WSRC-TR-95-00072
Rev. 0
February 1995

Site Geotechnical Services (SGS)

Geotechnical Seismic Assessment Report for Defense Waste Processing Facility (DWPF), (U)

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PREPARED FOR THE U.S. DEPARTMENT OF ENERGY UNDER CONTRACT DE-AC09-89R18035

This document was prepared in conjunction with work accomplished under Contract No. DE-AC09-96SR18500 with the U.S. Department of Energy.

Project: Defense Waste Processing Facility (DWPF)

Document: WSRC-TR-95-0072, Rev. 0

Title: Geotechnical Seismic Assessment Report for Defense Waste Processing Facility (DWPF), (U)

Approvals:

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EXECUTIVE SUMMARY

High level waste facilities at the Savannah River Site include several major structures that must meet seismic requirements, including the Defense Waste Processing Facility. Numerous geotechnical and geological investigations have been performed to characterize the in-situ static and dynamic properties of the soil sediments. These investigations have led to conclusions concerning the stability of foundation soils in terms of liquefaction potential and structure settlement. This report reviews past work that addresses seismic soil stability and presents the results of more recent analyses incorporating updated seismic criteria.

We conclude that there are neither geologic nor geotechnical hazards based on the design basis earthquake that would adversely affect the Defense Waste Processing Facility. Static and dynamic structure settlements are within tolerable limits and liquefaction susceptibility is negligible for the seismic events analyzed. Settlement monitoring of the major structures is recommended at regular intervals.

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1.0 INTRODUCTION

High level waste facilities in S-Area at the Savannah River Site (SRS) include several major structures and ancillary facilities (Figure 1). Category I structures include the Vitrification Building (S221), Glass Waste Storage Building (S250), Fan House (S292), and Sand Filter building (S294). Non-Category I facilities include service and administration buildings and facilities for water, sewage, electrical transmission, and storage (Mueser, 1984a).

During the late 1970's and early 1980's, extensive geotechnical investigations and analyses were completed for the Defense Waste Processing Facility (DWPF) and its support facilities. The first investigations and analyses were performed by D'Appolonia (1982a) and consisted of: 1) literature search, 2) aerial photographs and interpretation, 3) geological mapping, 4) seismic reflection survey, 5) soil drilling program, 6) geophysical logging of borings, 7) in-situ velocity measurements, and 8) engineering analyses. The field investigations were conducted by D'Appolonia (1982a) in three phases lasting from 1978 to 1981. Phases 1 and 2 were conducted from January to March and from September to November, 1978, respectively. Phase 3 was conducted from May to August, 1981, following the relocation of the DWPF site.

The subsurface investigations performed by D'Appolonia (1982a) consisted of 130 borings, 23 piezometers, in-situ pressuremeter tests, and geophysical surveys. Borings were drilled by Girdler Exploration and Foundation Inc. under the inspection of D'Appolonia. Approximately 100 of these 130 borings are found within the relocated DWPF site (Figure 1). Subsurface sampling consisted of 2 inch split-spoon samples and "undisturbed" samples recovered from 22 borings using Shelby, Osterberg, and Pitcher samplers. Continuous undisturbed sampling was performed in borings 24 and 86-L2, which are under the footprint of the Vitrification Building and Glass Storage Building, respectively (Figure 1).

In October 1982, Mueser, Rutledge, Johnstone and DeSimone (referred to as Mueser, hereafter) replaced D'Appolonia as the geotechnical subcontractor for DWPF. No additional drilling and sampling was performed by Mueser, except for those boreholes completed as part of the grouting program (Mueser, 1984a). Also, Mueser contracted with Geotechnical Engineers Inc. to perform the liquefaction analyses (GEI, 1983) and to write Section 3.6.4.8 "Liquefaction and Cyclic Mobility Potential" for the Preliminary Safety Analysis Report (duPont, 1982).

The previous geotechnical and geological investigations at DWPF are described herein with summaries of the methods used to determine the stability of the foundation soils - in particular, foundation settlement and liquefaction potential. The conclusions drawn from the aforementioned investigations are reviewed in light of current practice and knowledge at the SRS and are used to determine the technical adequacy of past work and the need for additional investigations and/or analyses.

2.0 GEOLOGIC AND GEOTECHNICAL SOIL CHARACTERIZATION

2.1 Geologic Characteristics of Soil Sediments at DWPF

Soils at DWPF consist of coastal plain sediments that are about 900 ft thick. These sediments range in age from late Cretaceous (about 65 million years ago) to Recent Quaternary. The Coastal Plain sediments are predominantly clastic and overlie a sequence of folded and faulted metamorphic rocks of Precambrian/Paleozoic age (up to 570 million years old) (duPont, 1982). The sedimentary sequence at DWPF consists mainly of interbedded, clayey sand, sand, silt, and silty clay with some thin carbonate units.

Five, shallow, geologic formations were identified by the initial seismic and geophysical surveys performed at DWPF (duPont, 1982). These formations were drilled and sampled during subsequent foundation explorations. In descending order, they are: Hawthorne, Barnwell, McBean, Congaree, and Ellenton Formations. New nomenclature has evolved for the various strata and are presented in Figure 2. Table 1 illustrates the relative positions of these strata at the DWPF site.

Table 1. Sedimentary Stratigraphy at the DWPF Site Based on Subsurface Exploration (Mueser, 1984a).

Stratum - **old nomenclature** (new nomenclature)	Soil Designation	Stratum Thickness (ft)	Stratum Elevation Range (ft, msl)	Geologic Soil Characterization
Hawthorne Formation (Altamaha Fm. - also Upland Unit)	**S1**	**0 +**	**surficial unit above 275 ±**	**poorly sorted, sandy with frequent lenses of gravel, pebbly sand; and oxidized, massive clay.**
Barnwell Formation (Tobacco Road Fm., Irwinton Sand Mbr., Tan Clay Mbr.)	**S2a S2b**	**80 ±**	**275 to 195**	**interbedded, clayey sand and sand with thin layers and lenses of clay or silt.**
Undifferentiated (Tan Clay Mbr. - included above)	**C2**	**5 to 20**	**215 to 195**	**stiff, silty clay.**
McBean Formation (Tinker Fm. and Santee Limestone)	**S3a S3b S3c**	**70**	**195 ± to 125 ±**	**alternating layers of sand, some clay and sand with trace clay or silt; discontinuous calcareous sand in lower strata.**
Undifferentiated (Green Clay)	**M1**	**10 ±**	**140 to 130**	**discontinuous, compact silt.**
Congaree Formation (same)	**S4**	**100 ±**	**125 to 30**	**continuous, dense sand and silty sand.**
Ellenton Formation	----	----	----	**dense, sandy to clayey silt with some silty sand.**

2.2 Geotechnical Characteristics of Soil at DWPF

SPT N-value profiles were prepared by D'Appolonia (1982a) for boreholes near the major facilities. The location of the borehole groups is shown in Figure 3 and the average ± 1 standard deviation profiles are given in Figure 4. Shear and compressional wave velocities for the DWPF soil profile are shown in Figure 5. Also, grain size distributions for soils in the Barnwell, McBean, Congaree, and Ellenton Formations are given in Figure 6. The geotechnical engineering characterization of the soils is given in Table 2.

Table 2. Geotechnical Engineering Characterization of Soil Strata at the DWPF Site (D'Appolonia, 1982a; Mueser, 1984a).

Soil Designation	Soil Description	Water Content (range, %)	SPT N-Value (bpf)
S1	**Clayey Sand, trace Gravel; or Sand, some Clay; organic Silt (see notes).**	**15 to 25**	**10 to 50 [20] (see notes)**
S2a	**Sand, trace Silt and Gravel; occasional Clay lenses.**	**12 to 26 [22]**	**4 to 50 (see notes)**
S2b	**Sand, trace Clay, occasional Silty Clay lenses.**	**15 to 28 [22]**	**8 to 45 (see notes)**
C2	**stiff, Silty Clay to Clayey Silt, trace Sand.**	**[53]**	**9 to 27**
S3a	**Sand, some Clay, trace shell fragments.**	**20 to 30 [23] U 25 to 35 [30] M,L (see notes)**	**10 to 40 U 10 to 60 M,L (see notes)**
S3b	**Sand, Trace Clay and Silt.**	**20 to 30 [25]**	**15 to > 100 [35]**
S3c	**Sand, some Silt, trace Clay**	**22 to 28 [25]**	**12 to 110 [40]**

Notes: *average values, where available, are given in [brackets].*

Stratum S1: organic Silt forms up to 20 ft thick lenses in depressions at ground surface. SPT N-values occasionally as low as 10 bpf in upper 5 ft of soil and as high as 50 bpf throughout possibly due to gravel.

Stratum S2a: typical SPT N-values range from 20 to 25 bpf. Five percent of N-values are below 10 bpf, but no loose, continuous layers were encountered.

Stratum S2b: seven percent of N-values below 8 bpf with continuous loose layer at about elevation 220 ft.

Stratum S3a: U, M, L denotes upper, middle, and lower portions of strata S3a, respectively. Isolated occurrences of SPT N-values as low as 2 bpf.

3.0 STRUCTURAL FAULTS AND SEISMICITY

3.1 Faulting at SRS

Subsurface mapping and seismic reflection surveys performed from 1988 to 1989 at SRS indicate a fault that displaces Cretaceous through Tertiary sediments with about 30 to 100 ft of vertical offset (WSRC, 1994a) (Figure 7). This fault, interpreted as a Cretaceous/Tertiary reactivation of earlier Mesozoic faulting, has been named the Pen Branch fault. The fault trends northwest across the site and closely parallels the fault that forms the northern boundary of the Dunbarton Basin. Based on deformation and sediment age, the fault is not capable (WSRC, 1994a).

Shallow faulting has been observed in the central area of SRS (F, H, and E-Areas). Current knowledge suggests these features are restricted generally to the Santee Formation and overlying sediments and generally do not extend with depth to basement (WSRC, 1994b). Based on profiles constructed from drilling and geophysical data, no capable faults were identified in the Cenozoic sediments at, or near, DWPF (duPont, 1982). Seismic reflection surveys indicate older faults with a maximum of about 50 ft of offset at the top of basement rock about 800 to 980 ft beneath the ground surface. However, reflecting horizons of Cretaceous age, and younger, are not displaced by these faults, which places a minimum age of about 80 to 85 million years before present (mybp) for these features (duPont, 1982).

3.2 Cenozoic Coastal Plain Province Fault Features

Cenozoic tectonic features in the Coastal Plain Province include gentle folding or uplifting of the crust that was both syn- and post-depositional. Several regional basement structures have been interpreted as uplifts or gentle folds and may be indicative of continued crustal movements into the Pleistocene. This deformation of the Coastal Plain may have been related to uplift of the Peninsular and Cape Fear Arches (WSRC, 1994a). Also, large-scale subsurface faulting of sediments as young as Oligocene (24 to 37 mybp) exists in the Atlantic Coastal Plain province near Charleston, SC and in Georgia (WSRC, 1994a).

Nearer to SRS, small-scale faulting and folding through Middle Eocene time has been recognized in the Coastal Plain sediments. Normal faulting, near Langley, SC (about 17 miles northwest of SRS), indicates about 4 ft of displacement associated with a small graben in the Eocene Barnwell Group (WSRC, 1994a).

3.3 Pre-Cenozoic Regional Fault Features

Three miles southeast of DWPF lies the Dunbarton Basin. It is a Triassic-Jurassic rift basin about 6 miles wide and 31 miles long (see Figure 7). The basin is bounded on the northwest by a southeast dipping normal fault and contains several intrabasinal faults (WSRC, 1994a). The lithologies within the basin consist of about 3000 ft of red conglomerates, sandstones, and shales. This is overlain by approximately 1100 ft of Cretaceous and Tertiary coastal plain sediments. Drilling within SRS indicates that these sediments, as well as the unconformity itself, are undeformed and show no evidence of movement since the development of the unconformity approximately 100 mybp (duPont, 1982).

3.4 Recent Seismicity Near SRS

The Charleston, SC area is the most significant source of recent seismicity affecting SRS, both in terms of the maximum ground shaking intensity and the number of earthquakes felt at SRS (WSRC, 1994a). The greatest intensity at SRS has been estimated at Modified Mercalli Iintensity (MMI) VI to VII and was produced by the intensity X earthquake that struck Charleston, SC on August 31, 1886 (WSRC, 1994a). Outside the Charleston area, the earthquake in Union County, SC (about 100 miles north-northeast of SRS) on January 1, 1913 is the event closest to SRS having a MMI ≥ VII. Earthquakes reported within 50 miles of SRS, with a MMI ≥ IV, include the July 26, 1945, MMI V event to the northeast of SRS; and the October 28, 1974, MMI IV earthquake to the north of SRS (WSRC, 1994a).

4.0 MISCELLANEOUS GEOLOGICAL FEATURES AND CONDITIONS

4.1 Depth to Ground Water

Twenty piezometers were installed in completed borings at the revised DWPF site. The piezometers were installed with bottom-screened intervals between elevations 245 and -49 ft (Mueser, 1984a).

During the preliminary investigations, ground water levels within the Tobacco Road Formation ranged from 30 to 50 ft below the ground surface, approximately paralleling the surface topography. The average water table elevation was 245 ft during the monitoring period from March 1978 to October 1981. During this period, the piezometric surface within the Santee Formation ranged from 35 to 55 ft below the ground surface and had a general flow direction to the northeast.

Current water levels (1992) near the Vitrification Building show water table elevations from 240 to 245 ft, suggesting a relatively static water table since the time of construction.

4.2 Clastic Dikes / Soil Fractures

The preliminary safety analysis report for DWPF noted numerous "clastic dikes" in the Altamaha Formation that appear to extend to a considerable depth. True clastic dikes often result from earthquake-induced ground failure and some investigators have suggested a possible liquefaction origin for these features at SRS (duPont, 1982).

More recent SRS publications have characterized these features as soil fractures and not as clastic dikes (WSRC, 1994a). It appears that the soil fractures have multiple origins and may be related to weathering, shrinkage, dissolution, or mid-Tertiary tensional faulting. In many outcrops there is little apparent vertical displacement along these features. Also, the primary strike orientation of the features parallels the strike of many of the shallow surface depressions at SRS, causing D'Appolonia to speculate that renewed, post-Eocene, tensional faulting may have taken place in the surficial sediments at SRS. This renewed tectonic faulting was postulated to have fractured the surficial sediments, causing preferential dissolutioning of the underlying calcareous deposits (duPont, 1982). The fractures appear to pose no obvious geological hazard to the DWPF.

4.3 Soft Zones

Low SPT N-values were found in three zones within the Santee Limestone (formerly the McBean Formation) and lower Dry Branch Formation (Tan Clay Member) from approximate elevations 140 to 157 ft, 167 to 171 ft, and 190 to 200 ft at DWPF (duPont, 1982). A review of the DWPF exploratory boring program in the Santee Formation by Mueser (1983) indicated soft zones of leached material having limited lateral extent between elevations 130 and 180 ft. These zones were identified by grout takes larger than the nominal borehole volume, low sampler penetration resistances, loss of drilling mud, and positive reaction of some soil samples to dilute hydrocholoric acid. Because leached conditions were indicated in the Santee Formation, Mueser recommended grouting beneath the proposed critical structures to minimize the risk of irregular building settlements (Mueser, 1983). More detail on the soft zones and the subsurface grouting program at DWPF is described in Appendix A.

It is important to note, however, that large static surface loads applied at ITP, H-Tank Farm, and DWPF have been unable to compress the soft zones. Settlement data throughout H- and S-Areas suggest no deep-seated settlement is taking place due to structure and embankment fill loading over this large area (WSRC, 1994b). Soft zones in the Santee and lower Dry Branch Formations, either because of their limited size and/or significant depth, do not appear to pose a risk of significant settlement.

5.0 STATIC SETTLEMENT PREDICTIONS AND MEASUREMENTS

Static building settlement at DWPF was predicted by D'Appolonia (1982a) and Mueser (1984a). The total measured static settlement has been about 1.5 to 3 inches during the monitoring period from November 1984 through July 1994. Table 3 shows the evolution and results of settlement predictions and actual measured settlement by various investigators. The post-construction settlement data at DWPF show good agreement with the settlement predictions of Mueser (Edinger, 1982; Mueser 1984a). This indicates the inherent, static stability of the foundation soils and the adequacy of the applied settlement analysis. Figure 8 shows estimated and measured settlements for Building S221 at DWPF and Figure 9 shows the load and settlement versus log time for two typical settlement points. A more detailed discussion of the settlement analyses and their historical incorporation is given in Appendix B.

Table 3. Static Total Settlement Predictions by Various Investigators for the Vitrification Building.

Investigator	Reference Year	Total Settlement (inches)	Notes
D'Appolonia - initial	1982a	**13**	predicted.
D'Appolonia - revised	1982a	**3. 5**	predicted.
Mueser (reference Edinger)	1982	**2 to 4**	predicted and based on earlier H-Area investigations.
Mueser	1984a	**3 to 5**	predicted.
Mueser	1991	**2.2 to 2.7 north** **1.3 to 2.0 south**	actual field measurements - total from November 1984 through April 1991.
WSRC SGS	1994c	**0.1 to 0.3**	actual field measurements - total from April 1991 through July 1994.
Measured Field Total	----	**1.4 to 3.0**	as of July 1994.

6.0 LIQUEFACTION SUSCEPTIBILITY AND CYCLIC MOBILITY

6.1 Liquefaction Studies Performed at SRS

Several liquefaction studies have been completed at SRS for the sandy soils of the Altamaha, Tobacco Road, and Dry Branch Formations.

Mueser (1976), at the request of duPont, reviewed unpublished studies in F, H, C, K, L, P, and R-Areas for the purpose of assessing the liquefaction susceptibility of the SRS soils. D'Appolonia (1979) completed an extensive subsurface investigation and liquefaction assessment at DWPF. In their draft report, the margin of safety against liquefaction at DWPF was considered to be "unacceptably low" and remedial measures were deemed necessary. However, because of concurrent changes in the DBE from 0.26 to 0.20 g, as recommended by URS/Blume, D'Appolonia (1982b) was requested to re-evaluate the liquefaction potential at DWPF for this lower peak ground acceleration. As a result of this reanalysis, D'Appolonia concluded that the DWPF profile had adequate factors of safety against liquefaction. The liquefaction susceptibility at DWPF was also reanalyzed and revised by Geotechnical Engineers, Inc. (GEI, 1983). At this time, GEI was subcontractor to Mueser and further evaluation was needed to independently assess the conclusions and recommendations of D'Appolonia (1979; 1982b). GEI concluded that cyclic strains would be very small for the 0.2 g DBE and the potential for liquefaction and cyclic mobility were negligible.

A detailed technical and historical discussion of these studies is included in Appendix C. The salient points of these studies are presented in Table 4. The studies state that the potential for soil liquefaction beneath the critical structures is negligible and foundation settlement is within allowable structural limits.

Table 4. Liquefaction Studies by Various Investigators.

Investigator (report year)	Conditions of Study	Results of Study
Mueser (1976)	• 0.2 g pga. • cohesionless, granular soils. • critical depth of 50 ft. • saturated soils. • loose soils with max. 20 to 30 bpf.	• possibility of small reduction in shear strength due to partial liquefaction was extremely remote. • no appreciable surface settlement.
D'Appolonia (1979) - initial analysis	• 0.26 g pga. • three western U.S., deep alluvium, seismic time histories. • 30 seconds duration with 20 seconds strong ground motion. • SHAKE computer analysis. • triaxial and cyclic torsional shear tests on sand with fines from 14 to 19 %. • used laboratory to in-situ correction factors.	• safety margin against liquefaction considered unacceptably low. • recommended soil replacement with compacted backfill beneath critical structures. • minimum factor of safety against liquefaction = 1.0 to 1.2.
D'Appolonia (1982b) - reanalyses	• 0.2 g pga proposed by URS/Blume. • three western U.S. seismic time histories. • field performance data. • laboratory-determined dynamic strength. • SHAKE computer analyses. • 30 seconds duration with 20 seconds strong ground motion.	• free-field cyclic stress ratio versus depth profile established for SRS using each of the three time history records.
reanalysis with field performance data	• SPT N-values averaged in a composite group.	• minimum factor of safety against liquefaction = 1.1 for all three earthquake records. • minimum FS = 1.1 in critical soil zone 50 to 70 ft below ground surface.
reanalysis with laboratory test data	• laboratory to in-situ correction factors applied to laboratory cyclic triaxial and cyclic torsional shear tests.	• minimum factor of safety against liquefaction = 1.3 for all three earthquake records. • minimum FS = 1.2 in critical soil zone 50 to 70 ft below ground surface.
Geotechnical Engineers, Inc. (1983)	• evaluated steady-state strength of sand. • SHAKE computer analyses. • 30 seconds duration with 20 seconds strong ground motion. • liquefaction failure assessment using laboratory tests. • assesssed cyclic mobility using SPT N-values, CPT values, and laboratory tests.	• structure not susceptible to bearing capacity failure due to liquefaction. • potential for cyclic mobility negligible using four types of analyses. • factor of safety = 2.1 to 2.5 against cyclic mobility using laboratory data.
WSRC Site Geotechnical Services (1995)	• SHAKE91 computer analyses. • controlling, site-specific earthquake event: Charleston, SC; M = 7.5, pga = 0.11g. • soil sediment aging considered.	• liquefaction potential negligible. • factor of safety against liquefaction = 1.2 to 4.0. • dynamic settlement < 0.5 inch.

6.2 Liquefaction and Dynamic Settlement Analyses by WSRC

Since the early 1980's, the state-of-the-art in liquefaction susceptibility analysis and the evaluation basis earthquakes at SRS have changed. New liquefaction susceptibility and settlement curves have been proposed for the SRS soils (RTF, 1993; WSRC, 1994d). This section discusses liquefaction susceptibility of the DWPF soils for local and distant, site-specific earthquakes (evaluation basis earthquakes, EBE) developed as part of the geotechnical investigations at the Replacement Tritium Facility (RTF) (BSRI, 1993) and the ITP Facility (WSRC, 1994d). The present liquefaction susceptibility analysis is performed using site-specific liquefaction curves developed from the RTF investigation for the Tobacco Road Formation (BSRI, 1993) and the EBE. The analysis was performed for 8 boreholes located under and near Building S221. Estimates of the dynamically-induced settlement are presented (Table 5), since it is now recognized that a small amount of dynamic settlement can occur without reaching the state of initial liquefaction. The results of these analyses are shown in Table 5. More detailed discussion of these analyses and their methodologies are presented in Appendix C.

Table 5. Liquefaction Determination and Estimates of Dynamic Settlement at DWPF by WSRC.

Borehole Number	Controlling Event (see note below)	Liquefaction Potential	Cumulative Settlement (inches)
BH-9	**distant**	**none**	**0.0**
BH-43	**distant**	**none**	**0.0**
BH-46	**distant**	**none**	**0.1**
BH-54	**distant**	**none**	**0.0**
BH-68	**distant**	**none**	**0.3**
BH-83	**distant**	**none**	**0.0**
BH-86	**distant**	**none**	**0.0**
BH-90	**distant**	**none**	**0.0**

__Note:__ the EBE (evaluation basis earthquake) used for RTF and ITP consists of a local controlling earthquake, M_w = 5 to 6, distance from SRS = 25 km and pga = 0.19 g; and a distant controlling earthquake, M_w = 7.5, distance from SRS = 120 km, and pga = 0.11 g. Previous studies show that the distant event controls liquefaction analysis, therefore the determination of liquefaction potential and associated dynamic settlement for this study utilized the distant event only.

The current WSRC analyses show the potential for liquefaction at DWPF is negligible for the seismic events analyzed and the methodologies employed. Furthermore, these analyses show the amount of dynamically-induced settlement is less than 0.5 inch due to the distant EBE.

7.0 CONCLUSIONS

WSRC Site Geotechnical Services has reviewed the geotechnical characterization and analyses performed by earlier investigators and has also performed additional analyses. The results are:

1. The previous investigations performed were thorough and well-planned,
2. Adequate characterization and analyses have been performed to assess the stability of the foundation soils,
3. The subsurface conditions at DWPF are similar and consistent with those found at other facilities in H- and S-Areas,
4. No design basis geologic nor geotechnical hazards that would adversely affect DWPF have been identified,
5. Post-construction settlement measurements at DWPF confirm the geotechnical parameters used in the settlement analysis and confirm the static stability of the subsurface soils,
6. Previous and current liquefaction susceptibility analyses show that the soils beneath DWPF will not liquefy for the seismic events analyzed, and
7. Dynamic settlement analyses for the current site, distant evaluation basis earthquake indicate that dynamic foundation settlement will be less than 0.5 inch.

8.0 RECOMMENDATIONS

Based on the results of this review, no further geotechnical work involving field characterization, laboratory testing, or engineering analyses is required for DWPF.

Continuation of the settlement monitoring program for the major structures is recommended. The monitoring period should be for the life of the structures at regular intervals not to exceed one year.

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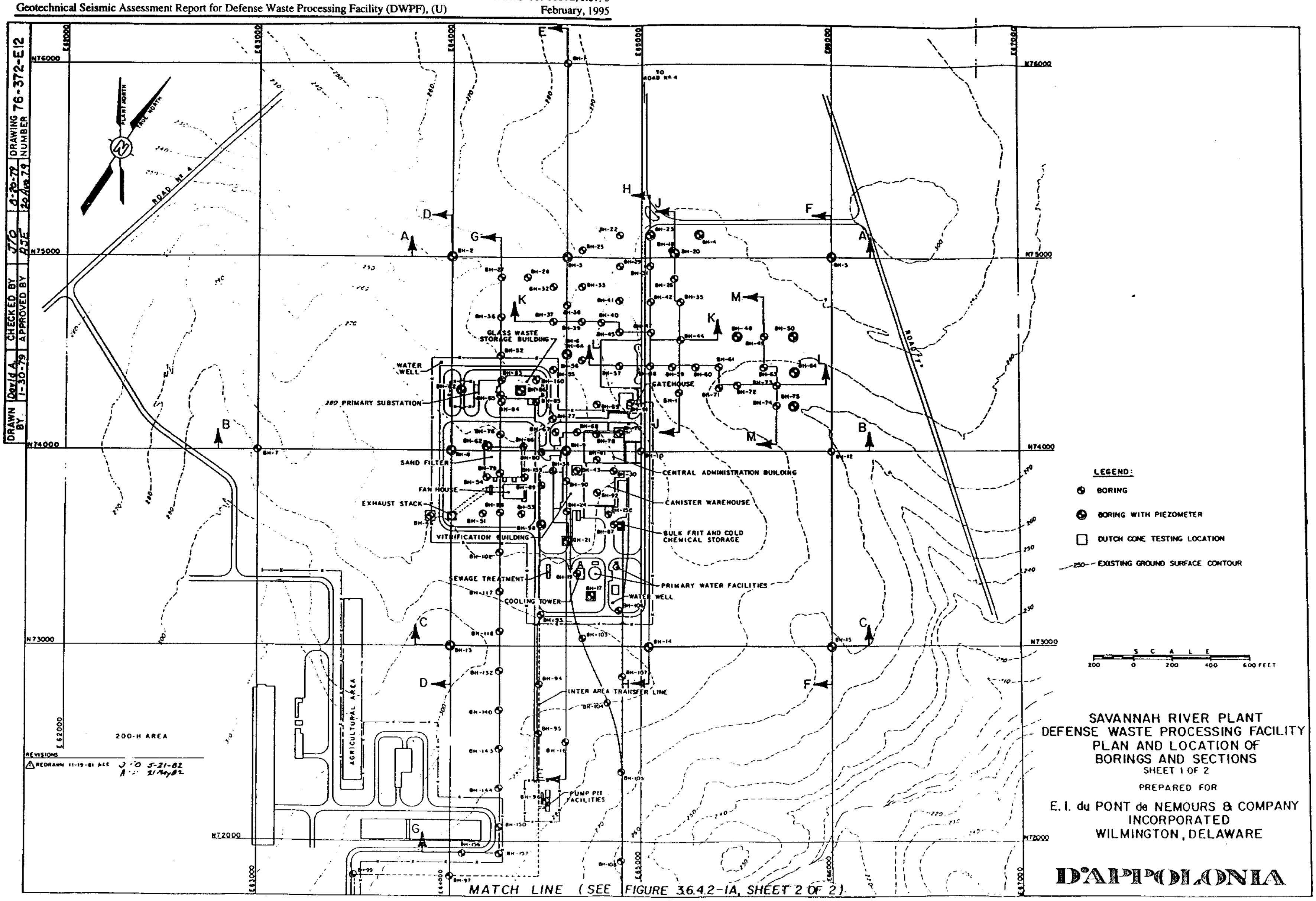

Figure 1. Plan and Location of Borings at the DWPF Site.

COMPARISON OF SHALLOW STRATIGRAPHY NOMENCLATURE

OLD NOMENCLATURE	NEW NOMENCLATURE		STRATUM ON SOIL LOGS
HAWTHORNE FM	ALTAMAHA FM (UPLAND UNIT)		S1
BARNWELL FM	TOBACCO ROAD FM		S2a or S2b
	DRY BRANCH FM	IRWINTON SAND MEMBER	
UNDIFFERENTIATED	DRY BRANCH FM	TAN CLAY MEMBER	C2
MCBEAN FM	TINKER FM / SANTEE LIMESTONE		S3a, S3b or S3c
UNDIFFERENTIATED	GREEN CLAY (INFORMAL)		M1
CONGAREE FM	CONGAREE FM		S4

Figure 2 Sediment Stratigraphic Sequence and Nomenclature

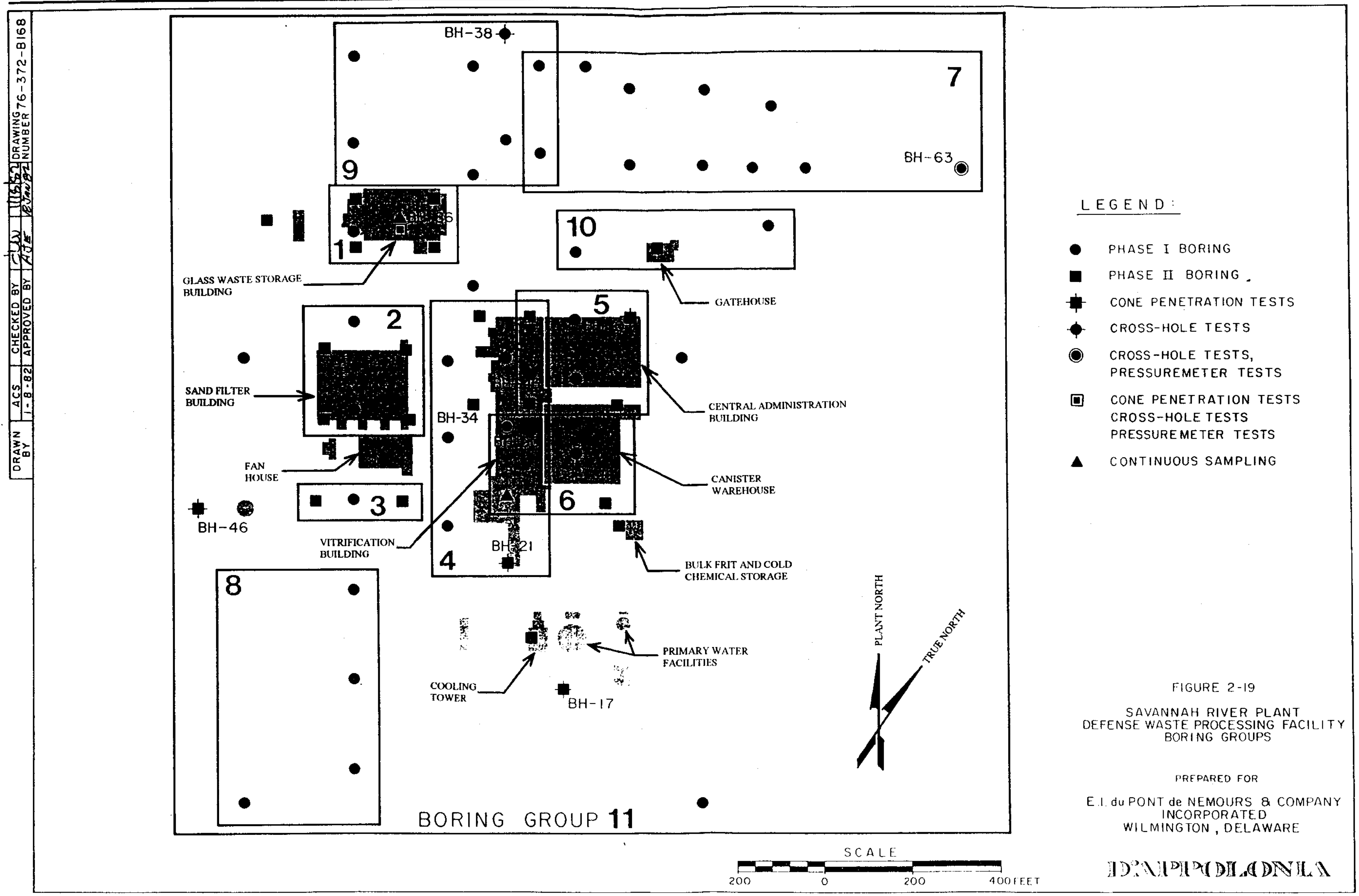

Figure 3. Location of Borehole Groups for SPT Profiles in Figure 4.

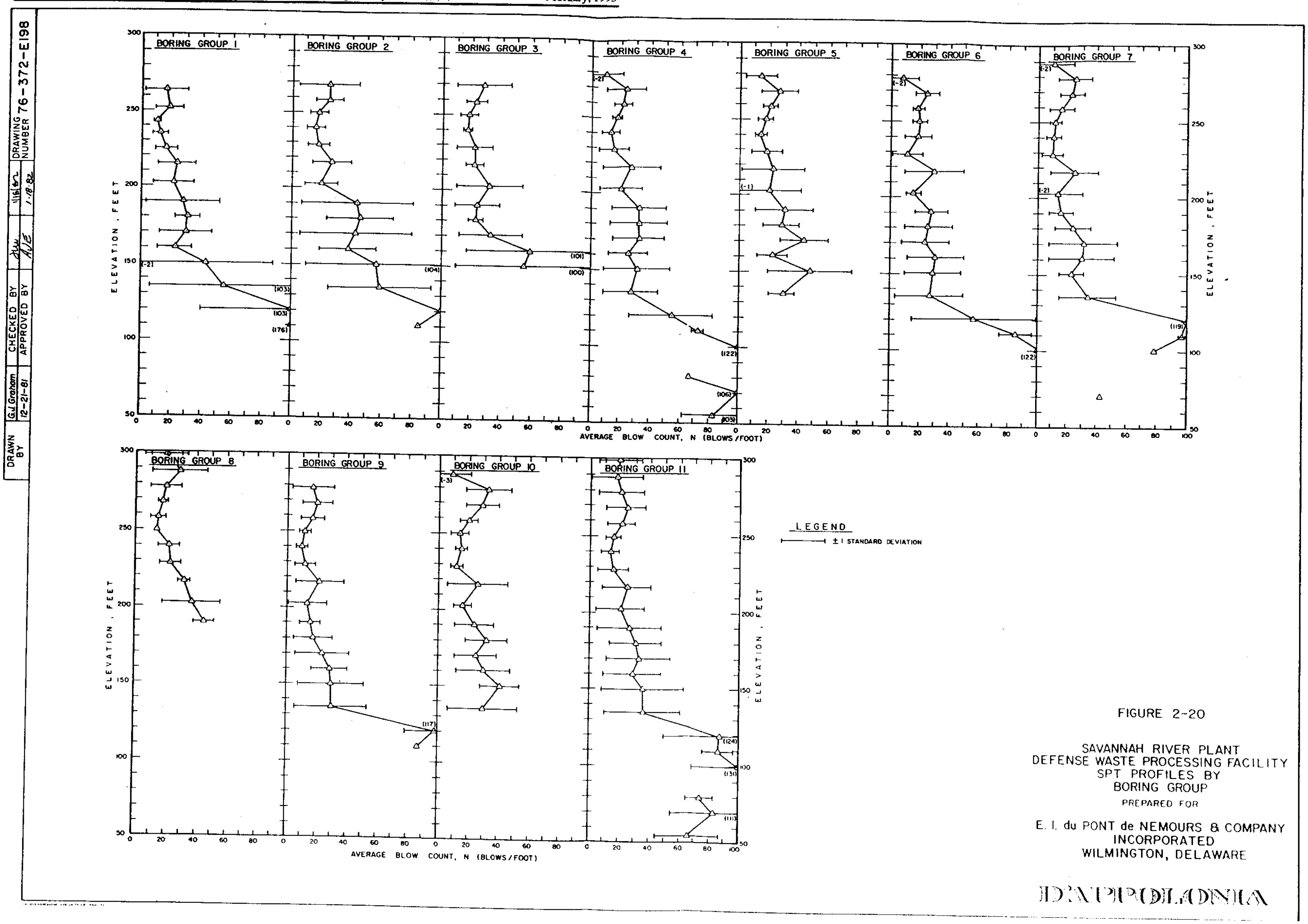

Figure 4. SPT N-Value Profiles for Borehole Groups in Figure 3.

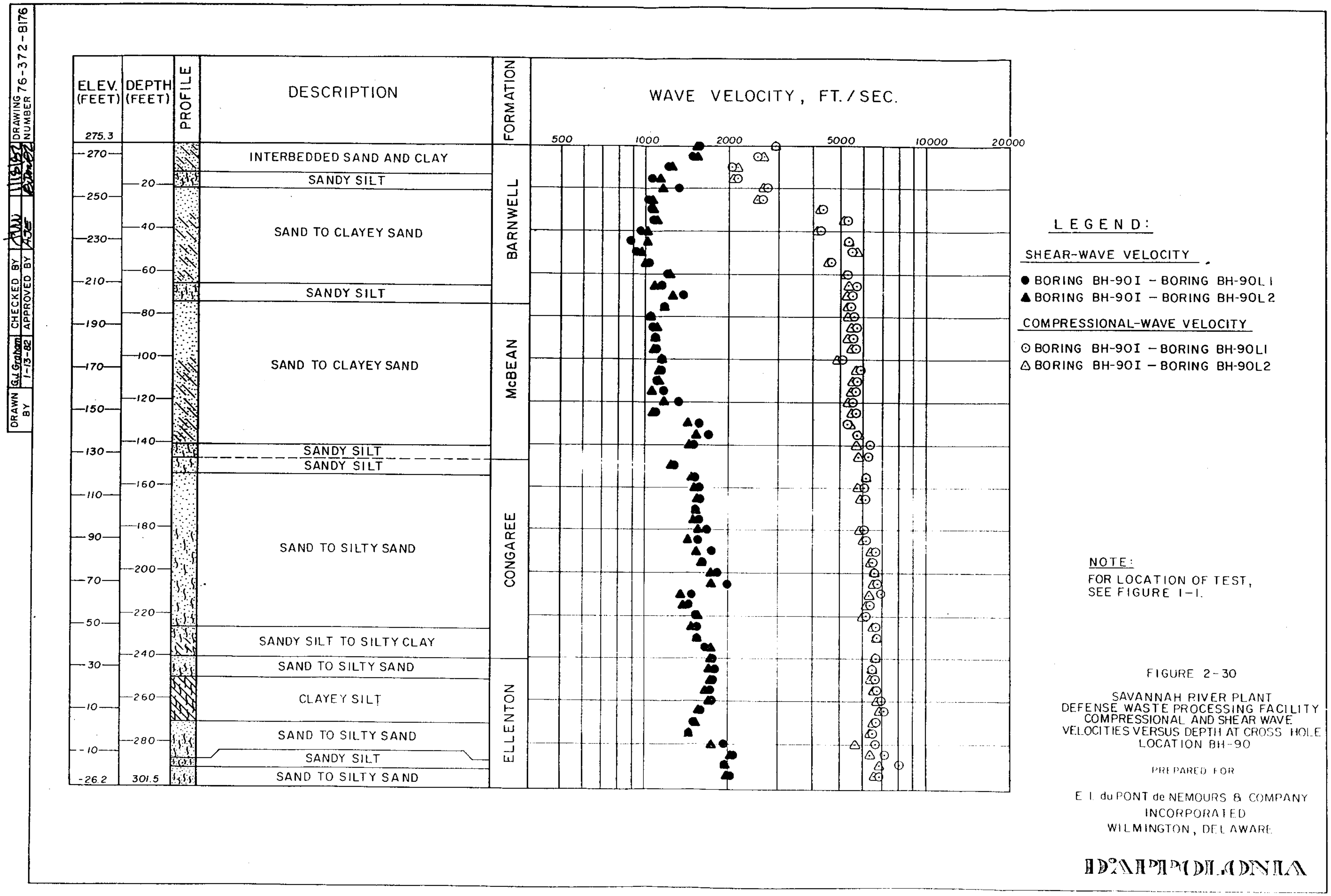

Figure 5. Shear and Compressional Wave Velocity Profiles at DWPF.

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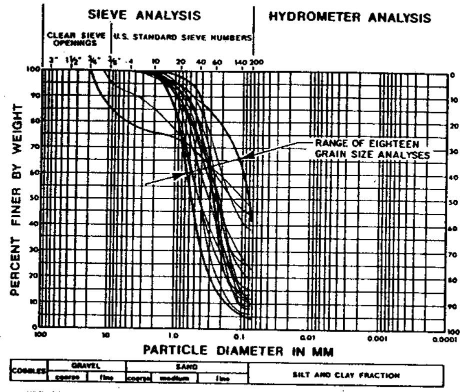
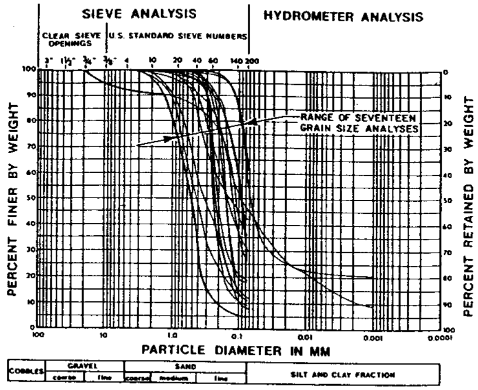
D.Weick 1-12-82	AJE	18 Jan 82	76-372-E216

BARNWELL FORMATION



BORING	SAMPLE	DEPTH	SOIL DESCRIPTION	U.S.C.S.	D_{50}	C_u	W.%
BH-20	S-4	7.5'-9.0'	RED AND WHITE SILTY FINE TO COARSE SAND-SOME FINE GRAVEL	sm	NA	NA	20.1
BH-20	S-6	12.5'-14.0'	RED AND PURPLE SILTY FINE TO COARSE SAND-TRACE OF FINE GRAVEL	sm	NA	NA	13.5
BH-20	S-11	35.0'-36.5'	TAN AND WHITE CLAYEY FINE TO COARSE SAND-TRACE OF SILT	sc	NA	NA	15.7
BH-20	S-17	65.0'-66.0'	TAN FINE TO MEDIUM SAND - TRACE OF SILT	SP-SM	NA	NA	21.9
BH-38	ST-1	36.8'-37.4'	TAN FINE TO MEDIUM SAND - SOME CLAY	sc-sm	NA	NA	37.3
BH-38	ST-2	51.5'-54.5'	RED FINE TO MEDIUM SAND - TRACE OF CLAY	SP-SC	0.10	2.9	24.2
BH-38	ST-5	71.5'-73.5'	LIGHT BROWN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	NA
BH-47	S-9	25.0'-26.5'	RED, BROWN AND PURPLE CLAYEY FINE TO COARSE SAND	sc	NA	NA	19.8
BH-55	S-15	55.0'-56.5'	TAN AND BROWN FINE TO COARSE SAND-TRACE OF SILT	SP-SM	0.079	7.6	20.6
BH-56	ST-3	56.5'-58.5'	TAN FINE TO MEDIUM SAND	SP-SM	0.16	3.9	NA
BH-55	ST-4	71.5'-72.1'	TAN CLAYEY FINE TO COARSE SAND - TRACE OF FINE GRAVEL	sc	NA	NA	23.6
BH-63	S-14	50.0'-51.5'	LIGHT BROWN FINE TO COARSE SAND-SOME CLAY	SW-SC	0.065	8.2	25.2
BH-75	S-11	35.0'-36.5'	TAN AND PINK CLAYEY FINE TO COARSE SAND	sc	NA	NA	57.9
BH-81	ST-1	36.5'-38.5'	ORANGISH BROWN FINE TO MEDIUM SAND-SOME CLAY	sc	NA	NA	NA
BH-81	ST-2	46.5'-48.3'	TAN FINE TO MEDIUM SAND - SOME CLAY	sc	NA	NA	22.6
BH-81	ST-3	56.5'-58.4'	REDDISH BROWN FINE TO MEDIUM SAND-TRACE OF CLAY	SP-SC	0.12	2.16	NA
BH-90	ST-1	37.0'-39.0'	REDDISH BROWN FINE TO MEDIUM SAND-TRACE OF CLAY	SP-SC	0.11	2.9	25.3
BH-90	ST-3	52.3'-52.6'	TAN FINE TO MEDIUM SAND-TRACE OF CLAY AND MICA	SP-SC	0.15	2.93	52.0

McBEAN FORMATION



BORING	SAMPLE	DEPTH	SOIL DESCRIPTION	U.S.C.S.	D_{50}	C_u	W.%
BH-20	S-26	110.0'-111.5'	TAN AND GRAY FINE TO COARSE SAND-TRACE OF CLAY	SP-SC	0.038	14.5	26.3
BH-38	S-21	85.0'-86.5'	TAN FINE TO MEDIUM SAND - TRACE OF CLAY	SP-SC	0.090	2.9	27.2
BH-38	ST-5	91.0'-94.0'	TAN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	25.3
BH-38	S-25	105.0'-106.5'	LIGHT BROWN FINE TO COARSE SAND-TRACE OF CLAY	SP-SC	0.23	3.0	27.8
BH-38	ST-6	109.4'	TAN FINE TO MEDIUM SAND-TRACE OF CLAY	SP-SC	0.097	4.4	27.1
BH-38	S-27	115.0'-116.5'	TAN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	32.2
BH-38	S-31	135.0'-136.5'	TAN SANDY SILT - SOME CLAY	ml-cl	NA	NA	33.4
BH-38	ST-7	138.1'	TAN SILTY FINE TO MEDIUM SAND-TRACE OF FINE GRAVEL	sm	NA	NA	NA
BH-47	S-22	90.0'-91.5'	TAN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	34.1
BH-47	ST-6	91.9'-92.9'	TAN FINE TO MEDIUM SAND-SOME CLAY-TRACE OF MICA	sc	NA	NA	36.0
BH-55	S-21	85.0'-86.5'	TAN FINE TO MEDIUM SAND-SOME CLAY	SP-SC	NA	NA	32.6
BH-55	ST-6	125.7'	TAN FINE SAND-TRACE OF SILT	SP-SM	NA	NA	30.6
BH-55	ST-8	143.0'-144.8'	TAN AND GREEN SILTY FINE TO COARSE SAND	sm	NA	NA	22.3
BH-63	S-32	140.0'-141.5'	LIGHT TAN SILTY FINE TO COARSE SAND-TRACE OF CLAY	sm	0.002	65.0	NA
BH-72	S-29	125.0'-126.5'	TAN SANDY CLAY	cl	NA	NA	47.9
BH-81	ST-8	144.3'-145.0'	TAN, WHITE AND ORANGE CLAYEY FINE TO COARSE SAND	sc	NA	NA	25.0
BH-92	S-32	140.0'-141.5'	TAN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	37

LEGEND:

INDEX PROPERTIES

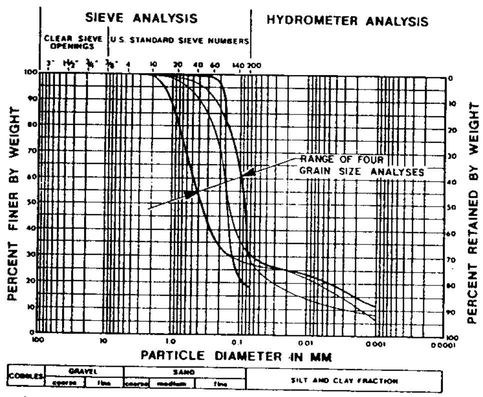
W — WATER CONTENT

D_{50} — DIAMETER AT WHICH 50% OF THE SOIL IS FINER

D_{60} — DIAMETER AT WHICH 60% OF THE SOIL IS FINER

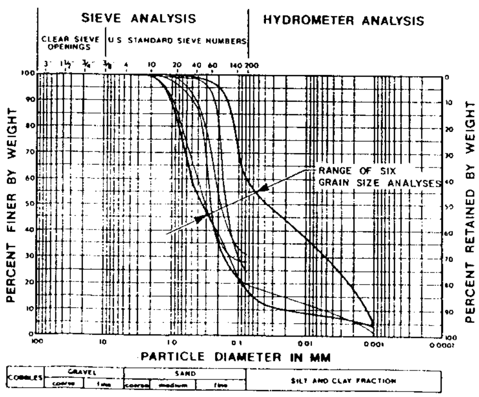
C_u — COEFFICIENT OF UNIFORMITY (D_{60}/D_{10})

CONGAREE FORMATION



BORING	SAMPLE	DEPTH	SOIL DESCRIPTION	U.S.C.S.	D_{50}	C_u	W.%
BH-20	S-33	145'-146'	TAN CLAYEY FINE TO MEDIUM SAND-TRACE OF SILT	sc	NA	NA	27.6
BH-38	S-43	240.0'-241.5'	GRAY SILTY FINE TO COARSE SAND-SOME CLAY	sm	0.018	94.4	19.7
BH-55	S-33	144.8'-146.3'	DARK GRAY & GREEN SILTY FINE TO COARSE SAND-SOME CLAY	sm	0.025	39.2	23.3
BH-63	S-41	220.0'-221.5'	LIGHT GRAY CLAYEY FINE TO COARSE SAND-SOME SILT	sc	NA	NA	14.4

ELLENTON FORMATION



BORING	SAMPLE	DEPTH	SOIL DESCRIPTION	U.S.C.S.	D_{50}	C_u	W.%
BH-20	S-44	250.0'-251.0'	GRAY SILTY FINE TO MEDIUM SAND - TRACE OF CLAY	sm	0.055	50.9	24.9
BH-20	S-47	280.0'-281.5'	GREEN CLAYEY FINE TO MEDIUM SAND	sc	NA	NA	19.8
BH-38	P-1	246.7'	DARK GRAY SILTY FINE TO MEDIUM SAND TRACE OF MICA	sm	NA	NA	25.1
BH-63	S-44	250.0'-251.5'	GRAY SILTY FINE TO COARSE SAND	sm	NA	NA	28.5
BH-63	S-47	280.0'-281.2'	GRAY SILTY FINE TO COARSE SAND-TRACE OF CLAY	sm	0.026	18.8	17.4
BH-90	S-47	290.0'-291.5'	LIGHT GRAY SANDY SILT - SOME CLAY	ml	NA	NA	21.2

NOTES:

1. FOR PLAN AND LOCATION OF BORINGS SEE FIGURE 1-1

FIGURE 2-12

SAVANNAH RIVER PLANT
DEFENSE WASTE PROCESSING FACILITY
SUBSURFACE INVESTIGATION
GRAIN SIZE DISTRIBUTIONS

PREPARED FOR

E.I. du PONT de NEMOURS & COMPANY
INCORPORATED
WILMINGTON, DELAWARE

DAMES & MOORE

Figure 6. Grain Size Distribution Curves for Soils in Various Strata.

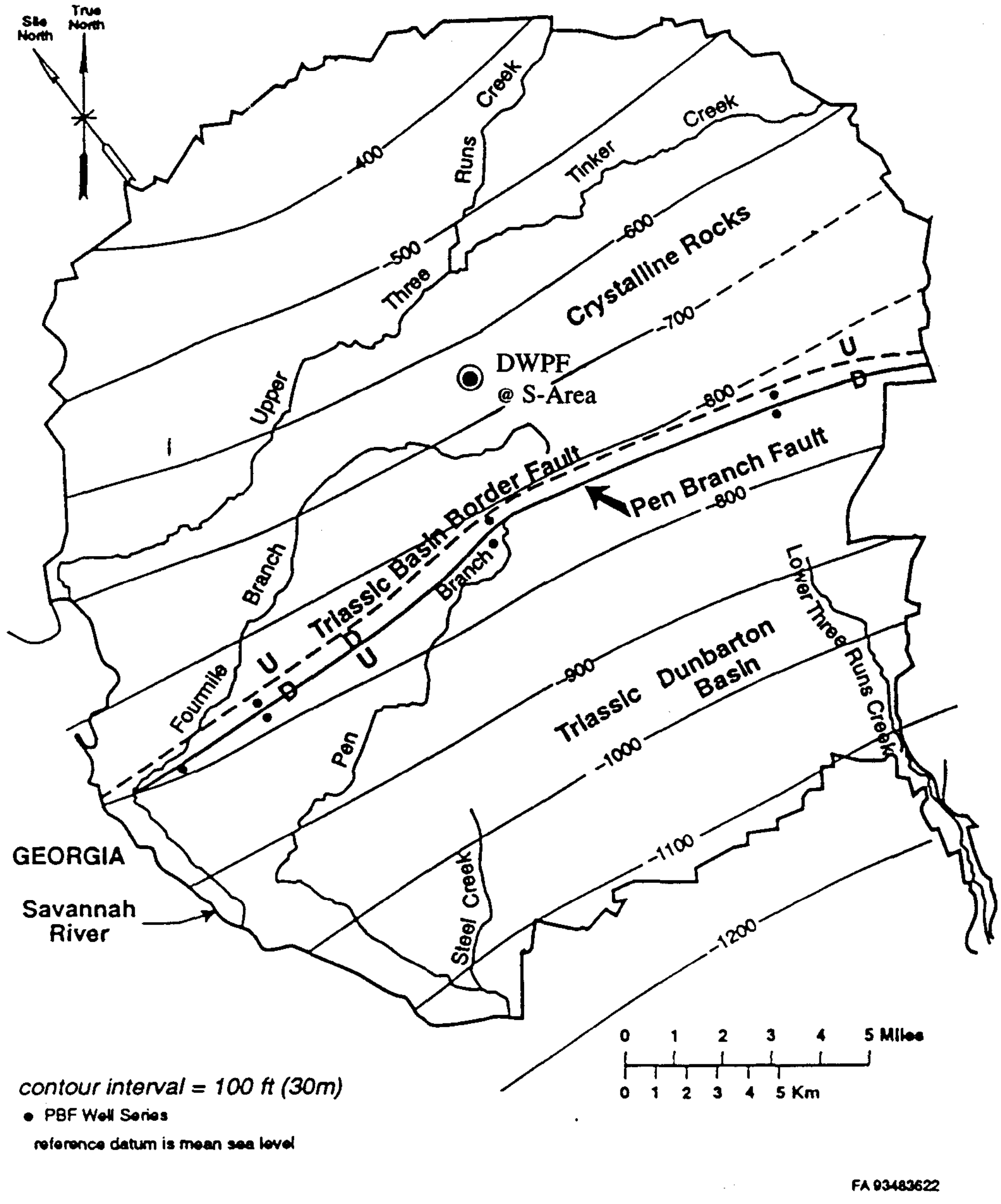


Figure 7. Dunbarton Rift Basin.

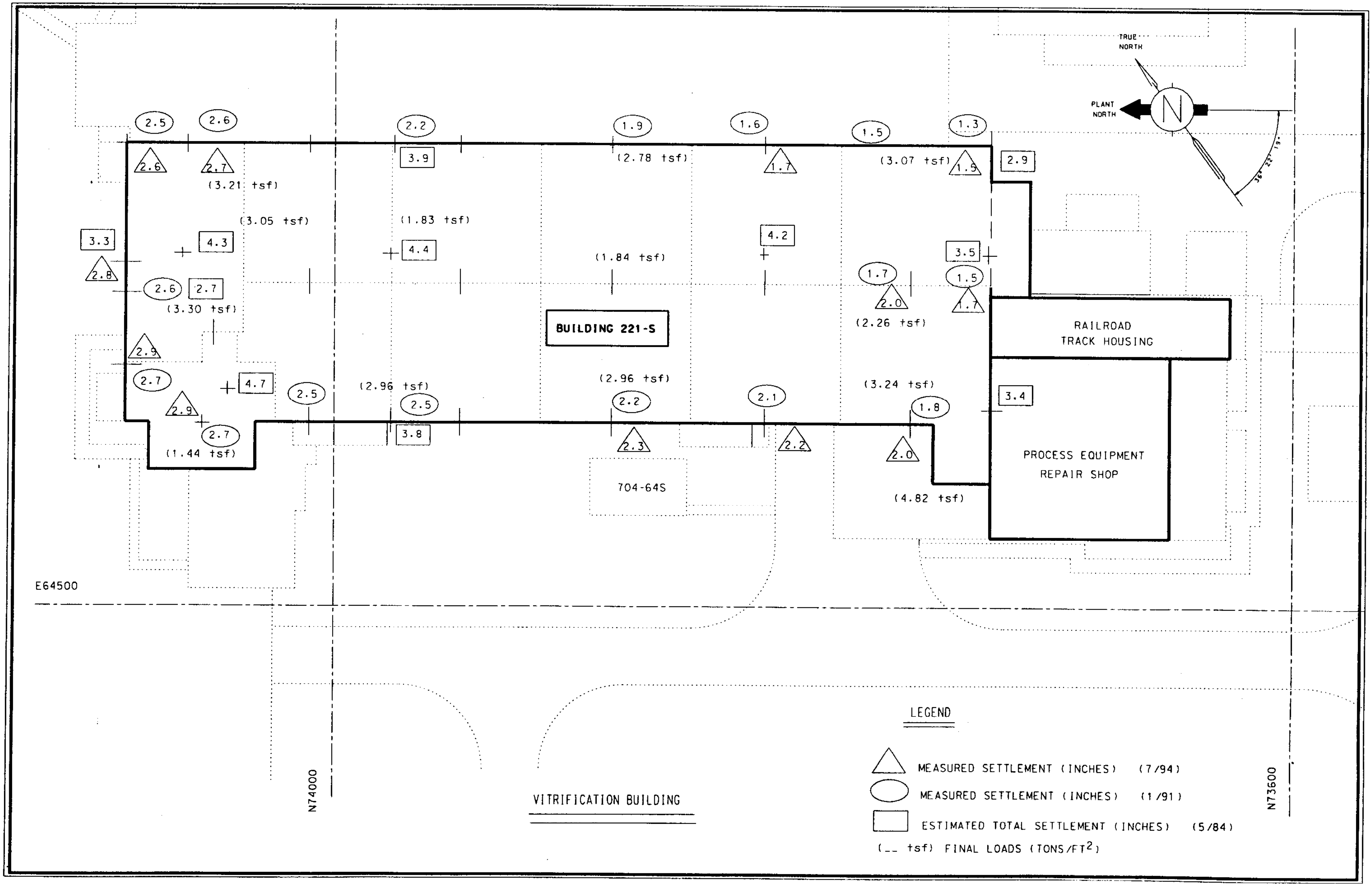


Figure 8. Plan View of Vitrification Building with Estimated and Actual Settlements.

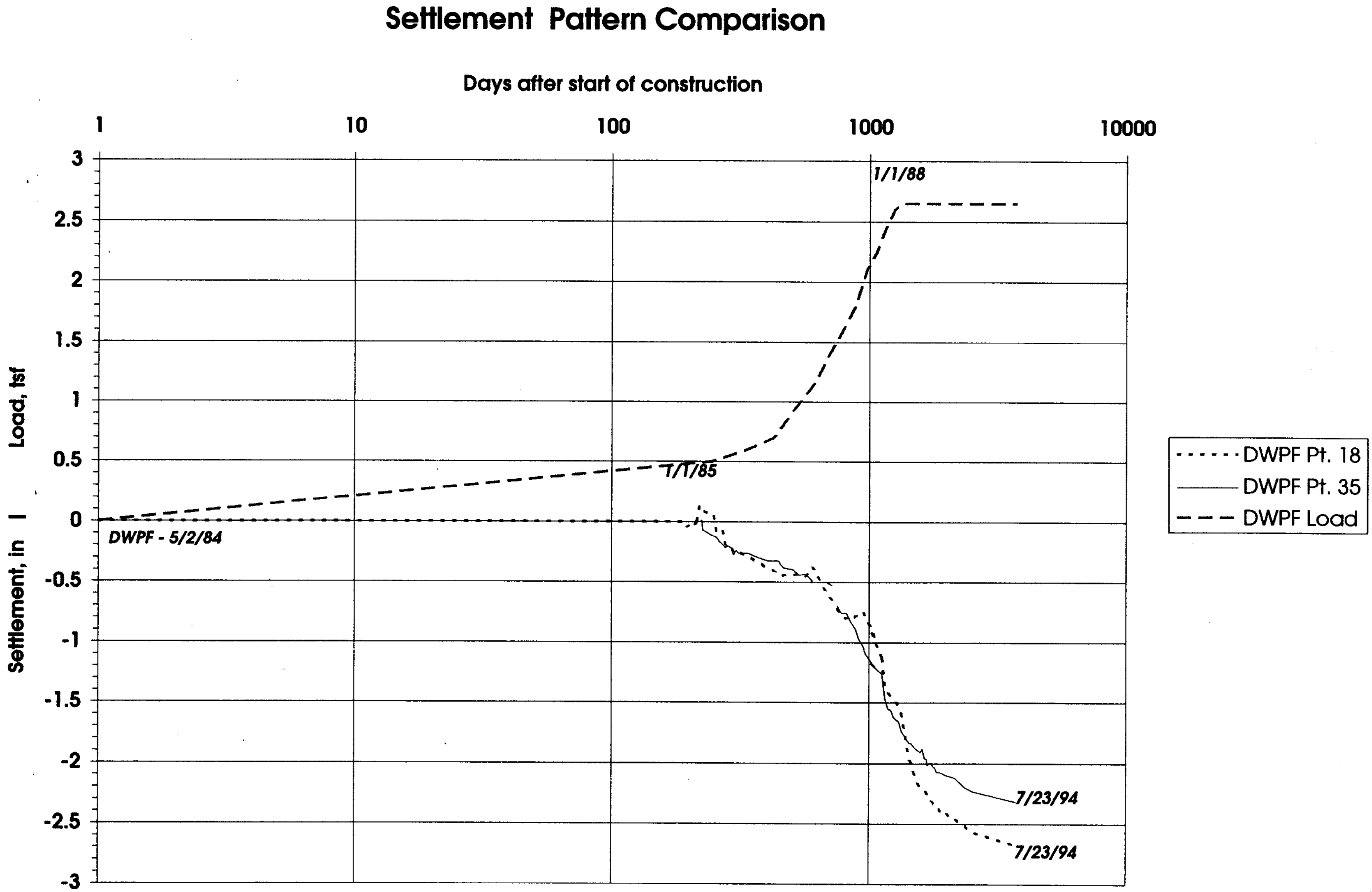


Figure 9. Settlement of DWPF During Construction.

APPENDICES

Note: appendices use figure numbers and references from main text.

APPENDIX A - SOFT ZONES

A.1 Occurrence of Soft Zones

The occurrence of soft or underconsolidated zones and rod drops has been described in numerous drilling reports throughout the central portion of SRS (WSRC, 1994b). The prevailing assumption for the cause of these zones has been the dissolutioning of carbonate rich sediments, resulting in vugular porosity where drill rods meet little penetration resistance. Frequently associated with these soft zones are localized beds of silica-cemented sand or indurated limestone where drilling becomes much harder. For example, during recent drilling in the General Separations Area (E, F, H-Areas), drillers reported rod drops immediately after drilling through a well cemented siliceous or calcareous bed or beds (WSRC, 1994b). Furthermore, rod drops associated with well-cemented, silicified units (hard grounds) have been noted in clastic lithofacies where carbonate percentage is low, but generally present. The silica cemented horizons are generally noted at, or near, the top of the Santee Formation at, or near, an unconformity that separates the Santee Formation from the overlying Dry Branch Formation (WSRC, 1994b). This cemented horizon at SRS appears to be similar to cemented zones found in modern coastal environments where rapid lateral and vertical movement of fresh to saline ground water occurs, causing a high variability in cementation (WSRC, 1994b). Thus, it is possible that the hard grounds, and the underlying soft zones, may have developed during the unconformity at the end of Santee times (WSRC, 1994b). In fact, the hard zones may act as a resistant cap for the underlying soft zones, allowing them to persist as pockets of underconsolidated material through geologic time (WSRC, 1994b).

Three dimensional mapping of soft zones at the In Tank Precipitation Facility (ITP), located about 3/4 of a mile to the southwest of DWPF in H-Area, indicate that low penetration resistances tend to occur in the Santee Formation in a fine to medium-grained, silty or clayey quartz sand having varying amounts of shell fragments and cementation. At ITP, this sandy facies appears to flank a more competent limestone body in a band that is 100 to 200 ft wide. Although the soft zones shadow the limestone in a relatively wide band, individual penetration resistances within this band are highly variable. The soft material appears to occur as pockets or stringers of calcareous material that are laterally discontinuous. For example, at ITP, no lateral continuity in SPT and CPT penetration resistances were found that exceeds of few tens of feet. This high degree of variability is probably attributable to either rapid lateral and vertical changes in the amount of carbonate originally deposited or to subsequent changes in the diagenetic history of the sediments, or both. In cross-sectional view, the soft zones at ITP appear to be concentrated in two horizons that dip approximately 5 % to the west. These horizons roughly parallel the dip of the overlying strata, suggesting that soft zones are not random, but are depositionally controlled. Near the ITP tanks, the upper horizon has a maximum thickness of approximately 10 to 15 ft and is typically found between elevations 170 and 160 ft. The lower horizon is thinner and less continuous. It has a maximum thickness of about 5 ft and is typically found between elevations 138 and 133 ft.

A.2 Grouting Program at DWPF

Because of the critical nature of DWPF and the uncertainty in the characterization data, Mueser (1983) recommended grouting of the Santee Formation underneath all Category I structures. Figure A1 (Mueser, 1984a) summarizes the abandonment grouting data compiled during the subsurface investigation initiated by D'Appolonia (1982a). This figure includes the grout take ratio (grout take divided by nominal borehole volume), grouted depth, borehole diameter, number of bags of cement, and the time charged to grouting. Grout takes were determined by assuming one bag of cement yielded 2.2 cubic ft of grout. This assumed average grout yield per cement bag was later confirmed by Girdler Exploration and Foundation Inc, the drilling subcontractor for D'Appolonia. The grout take ratios presented in Figure A1 are shown within the squares posted adjacent to the boring symbol.

In three of the four borings drilled at the proposed location of the Glass Waste Storage building (Bldg. S250), rod drops and/or calcareous materials were encountered (Mueser, 1983). In Boring No. 86L1P (Figure 1), a rod drop of 5 ft occurred between elevations 143 and 138 ft. Calcareous materials were noted on the field logs in Boring No. 83 and 84. Positive reaction to dilute HCl occurred with some samples from Boring No. 84 between elevations 162 and 126 ft. In Boring No. 62, the only boring drilled within the limits of the Sand Filter building (Bldg. S294), samples from elevations 164 to 139 ft, reacted positively to HCl. However, in Boring No. 66, located immediately north of the Sand Filter building, no rod drops or calcareous materials were noted. In Boring No. 159, located on the east wall of the Fan House (Bldg. S292), a rod drop of 4 ft and low SPT blow counts occurred and calcareous materials were noted by the field inspector. None of these indications of leached conditions were noted in the borings beneath the Vitrification Building (Bldg. S221). However, in Boring No. 24, located adjacent to the south side of the building, calcareous material was noted between elevations 163 and 158 ft (Mueser, 1983).

Mueser (1983), after reviewing the field data, noted a paucity of notations regarding the loss of drilling fluid on the field logs. Drawing on experience in F- and H-Areas, they concluded that a loss of drilling mud typically precedes a rod drop or a zone of unusually low penetration resistance. They also suggested for these zones that the amount of fluid loss is an indicator of the size or extent of the highly porous materials. Because of the similarity of the subsurface conditions in F-, H-, and S-Areas, Mueser also suggested that losses of circulation probably had occurred in conjunction with rod drops in S-Area. However, loss of drilling mud was noted by D'Appolonia for only one of the thirty-three borings which had indications of soft zones. Temporary loss of 90 % circulation was noted in Boring No. 25 at approximately elevation 150 ft, immediately above a rod drop of 5 ft. Neither the quantity of lost mud, the depth, nor the time of circulation return was recorded for that boring.

The scarcity of notations of fluid losses on the field logs implied to Mueser that either no other mud losses had occurred or that mud losses had not been recorded. The latter explanation was preferred by Mueser et al. because some borings were drilled without continuous inspection by D'Appolonia. D'Appolonia's field logs indicated that at least 10 borings were drilled between elevations 130 and 160 ft by the drillers without the presence of D'Appolonia field personnel (Mueser, 1983). Also, Mueser et al. commented that in their experience, very few drillers will note the loss of drilling mud unless specifically requested.

On the basis of the data presented in Figure A1, Mueser (1983) concluded that some soft zones or "voids" of limited lateral extent may exist within the area of Category I structures. They concluded that although these indications are not sufficient by themselves to definitely conclude that extensive voids are present, or that the "voids" represent a severe threat to the performance of the structures, the general lack of notations on the field logs of mud loss during drilling was suspect and lead to a lack of confidence in the recorded information. This implied that perhaps other important occurrences (rod drops or calcareous materials) were unreported.

Drill rigs arrived at the site on March 7, 1984, and began drilling grout holes at the northern end of the Vitrification Building (S221) (Mueser, 1984b). Drilling continued southward within the footprint of the Vitrification Building with grouting closely following the drilling operation. The grout holes for the remaining Category I structures were drilled starting at the Fan House (S292) and continued north to the Sand Filter (S294) and Glass Waste Storage buildings (S250). Also, four secondary grout holes were drilled and grouted in the Vitrification Building. In all, 39 grout holes were completed by March 30, 1984 (Mueser, 1984b).

The grout holes were drilled with truck-mounted drill rigs using four inch nominal diameter tricone roller bits and drilling mud to maintain a stable borehole. During the contract period, the location of the Fan House was moved 10 ft to the south. Therefore, some grout holes in this area were drilled outside the building limits. Grout holes were generally advanced without sampling from the ground surface to elevation 180 ft. The boreholes were then advanced with split-spoon sampling at 5 ft intervals to a minimum elevation of 130 ft, or approximately 10 ft below the base of the calcareous materials. All split spoon samples were logged and classified by Mueser's resident engineer and tested for the presence of calcareous materials (Mueser, 1984b).

The grout was batched one tank at a time. Each tank was calibrated so that grout quantities could be accurately recorded. A typical batch of grout was composed of one 94 lb bag of Type I cement; eight, 5 gallon buckets of sand (calibrated for 0.67 cf per bucket); one-third of a 50 lb bag of bentonite, and 30 gal of water (Mueser, 1984b). The grout yield per batch was approximately 7 cf. The bentonite was first premixed with water in a 130 gal tub to yield sufficient bentonite-water slurry for 4.5 batches.

When grouting could not begin on the same day a hole was drilled, the open hole was flushed prior to grouting with water to remove any heavy drilling mud and cuttings that had settled to the bottom. During the flushing operations and at the start of grouting, the grout pipe was positioned approximately 1 ft above the bottom of the borehole. Before the grout was pumped, the levels of the grout mix within each tank were recorded (Mueser, 1984b).

Generally, grout was pumped at an initial pressure of less than 5 psi, measured at the top of the grout pipe. When the pressure exceeded approximately five psi, pumping ceased and the grout take and depth of the bottom of the grout pipe were recorded. The grout pipe was then raised approximately 10 ft and grouting was restarted. Occasionally, a pressure of 20 psi was required to initiate grout flow, but once flow was obtained, the pressure was decreased to a maximum of 5 psi until the grout take ceased (Mueser, 1984b).

The small grout take ratios (Figures A2 and A3) indicate that large leached zones do not exist underneath DWPF facilities in the Santee Formation. Grout take ratios in all holes, except Hole No. 2, varied from 0.9 to 2.7, with most of the ratios close to 1.3, indicating stable ground conditions. Grout take ratios were calculated by dividing the grout take by the theoretical or nominal volume of the borehole. The grout take ratio in Grout Hole No. 2 was 7.7, which appeared to be excessively high. Two secondary grout holes (Nos. 36 and 37) were installed adjacent to Grout Hole No. 2 (Figure A2) to explore the reasons for this relatively high grout take. The resulting grout take ratios in these secondary holes were approximately 1.0, indicating that a large pocket was not present (Mueser, 1984b). It was later hypothesized during a review of the records that the high grout take in Grout Hole No. 2 was probably due to inadvertent high grouting pressure which caused local fracturing or compressing of the soil mass (Mueser, 1984b).

During drilling of the grout holes, isolated cases of rod drops, mud losses, and calcareous materials were encountered in the north end of the Vitrification and Glass Waste Storage buildings (Mueser, 1984b). These observations suggested the presence of thin layers of calcareous materials that had been leached in the past. However, no voids or continuous layers of loose soil with a fragile structure were encountered (Mueser, 1984b). At the Sand Filter Building, calcareous material was encountered in six grout holes between elevations 131 and 163 ft in layers up to 12 ft thick. This material and the overlying soils were typically medium dense to dense, indicating significant leaching had not occurred (Mueser, 1984b).

Calcareous material was also encountered in all five grout holes at the Fan House between elevations 142 and 162 ft in layers up to 7 ft thick. Very loose materials, including two cases of rod drops, were encountered in the soils immediately overlying the calcareous material. This suggested some leaching of the calcareous material and possible raveling of the overlying soil into the leached zone. Like the Sand Filter building, the calcareous soil underneath the Fan House is generally medium dense to dense. Significant settlement due to further leaching of this soil is not anticipated (Mueser, 1984b).

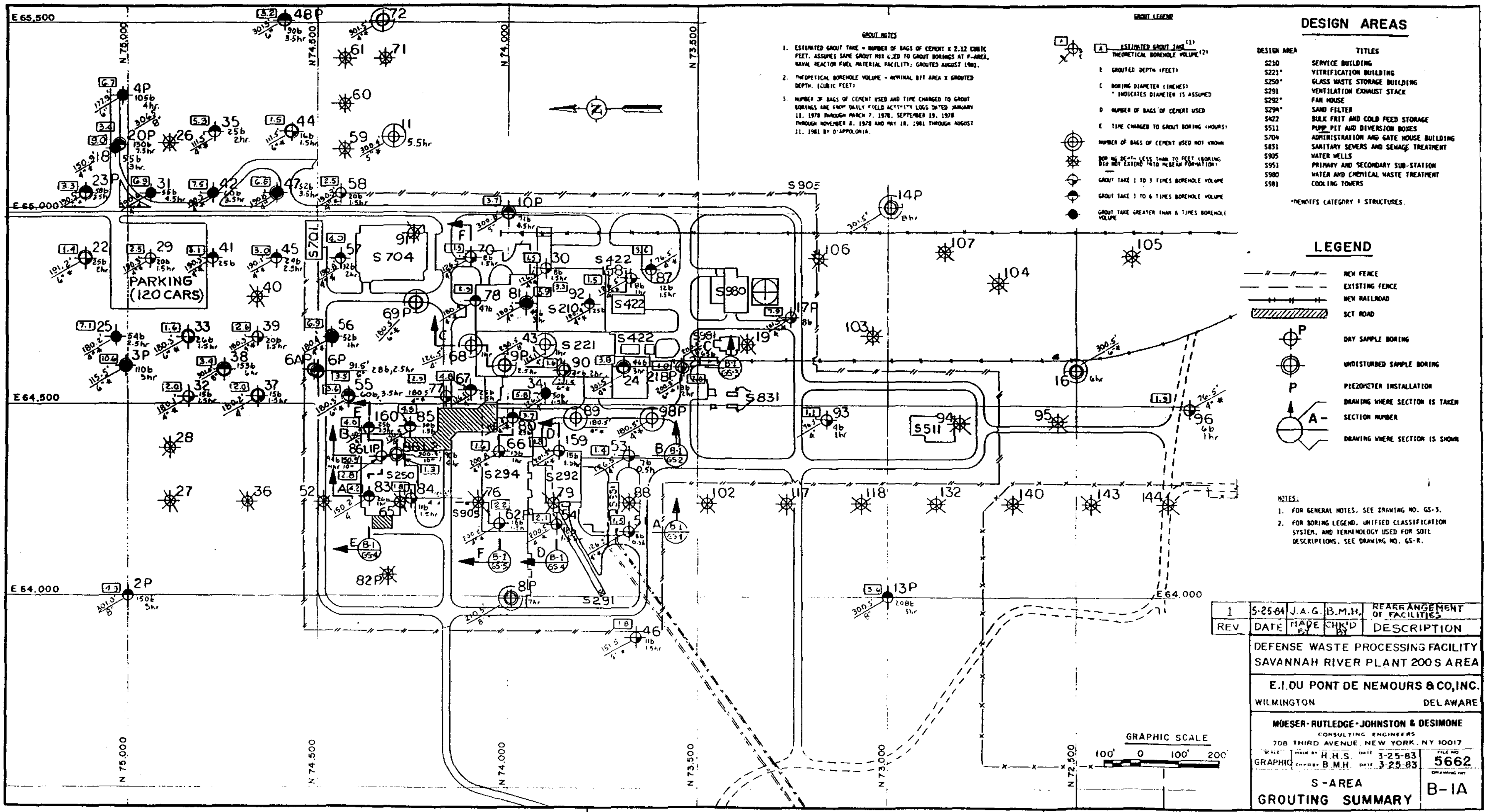


Figure A1. Grouting Summary.

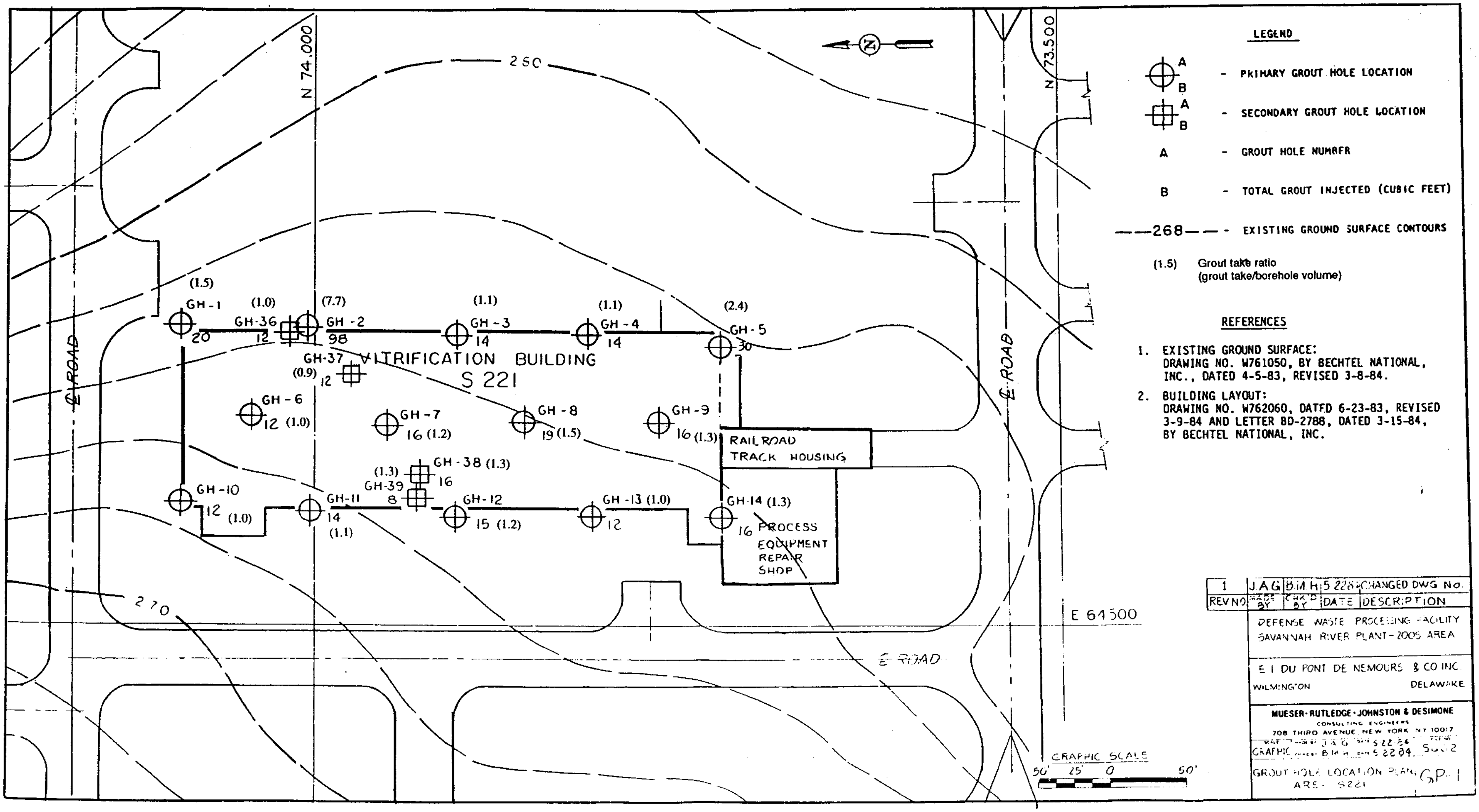


Figure A2. Grout Hole Location Plan and Grout Takes for the Vitrification Building.

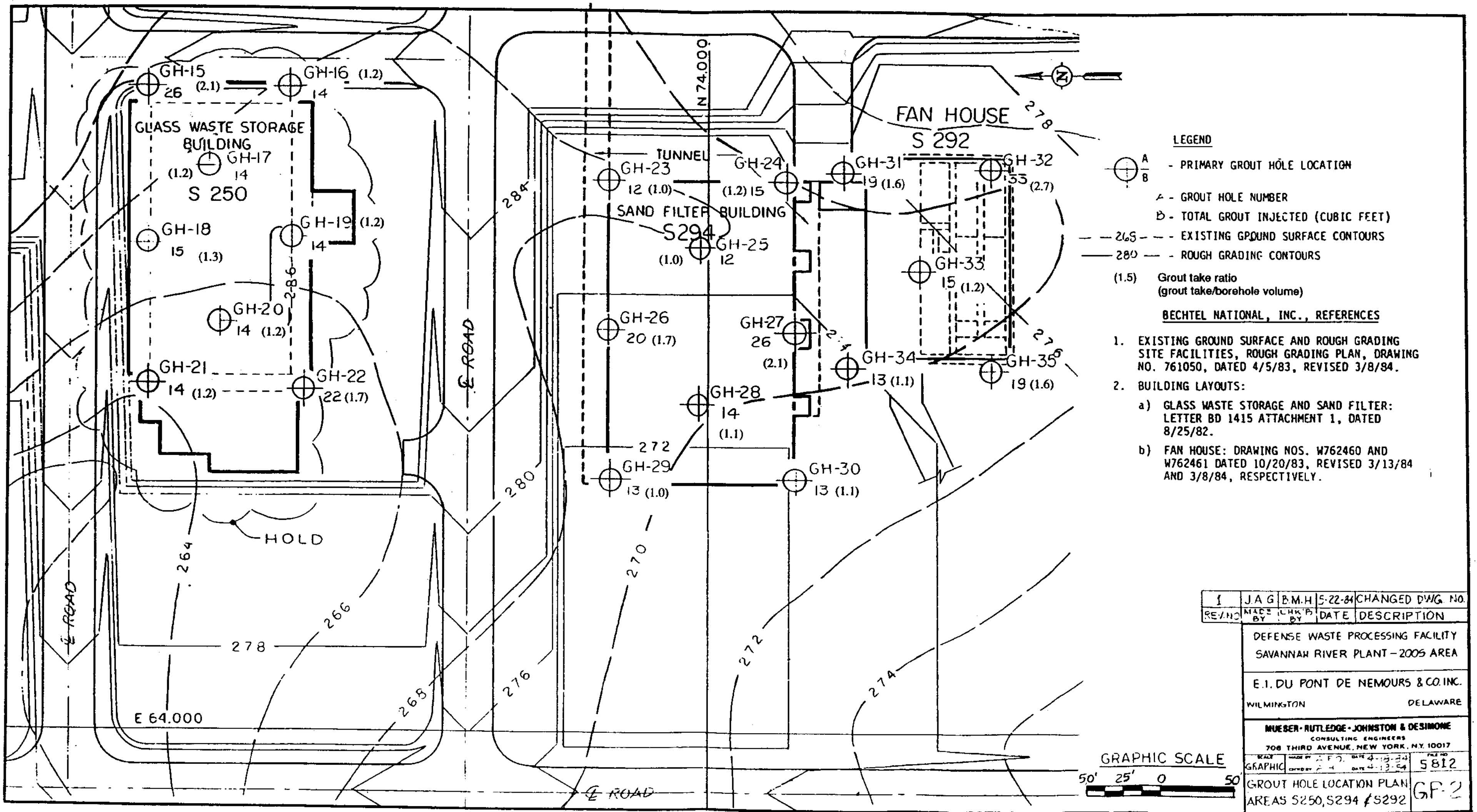


Figure A3. Grout Hole Location Plan and Grout Takes for Various S-Area Buildings.

APPENDIX B - STRUCTURE SETTLEMENT

B.1 Preliminary Settlement Estimates by D'Appolonia

The consolidation state and compression indices of the SRS soil profile has been thoroughly reviewed and discussed by several investigators in F, H, and S-Areas.

D'Appolonia (1982a) performed an extensive field and laboratory investigation on the soils in the Tobacco Road and Dry Branch Formations for the preliminary safety analysis of DWPF. The compiled data included SPT and undisturbed sample borings, piezometers, pressuremeter tests, cross-hole seismic tests, cone penetrometer tests, and laboratory testing. Using one dimensional consolidometer and compressibility results at DWPF, D'Appolonia concluded that the clay layers within the Tobacco Road and Dry Branch Formations were "normally consolidated" (Richards, 1982). Static settlements of the mat foundations were estimated by D'Appolonia (1982a) using a three dimensional elastic stress analysis based on soil moduli obtained from the results of field and laboratory tests. Based on this analysis, approximately 13 inches of static settlement was predicted for the Vitrification Building (D'Appolonia, 1982a). This was based on an average 4.7 ksf surface load and a 2.7 ksf induced load at the top of the Santee Formation (Richards, 1982). Further, because of the large predicted settlement, D'Appolonia recommended precompression of the Tan Clay and Santee Formation by dewatering the Tobacco Road and Dry Branch Formations and designing the Vitrification Building to withstand large differential settlements (Richards, 1982).

Prior to their involvement in the DWPF soil investigation, Mueser had conducted several subsurface investigations in the nearby H-Area and had estimated much smaller settlement (2 to 4 in of total settlement) for similar foundation loads. As a result, DuPont requested Mueser to review the preconsolidation state of the DWPF soils and D'Appolonia's static settlement calculations (Richards, 1982). To complete this review, Mueser (1984a) compared the results of laboratory tests from the DWPF investigation (S-Area) with results taken from investigations in H-Area (Figure B1). The data in this figure are from H-Area, except for the solid hexagonal markers, which indicate S-Area tests. In reviewing the S-Area consolidation tests, Mueser (1984a) noted that distinct breaks in the slope of e vs. log p curves were apparent only in silty clay samples taken from the Tan Clay (stratum C2). Consolidation test results from more sandy strata either did not show a well defined break or were not loaded to a sufficiently high pressure to clearly establish the slope of the virgin compression curve (Mueser, 1984a). Therefore, for S-Area, Mueser (1984a) estimated the preconsolidation pressure for only those samples that were clearly plastic (solid hexagonal markers for stratum C2 in S-Area on Figure B1). Based on these consolidation tests, tank settlement data in H-Area (Tank Nos. 38 through 42), and the fact that the subsurface conditions in H- and S-Areas are similar, Mueser concluded that the soils in both S-Area and H-Area were overconsolidated and that any settlement of the DWPF structures would occur on the recompression part of the consolidation curve (Richards, 1982).

An additional consulting engineer, R. M. Pyke, was also requested to review D'Appolonia's foundation report (Glisson, 1982). Pyke made the following comments about the consolidation state of the S-Area soils:

1. The geological history suggests that the soil profile is overconsolidated. George Siple, in a conversation with Pyke, estimated that 50 to 100 ft of overburden may have been removed from the site.

2. Data from consolidation tests made of samples from B and S-Areas indicate preconsolidation. Some of the samples have enough fines to give a good curve definition of preconsolidation. From these results, Pyke found OCR ranging from 1 to 5.2. He recommended that 2 be used as a representative value.

3. Shear wave velocity measurements from geophysical logs are inconsistent with normal consolidation (i.e., OCR = 1.0).

To resolve differences on the consolidation state of the S-Area and H-Area soils, two meetings were held between D'Appolonia, Mueser, and DuPont on September 8, 1982 and September 23, 1982 (Edinger, 1982). In the latter meeting, Mueser presented an expanded summary of all H-Area consolidation tests and loading versus time plots for Tanks 38 through 42. Mueser had used these data to estimate approximately 3.5 inches of recompression settlement under the DWPF structure. As a result of these meetings, D'Appolonia also agreed that the settlements under DWPF should be determined from moduli computed from observed tank settlements. They revised their original settlement estimates of the Vitrification Building from 13 inches to about 3 to 5 inches of total settlement. D'Appolonia further agreed that the final geotechnical report on DWPF would avoid characterizing the soils as "normally consolidated." They maintained, however, that the high moduli have only been confirmed for loadings up to 4 ksf and for structures where soil disturbance is small (Edinger, 1982).

B.2 Discussion of Consolidation Tests: S-Area Profile

The five tests on clayey soils in S-Area (Stratum C2, Figure B1) exhibited preconsolidation stresses in excess of the existing overburden (Mueser, 1984a). These data show overconsolidation ratios that range from about 1.2 to 2.0. Two tests, however, indicated the possibility that a portion of the Tan Clay is not preconsolidated beyond the final loads imposed by the Vitrification Building (S221). These tests appear to be correlated with similar plastic clays encountered at slightly lower depths in H-Area. At both sites, there appear to be pockets of clayey soils exhibiting normally to slightly overconsolidated behavior (Mueser, 1984a). Two tests on the sandy soils of stratum S3a (Santee/Tinker Fm.) in S-Area also show overconsolidation ratios of about 2 (Figure B1).

B.3 Settlement Estimates by Mueser (1984a)

Final design total settlements were estimated assuming one-dimensional consolidation of the soil profile under applied loads where excavation was treated as a negative loading to determine heave of the excavation bottom. Stress changes in the soil mass resulting from applied loads were computed using a Boussinesq solution. Average ground water levels were assigned to each formation for computation of in-situ effective stresses. Based on available data, the average piezometric surfaces of Tobacco Road, Santee, and Congaree Formations were elevations 245, 240, and 170 ft, respectively.

Soil properties were assigned to each stratum based on an evaluation of the available S-Area laboratory test data and soils data from previously mentioned H-Area studies (Figure B1). Table B1 lists the parameters used in the settlement calculations. Because tank settlement performance data established that the clayey soils of the Tobacco Road and Dry Branch Formations are preconsolidated to stresses greater than the stresses developed in the soil under the foundation, recompression indices were used for all strata except Stratum C2 (Mueser, 1984a).

Table B1. Soil Consolidation Properties Used for DWPF (Mueser, 1984a).

Stratum	Unit Weight (pcf)	Average Water Content (%)	Compression Index (C_c)	Recompression Index (C_r)	Initial Void Ratio (e_o)
S1 (Tobacco Road)	**130**	**21**	-----	**0.009**	**0.56**
S2 (Tobacco Road)	**128**	**22**	-----	**0.008**	**0.59**
S2b (Dry Branch)	**128**	**22**	-----	**0.009**	**0.59**
C2 (Tan Clay)	**106**	**53**	**0.85***	**0.070**	**1.4**
S3a (Santee)	**127**	**23**	-----	**0.009**	**0.62**

** denotes this value was only recommended for the CH soils in the C2 stratum that were found to be normally consolidated. For DWPF, one-half of the C2 thickness was treated as overconsolidated and the other half was treated as normally consolidated.*

In Stratum C2 (Tan Clay), two of the five consolidation tests exhibited preconsolidation stresses less than the final load of the Vitrification Building, indicating that some virgin compression may occur within this layer. To account for this, Mueser (1984a) adopted a conservative approach for the settlement analyses and assumed that one-half of the C2 layer would experience virgin compression and one-half would experience recompression under the net stresses imposed by the structures (Mueser, 1984a).

Total settlements, including soil heave plus settlements due to the net structural loads were estimated for all Category I structures (Mueser, 1984a). Settlement of the Vitrification Building at four stages of construction was also estimated. Figure 8 shows the estimated total settlements (values shown in rectangles) due to final structural loads (values in parentheses) for the Vitrification Building.

B.4 Settlement Monitoring Program by Mueser (1984a)

A pre-construction heave and post-construction settlement monitoring program was initiated at DWPF in June 1984. Excavation was completed by October 1984 and the full structural loads were in place by January 1988. In the last progress report of this program, Mueser Rutledge (1991) reported: "The total cumulative settlement of the mat from November 1984, the date of the mat placement, through April 1991 varied from 2.2 to 2.7 inches in the northern section of the building where the heaviest loads were applied and 1.3 to 2.0 inches in the southern section." Actual cumulative settlement through January 1991 of the mat and differential settlement across the mat were approximately 70 to 95 % of the total predicted settlement. Figure 8 also shows total measured settlements (values in ellipses) as of January 1991 (Mueser, 1991). An additional survey was made at select locations by WSRC Site Geotechnical Services in April 1994 (values in triangles on Figure 8). The measurements show additional settlements of about 0.1 to 0.3 inches since the last measurements taken in January 1991.

In summary, the settlement performance data show good agreement with the settlement prediction, which confirms the reasonableness of the consolidation parameters in Table B1 and the methodology used for the settlement prediction.

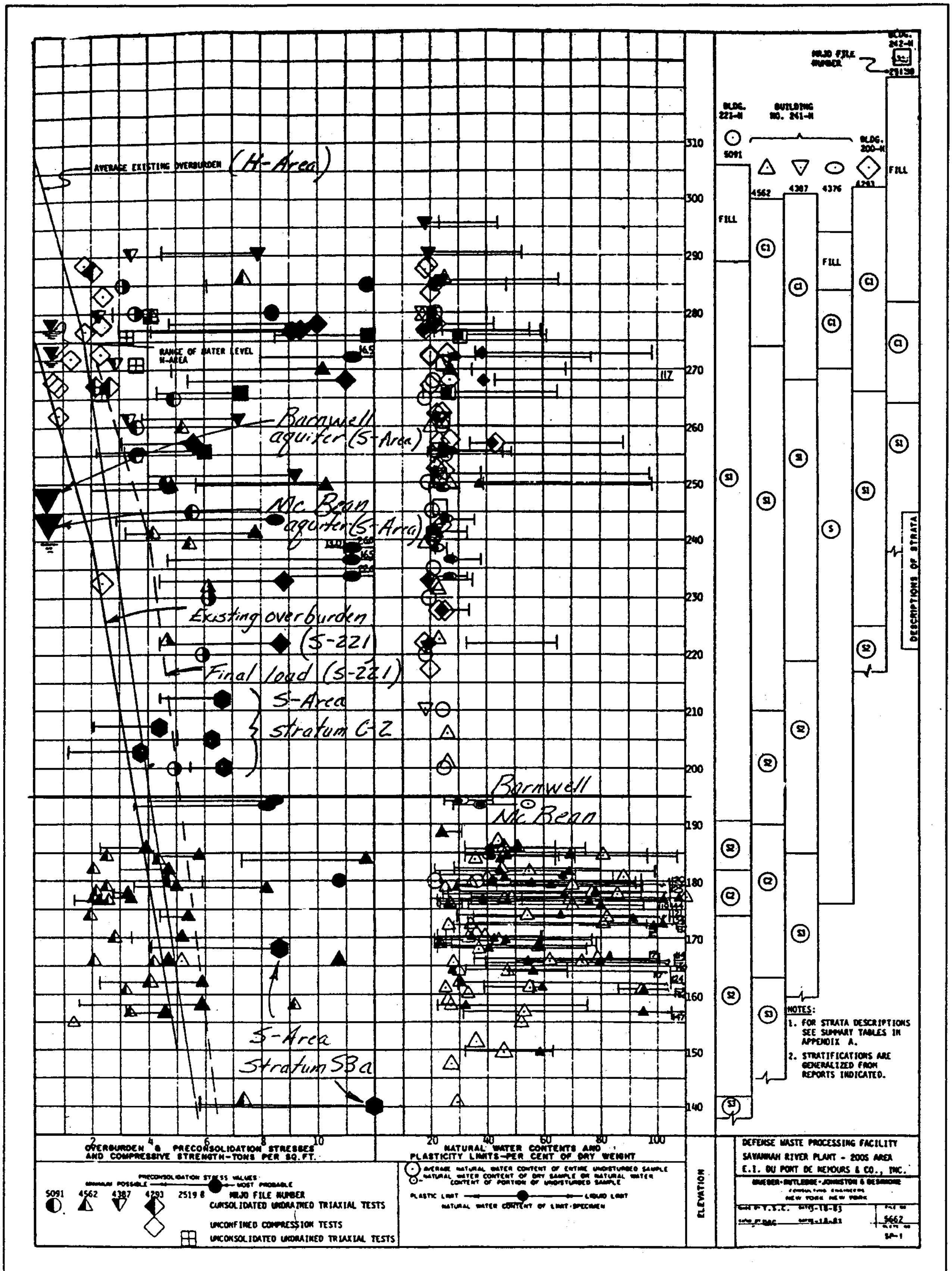


Figure B1. Consolidation State Profile of Soils at DWPF.

APPENDIX C - LIQUEFACTION ASSESSMENT

C.1 Liquefaction - Mueser (1976) Review

Several liquefaction studies have been completed at SRS for the sandy soils of the Upland Unit and the Tobacco Road and Dry Branch Formations. Mueser (1976), at the request of duPont, reviewed unpublished studies in F, H, C, K, L, P, and R-Areas for the purpose of assessing the liquefaction susceptibility of the SRS soils. This report evaluated the possibility of liquefaction for a 0.2 g peak ground acceleration (pga) using the following criteria:

1) relatively cohesionless, granular soils; note that clayey sands and clayey silts with measurable plasticity indices have sufficient cohesion to prevent rapid rearrangement of grain structure and were not considered susceptible;
2) saturated soils; the critical zone must be below the water-table;
3) relatively shallow soils; Mueser (1976) conservatively used 0.2 g as the peak ground acceleration and 50 ft as the critical depth, below which the soils were considered not susceptible;
4) relatively loose soils; for depths less than 30 ft, the critical SPT N-value was considered less than 20 bpf, and for depths between 30 and 50 ft, the critical SPT N-value was considered to be less than 30 bpf.

In their assessment of these criteria, Mueser (1976) reviewed 291 borings - 224 borings in the reactor areas, 42 borings in F-Area, and 25 borings in H-Area. Of these 291 borings, 7 borings revealed intervals considered as susceptible to liquefaction. Another 21 borings were labeled as "questionable" because the soil appeared to be potentially liquefiable based on its low SPT N-values, but it was unknown whether these zones occurred below the water table and what soil type was sampled in this interval.

From the detailed review of C, K, L, P, and R-Areas, Mueser (1976) concluded that the possibility of even a small reduction in shear strength due to partial liquefaction under the effects of a 0.2 g pga earthquake was extremely remote. For F- and H-Area, it was concluded that the only soil layers with a credible potential for reduction in strength during a major earthquake was a 2 ft thick, saturated layer of silty sand at depths of 10 to 12 ft found in two borings in H-Area and a 2 ft thick layer of nonplastic silt at a depth of 70 ft found in two borings in H-Area. Mueser postulated that any possible loss in strength in this deep silt layer would not cause any appreciable surface settlement.

C.2 Initial Liquefaction Analysis by D'Appolonia (1979)

D'Appolonia (1979) completed extensive subsurface investigation and liquefaction assessment at DWPF and wrote its findings in Section 3.6.4 of the Preliminary Safety Analysis Report (PSAR) "Stability of Subsurface Materials". In this draft report, the safety margin against liquefaction for DWPF was considered to be "unacceptably low" and remedial measures were deemed necessary.

This initial liquefaction analysis for DWPF was performed using a 0.26 g pga design basis earthquake (DBE) to determine the dynamically induced stresses. For the dynamic analysis, three different western U.S. time histories were selected from deep alluvium sites. The selected records were: El Centro, M = 6.7, R = 58 km, pga = 0.348 g; San Fernando, M = 6.7, R = 38 km, pga = 0.202 g; Seattle (Olympia), M = 7.1, R = 21 km, pga = 0.280 g; where M = earthquake magnitude, R = epicentral distance, and pga = peak ground acceleration (D'Appolonia, 1979). The duration of strong ground motion was 16 seconds. For additional conservatism, D'Appolonia (1979) chose to use full 30 second duration events with 20 seconds of strong ground motion. For the dynamic analysis, these three time histories were scaled to 0.26 g pga and acceleration time histories and dynamic shear stresses were determined using the computer program SHAKE (Schnabel et al., 1972).

To complete the liquefaction assessment, a program of "undisturbed" soil sampling and laboratory testing using representative samples trimmed from thin-wall tube samplers was carried out. All undisturbed samples for triaxial and cyclic torsional shear tests were recovered from depths of 32 to 57 ft in borings BH-38, BH-47, BH-55, BH-81 (Figure 1). The soils in this interval consist of fine to medium-grained sands with fines contents that varied from 14 to 19 % (D'Appolonia, 1982b).

No comprehensive assessment of liquefaction susceptibility was performed using field performance data (i.e., empirical cyclic stress ratio vs. $(N_1)_{60}$ charts). It is apparent that D'Appolonia had decided laboratory testing was preferable to empirical techniques because of comments made in a later report (D'Appolonia, 1982b): "It should be mentioned that D'Appolonia emphasized the foregoing approach of using site-specific cyclic triaxial test data, rather than one based strictly on standard penetration test (SPT) data for two reasons. First, it was evident that the small amounts of silt and clay present in the predominately sandy soils provided enough of a binder to permit reasonably undisturbed thin-wall tube samples to be recovered and tested. Second, it was apparent that an analysis incorporating only SPT resistance would lead to considerably lower factors of safety. As less confidence was placed at that time on the use of penetration resistance, it was believed that an assessment based on SPT resistance would be overly conservative."

The laboratory samples were subjected to cyclic triaxial shear testing. To correct the results of these laboratory tests to in-situ or field conditions, the factors listed in Table C1 were applied.

Table C1. Correction Factors for Cyclic Triaxial Shear Tests (D'Appolonia, 1979).

Correction	Correction Factor
Soil structure	**1.0**[1]
Geologic time	**1.0**[1]
Overconsolidation ratio	**1.0**[1]
Previous seismic history	**1.25 to 1.5**
Two- versus one-dimensional shaking	**0.9**
State of stress during shaking	**0.7**

[1] *no adjustment was necessary for the first three correction factors because the soils were considered "normally" consolidated.*

The liquefaction assessment consisted of determining the factor of safety against liquefaction by comparing the dynamic strength of the soil to the dynamic shear stress induced by the earthquake (i.e., FS = dynamic soil strength / dynamic shear stress). Based on the SHAKE dynamic analyses and the corrected, in-situ dynamic strength, the minimum factor of safety against liquefaction occurred at a depth of approximately 50 ft and varied from 0.98 for the El Centro record to 1.12 for the Seattle record. D'Appolonia (1979) concluded that for the most part, factors of safety against liquefaction approached acceptable values near the top of the Tan Clay. However, for the soils within the upper Dry Branch and Tobacco Road Formations, the factors of safety ranged from 1.0 to 1.2, which was deemed to be unacceptably low for a nuclear facility. To eliminate the potential for liquefaction, D'Appolonia recommended the soil beneath Category I structures be removed to approximately elevation 215 ft, a depth 5 ft above the Tan Clay, and replaced with compacted backfill (D'Appolonia, 1979). An extensive dewatering program was also proposed to allow for excavation beneath the water table.

C.3 Reanalysis of Liquefaction by D'Appolonia (1982b)

D'Appolonia (1982b) was requested by duPont to re-evaluate the liquefaction potential at DWPF for the 0.2 g pga design basis earthquake that was being proposed at the time by URS/Blume (1982). The initial D'Appolonia (1979) analysis was performed at a peak ground acceleration of 0.26 g. Like the GEI (1983) analysis, D'Appolonia's re-analysis used two basic approaches: 1) empirical methods using field performance data (Section C.3.1) based on the Simplified Procedure (Seed and Idriss, 1981a), and 2) analytical methods (Section C.3.2) using dynamic strengths determined from laboratory tests on undisturbed samples.

Dynamic analyses using SHAKE were performed to obtain the earthquake loading for both approaches. For the SHAKE analyses, the three earthquake records given in Section C.2 were scaled to a peak ground acceleration of 0.2 g. The records were analyzed using 30 seconds of duration with 20 seconds of strong ground motion (D'Appolonia, 1982b). From these results, a free-field cyclic stress ratio versus depth profile was established for the site for each of the three records.

C.3.1 Reanalysis by D'Appolonia (1982b) Using Field Performance Data

D'Appolonia concluded that the DWPF SPT N profile was statistically homogeneous and that the re-analysis should be based on average properties (D'Appolonia, 1982b). They decided to combine the SPT N-values given in Groups 1 through 10 (Figures 3 and 4) into a composite group, Group 11, and perform the liquefaction analysis on that group. Other in-situ properties, including grain-size distribution, soil classification, soil unit weight, and moisture content, were obtained from previous investigations (D'Appolonia, 1982a).

Based on these data and analyses, the minimum factor of safety against liquefaction, averaged for the three accelerogram records, was 1.1. The minimum factor of safety for the empirical evaluation occurred at approximately elevation 225 ft at the apparent contact between the Tobacco Road and Dry Branch Formations.

C.3.2 Reanalysis by D'Appolonia (1982b) Using Laboratory Test Results

The results of the cyclic triaxial and cyclic torsional shear test conducted on the DWPF soils are shown on Figure C1 and have been discussed by D'Appolonia (1979), GEI (1983), and by this report. Figure C2 shows these same data after they have been corrected for membrane compliance (D'Appolonia, 1982b). GEI (1983) concluded that correction for membrane compliance was not warranted (see Section C.4.2.3) D'Appolonia (1982b) used the lower bound curve in Figure C2 to determine the cyclic stress ratio for an unstated number of cycles. It is not clear from the report how many equivalent cycles, N_{eq}, were used in the analysis by D'Appolonia. The empirical analysis described in the previous section was analyzed for a M = 6.0 earthquake. The correction factors listed in Table C2 were used to adjust the laboratory cyclic shear ratio to field conditions (D'Appolonia, 1982b).

Table C2. Correction Factors for Cyclic Triaxial Shear Tests (D'Appolonia, 1982b).

Correction	Correction Factor
Soil structure	**1.0**[1]
Geologic time	**1.0**[1]
Overconsolidation ratio	**1.0**[1]
Previous seismic history	**1.25 to 1.50**
Two- versus one-dimensional shaking	**0.9**
State of stress during shaking	**0.70**

[1] *It was considered inappropriate to infer correction factors greater than 1.0 (i.e., increased strength) to account for effects of soil structure, geologic time, and overconsolidation. Because the tests were conducted on "high-quality trimmed samples", rather than on reconstituted samples, D'Appolonia (1982b) considered the effects of the first three correction factors as retained by the samples.*

The multiplication of the correction factors in Table C2 yields a net correction factor that ranges from approximately 0.8 to 0.95. Using the most conservative net correction factor of 0.8, the minimum factor of safety is approximately 1.2 to 1.3 for the three earthquake records. For a net correction factor of 0.95, the minimum factor of safety is approximately 1.3 for all three earthquake records (D'Appolonia, 1982b).

It was concluded that the DWPF profile has adequate resistance against liquefaction for the 0.2 g design earthquake (D'Appolonia, 1982b). The minimum factors of safety were found at depths from 50 to 70 ft below ground surface. The minimum factors of safety in this zone for the SPT N and laboratory test data were 1.1 and 1.2, respectively.

C.4 Liquefaction - Geotechnical Engineers Inc. (1983)

The liquefaction assessment in the draft PSAR by D'Appolonia (1979 and 1982b) was redone and revised by Geotechnical Engineers Inc. (GEI, 1983), which was acting as a subcontractor to Mueser. This reevaluation appears to stem from the need to independently assess the conclusions and recommendations of D'Appolonia and to revise accordingly Section 3.6.4.8 "Liquefaction and Cyclic Mobility Potential" of the PSAR.

Ground failure analysis performed by GEI (1983) consisted of two steps. First, the steady-state strength of the sand was determined. If the sand was found to be contractive (i.e., undergoes a volume decrease during undrained shear), it was considered to be susceptible to liquefaction failure (Section C.4.1). Second, if the sand was found to be dilative (i.e., undergoes a volume increase during undrained shear), it was not considered to be susceptible to liquefaction failure, but was evaluated for "cyclic mobility" (Section C.4.2).

Like D'Appolonia, GEI used SHAKE to complete the dynamic analysis. The dynamic analyses showed that the El Centro accelerogram was the most critical case and provided the closest fit to the recommended URS/Blume response spectrum. Like D'Appolonia (1979), the duration of strong ground motion was determined as 16 seconds. This duration enveloped the requirements of Bolt (1973?), Trifunac and Brady (1975?), and URS/Blume (1982) for deep, cohesionless material. For additional conservatism, a full 30 second duration with 20 seconds of strong ground motion was used.

C.4.1 Liquefaction Failure Assessment

Liquefaction failure, as used by GEI, means that the static and dynamic driving shear stresses are greater than the steady-state strength of the liquefied soil, thus essentially unmitigated deformation can occur resulting in bearing capacity failure.

To determine the steady-state strength of the DWPF soils, nine series of consolidated-undrained triaxial tests were performed on undisturbed samples taken between elevations 200 and 250 ft. Each series consisted of three tests, with each of these tests performed at different effective confining pressures. Of the nine series of tests, four had been performed on soils that were classified as clay and were not used to determine the steady-state strength. The remaining five series (fifteen individual tests) were performed on sands with less than 20 % clayey fines and on silty sands (GEI, 1983).

Prior to testing, an estimate of the density change between in-situ conditions and reconsolidation of the specimen to final effective confining pressure was made to determine the in-situ void ratio. Each specimen was then sheared, undrained, to axial strains on the order of 10 to 15 %. From the axial stress versus strain data and the pore pressure versus axial strain data, estimates of the steady -state shear strength were made and plotted as a function of the in-situ void ratio to define the steady-state line (GEI, 1983).

The driving shear stresses were computed for the Vitrification Building on the basis of a static building load of 7.0 ksf using the computer program SSTAB1 (Wright, 1974). The distribution of vertical stress beneath the structure to account for building acceleration during the seismic event was computed using a trapezoidal distribution. For the maximum case, the vertical stress varied from 2.1 ksf on one side of the building to 13.1 ksf on the other (GEI, 1983).

The driving stresses imposed by the Vitrification Building were compared with the steady-state strengths determined from the steady-state line. The liquefaction failure assessment showed the steady-state shear strength to be higher than the combined peak dynamic shear stress and the static load of the Vitrification Building for 13 of the 15 tests performed. As a result, GEI (1983) concluded that the structure was not susceptible to bearing capacity failure.

C.4.2 Assessment of Cyclic Mobility

Cyclic mobility, as used by GEI, is a transient softening of the soil due to increased pore pressures, which can lead to some limited deformation during strong ground motion. To assess the cyclic mobility potential of the DWPF soils, GEI used two general approaches: 1) empirical methods (Section C.4.2.1), and 2) analytical methods (Sections C.4.2.2 and C.4.2.3). The empirical methods are based on empirical curves of field performance data for past earthquakes and the analytical methods are based on laboratory testing of undisturbed samples primarily collected by D'Appolonia (1979).

C.4.2.1 Empirical Methods (SPT N-values)

Previously, D'Appolonia had subdivided the DWPF site into 11 boring groups as shown in Figure 3. The corresponding SPT N-value profiles for these groups are given in Figure 4. Group Number 4 contains the borings near the Vitrification Building. Ultimately, the SPT N-values from the 24 borings comprising Groups 1, 2, and 4 were used in the cyclic mobility analysis. GEI noted that the trends in penetration resistance with depth are similar for all borehole groups, suggesting that the profile is reasonably homogeneous. Above elevation 250 ft, the materials are predominately clays or clayey sands and have mean SPT N-values exceeding 15 to 20 bpf. Also, this zone has relatively high mean shear wave velocities ($V_S > 1150$ ft/s) and high, mean CPT resistances ($q_C > 100$ kg/cm^2), thus cyclic mobility was not evaluated in this zone (GEI, 1983).

Between elevations 200 to 250 ft, the mean SPT N-values are less than the overlying material and the soil contains significantly less fines. This interval was identified as the critical zone for cyclic mobility analysis. The soils in this zone probably comprise the sands of the Tobacco Road Formation and the Irwinton Sand Member of the Dry Branch Formation and consist of the soil strata S1, S2a, and S2b identified by Mueser (1984a). These soils typically consist of clayey fine to coarse sand (S1), fine to medium sand with a trace of silt (S2a), and fine to medium sand with a trace of clay (S2b). Extensive grain-size and hydrometer tests performed at DWPF indicate that for soils with more than 10 % fines, the majority of these fines tend to be in the clay-size range (GEI, 1983). Specifically, these tests show that samples with more than 20 % fines (less than 0.074 mm) have at least a 15 % clay (less than 0.005 mm) (GEI, 1983). These data suggest that sands with more than 20 % fines are not susceptible to ground failure according to the criteria given by Wang (1979).

At approximately elevation 200 ft, the Tan Clay is found in many boreholes and its base forms the contact between the Santee and Dry Branch Formations. The Santee Formation at DWPF is generally characterized as a slightly silty to clayey sand with intermittent calcareous zones. SPT N-values in the Santee range from 25 to 35 bpf (GEI, 1983). This formation was considered to be less susceptible to cyclic mobility for the following reasons (GEI, 1983):

1. The corrected SPT N-values for the Santee Formation are generally greater than for the Dry Branch and Tobacco Road Formations.
2. The earthquake induced cyclic stress ratio is less for this zone than in the overlying formations.
3. The Santee Formation tends to have a slightly higher fines content than the critical zone in overlying formations.

Consequently, only the materials above the Tan Clay (about elevation 200 ft) and below the groundwater table (about 245 ft) are considered in the assessment.

Because the fines content is known to affect pore pressure generation and the dynamic response of granular soils, GEI (1983) further divided the soils into three general groups based on grain-size data and field soil descriptions as presented in Table C3.

Table C3. Soil Group, Percent Fines, and Unified Soil Classification for DWPF Soils.

Soil Group	Percent Fines	Unified Soil Classification
A	**> 10 %**	**SC, SM**
B	**7 to 10 %**	**SP-SC, SP-SM**
C	**< 7 %**	**SP**

In each group, a significant number of split spoon samples with the lowest SPT N-values were subject to grain-size and hydrometer analysis to confirm the grouping based on the visual classification. Ultimately, the empirical curves proposed by Tokimatsu and Yoshimi (1982) were selected to determine the soil's resistance to cyclic mobility (Figure C3). GEI concluded that the 10 % shear strain curve shows good agreement with the field evidence of ground failure and selected a slightly conservative line (dashed straight line in Figure C3) to represent the boundary between ground failure and non-ground failure. These curves were normalized to the M = 6.6 evaluation basis earthquake according to the methodology recommended by Seed et al. (1983).

Figure C4 shows the ground failure criterion lines as a function of depth for soil groups A, B, and C. Data points lying to the right side of these lines do not meet the ground failure criteria and those lying to the left side are considered to be susceptible to cyclic mobility. Table C4 summarizes the percentage of SPT N-values that are below the failure criteria as a function of elevation (GEI, 1983).

Table C4. Percentage of SPT N-Values Meeting the Ground Failure Criteria.

Elevation (ft, msl)	Number and Percentage of SPT N-values Below Failure Criteria
240 to 250	**0 out of 43, or 0 %**
230 to 240	**0 out of 42, or 0 %**
220 to 230	**6 out of 38, or 16 %**
210 to 220	**3 out of 39, or 8 %**
200 to 210	**1 out of 33, or 3 %**

GEI (1983) concluded the SPT N-values falling below the ground failure criteria occur randomly between elevations 200 and 230 ft and do not correspond to a vertically or horizontally continuous zone, but occur as isolated pockets. Thus, the potential for cyclic mobility under the evaluation basis earthquake was considered to be negligible.

In addition, GEI used the cone penetrometer (CPT) data to evaluate the liquefaction potential. The CPT data were evaluated using the relationship proposed by Zhou (1980). Based on the results of the CPT data, GEI (1983) concluded that essentially all material penetrated by the cone penetrometer exceeds the ground failure criteria and the potential for cyclic mobility was negligible.

C.4.2.2 Analytical Methods (Undrained Cyclic Triaxial and Torsional Shear Testing)

Procedures outlined by Seed (1979) were used to assess the dynamic shear strength of the DWPF soils (GEI, 1983). Results of cyclic triaxial and cyclic torsional shear tests performed previously by D'Appolonia (1979) were reanalyzed and plotted in Figure C1. These results are plotted in terms of cyclic stress ratio ($\sigma_d/2\sigma'_o$ or τ/σ'_o) versus the number of cycles to 5 % double amplitude strain. The resistance to cyclic loading measured in the torsional tests is somewhat higher than the triaxial tests; thus, a low average line (solid heavy line on Figure C5) was drawn based on the triaxial tests (GEI, 1983).

The magnitude of membrane penetration for the DWPF soils was estimated by performing a series of four tests on each of two soil mixtures obtained from Boring BH-24. The soil mixtures were made by combining several zones of soil that were similar in grain-size characteristics to obtain large enough samples to perform the series of tests (GEI, 1983). The tests were conducted on 1.4 in diameter and 4 in diameter specimens using a procedure by Martin et al. (1978). A review of the data by GEI (1983) indicated that no significant correction was needed for either the 1.4 in diameter or the 4 in diameter samples. Thus, no correction for membrane penetration was applied.

GEI (1983) used nine equivalent cycles for a M = 6.6 earthquake. Consequently, value of 0.26 was used for the cyclic stress ratio induced by the earthquake ($\tau/\sigma'_o = 0.26$ from Figure C1). This value was corrected to field conditions using the correction factors in Table C5.

Table C5. Correction Factors for Cyclic Triaxial Shear Tests (GEI, 1983).

Correction	Correction Factor
Soil Structure	**1.0**
Geologic time	**1.5**[1]
Overconsolidation ratio	**1.4**[2]
Previous seismic history	**1.0**[1]
Two- versus one-dimensional shaking	**0.9**
State of stress during shaking	**0.65**

[1] *for the DWPF analysis, GEI conservatively combined the effects of geologic time and previous seismic history into a combined correction factor of 1.5. In this table the combined correction factor of 1.5 is shown in the geological time column and the previous seismic history factor is 1.0.*

[2] *consolidation tests on the Barnwell Formation generally show OCR ranging from 2 to 4. A conservative OCR value of 2 was used for this analysis. This gives an OCR correction factor of 1.4.*

The multiplication of all the above correction factors yields a net correction factor of 1.23. The cyclic stress ratio adjusted to field conditions was determined as 0.32.

Using the computer program SHAKE, GEI (1983) concluded that the average cyclic stress ratio for the soils between elevation 200 and 250 ft produced by the M = 6.6 design basis earthquake ranged from approximately 0.13 to 0.15 (see El Centro Adjusted Line in Figure C6). Based on laboratory test data, the average computed factor of safety against cyclic mobility ranged from 2.1 to 2.5. GEI therefore concluded that the potential for cyclic mobility was negligible.

C.4.2.3 Analytical Methods (Threshold Strain Method)

The Threshold Strain Method proposed by Dobry et al. (1982) for predicting pore water pressure build-up as a function of shear strain was utilized. Dobry et al. have found that the generation of excess pore pressure in laboratory samples of saturated sand can be predicted by the amount of shear strain, γ. For the same level of shear strain, they also observed that the induced pore pressure is not dependent upon the density of the sand or the induced stress level.

The method is based on a threshold cyclic strain, γ_t, in which about 0.01 % is required to initiate a build-up of dynamically-induced pore pressure. Below this threshold shear strain, there is no significant porewater pressure build up and no cyclic mobility can occur. This method requires in-situ measurements of the low strain shear modulus (G_{max}), laboratory measurements of G/G_{max} versus γ, γ_t, and the pore pressure versus cyclic shear strain relationship. The level of cyclic shear strain induced by the earthquake, γ_c, is computed as a function of the peak surface acceleration and the G/G_{max} versus γ relation. This value is then compared to the threshold cyclic shear strain to assess the potential of cyclic mobility. Values of γ_c were computed for the design earthquake using SHAKE (GEI, 1983). Levels of γ_c in the critical zone reached a maximum value of approximately 0.06 %, which is just slightly above the threshold cyclic strain, γ_t. For the computed γ_c, the pore pressure build-up is approximately 15 % of the effective confining stress. Results from cyclic triaxial shear tests show that pore pressure build-up at this small level of strain does not produce any significant softening of sands. GEI (1983) concluded that cyclic strain would be very small for the design basis earthquake and that the potential for cyclic mobility is negligible.

C.5 WSRC Analysis - Evaluation Basis Earthquakes and Dynamic Response Analysis

At the Replacement Tritium Facility (RTF) (BSRI, 1993) and the In-Tank Precipitation Facility (ITP) (WSRC, 1994c), design basis enveloping spectra such as the URS/Blume Spectrum (1982) were not recommended for liquefaction evaluation because these design spectra do no represent a specific earthquake (Stephenson et al., 1993). Results of a probabilistic seismic hazard study show the seismic hazard at SRS can be characterized by a local M = 5 to 6 event with a pga of 0.19 g and a distant M = 7.5 event with a pga of 0.11 g (Stephenson et al., 1993). These controlling earthquakes were also selected as consistent with DOE probabilistic hazard acceptance criteria adopted at that time. A spectral shape was taken from the local event spectrum developed for K reactor by Geomatrix (1991). The distant event spectrum was recommended as an unscaled spectra (Stephenson et al., 1993).

Time histories have been fitted to the local and distant spectra for use in SHAKE analysis (BSRI, 1993). For this analysis, the dynamic stresses induced by the earthquake were calculated using SHAKE91. The shear wave velocity profile from GEI (1983) was input into SHAKE91 along with the shear modulus and damping curves developed for ITP.

Figure C6 shows the cyclic stress ratio (CSR) profile at DWPF for the distant event (Charleston M = 7.5 line), which was used for this analysis. Based on previous studies at RTF and ITP, the distant earthquake is the controlling earthquake for liquefaction evaluation, thus the local event will not be discussed further.

C.6 WSRC Analysis - Development of Liquefaction Curves and Correction Factors

To account for significant differences in the genesis, age, and composition of the older SRS sediments, it was decided at the onset of the RTF investigation to develop site-specific liquefaction curves for the Tobacco Road Formation. Many researchers have proposed empirical curves to calculate liquefaction resistance as a function of SPT $(N_1)_{60}$; however, these curves are mainly based on case histories of Holocene sand liquefaction. Such curves do not reflect the apparently higher resistance to liquefaction evidenced by older sand deposits. Schmertman (1991) summarizes many of these studies and has attempted to quantify the effects of soil aging. He concludes that soil aging over engineering time (40 to 60 years) can cause a general 50 to 100 % improvement in many key soil properties such as static and dynamic shear strength. Furthermore, detailed geotechnical characterizations at RTF and ITP suggest that the Tobacco Road sediments are significantly less susceptible to liquefaction than the recent sediments comprising the standard liquefaction susceptibility curves proposed by Seed et al. (1983). The Tobacco Road Formation is about 40 million years old (late Eocene), whereas the sediments in the Seed et al. database are generally less than 10,000 years old (Holocene). Arango et al. (in press, 1994) has shown that the liquefaction resistance of the Tobacco Road sands is about 2 to 3 times greater than that of clean, Holocene sands.

The RTF liquefaction curve (Figure C5) was determined from laboratory cyclic triaxial shear testing and corrected to in-situ conditions using standard techniques (BSRI, 1993). Table C6 presents the correction factors adopted.

Table C6. Correction Factors for Cyclic Triaxial Shear Tests (BSRI, 1993).

Correction	Correction Factor
Static overburden correction factor, K_σ (Seed and Harder, 1990)	**varies with depth (see eqn. 5)**
Static driving shear stress correction factor, K_α (Seed and Harder, 1990)	**1.0**
Sample disturbance	**1.0**[1]
State of stress during shaking	**0.64**
Membrane compliance	**1.0**

[1] *no credit was taken for sample disturbance. Some investigators have suggested factors as high as 1.5.*

Using best-fit approximation of the RTF cyclic stress ratio curve (Figure C5), and the correction factors given in Table C6, the factor of safety against liquefaction for a given value of $(N_1)_{60}$ is calculated using:

$$FS = CSR \text{ (from Figure C5) } / \text{ CSR (from Figure C6, Charleston Event).}$$

Boreholes BH-43, 46, 54, 68, 83, 86, 90 and 9 were selected for the liquefaction analyses. These boreholes were selected because they are found near, or beneath, Category I structures and grain-size distribution data were available to complete the liquefaction analysis. The results show factors of safety against liquefaction ranging from about 1.2 to 4.0. Thus, for the evaluation basis earthquake used, the potential for liquefaction at DWPF is negligible.

C.7 WSRC Analysis - Volumetric Strain and Settlement

The total cumulative settlement resulting from a seismic event is the sum of settlement due to liquefaction and partial liquefaction. As with previous studies (BSRI, 1993; WSRC, 1994c), liquefaction is assumed to occur for $FS \leq 1.15$ and partial liquefaction is assumed to occur for $1.15 < FS \leq 2.2$ of each increment.

Utilizing the methodology described in BSRI (1993) and the results from the eight boreholes discussed in the previous section, dynamic settlement due to the distant Charleston event were computed. The results show that dynamic settlement is less than about 0.5 inch for all cases.

C.8 Liquefaction and Settlement Summary

Liquefaction and settlement results performed by WSRC Site Geotechnical Services are presented in Appendix C. The analyses carried out for DWPF were described in the previous sections. WSRC liquefaction analysis indicates no liquefaction for the distant Charleston event ($M = 7.5$, pga = 0.11 g). Furthermore, dynamic settlement due to the same event is expected to be less than 0.5 inch. As with previous liquefaction studies (GEI, 1983; D'Appolonia, 1982b), WSRC concludes that there is strong evidence the DWPF soils will not experience liquefaction nor undergo significant dynamic settlement for the parameters assumed, the analyzed earthquake, and the methodology used.

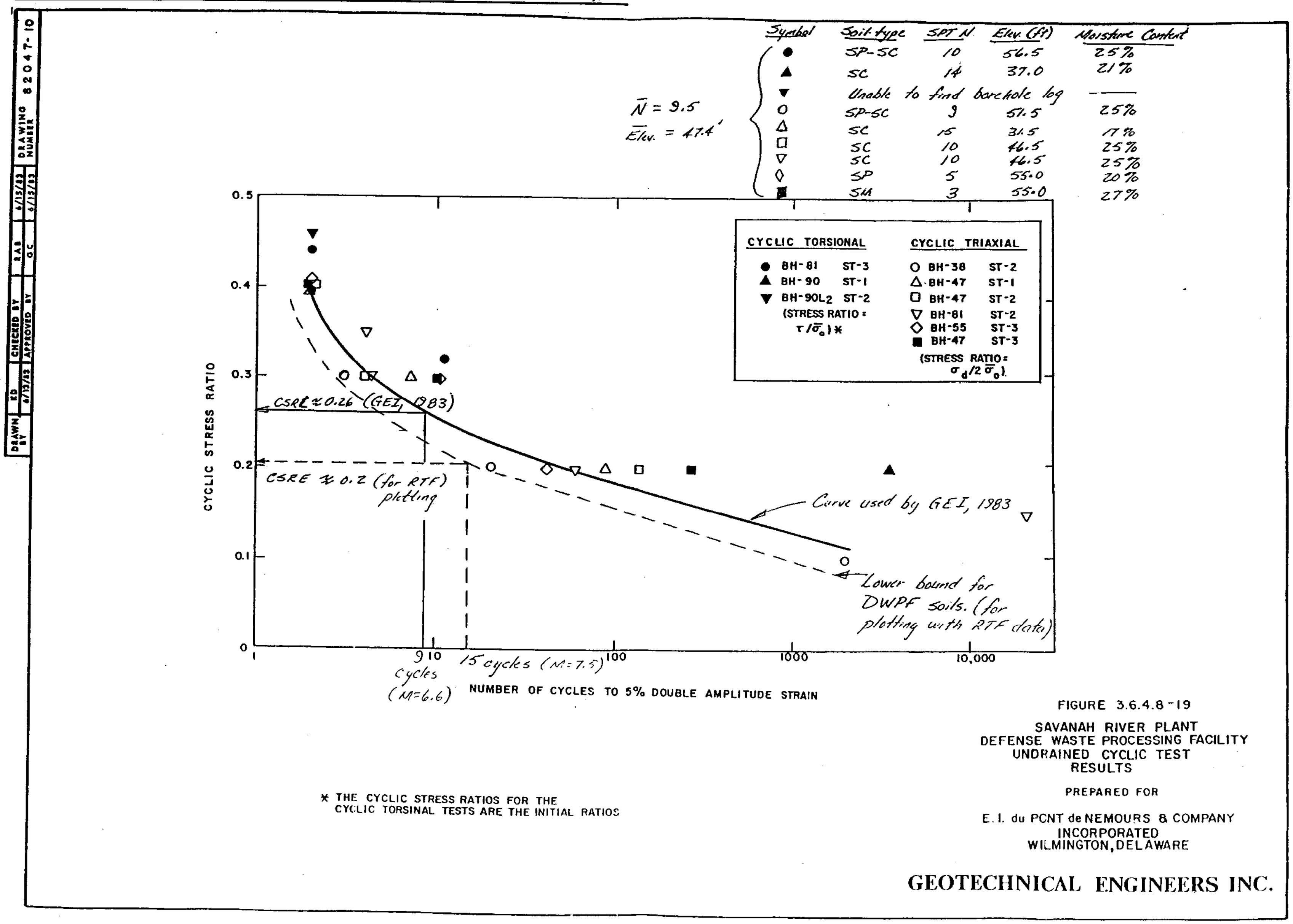


Figure C1. Undrained Cyclic Test Results.

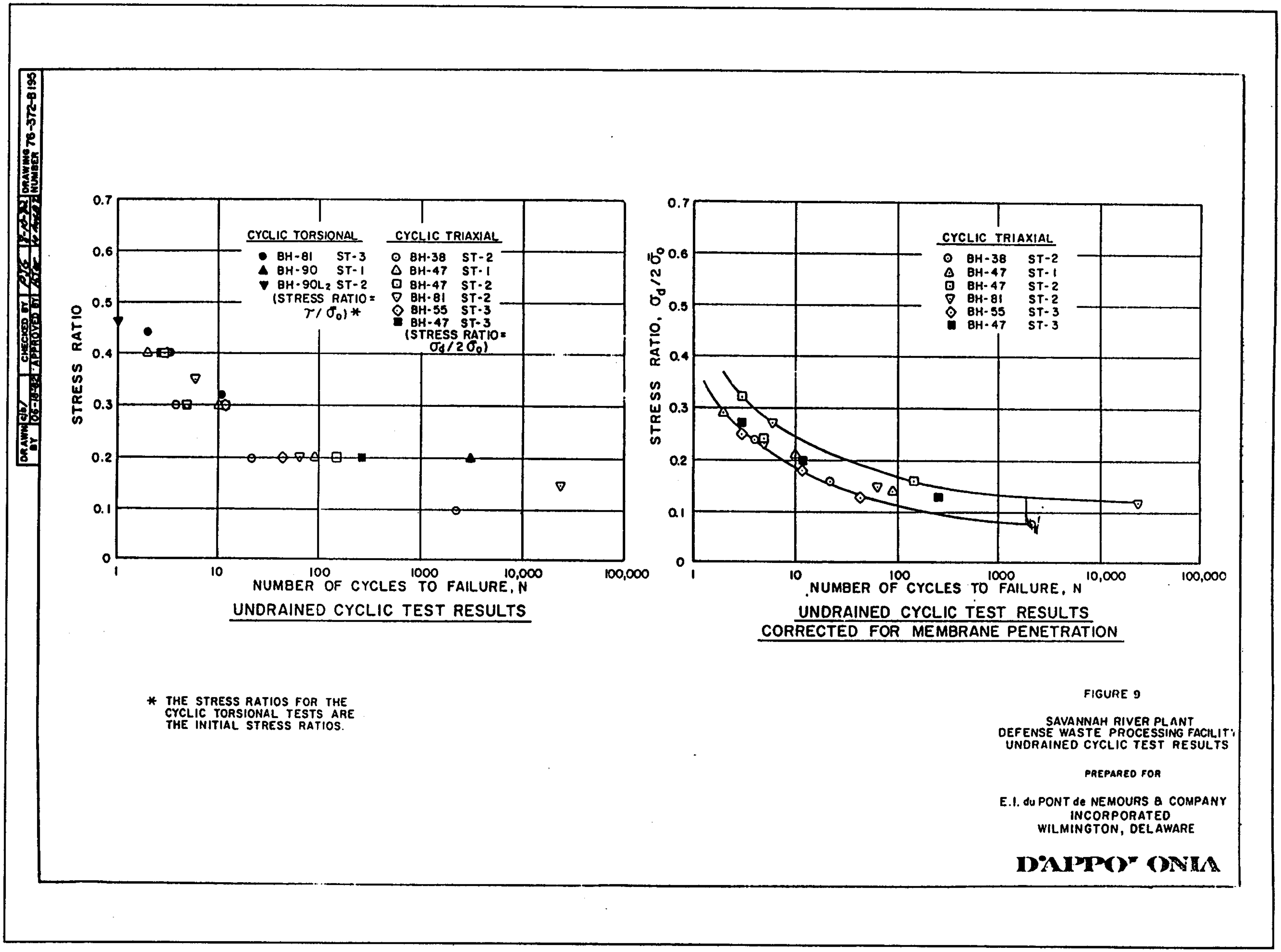


Figure C2. Undrained Cyclic Test Results Corrected for Membrane Compliance.

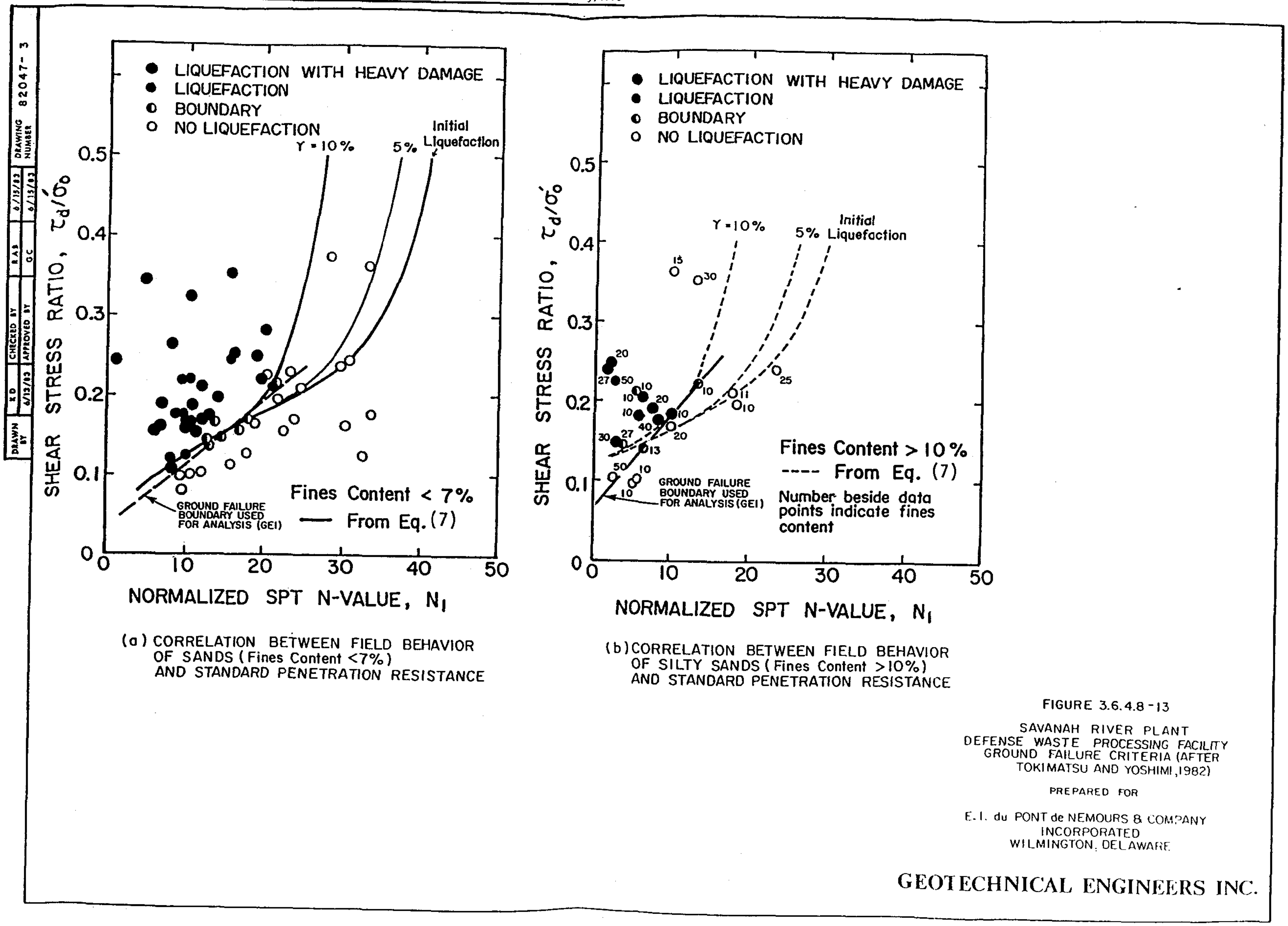
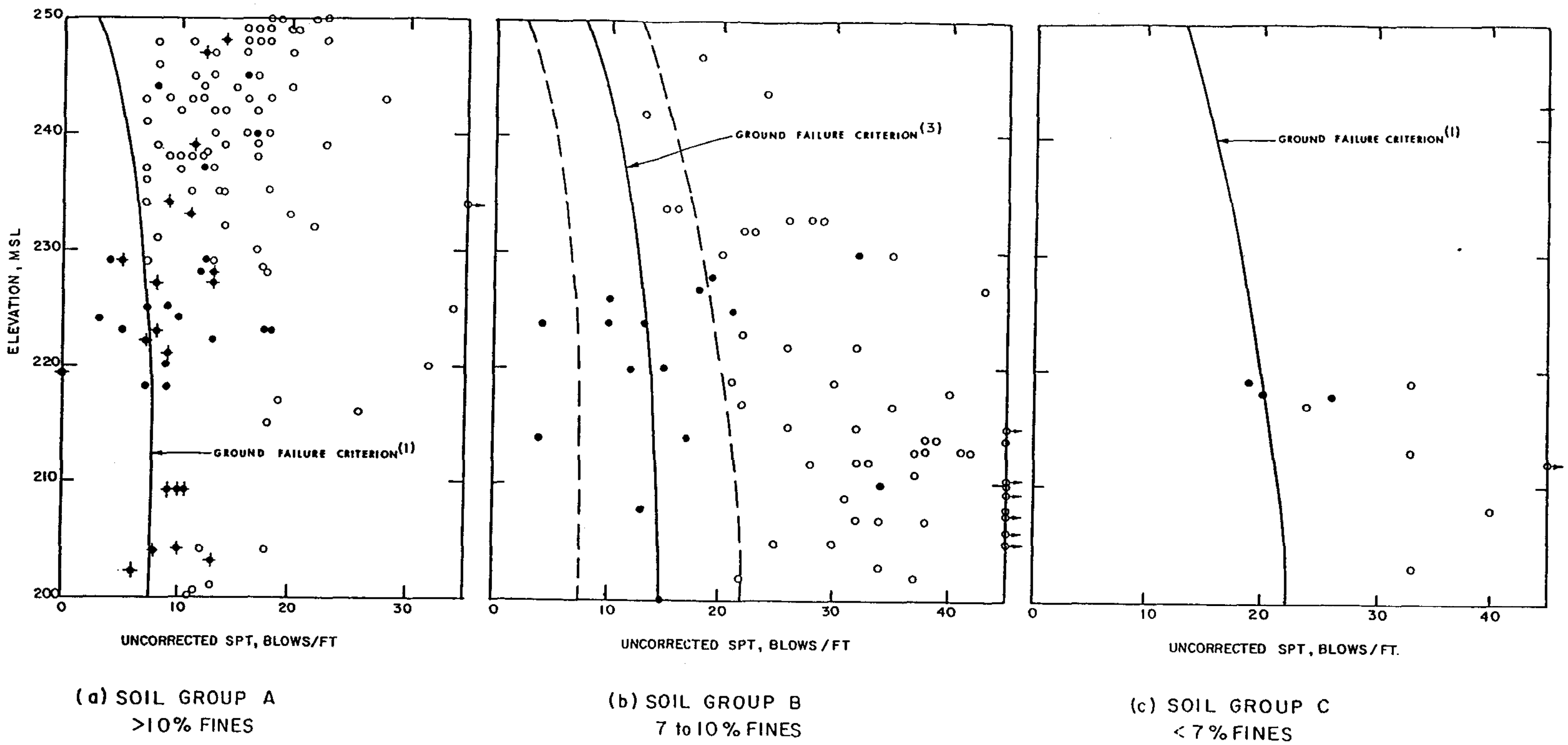


Figure C3. Empirical Ground Failure Criteria Curves.



LEGEND

- ● GRAIN SIZE ANALYSIS OR PERCENT FINES TEST PERFORMED.
- O SOIL CLASSIFICATION BASED ON VISUAL DETERMINATION OF USCS SYMBOL.
- ✦ GRAIN SIZE ANALYSIS PERFORMED IN WHICH THE PERCENT FINER THAN .005mm IS GREATER THAN 15% (Non liquèfiable)

NOTES

1. GROUND FAILURE CRITERIA AFTER TOKIMATSU AND YOSHIMI (1982)
2. GROUND SURFACE ASSUMED AT EL. 280 FOR GROUND FAILURE CRITERIA.
3. GROUND FAILURE CRITERION IS AVERAGE OF DASHED LINES WHERE DASHED LINES ARE CRITERIA FOR SOIL GROUPS A AND C.

FIGURE 3.6.4.8-15

SAVANAH RIVER PLANT
DEFENSE WASTE PROCESSING FACILITY
STANDARD PENETRATION RESISTANCE
VS. GROUND FAILURE CRITERIA

PREPARED FOR

E I du PONT de NEMOURS & COMPANY
INCORPORATED
WILMINGTON, DELAWARE

GEOTECHNICAL ENGINEERS INC.

DRAWN BY K.D 6/13/83 | CHECKED BY R.A.B 6/13/83 | APPROVED BY G.C 6/13/83 | DRAWING NUMBER 82047-5

Figure C4. Elevation Profile of Ground Failure Criterion Curves and SPT Resistance.

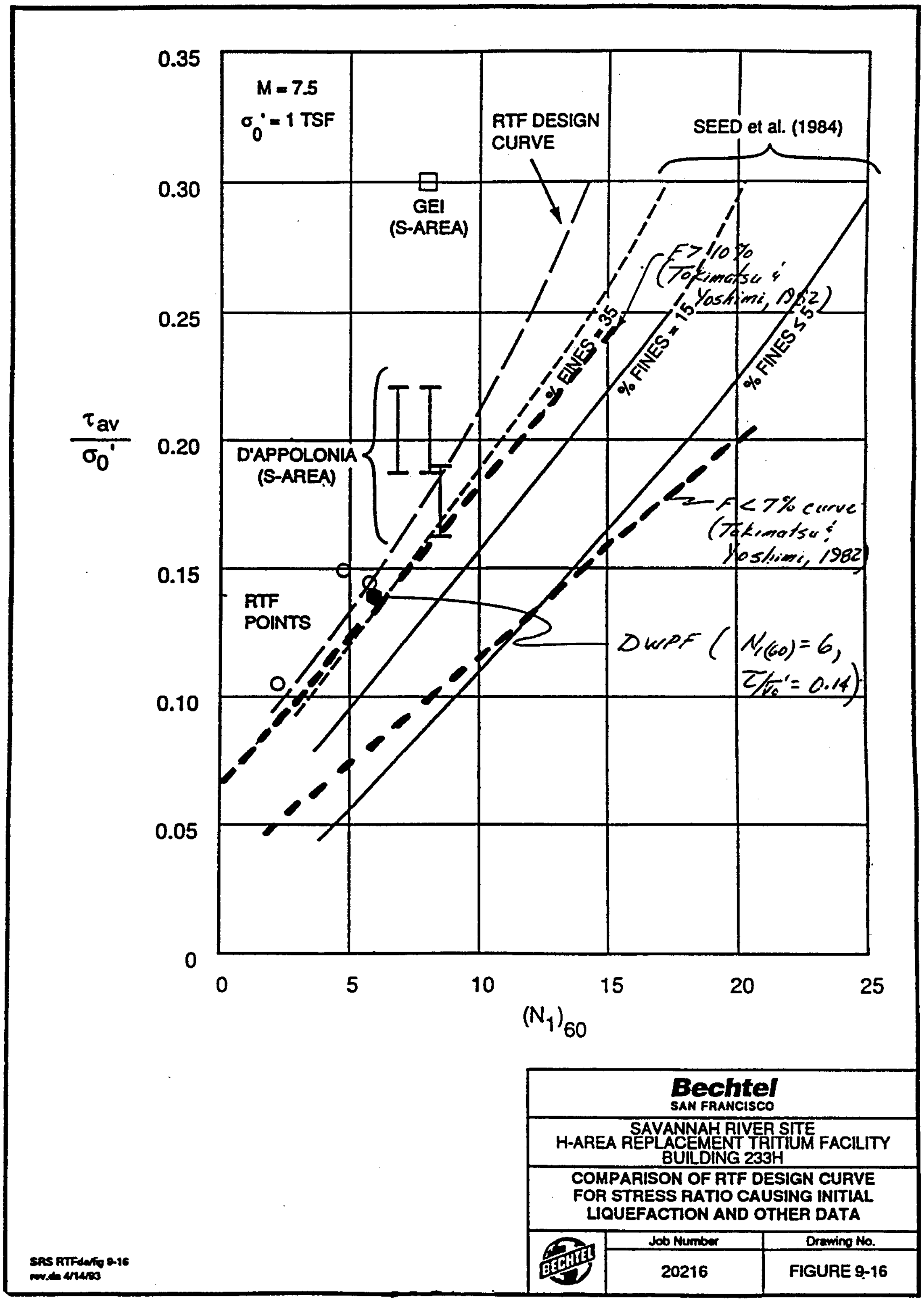


Figure C5. Comparison of RTF Design Curve for Stress Ratio Causing Initial Liquefaction.

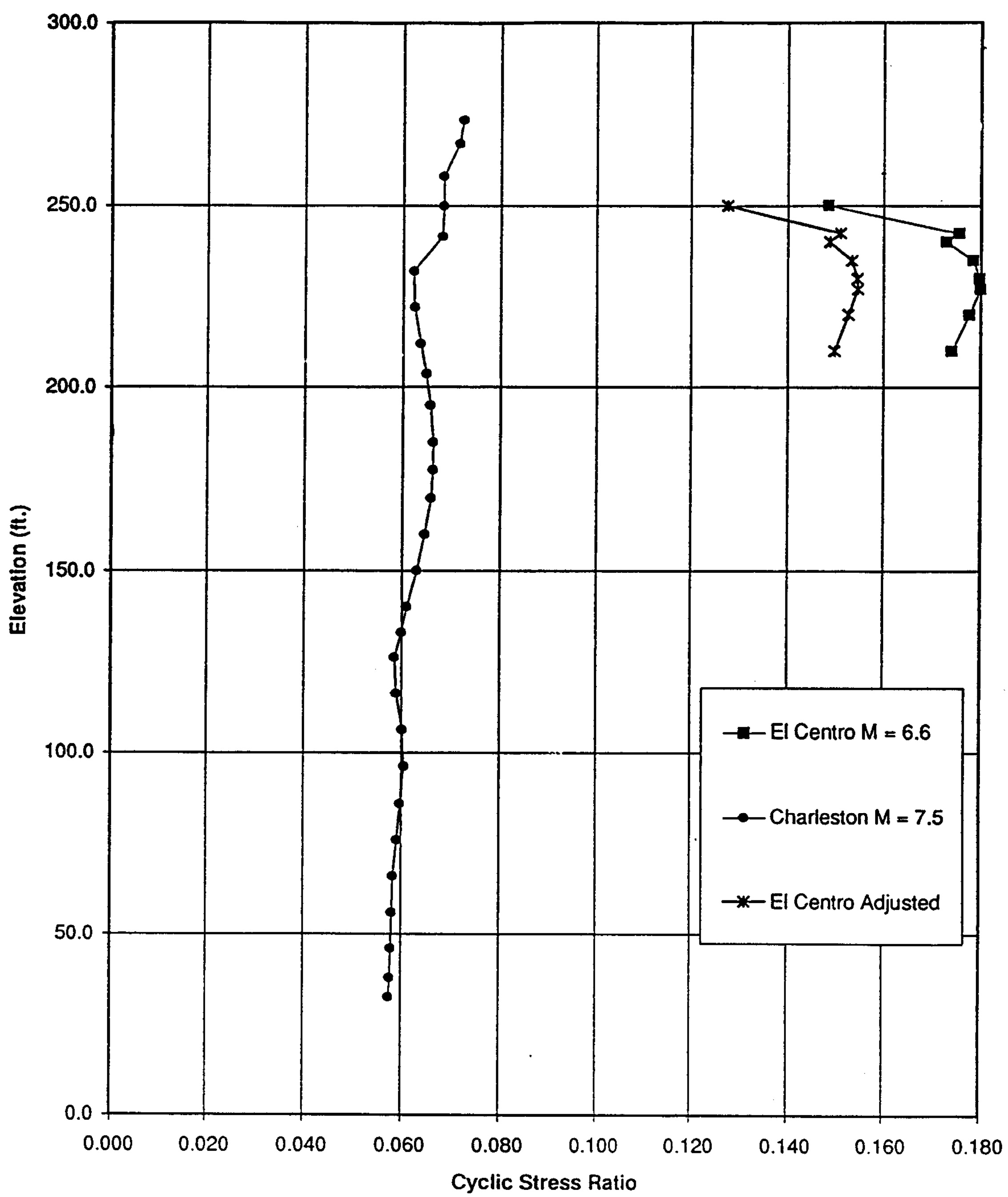


Figure C6. Elevation Profile of Cyclic Stress Ratio Using Charleston and El Centro Events at DWPF.