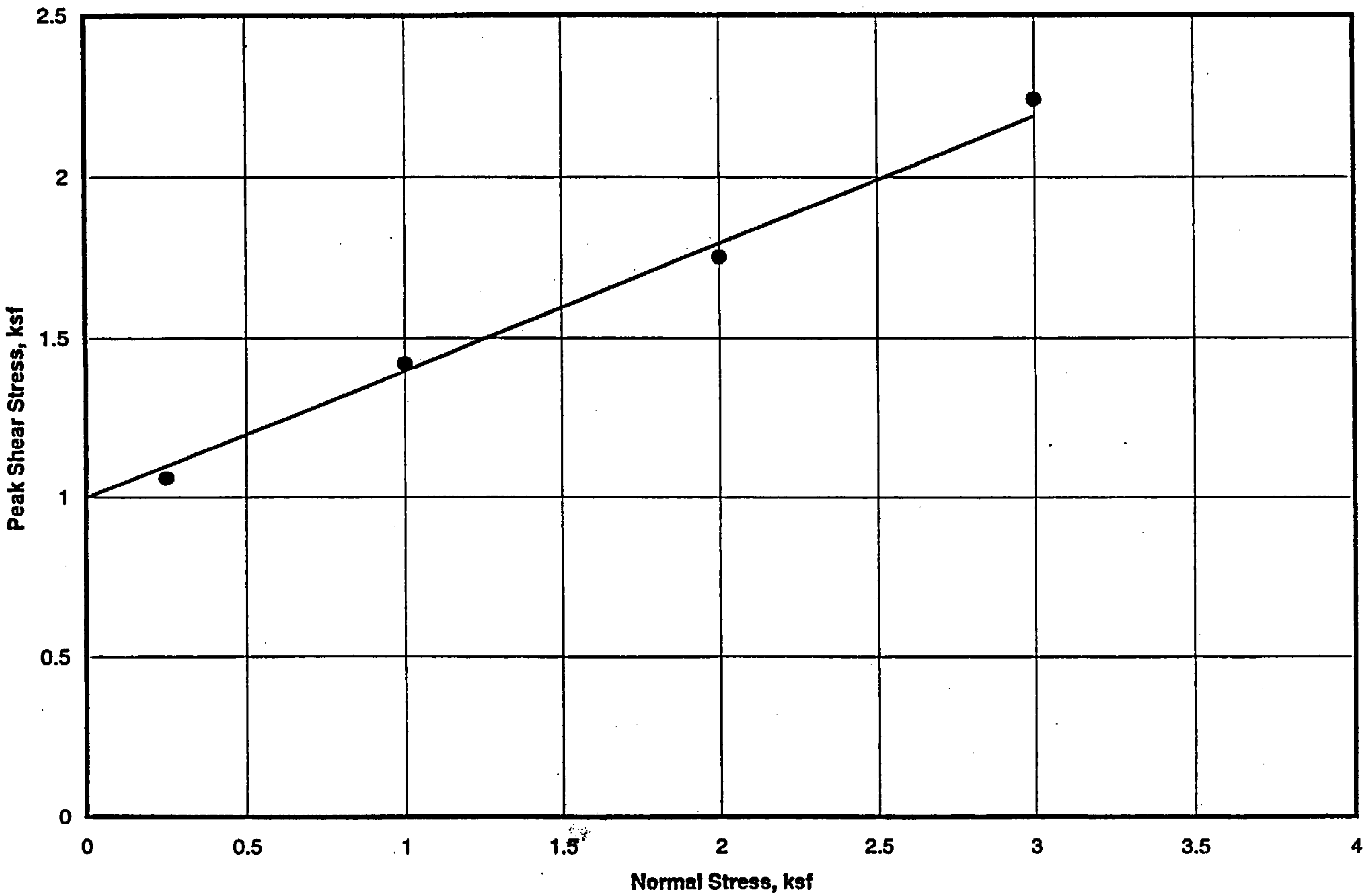
Boring	Sample	Depth (ft)	Elevation (ft)	Atterberg Limits LL	PL	PI	USC Code	Water Content (%)	Initial γ_m (pcf)	γ_d (pcf)	Void Ratio	After Consolid. γ_d (pcf)	Void Ratio	Normal Stress (ksf)	Peak Shear (ksf)	Cohesion (ksf)	Tan ϕ	ϕ (deg)
CTB-S	U-1AA3	5.1	4469.4					80.9	75.7	41.8	3.06	42.6	2.98	3.0	2.24			
CTB-S	U-1AA2	5.3	4469.2	82.7	44.8	37.9	MH	84.6	73.1	39.6	3.29	39.9	3.25	2.0	1.75	1.00	0.397	21.6
CTB-S	U-1AA1	5.4	4469.1					86.8	70.9	37.9	3.48	38.1	3.45	1.0	1.42			
CTB-S	U-1C	6.1	4468.4	79.0	44.8	34.2	MH	69.2	78.8	46.5	2.65	46.6	2.64	0.25	1.05			

Consolidated-Undrained Triaxial Compression Test Results

Boring	Sample	Depth (ft)	Elevation (ft)	Atterberg Limits LL	PL	PI	USC Code	Water Content (%)	Initial γ_m (pcf)	γ_d (pcf)	Void Ratio	After Consolid. γ_d (pcf)	Void Ratio	Confin'g Stress (ksf)	Peak Shear (ksf)	Cohesion (ksf)	Tan ϕ	ϕ (deg)
B-1	U-2B	5.3	4459.5	80.6	40.9	39.7	MH	52.9	70.8	46.3	2.67	46.3	2.67	1.0	2.21	1.4	0.390	21.3
B-1	U-2C	5.9	4458.9	66.1	33.4	32.7	MH	47.1	79.3	53.9	2.15	53.9	2.15	0.0	2.03	1.15	0.390	21.3
B-1	U-2D	6.5	4458.3	59.8	34.7	25.1	MH	45.2	76.7	52.8	2.22	52.8	2.22	2.10	3.26	1.4	0.390	21.3

STONE & WEBSTER ENGINEERING CORPORATION

DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA&C



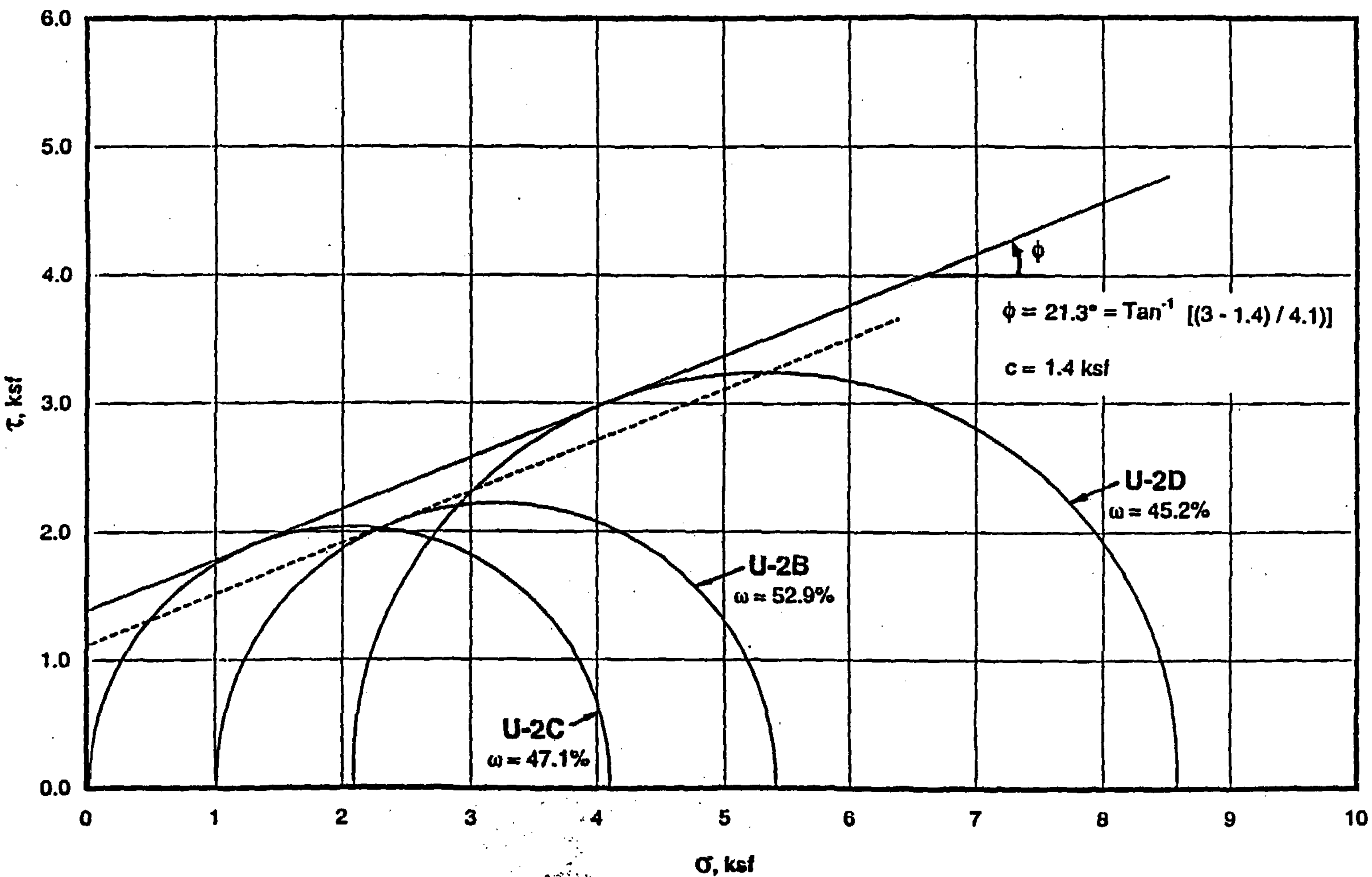
Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO 05996.02
November 1999

STONE & WEBSTER ENGINEERING CORPORATION

Total Stress Mohr's Circles

Boring B-1, Sample U-2

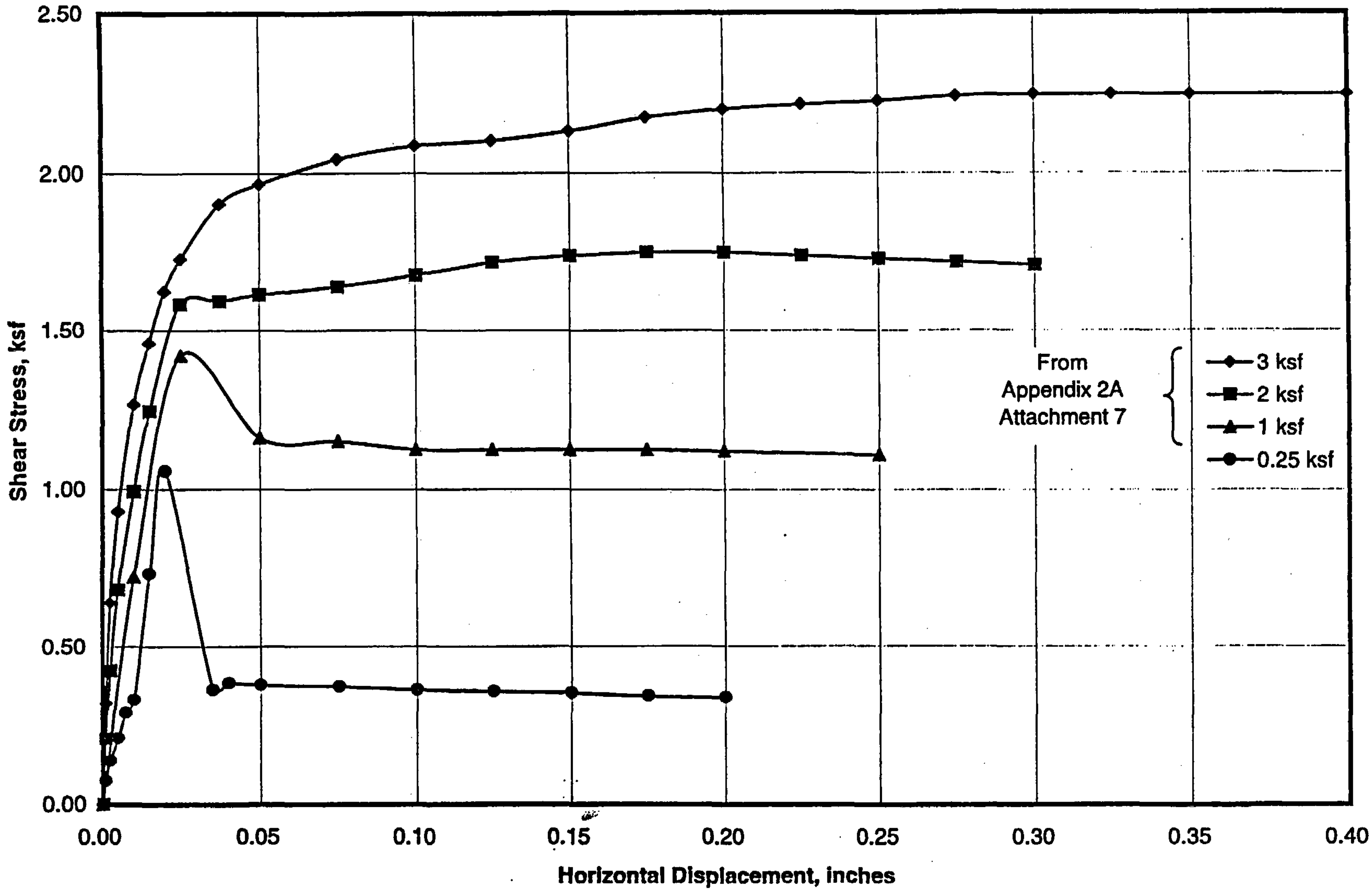


APPENDIX A

Direct Shear Test Plots and Data

STONE & WEBSTER ENGINEERING CORPORATION

DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA&C



DIRECT SHEAR TEST DATA

SAMPLE INFORMATION:

BORING:	CTB-S	DATE:	11/11/99
SAMPLE:	U-1C	TESTED BY:	ACS
DEPTH:	6.1 ft	CHECKED:	TLC
DESCRIPTION:	Clayey SILT (MH)		
HEIGHT:	0.99 inches	WATER CONTENT:	69.2 %
DIAMETER:	2.50 inches	DRY WEIGHT SOIL:	59.32 g
AREA:	4.90 sq. in.	SPECIFIC GRAVITY:	2.72

	INITIAL	AFTER CONSOLIDATION
MOIST UNIT WEIGHT:	78.8 pcf	
DRY UNIT WEIGHT:	46.5 pcf	46.8 pcf
VOID RATIO:	2.65	2.64

TEST DATA:

NORMAL STRESS: 1.0 ksf
STRAIN RATE: 0.012 in/min
PEAK SHEAR STRESS: 1.42 ksf

ELAPSED TIME min.	HORIZONTAL DISPLACEMENT in.	VERTICAL DEFORMATION mm	SHEAR FORCE div.	SHEAR FORCE lb.	SHEAR STRESS ksf	STRESS RATIO
0.00	0.000	1.00	0.0	0.0	0.00	0.00
0.23	0.001	1.00	7.0	2.5	0.07	0.07
0.55	0.0025	1.00	13.0	4.6	0.13	0.13
0.85	0.005	1.01	20.0	7.0	0.21	0.21
1.14	0.0075	1.025	28.0	9.9	0.29	0.29
1.42	0.010	1.03	32.0	11.3	0.33	0.33
2.27	0.015	1.10	71.0	25.0	0.73	0.73
3.12	0.020	1.21	102.0	35.9	1.05	1.05
3.90	0.035	1.20	35.0	12.3	0.36	0.36
4.30	0.040	1.20	37.0	13.0	0.38	0.38
5.15	0.050	1.24	36.5	12.8	0.38	0.38
7.35	0.075	1.31	36.0	12.7	0.37	0.37
9.46	0.100	1.38	35.0	12.3	0.36	0.36
11.60	0.125	1.46	34.5	12.1	0.36	0.36
13.73	0.150	1.50	34.0	12.0	0.35	0.35
15.87	0.175	1.55	33.0	11.6	0.34	0.34
18.02	0.200	1.58	32.5	11.4	0.34	0.34

APPENDIX B

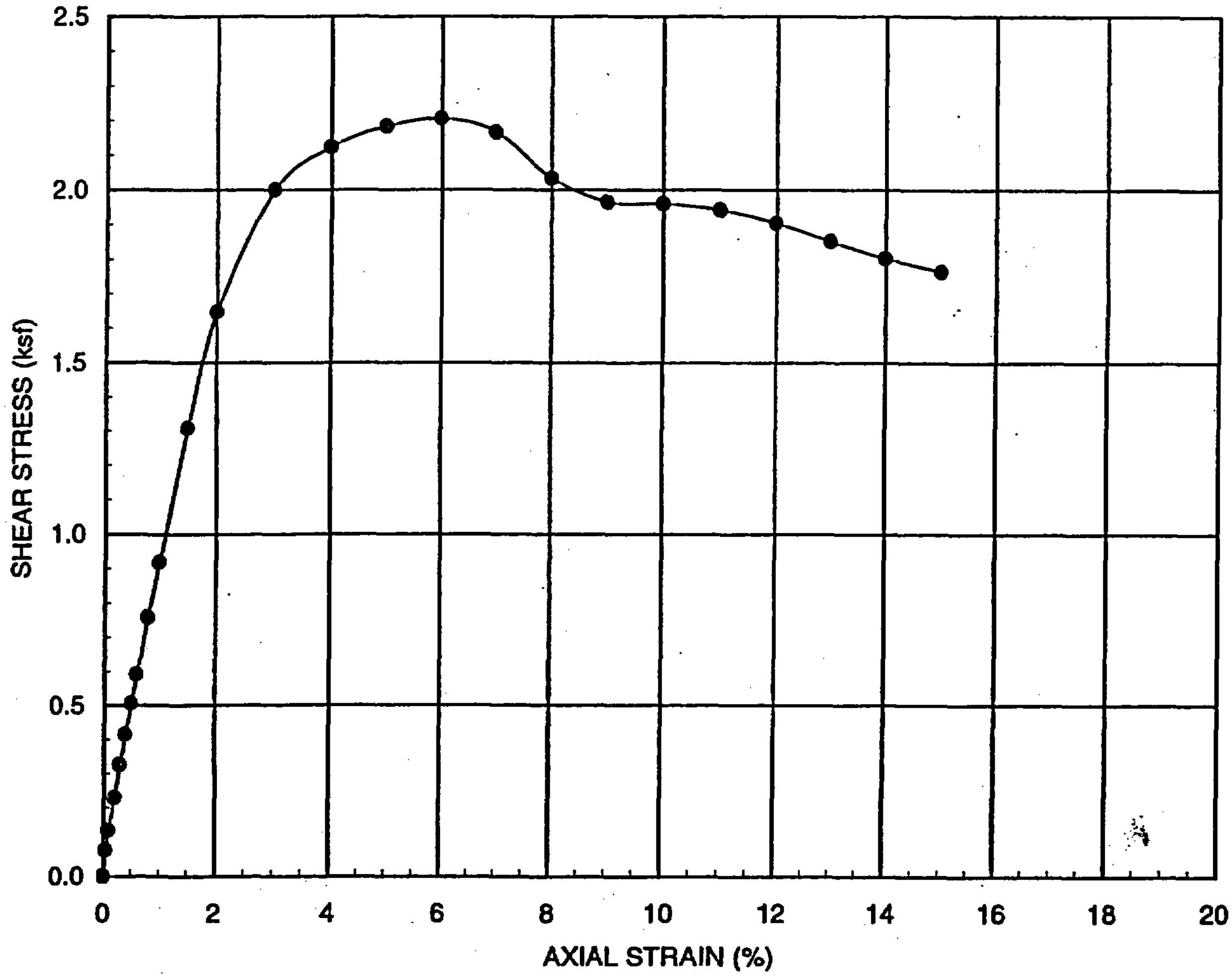
Consolidated Undrained Triaxial Test Plots and Data

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
November 1999

CONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2B	TESTED BY:	ACS
DEPTH:	5.3 ft	CHECKED:	TGC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.555 ft	WATER CONTENT:	52.9 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT:	46.3 pcf
AREA:	0.0440 ft^2	INITIAL VOID RATIO:	2.67

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.0 ksf		

UNDRAINED SHEAR STRENGTH:	2.21 ksf
COMPRESSIVE STRENGTH:	4.41 ksf
FAILURE STRAIN:	6.0 %

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2B	TESTED BY:	ACS
DEPTH:	5.3 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.555 ft	WATER CONTENT:	52.9 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT	46.3 pcf
AREA:	0.0440 ft^2	INITIAL VOID RATIO:	2.67

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	1.0 ksf		

UNDRAINED SHEAR STRENGTH: 2.21 ksf
COMPRESSIVE STRENGTH: 4.41 ksf
FAILURE STRAIN: 6.0 %

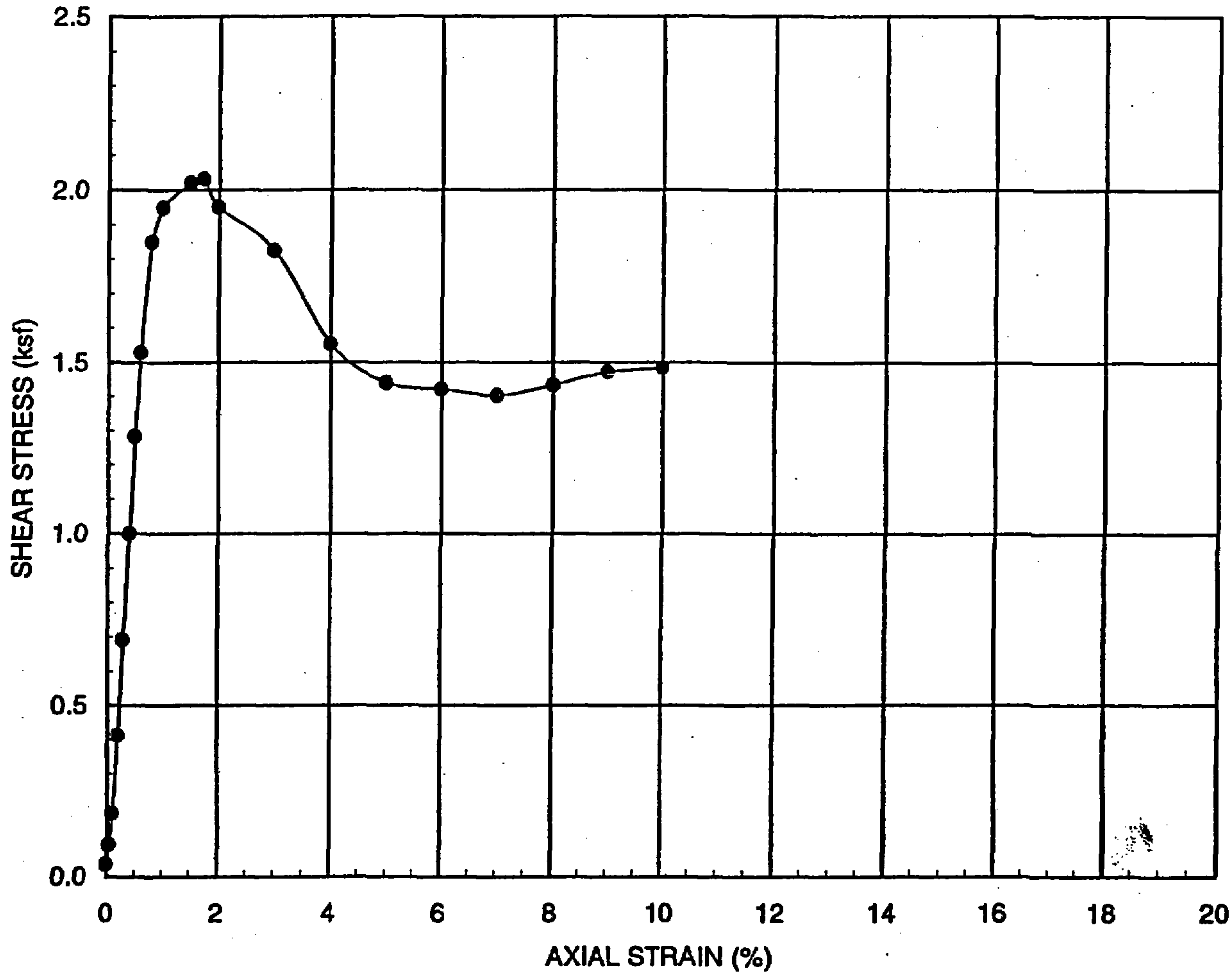
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.49	0.34	0.00	0.000	0.0440	0.00	0.00
0.57	2.50	0.05	0.007	0.0440	0.15	0.08
0.66	4.15	0.10	0.012	0.0440	0.27	0.13
0.83	6.83	0.20	0.020	0.0441	0.46	0.23
1.00	9.58	0.30	0.029	0.0441	0.65	0.33
1.17	12.12	0.40	0.037	0.0442	0.83	0.41
1.34	14.71	0.50	0.045	0.0442	1.01	0.51
1.51	17.16	0.60	0.052	0.0443	1.18	0.59
1.84	21.96	0.80	0.067	0.0443	1.52	0.76
2.18	26.58	1.00	0.082	0.0444	1.84	0.92
3.03	37.89	1.50	0.117	0.0447	2.62	1.31
3.87	47.87	2.00	0.148	0.0449	3.29	1.65
5.57	58.67	3.00	0.181	0.0453	4.00	2.00
7.26	62.90	4.00	0.195	0.0458	4.25	2.12
8.95	65.33	5.00	0.202	0.0463	4.37	2.18
10.64	66.71	6.00	0.206	0.0468	4.41	2.21
12.33	66.20	7.00	0.205	0.0473	4.33	2.17
14.03	62.85	8.00	0.194	0.0478	4.07	2.03
15.72	61.42	9.00	0.190	0.0483	3.93	1.97
17.41	61.94	10.00	0.192	0.0489	3.92	1.96
19.10	62.07	11.00	0.192	0.0494	3.88	1.94
20.79	61.55	12.00	0.190	0.0500	3.81	1.90
22.49	60.55	13.00	0.187	0.0506	3.70	1.85
24.18	59.66	14.00	0.184	0.0511	3.61	1.80
25.87	59.04	15.00	0.183	0.0517	3.53	1.78

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
November 1999

UNCONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	5.9 ft	CHECKED:	TLC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.550 ft	WATER CONTENT:	47.1 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT:	53.9 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	2.15

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	0.0 ksf		

UNDRAINED SHEAR STRENGTH:	2.03 ksf
COMPRESSIVE STRENGTH:	4.06 ksf
FAILURE STRAIN:	1.7 %

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	5.9 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.550 ft	WATER CONTENT:	47.1 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT	53.9 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	2.15

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	0.0 ksf		

UNDRAINED SHEAR STRENGTH: 2.03 ksf
COMPRESSIVE STRENGTH: 4.06 ksf
FAILURE STRAIN: 1.7 %

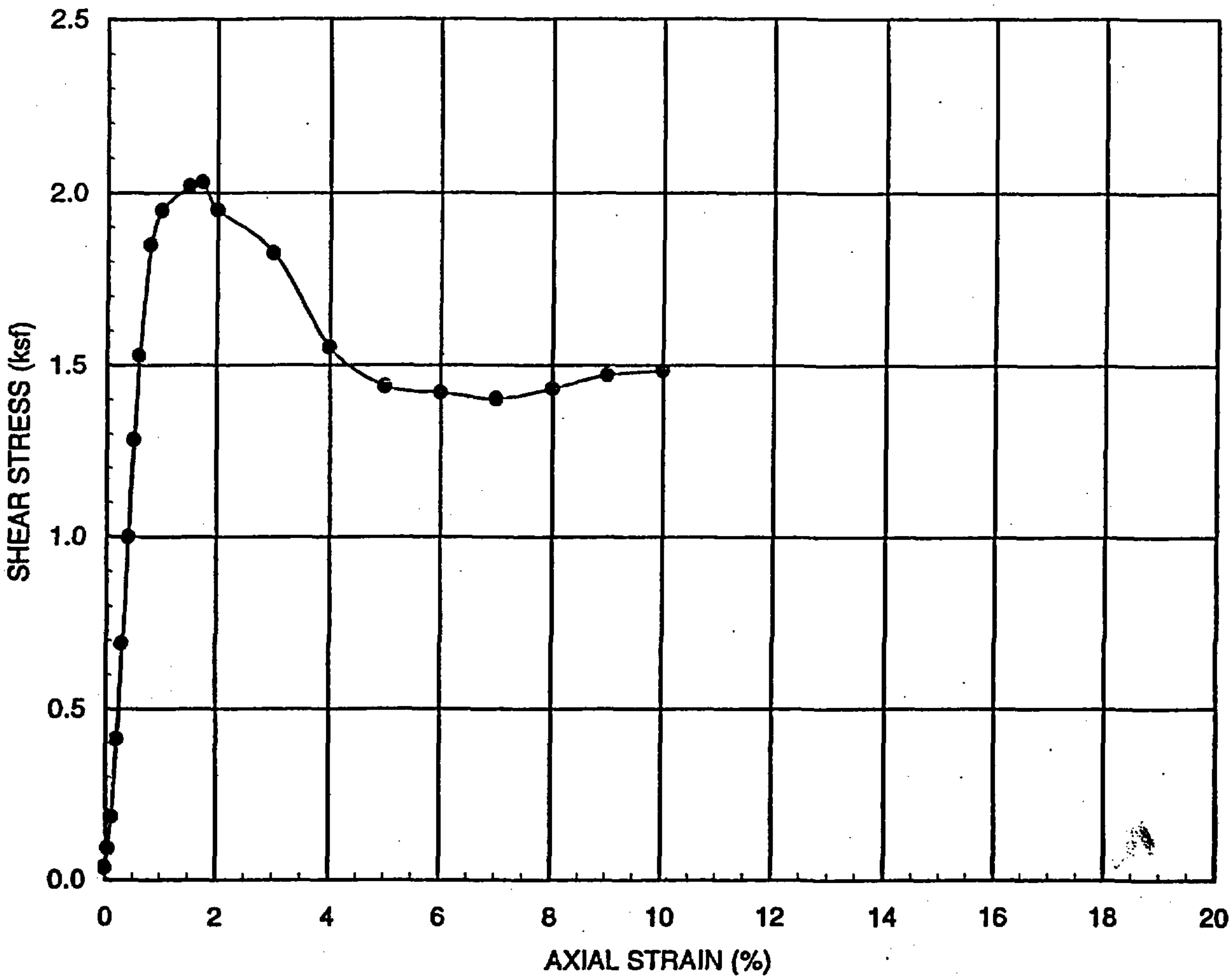
DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.00	-0.23	0.00	0.003	0.0442	0.07	0.04
0.08	1.38	0.05	0.008	0.0442	0.18	0.09
0.17	4.03	0.10	0.016	0.0442	0.37	0.18
0.34	10.53	0.20	0.037	0.0443	0.83	0.41
0.50	18.49	0.30	0.061	0.0443	1.38	0.69
0.67	27.30	0.40	0.089	0.0444	2.00	1.00
0.84	35.42	0.50	0.114	0.0444	2.57	1.28
1.01	42.44	0.60	0.136	0.0444	3.08	1.53
1.34	51.72	0.80	0.165	0.0445	3.70	1.85
1.68	54.70	1.00	0.174	0.0446	3.90	1.95
2.51	57.00	1.50	0.181	0.0449	4.04	2.02
2.90	57.44	1.73	0.182	0.0450	4.06	2.03
3.35	55.31	2.00	0.176	0.0451	3.90	1.95
5.03	52.21	3.00	0.166	0.0455	3.65	1.82
6.70	44.70	4.00	0.143	0.0460	3.10	1.55
8.38	41.80	5.00	0.134	0.0465	2.88	1.44
10.06	41.70	6.00	0.134	0.0470	2.84	1.42
11.73	41.60	7.00	0.133	0.0475	2.80	1.40
13.41	43.00	8.00	0.138	0.0480	2.86	1.43
15.08	44.70	9.00	0.143	0.0485	2.94	1.47
16.76	45.60	10.00	0.146	0.0491	2.97	1.48

STONE & WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, LLC.
PFSF, Skull Valley, UT

JO: 05996.02
November 1999

UNCONSOLIDATED UNDRAINED TRIAXIAL



SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	5.9 ft	CHECKED:	TKC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.550 ft	WATER CONTENT:	47.1 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT:	53.9 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	2.15

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	0.0 ksf		

UNDRAINED SHEAR STRENGTH:	2.03 ksf
COMPRESSIVE STRENGTH:	4.06 ksf
FAILURE STRAIN:	1.7 %

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST DATA

SAMPLE INFORMATION:

BORING:	B-1	DATE:	11/11/99
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	5.9 ft	CHECKED:	TYC
DESCRIPTION:	Clayey SILT		
HEIGHT:	0.550 ft	WATER CONTENT:	47.1 %
DIAMETER:	0.237 ft	INITIAL DRY UNIT WEIGHT	53.9 pcf
AREA:	0.0442 ft^2	INITIAL VOID RATIO:	2.15

TEST DATA:

LOADING:	Axial Compression	STRAIN RATE:	0.8 %/min
CELL PRESSURE:	0.0 ksf		

UNDRAINED SHEAR STRENGTH: 2.03 ksf
COMPRESSIVE STRENGTH: 4.06 ksf
FAILURE STRAIN: 1.7 %

DIAL READING mm	LOAD CELL mV	AXIAL STRAIN %	FORCE kip	AREA sq ft.	AXIAL STRESS ksf	SHEAR STRESS ksf
0.00	-0.23	0.00	0.003	0.0442	0.07	0.04
0.08	1.38	0.05	0.008	0.0442	0.18	0.09
0.17	4.03	0.10	0.016	0.0442	0.37	0.18
0.34	10.53	0.20	0.037	0.0443	0.83	0.41
0.50	18.49	0.30	0.061	0.0443	1.38	0.69
0.67	27.30	0.40	0.089	0.0444	2.00	1.00
0.84	35.42	0.50	0.114	0.0444	2.57	1.28
1.01	42.44	0.60	0.136	0.0444	3.06	1.53
1.34	51.72	0.80	0.165	0.0445	3.70	1.85
1.68	54.70	1.00	0.174	0.0446	3.90	1.95
2.51	57.00	1.50	0.181	0.0449	4.04	2.02
2.90	57.44	1.73	0.182	0.0450	4.06	2.03
3.35	55.31	2.00	0.176	0.0451	3.90	1.95
5.03	52.21	3.00	0.166	0.0455	3.65	1.82
6.70	44.70	4.00	0.143	0.0460	3.10	1.55
8.38	41.80	5.00	0.134	0.0465	2.88	1.44
10.06	41.70	6.00	0.134	0.0470	2.84	1.42
11.73	41.60	7.00	0.133	0.0475	2.80	1.40
13.41	43.00	8.00	0.138	0.0480	2.86	1.43
15.08	44.70	9.00	0.143	0.0485	2.94	1.47
16.76	45.60	10.00	0.146	0.0491	2.97	1.48

APPENDIX 2A

ATTACHMENT 9

BORING LOGS FOR TEST PITS - JANUARY 2001

TABLE OF CONTENTS

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-1 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,176 E 1,281,860
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16/01
Ground Elevation: 4470 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 0.5-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT, moist, brown.
		G	2			MH	ELASTIC SILT, moist, pale-brown.
		G	3			CL	LEAN CLAY, moist, light brownish-grey.
4465	5						

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-2 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,476 E 1,281,865
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16.
Ground Elevation: 4468 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 0.5-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			CH	FAT CLAY with sand, moist, dark yellowish-brown.
		G	2			MH	ELASTIC SILT, moist, light grey.
4465							
		G	3			CL	LEAN CLAY, moist, light brownish-grey.
	5						
4460							

Legend/Notes
- Datum is 1929 NGVD.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-3 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility | Logged by: P. J. Trudeau
Client: Private Fuel Storage, L.L.C. | Date Start - Finish: 01/16/01 - 01/16/01
Coordinates: N 7,322,181 E 1,281,510 | Ground Elevation: 4470 ft
Groundwater Depth: | Depth to Bedrock: | Total Depth Drilled: 6 ft
Contractor: AGEC | Driller: | Rig Type:

Methods: | Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 0.5-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT with sand, moist, yellowish-brown.
		G	2			MH	ELASTIC SILT, moist, light yellowish-brown.
		G	3			CL	LEAN CLAY, moist, light olive-brown.
4465	5						

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-4 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility — Logged by: P. J. Trudeau

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 01/16/01 - 01/16/0

Coordinates: N 7,322,481 E 1,281,515 — Ground Elevation: 4469 ft

Groundwater Depth: — Depth to Bedrock: — Total Depth Drilled: 6 ft

Contractor: AGEC — Driller: — Rig Type:

Methods: — Casing Used:

Drilling Soil:

Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 0.5-ft intervals.

Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT with sand, moist, yellowish-brown.
		G	2			CH	FAT CLAY, moist, light yellowish-brown.
4465		G	3			CL	LEAN CLAY, moist, light yellowish-brown.
	5						

Legend/Notes

- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P.J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-5 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,188 E 1,280,990
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16/01
Ground Elevation: 4470 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Casing Used:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT with sand, moist, brown.
		G	2			MH	ELASTIC SILT, moist, light olive-brown.
		G	3			CL	LEAN CLAY, moist, greyish-brown.
4465	5						

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P.J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-6 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,488 E 1,280,995
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16/
Ground Elevation: 4468 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Casing Used:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT, moist, yellowish-brown.
4465		G	2			CL	LEAN CLAY, moist, light yellowish-brown.
		G	3			CL	LEAN CLAY, moist, light brownish-grey.
	5						
4460							

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-7 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,193 E 1,280,641
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:
Casing Used:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16/01
Ground Elevation: 4469 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SANDY SILT, moist, dark yellowish-brown.
						ML	CLAYEY SILT, moist, light brown.
		G	2			CL	LEAN CLAY, moist, light brownish-grey.
4465		G	3			CL	LEAN CLAY, moist, light grey.
	5						

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
PJ Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-8 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,322,493 E 1,280,645
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:
Casing Used:

Logged by: P. J. Trudeau
Date Start - Finish: 01/16/01 - 01/16/01
Ground Elevation: 4468 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT with sand, moist, yellowish-brown.
4465		G	2			MH	ELASTIC SILT, moist, light yellowish-brown.
		G	3			CL	LEAN CLAY, moist, light brownish-grey.
	5						
4460							

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P J Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-9 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Logged by: P. J. Trudeau
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 01/17/01 - 01/17/01
Coordinates: N 7,323,028 E 1,280,652
Ground Elevation: 4464 ft
Groundwater Depth:
Depth to Bedrock:
Total Depth Drilled: 6 ft
Contractor: AGEC
Driller:
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			MH	SANDY ELASTIC SILT, moist, brown.
		G	2			CL	LEAN CLAY, moist, greyish-brown.
4460							
	5	G	3			MH	ELASTIC SILT, moist, light yellowish-brown.

Legend/Notes

- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-10 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,323,328 E 1,280,656
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/17/01 - 01/17/0
Ground Elevation: 4461 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			ML	SILT with sand, moist, dark yellowish-brown.
4460							
		G	2			CL	LEAN CLAY, moist, light olive-brown.
		G	3			CH	FAT CLAY, moist, light grey.
	5						
4455							

Legend/Notes
- Datum is 1929 NGVD.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P J Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-11 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,323,023 E 1,281,002
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/17/01 - 01/17/07
Ground Elevation: 4463 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Casing Used:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			MH	ELASTIC SILT with sand, moist, yellowish-brown.
						ML	CLAYEY SILT, moist, light brown.
		G	2			CL	LEAN CLAY, moist, light olive-brown.
4460							
		G	3			MH	ELASTIC SILT, moist, light yellowish-brown.
	5						
4455							

Legend/Notes
- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-12 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility — Logged by: P. J. Trudeau

Client: Private Fuel Storage, L.L.C. — Date Start - Finish: 01/17/01 - 01/17/01

Coordinates: N 7,323,323 E 1,281,006 — Ground Elevation: 4462 ft

Groundwater Depth: — Depth to Bedrock: — Total Depth Drilled: 6 ft

Contractor: AGEC — Driller: — Rig Type:

Methods: — Casing Used:

Drilling Soil:

Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.

Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			CL	LEAN CLAY with sand, moist, dark yellowish-brown.
4460		G	2			CL	LEAN CLAY, moist, light olive-brown.
		G	3			MH	ELASTIC SILT, moist, light yellowish-brown.
	5						
4455							

Legend/Notes

- Datum is 1929 NGVD.
- ⊻ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-13 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,323,016 E 1,281,522
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/17/01 - 01/17/01
Ground Elevation: 4464 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Casing Used:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			MH	SANDY ELASTIC SILT, moist, yellowish-brown.
						ML	CLAYEY SILT, moist, light brown.
		G	2			CL	LEAN CLAY, moist, light olive-brown.
4460		G	3			CL	LEAN CLAY, moist, light grey.
	5						

Legend/Notes

- Datum is 1929 NGVD.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P.J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-14 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,323,316 E 1,281,526
Groundwater Depth:
Contractor: AGEC

Logged by: P. J. Trudeau
Date Start - Finish: 01/17/01 - 01/17/0
Ground Elevation: 4462 ft
Depth to Bedrock:
Total Depth Drilled: 6 ft
Driller:
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			CH	FAT CLAY, moist, yellowish-brown.
4460		G	2			CL	LEAN CLAY, moist, light olive-brown.
		G	3			CH	FAT CLAY, moist, light olive-brown.
	5						
4455							

Legend/Notes
- Datum is 1929 NGVD.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P. J. Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-15 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Logged by: P. J. Trudeau
Client: Private Fuel Storage, L.L.C.
Date Start - Finish: 01/17/01 - 01/17/01
Coordinates: N 7,323,011 E 1,281,872
Ground Elevation: 4465 ft
Groundwater Depth:
Depth to Bedrock:
Total Depth Drilled: 6 ft
Contractor: AGEC
Driller:
Rig Type:

Methods:
Casing Used:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			CL	LEAN CLAY with sand, moist, dark yellowish-brown.
		G	2			CH	FAT CLAY, moist, light olive-brown.
		G	3			CL	LEAN CLAY, moist, light grey.
4460	5						

Legend/Notes
- Datum is 1929 NGVD.
- ∇ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.

* indicates use of 300 pound hammer.

- Sample Type:
  G = Grab sample

Approved	Date
P J Trudeau	04/30/01

Stone & Webster Engineering Corporation	BORING LOG	Boring TP-16 J.O. 05996.02 Sheet 1 of 1

Site: Private Fuel Storage Facility
Client: Private Fuel Storage, L.L.C.
Coordinates: N 7,323,311 E 1,281,876
Groundwater Depth:
Contractor: AGEC

Depth to Bedrock:
Driller:

Logged by: P. J. Trudeau
Date Start - Finish: 01/17/01 - 01/17/0
Ground Elevation: 4463 ft
Total Depth Drilled: 6 ft
Rig Type:

Methods:
Drilling Soil:
Sampling Soil: Test pits excavated with backhoe. Grab samples obtained at 2-ft intervals.
Drilling Rock:

Casing Used:

Comments: Coord's refer to Modified Project Datum and were measured using a Garmin III-Plus GPS, accurate to ~10 ft. Elev's estimated based on topo from aerial survey.

Elev (ft)	Depth (ft)	Sample Type	Sample No.	Blows or Recovery / RQD	SPT N Value	USC Symbol	Sample Description
*****	0	G	1			CH	FAT CLAY, moist, dark yellowish-brown.
		G	2			MH	ELASTIC SILT, moist, yellowish-brown.
4460							
		G	3			CL	LEAN CLAY, moist, light olive-brown.
	5						
4455							

Legend/Notes

- Datum is 1929 NGVD.
- ▽ indicates groundwater level.
- ▌ indicates location of samples.
- Blows = number of blows required to drive 2" O.D. sample spoon 6" or distance shown using 140 pound hammer falling 30".
- ( ) = inches of sample recovery.
- Recovery = % rock core recovery.
- RQD = Rock Quality Designation.
- SPT N = Standard Penetration Test resistance to driving, blows/ft.
- USC = Unified Soil Classification system.
- * indicates use of 300 pound hammer.

Sample Type:
G = Grab sample

Approved	Date
P.J. Trudeau	04/30/01