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Subject: Docket Number 070-03098
Duke Cogema Stone & Webster
Mixed Oxide Fuel Fabrication Facility
Geotechnical Report

Enclosed are six (6) copies of the DCS Report DCS01-WRS-DS-NTE-G-00005-C, *MOX Fuel Fabrication Facility Site Geotechnical Report*, dated August 2001. The information presented in this report documents the rationale for selection of the design bases described in the Construction Authorization Request (CAR) submitted to the U.S. Nuclear Regulatory Commission (NRC) on 28 February 2001; it also presents the results of analyses performed as part of developing the design bases into a facility design. In particular, the report describes investigation activities performed at the site, and evaluations to demonstrate the adequacy of the facility with respect to considerations such as the potential for liquefaction and the effect of soft zones.

The enclosed report constitutes supporting information to the CAR that would normally be maintained at DCS' facility and available for NRC review onsite (i.e., in Charlotte, NC). However, DCS presumes that making the report available directly will facilitate the Staff's review and preparation for subsequent discussions. DCS does not consider it to be part of the CAR.

As we have discussed with the NRC Staff, DCS would like to schedule interface meetings to discuss the topics of geology and seismology (a one-day meeting) and geotechnical engineering (a second one-day meeting) with appropriate subject matter experts from the NRC Staff. We will be working with your staff to set up a reasonable timetable for these exchanges, allowing your representatives the opportunity to assimilate the contents of the Geotechnical Report. If you have any questions, please contact me at (704) 373-7820.

Sincerely,

Peter S. Hastings, P.E.
Licensing Manager

NMSS01 Public

PO Box 31847
Charlotte, NC 28231-1847

400 South Tryon Street, WC-32G
Charlotte, NC 28202

Enclosure:

MFFF Geotechnical Report (one copy for Document Control/PDR; five copies for distribution)

xc (without enclosure):

David Alberstein, USDOE/HQ
Edward J. Brabazon, DCS
Joseph G. Giitter, USNRC/HQ
Robert H. Ihde, DCS
James V. Johnson, USDOE/HQ
Eric J. Leeds, USNRC/HQ
John E. Matheson, DCS
Andrew Persinko, USNRC/HQ
Robert C. Pierson, USNRC/HQ
Donald J. Silverman, Esq., DCS
Thomas E. Touchstone, DCS
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MOX FUEL FABRICATION FACILITY
SITE GEOTECHNICAL REPORT

QL-1A (IROFS)

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APPROVALS: (* Mark N/A if not applicable per implementing procedure)

Preparer:	James K. Meisenheimer	Date: 8/8/01	*Reviewer:	NA	Date:
Reviewer:	Rich Wentz by James K Meisenheimer per phone approval	Date: 8/8/01	*Reviewer:	NA	Date:
*Reviewer:	NA	Date:	*Reviewer:	NA	Date:
*Compliance:	NA	Date:	*Specialist:	NA	Date:

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*DV:	JM McConaghy for William L Eakely	Date: 09Aug01	LDE:	John McConaghy	Date: 10Aug01
*PDM:	NA	Date:	*FDM:	Richard O. Berry	Date: 10 Aug 01
*EM:	NA	Date:	*QA Manager:	NA	Date:

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ATTACHMENTS

ATTACHMENT 1	Exploration Program and Log of Borings
ATTACHMENT 2	Cone Penetration Testing Final Report
ATTACHMENT 3	Final Report Seismic Downhole Surveying
ATTACHMENT 4	Laboratory Testing Program and Test Results

1. EXECUTIVE SUMMARY

This report presents the results of the geotechnical assessment for the Mixed Oxide Fuel Fabrication Facility (MFFF) site. The results of exploration boring, cone penetration testing (CPT) and laboratory classification testing program for of soils at the MFFF site are presented in this report. The results of the static and dynamic laboratory testing of soils, engineering analysis, and establishment of final static and dynamic geotechnical design criteria and site preparation requirements are included in this report.

The results of the field exploration program and laboratory testing program and a detailed assessment of site conditions indicate that subsurface conditions encountered at the MFFF site are consistent with subsurface conditions reported in previous geotechnical investigations for the F-Area. Regional and SRS site specific hydrological, geological, tectonic, and seismic conditions described for the Savannah River Site (SRS) in WSRC (2000a) are considered applicable to the MFFF site. No unusual subsurface or conditions were encountered at the MFFF site. The geologic, groundwater and seismic conditions described in SRS reports for F-Area are applicable for the MFFF site.

An assessment of the subsurface conditions encountered at the MFFF site indicates that the site is considered suitable to support the proposed structures. It is anticipated that an engineered structural fill will be required beneath the MFFF and emergency diesel generator buildings to properly control settlement of the structures and to provide a uniform foundation bearing material for static and seismic load conditions. Planned foundation preparation and treatment will not have any adverse affect to the existing groundwater conditions at the site.

Some isolated soft zones were identified at depth on the MFFF site and are consistent with soft zones encountered in previous investigations in the F-Area. The exploration borings and CPT holes were used to define approximate limits of any substantial soft zones encountered. Critical structures, such as the MOX Fuel Fabrication Building (MOX) and the Emergency Diesel Generator Building (EDG), have been located on the MFFF site so that only a limited number of smaller isolated soft zones (≤4 feet thick) occur beneath critical structures. The static and dynamic analysis has included any soft zones or soft materials that have been identified beneath and adjacent to the MOX and EDB buildings. It has been determined that the identified soft zones should not have any adverse affect to these structures.

The site geological conditions encountered at the MFFF site are consistent with subsurface conditions at the adjacent F-Area, therefore, the deep soil and bedrock profile used at SRS F-Area is applicable for the MFFF site. The subsurface conditions beneath critical structures at MFFF site were analyzed for the PC-3+ and 1886 Charleston earthquakes and it was determined that liquefaction is limited to isolated pockets at depth. Post earthquake settlements and liquefaction should not have any adverse affect to critical structures.

2. INTRODUCTION

The Mixed Oxide Fuel Fabrication Facility (MFFF) site is located adjacent to the F-Area, in the Separations Area of the Department of Energy's (DOE) Savannah River Site (SRS) in South Carolina. The MFFF site geotechnical program was performed on a land area set aside for the MFFF. DOE assigned this site for the MFFF after an evaluation of five sites in the vicinity of the F-Area. This Geotechnical Report presents the results of the detailed geotechnical investigation performed at the MFFF site location.

The detailed field exploration program for the MFFF site has been completed. A total of thirteen (13) exploration borings and sixty-three (63) cone penetration test (CPT) holes were used to define subsurface conditions at the MFFF site. Laboratory testing to determine the classification and engineering properties for representative soil samples at the site has been completed. Previous site geotechnical programs, performed by others adjacent to and on this site, were also used to evaluate site subsurface geologic and groundwater conditions. Exploration boring logs, CPT logs and initial soil classification test results are presented in this report.

The results of the geotechnical program have been used to establish a database to compare with previous investigations performed at SRS and the F-Area. This analysis confirms that subsurface conditions at the MFFF site are consistent with other geotechnical investigations performed in the F-Area adjacent to the MFFF site.

The Geotechnical Exploration and Testing Program was performed under the engineering oversight of the Duke Cogema Stone & Webster (DCS) Lead Geotechnical Engineer and geotechnical staff.

2.1 PURPOSE AND SCOPE

The purpose of the Geotechnical Exploration and Testing Program was to obtain geotechnical information to characterize subsurface conditions at the MFFF site and to compare these results with subsurface conditions reported in and around the adjacent F-Area. Specific objectives include:

- Define geologic stratigraphy and compare the continuity, thickness and relative elevation to stratigraphic units defined across the F-Area and at SRS;
- Define the index properties of each stratigraphic layer and make a comparison to geotechnical properties determined for the F-Area stratigraphy;
- Evaluate the subsurface conditions to define relative geotechnical conditions and suitability to support the proposed MFFF foundations systems;
- Define any subsurface conditions that may be detrimental to support the proposed MFFF foundation systems; and
- Develop geotechnical design criteria for the MFFF site

3. GEOLOGY

3.1 REGIONAL GEOLOGY

The following discussion on the regional and MFFF site geology is based on detailed discussions presented in Section 1.4.3 of WSRC (2000a). The area of interest evaluated includes a radius of about 200 miles from the SRS and MFFF site. The information also provides the basis for understanding the regional and SRS geology as applied to subsurface conditions encountered at the MFFF site.

Many SRS investigations and an extensive literature review reach the conclusion that there are no geologic threats affecting the MFFF site, except the Charleston Seismic Zone and the minor random Piedmont earthquakes. The Pen Branch fault has been regarded as the primary structural feature at SRS that has the characteristics necessary to pose a potential seismic risk. Studies have indicated that, despite this potential, the fault is not capable.

3.1.1 Atlantic Coastal Plain Stratigraphy

The SRS is located on the sediments of the Upper Atlantic Coastal Plain in South Carolina. The Coastal Plain are stratified sand, clay, limestone, and gravel that dip gently seaward and range in age from Late Cretaceous to Recent. The sedimentary sequence thickens from essentially zero at the Fall Line to more than 1,219 meters (4,000 feet) at the coast. Regional dip is to the southeast, although beds dip and thicken locally in other directions because of locally variable depositional regimes and differential subsidence of basement features such as the Cape Fear Arch and the South Georgia Embayment. A map depicting these regional features and the study area discussed in the following sections is presented in Figure 3-1.

The Coastal Plain sedimentary sequence near the center of the region (i.e., SRS) consists of about 213 meters (700 feet) of Late Cretaceous quartz sand, pebbly sand, and kaolinitic clay, overlain by about 18 meters (60 feet) of Paleocene clayey and silty quartz sand, glauconitic sand, and silt. The Paleocene beds are in turn overlain by about 107 meters (350 feet) of Eocene quartz sand, glauconitic quartz sand, clay, and limestone grading into calcareous sand, silt, and clay. The calcareous strata are common in the upper part of the Eocene section in down dip parts of the study area. In places, especially at higher elevations, deposits of pebbly, clayey sand, conglomerate, and clay of Miocene or Oligocene age cap the sequence. Lateral and vertical facies changes are characteristic of most of the Coastal Plain sequence, and the lithologic descriptions below are therefore generalized. The stratigraphic section, which delineates the coastal plain lithology (see Figure 3-2), is divided into several formations and groups, based principally on age and lithology.

The following sections describe regional stratigraphy and lithologies of the Coastal Plain sediments, with emphasis on variations near the SRS. The data presented are based upon direct observations of surface outcrops; geologic core obtained during drilling of bore holes; microfossil age dating; and borehole geophysical logs. Several key boring locations within the SRS boundaries and in the adjacent regions (see Figure 3-3) are referenced throughout the following discussions.

3.1.1.1 Upper Cretaceous Sediments

At the F-Area and the MFFF site, Upper Cretaceous sediments overlie Paleozoic crystalline bedrock at about El -585 feet, (WSRC, 1996). The Upper Cretaceous sequence includes the basal Cape Fear Formation and the overlying Lumbee Group, which is divided into three formations (see Figure 3-2). The sediments in this region consist predominantly of poorly consolidated, clay-rich, fine- to medium-grained, micaceous sand, sandy clay, and gravel, and is about 264 meters (865 feet) thick at the F-Area and MFFF site. Thin clay layers are common. In parts of the section, clay beds and lenses up to 21 meters (70 feet) thick are present. Depositional environments were fluvial to prodeltaic.

3.1.1.2 Tertiary Sediments

Tertiary sediments range in age from Early Paleocene to Miocene and were deposited in fluvial to marine shelf environments. The Tertiary sequence of sand, silt, and clay generally grades into highly permeable platform carbonates in the southern part of the study area and these continue southward to the coast. The Tertiary sequence is divided into three groups, the Black Mingo Group, Orangeburg Group, and Barnwell Group, which are further subdivided into formations and members (see Figure 3-2). The ubiquitous Upland unit overlies these groups.

3.1.1.2.1 Black Mingo Group

The Black Mingo Group underlies SRS and the MFFF site and consists of quartz sand, silty clay, and clay that suggest upper and lower delta plain environments of deposition generally under marine influences. In the southern part of the study area, massive clay beds, often more than 50 feet (15 meters) thick, predominate.

Basal Black Mingo sediments were deposited on the regional "Cretaceous-Tertiary" unconformity of Aadland that defines the base of Sequence Stratigraphic unit I. There is no apparent structural control of this unconformity. Above the unconformity, the clay and clayey sand beds of the Black Mingo Group thin and often pinch out along the traces of the Pen Branch and Crackerneck Faults. This suggests that coarser-grained materials were deposited preferentially along the fault traces, perhaps due to shoaling of the depositional surface. This, in turn, suggests movement (reactivation) along the faults. This reactivation would have occurred during Black Mingo deposition, that is, in Paleocene and lower Eocene time.

3.1.1.2.2 Orangeburg Group

The Orangeburg Group underlies SRS and the MFFF site and consists of the lower middle Eocene Congaree Formation (Tallahatta equivalent) and the upper middle Eocene Warley Hill Formation and Santee Limestone (see Figure 3-2). Over most of the study area, these post-Paleocene units are more marine in character than the underlying Cretaceous and Paleocene units; they consist of alternating layers of sand, limestone, marl, and clay.

The group crops out at lower elevations in many places within and near SRS. The sediments thicken from about 26 meters (85 feet) at well P-30 near the northwestern SRS boundary to 61 meters (200 feet) at well C-10 in the south (see Figure 3-3). Dip of the upper surface is 2 m/km (12 ft/mile) to the southeast.

In the central part of the study area the group includes, in ascending order, the Congaree, Warley Hill, and Tinker/Santee Formations (see Figure 3-2). The units consist of alternating layers of sand, limestone, marl, and clay that are indicative of deposition in shoreline to shallow shelf environments. From the base upward, the Orangeburg Group passes from clean shoreline sand characteristic of the Congaree Formation to shelf marl, clay, sand, and limestone typical of the Warley Hill and Santee Limestone. Near the center of the study area, the Santee sediments consist of up to 30 % carbonate. The sequence is transgressive, with the middle Eocene Sea reaching its most northerly position during Tinker/Santee deposition.

Toward the south, near wells P-21, ALL-324, and C-10 (see Figure 3-3), the carbonate content of all three formations increases dramatically. The shoreline sand of the Congaree undergoes a facies change to interbedded glauconitic sand and shale, grading to glauconitic argillaceous, fossiliferous, sandy limestone. Downdip, the fine-grained, glauconitic sand, and clay of the Warley Hill become increasingly calcareous and grades imperceptibly into carbonate-rich facies comparable to both the overlying and underlying units. Carbonate content in the glauconitic marl, calcareous sand, and sandy limestone of the Santee increases towards the south. Carbonate sediments constitute the vast majority of the Santee from well P-21 southward.

3.1.1.2.2.1 Congaree Formation

The early middle Eocene Congaree Formation has been traced from the Congaree valley in east central South Carolina into the study area. It has been paleontologically correlated with the early and middle Eocene Tallahatta Formation in neighboring southeastern Georgia.

The Congaree is about 9 meters (30 feet) thick near the center of the SRS study area and consists of yellow, orange, tan, gray, green, and greenish gray, well-sorted, fine to coarse quartz sand, with granule and small pebble zones common. Thin clay laminae occur throughout the section. The quartz grains tend to be better rounded than those in the rest of the stratigraphic column. The sand is glauconitic in places suggesting deposition in shoreline or shallow shelf environments. To the south, near well ALL-324 (see Figure 3-3), the Congaree Formation consists of interbedded glauconitic sand and shale, grading to glauconitic, argillaceous, fossiliferous sandy limestone suggestive of shallow to deeper shelf environments of deposition. Farther south, beyond well C-10, the Congaree grades into platform carbonate facies of the lower Santee Limestone.

The Congaree Formation is present across the MFFF site and was encountered in all of the exploration borings. It is a very dense sand and was a consistent geologic marker bed.

3.1.1.2.2.2 Warley Hill Formation

Unconformably overlying the Congaree Formation are 3 meters (10 feet) to 6 meters (20 feet) of fine-grained, often glauconitic sand and green clay beds that have been referred to respectively as the Warley Hill and Caw Members of the Santee Limestone. The green sand and clay beds are referred to informally as the "green clay" in previous SRS reports. Both the glauconitic sand and the clay at the top of the Congaree are assigned to the Warley Hill Formation. In the updip parts of the study area, the Warley Hill apparently is missing or very thin, and the overlying Tinker/Santee Formation rests unconformably on the Congaree Formation.

The Warley Hill Formation (GC) is present at the MFFF site and averages 1.5 meters (5 feet) in thickness.

The Warley Hill sediments indicate shallow to deeper clastic shelf environments of deposition in the study area, representing deeper water than the underlying Congaree Formation. This suggests a continuation of a transgressive pulse during upper middle Eocene time. To the south, beyond well P-21, the green silty sand, and clay of the Warley Hill undergo a facies change to the clayey micritic limestone and limey clay typical of the overlying Santee Limestone. The Warley Hill blends imperceptibly into a thick clayey micritic limestone that divides the Floridan Aquifer System south of the study area. In the SRS study area, the thickness of the Warley Hill Formation is generally less than 6 meters (20 feet).

3.1.1.2.2.3 Tinker/Santee Formation

The Tinker/Santee (Utley) interval is about 21 meters (70 feet) thick near the center of SRS. Sediments of the Tinker/Santee indicate deposition in shallow marine environments. Often found within the Tinker/Santee sediments, particularly in the upper third of the interval, are weak zones interspersed in stronger carbonate-rich matrix materials. The weak zones, which vary in apparent thickness and lateral extent, were noted where rod drops and/or lost circulation occurred during drilling, low blow counts occurred during SPT pushes, etc. These weak zones have variously been termed in SRS reference documents as “soft zones”, the “critical layer”, “underconsolidated zones”, “bad ground”, and “void”.

The Tinker/Santee Formation is present at the MFFF Site and is about 12 meters (40 feet) thick. Soft zones, typical to this formation, were also encountered at the MFFF site.

3.1.1.2.3 Barnwell Group

Upper Eocene sediments of the Barnwell Group (see Figure 3-2) represent the Upper Coastal Plain of western South Carolina and eastern Georgia. Sediments of the Barnwell Group are present at the MFFF site and overlie the Tinker/Santee Formation and consist mostly of shallow marine quartz sand containing sporadic clay layers. The group is about 21 meters (70 feet) thick near the northwestern boundary of SRS and 52 meters (170 feet) near its southeastern boundary. The regionally significant Santee Unconformity that defines of boundary between Sequence Stratigraphic units II and III (Figure 3-2) separates the Clinchfield Formation from the overlying

Dry Branch Formation. The Santee Unconformity is a pronounced erosional surface observable throughout the SRS region (Figure 3-2).

3.1.1.2.3.1 Clinchfield Formation

The basal late Eocene Clinchfield Formation consists of light colored quartz sand and glauconitic, biomoldic limestone, calcareous sand, and clay. Sand beds of the formation constitute the Riggins Mill Member of the Clinchfield Formation and are composed of medium to coarse, poorly to well sorted, loose and slightly indurated, tan, clay, and green quartz. The sand is difficult to identify unless it occurs between the overlying carbonate layers of the Griffins Landing Member and the underlying carbonate layers of the Santee Limestone. The Clinchfield is about 8 meters (25 feet) thick in the southeastern part of SRS and pinches out or becomes unrecognizable at the center of the site, at the MFFF site location.

The Clinchfield Formation is not present at the MFFF site.

3.1.1.2.3.2 Dry Branch Formation

The late Eocene Dry Branch Formation is divided into the Irwinton Sand Member, the Twiggs Clay Member, and the Griffins Landing Member. The unit is about 18 meters (60 feet) thick near the center of the study area. The Dry Branch sediments overlying the Tinker/Santee (Utley) interval in the central portion of SRS were deposited in shoreline/lagoonal/tidal marsh environments. The shoreline retreated from its position in northern SRS during Tinker/Santee (Utley) time to the central part of SRS in Dry Branch time. Progradation of the shoreline environments to the south resulted in the sands and muddy sands of the Dry Branch being deposited over the shelf carbonates and clastics of the Tinker/Santee (Utley) sequence.

The Dry Branch Formation is present at the MFFF site and is about 9 meters (30 feet) thick.

3.1.1.2.3.3 Tobacco Road Formation

The Late Eocene Tobacco Road Formation consists of moderately to poorly sorted, red, brown, tan, purple, and orange, fine to coarse, clayey quartz sand. Pebble layers are common, as are clay laminae and beds. Ophiomorpha burrows are abundant in parts of the formation. Sediments have the characteristics of lower Delta plain to shallow marine deposits. The top of the Tobacco Road is characterized by the change from a comparatively well sorted sand to the more poorly sorted sand, pebbly sand, and clay of the "Upland unit." Contact between the units constitutes the "Upland" unconformity. The unconformity is very irregular due to fluvial incision that accompanied deposition of the overlying "Upland unit" and later erosion.

The Tobacco Road Formation is found at the MFFF site and is about 13 meters (43 feet) thick. The Tobacco Road Formation is overlain by the "Upland Unit" at the MFFF site.

3.1.1.2.3.4 "Upland Unit"/Hawthorn/Chandler Bridge Formations

Deposits of poorly sorted silty, clayey sand, pebbly sand, and conglomerate of the "Upland unit" cap many of the hills at higher elevations over much of the study area. Weathered feldspar is abundant in places. The color is variable, and facies changes are abrupt. The "Upland unit", is generally considered to be Miocene age. The unit is up to 18 meters (60 feet) thick. The environment of deposition appears to be fluvial, and the thickness changes abruptly owing to channeling of the underlying Tobacco Road Formation during "Upland" deposition and subsequent erosion of the "Upland" unit itself. This erosion formed the "Upland" unconformity. The unit is up to 18 meters (60 feet) thick at SRS.

The "Upland Unit" is the upper soil unit at the MFFF site. The thickness of the unit is about 9 meters (30 feet).

3.2 SOFT ZONES

Much of SRS, including the F-Area and MFFF site, contains intermittent and isolated weak soil zones interspersed in stronger carbonate-rich matrix materials within the Tinker/Santee (Utley) section, as discussed in Section 3.1.1.2.2.3. These weak zones, which vary in apparent thickness and lateral extent, were recorded where rod drops and/or lost circulation occurred during drilling and low blow counts occurred during SPT sampling. Also they have been identified as low penetration resistance during CPT pushes. These week zones within the Tinker/Santee sediments have variously been termed at SRS as "soft zones", "the critical layer", "underconsolidated zones", "bad ground", and "void". The preferred term used to describe these zones is "soft zones". Other soft material zones can be found in other formations at the F Area and MFFF site, however they do not have the same origin or physical behavior as the Tinker/Santee "soft zones". Since the "soft zones" are considered significant for siting critical structures such as the MOX and EDG buildings, they will be discussed as a separate subject in this report and all discussions related to soft zones will be in reference to the "soft zones" of the Tinker/Santee sediments.

Extensive studies and analysis of the soft zones at SRS have been conducted since 1952. Eighteen of studies, including outside peer consultant review comments, have been summarized are presented in comprehensive detail in WSRC 1999b. WSRC 1999b also presents current conclusions on the origin, development and stability of the soft zones, along with an assessment of the engineering properties of the soft zones. WSRC 2000a also presents a detailed assessment of the soft zones found at SRS. These studies have been used for MFFF site to develop the approach for field exploration programs, laboratory testing and analysis of soft zones encountered at the MFFF site. The results of the previous investigations for the F Area and the adjacent APSF site led to the decision to relocate the MOX and EDG buildings after initial MFFF site investigations identified the presence of major soft zone areas in the eastern portion of the site.

Isopach maps reveal that carbonate thickness and concentration is directly related to the isopach thickness of the Tinker/Santee (Utley) interval. Where the Santee-Utley interval is thick, carbonate is more concentrated, where the interval is thin, carbonate thickness and concentration

is reduced. It is further observed that where carbonate is concentrated in the Santee-Utley section the overlying "upland unit", Tobacco Road/Dry Branch section is generally structurally high, and where the carbonate content is reduced or absent the overlying "upland unit," Tobacco Road/Dry Branch section is generally structurally low. This indicates that the removal (dissolution) of carbonate and the thinning of the Santee-Utley interval occurred in post Tobacco Road time. (WSRC 2000a)

The origin of the carbonates in the Tinker/Santee (Utley) interval is fairly clear. The carbonate content ranges from zero to approximately 90 percent. The presence of glauconite along with a normal marine fauna including foraminifers, molluscs, bryozoans, and echinoderms, indicates that the limestones and limey sandstones were deposited in clear, open-marine water of normal salinity on the inner to middle shelf. The abundance of carbonate mud (micrite) in the limestones suggests deposition in quiet water below normal marine wave base. The presence of abraded and well-worn skeletal grains indicates that bottom transport by currents or storm-generated waves alternated with quiet-water conditions in which the sediments accumulated. (WSRC 2000a)

The original thoughts were that the soft zones were the result of the dissolution of the shell debris concentrated in bioherms (oyster banks). This premise has since been proven to be false. Significant study of the deposition of the Tinker/Santee sediments precludes the formation of bioherms. Several hypotheses exist concerning the origin of the soft zones: one being that these zones consisted of varying amounts of carbonate material that has undergone dissolution over geologic time leaving sediments that are now subjected to low vertical effective stresses due to arching of more competent soils above the soft zone intervals. (WSRC 2000a)

A second hypothesis is based on recent studies that indicate that soft zones occur where silica replacement/cementation of the carbonate occurred. The silicification (by amorphous opaline silica) of the enclosing carbonate sediment would follow and spread along bedding planes, along microfractures of varied orientations and along corridors of locally enhanced permeability. The resulting "soft zone" could be in the form of irregular isolated pods, extended thin ribbons or stacked thin ribbons separated by intervening unsilicified parent sediment. Careful observations of the grouting programs conducted by the COE in the early 1950s, and more recently for the restart of K Reactor, corroborate these recent findings. They observed that the grout was not having the desired effect on the subsurface soft zones as was previously thought. The results showed that the grout traveled in thin sheets along preferential pathways. Soft zones that existed prior to grouting still existed after grouting was completed. Soft zones encountered in one cone penetrometer test (CPT) sounding could be absent in the neighboring CPT only a few feet away. Only where silicification has spread far enough away from the bedding planes and/or fractures along which the silica replacement has taken place, where all the intervening sediment is replaced, would the soft zones be large enough and coherent enough to pose a question for the siting of new facilities. In all likelihood this would be a most uncommon event. (WSRC 2000a)

Two primary episodes of freshwater flushing of the Tinker/Santee section in two or more stages of carbonate dissolution are hypothesized based on the multiple episodes of erosion/dissolution of the section. The first occurred at the time of the Santee unconformity, the second at the time

of deposition of the "Upland Unit" following the Upland unconformity. During the interim period between the two primary episodes of fresh water flushing of the Tinker/Santee section, dissolution of carbonate and precipitation and replacement of carbonate by silica contiued albeit at a slower rate during the deposition of the Dry Branch and Tobacco Roads section. (WSRC 1999b)

At the F Area and MFFF site, the Tinker/Santee section is in the saturated zone, well below the water table. Here the sediments are in a stable chemical environment and the carbonate dissolution is minimal. The further dissolution and removal of the Tinker/Santee carbonate in the next 100 years is a non-issue. (WSRC1999b)

3.3 FAULTING

The location of mapped faults at SRS in the vicinity of the F-Area, where the MFFF site is located, is shown on Figure 3-4. Faulting at SRS is discussed in detail in Section 1.4.3.2 of WSRC (2000a). The MFFF site is located just north of the F area shown on Figure 3-4. Many SRS investigations and an extensive literature review reach the conclusion that there are no geologic threats affecting the SRS, which also applies to the MFFF site, except the Charleston Seismic Zone and the less significant Piedmont and Coastal earthquakes. The Pen Branch fault is a significant bedrock feature at SRS and has been the subject of a number of investigations to establish its potential seismic risk. These investigations at SRS have not indicated any Quaternary movement. It has been determined that the Pen Branch fault is not capable per 10 CFR, Appendix A (WSRC 2000a).

3.4 SEISMOLOGY

Significant studies of the local and regional seismology for SRS have been conducted to support operation of DOE facilities there. The MFFF project has used these studies as a starting point in establishing appropriate design inputs for the MFFF site. Section 1.4.4 of WSRC 2000a presents a detailed discussion for the seismology at SRS, criteria that have been developed for the DOE facilities at SRS, and their application to develop design criteria for the MFFF. Pertinent sections of WSRC 2000a are included in this report.

3.4.1 Earthquake History of the General Site Region

This section includes a broad description of the historic seismic record (non-instrumental and instrumental) of the southeastern U.S. and SRS. Aspects that are of particular importance to SRS include the following:

- The Charleston, SC, area is the most significant seismogenic zone affecting the SRS.
- Seismicity associated with the SRS and surrounding region is more closely related to South Carolina Piedmont-type activity. This activity is characterized by occasional small shallow events associated with strain release near small scale faults, intrusive bodies, and the edges of metamorphic belts.

3.4.1.1 Regional History

The earthquake history of the southeastern U.S. (of which the SRS is a part) spans a period of nearly three centuries, and is dominated by the catastrophic Charleston earthquake of August 31, 1886. The historical database for the region is essentially composed of two data sets extending back to as early as 1698. The first set is comprised of pre-network, mostly qualitative data (1698-1974), and the second set covers the relatively recent period of instrumentally recorded or post-network seismicity (1974-present). Today seismic monitoring results from all southeastern seismic networks are cataloged annually in the Southeast U.S. Seismic Network bulletins. Significant earthquakes within 200 miles of SRS (Intensity >4 for Magnitude >3) are presented in WSRC 2000a.

The Charleston, SC, area is the most significant source of seismicity affecting SRS, in terms of both the maximum historical site intensity and the number of earthquakes felt at SRS. The greatest intensity felt at the SRS has been estimated at MMI VI-VII and was produced by the intensity X earthquake that struck Charleston, SC, on August 31, 1886. An earthquake that struck Union County, South Carolina (about 100 miles [160 km] north-northeast of SRS), on January 1, 1913, is the largest event located closest to SRS outside of the Charleston area. It had an intensity greater than or equal to MMI VII. This earthquake was felt in the Aiken-SRS area with an intensity of MMI II-III. Several other earthquakes, including some aftershocks of the 1886 Charleston event, were felt in the Aiken-SRS area with intensities estimated to be equal to or less than MMI IV. The location of historical seismic events are presented in WSRC 2000a.

Several large earthquakes outside the region were probably felt at SRS, including the earthquake sequence of 1811 and 1812 that struck New Madrid, Missouri (about 535 miles west-northwest of SRS) and the earthquake that struck Giles County, Virginia (about 280 miles north of SRS), on May 31, 1897.

3.4.1.2 SRS Activity Within 50 Mile Radius

The SRS is located within the Coastal Plain physiographic province of South Carolina. However, seismic activity associated with SRS and the surrounding region displays characteristics more closely associated with the Piedmont province, that is, a marked lack of clustering in zones. The activity is more characteristic of the occasional energy strain release occurring through a broad area of central Piedmont of the state. Epicenter locations for events near (within 50 miles from center of site) SRS are presented in WSRC 2000a.

3.4.2 Paleoseismic Data

Estimating seismic recurrence intervals of moderate to large earthquakes within the southeastern U.S. is difficult. These difficulties stem from the relatively short (300 years) historical record coupled with an absence of surface faulting, offset features, or prehistoric ruptures.

Geologic field study methods developed to extend the seismic record assess both the temporal and spatial distribution of past moderate and large earthquakes. This assessment is carried out through identification and dating of secondary deformation features resulting from strong ground shaking. In the southeast, this extension of the seismic record has been accomplished through

field search for earthquake-induced liquefaction flowage features called "sand blows" associated with prehistoric earthquake-induced paleoliquefaction features.

These features are attributed to prehistoric earthquake induced liquefaction as defined by the transformation of sediments from solid to liquid state caused by increased pore water pressure (WSRC 2000a). The increased pore pressure is caused during or immediately after an earthquake. "Sand blows" are features formed where earthquake shaking causes liquefaction at depth followed by the venting of the liquefied sand and water to the surface.

The following summarizes paleoliquefaction studies in the southeastern United States. Aspects that are of particular importance to SRS include the following (WSRC 2000a):

- No conclusive evidence of large prehistoric earthquakes originating outside of coastal South Carolina has been found.
- Young fluvial terraces at or slightly above the level of the modern floodplain and Carolina bays are the most likely depositional environments for potentially liquefiable deposits in the SRS region.

3.4.2.1 Paleoliquefaction Studies in the Eastern United States

Widespread occurrence of earthquake-induced sand blows have been documented throughout the meizoseismal area of the 1886 Charleston, SC, earthquake (WSRC 2000a). Excavation and detailed analyses of these liquefaction flow features provided the first insight into the pre-history of the Charleston earthquake. Other pre-1886 liquefaction flow features (mostly sand blows) were discovered and investigated near the town of Hollywood, about 25 km (15 miles) west of Charleston. Searches for sand blows were continued throughout the Charleston area and expanded to the remaining coastal South Carolina areas. Eventually, areas of study were broadened to include Delaware, Virginia, North Carolina, and Georgia. The objective was to identify other epicentral regions, if they existed, and to estimate the sizes of pre-1886 earthquakes assuming the areal extent of sand blows caused by an earthquake are a function of earthquake intensity in areas of similar geologic and groundwater settings. Figure 3-5 shows the study region of current paleoliquefaction areas of interest (WSRC 2000a). To date, no conclusive evidence of large prehistoric earthquakes originating outside of coastal South Carolina have been found (WSRC 2000a).

In coastal South Carolina investigations, identification of paleoliquefaction features generally adheres to specific local geologic criteria. Some specific relations between liquefaction susceptibility and subsequent formation of liquefaction features (sand blows) are summarized below (WSRC 2000a):

- A water-table very near the ground surface greatly increases susceptibility to liquefaction (depth <1 m [<3 feet]).

- Virtually all seismically induced liquefaction sites are located in either beach-ridge, backbarrier, or fluvial depositional environments. Of these, beach-ridge deposits were found to be the most favorable for the generation and preservation of seismically induced liquefaction features.
- Due primarily to the effects of chemical weathering, materials older than about 250 ka were less susceptible to liquefaction than were younger deposits. This indicates that the probabilities of sand blows forming in deposits of late Pleistocene and early Holocene age are extremely low.
- The liquefied materials are generally fine-grained, well-sorted (i.e., uniformly graded), clean beach sand. The principal properties of sand that control liquefaction susceptibility during shaking are degree of compaction (measured as relative density by geotechnical engineers), sand-grain size and sorting, and cementation of the sand at grain-to-grain contacts. Fine grained well-sorted sand of ancient and modern beaches are much more susceptible to liquefaction than standard sand used for engineering analyses.
- Features large enough to be interpreted as possibly having an earthquake origin in the low country were found only in sand deposits having total thickness greater than 2 to 3 meters (7 to 10 feet).
- The depth of the probable source beds at liquefaction sites is generally less than 6 to 7 meters (20 to 23 feet), and the groundwater table is characteristically less than 3 meters (10 feet) beneath present ground surface.

Utilizing the above methods, at least four pre-1886 liquefaction episodes occurred at approximately 580 +104 (CH-2), 1311+ 114 (CH-3), 3250+ 180 (CH-4), and 5124+ 700 (CH-5) years before the present (WSRC 2000a). CH refers to Charleston source with CH-1 designated as the 1886 earthquake. An even older episode (CH-6) was found to be cut by a CH-5 feature.

Changes in hydrologic conditions (groundwater levels) play an important role in determining an area's susceptibility to liquefaction. On the basis of published sea-level curves, groundwater levels in the southeastern U.S. have been assumed at or near present levels for only the past 2,000 years. Consequently, the paleoliquefaction record is probably most complete for this period (WSRC 2000a). However, beyond the 2,000-5,000 year range, knowledge of groundwater conditions is considerably less reliable, making gaps in the paleoseismic record much more probable.

3.4.2.2 Paleoliquefaction at SRS

A paleoliquefaction assessment of SRS was prepared by WSRC in 1996. This investigation indicated that several hydrologic, sedimentological, and logistical conditions must be met for seismically induced liquefaction (SIL) to occur and be identified. These included: (1) the presence of Quaternary-age deposits; (2) the presence of a shallow groundwater table; (3) proximity to potential seismogenic features; (4) geologic sections of several different types of unconsolidated deposits; and (5) quality and extent of exposure.

Based on these considerations, the floodplains of the Savannah River and its tributaries were identified as the areas on the SRS with the highest potential for generating and recording Holocene SIL features. The terraces of the Savannah River and tributaries were also considered potential areas for recording Quaternary SIL features, though these features would likely be older than ones in the floodplains. The upland areas on the SRS have a low potential for recording Quaternary SIL because they are pre-Quaternary in age, partially indurated, and generally high above the water table. Paleoliquefaction investigations in the SRS uplands, therefore, only targeted those sites postulated by previous workers as containing evidence of SIL.

Conclusions from this paleoliquefaction assessment fell into two categories: (1) field studies of floodplain deposits along the Savannah River, and (2) evaluation of previously reported paleoliquefaction and neotectonic features located in pre-Quaternary sediments. A brief summary of findings in these two areas follows.

Investigation of banks along 110 km (68 miles) of the Savannah River adjacent to the SRS revealed a large number of excellent exposures of floodplain deposits. Most of the exposed deposits were clay and silt, and had a low liquefaction potential. Locally however, clean sand deposits with a high liquefaction potential were present. Given the extensive amount of exposure and the local presence of liquefiable materials, SIL features would likely be present in these deposits if strong earthquakes had occurred after they were deposited. However, the presence of buried historical objects and radiocarbon dates from these materials illustrated that most or all of the exposed floodplain deposits were historical in age. As no strong ground motions have occurred in historical times in the SRS area, SIL features could not exist in these deposits. Furthermore, the fact that they date to historical times precludes them from providing any information of earlier earthquake history.

The absence of SIL features in the bank exposures does not preclude the possibility that SIL features exist deeper in the section or on the older, higher terraces. In fact, the local presence of liquefiable materials in the Modern floodplain deposits suggests that, if strong prehistoric earthquakes had occurred, SIL features are probably present at depth in the floodplain deposits or on the older/higher terraces. These key areas were not investigated, and exposure is limited.

The upland areas of the SRS were considered to have a low potential for recording Quaternary SIL because the deposits are old (pre-Quaternary), generally high above the water table (>10 meters [>30 feet]), and are indurated. However, previous investigators described several features in the Tertiary section as clastic dikes, and attributed them to SIL and/or neotectonic activity. The sites were evaluated to determine if they have the diagnostic characteristics that have recently been documented for true SIL.

Four types of post-depositional features were identified during the investigations at SRS: (1) irregularly shaped cutans; (2) structurally controlled cutans; (3) joints; and (4) faults. Cutans are a modification of the texture, structure, or fabric of the host material by pedogenic (soil) processes, either by a concentration of particular soil constituents or in-situ modification of the matrix. These features were interpreted through the process of elimination procedure of multiple working hypotheses. None were thought to be the result of SIL. (WSRC 2000a)

3.4.3 Development of SRS Design Basis Earthquake

The basic approach that has been used to develop the Design Basis Earthquake (DBE) spectra at SRS is described in detail in Section 1.4.4.3 of WSRC 2000a. The following sections present a summary of the process that was used to develop the present DBE spectra in use at SRS.

3.4.3.1 Design Basis Earthquake for SRS

Because maximum potential causative fault structures within the Coastal Plain, Piedmont, and Blue Ridge provinces are not clearly delineated by lower-level seismicity or geomorphic features, past regulatory guidance prescribes the use of an assumed local earthquake. The magnitude/intensity is conservatively assumed to be a repeat of the largest historic event in a given tectonic province located at that province's closest approach to the site. Application of this guidance has resulted in the definition of two controlling earthquakes for the seismic hazard at SRS. One earthquake is a local event comparable in magnitude and intensity to the Union County earthquake of 1913 but occurring within a distance of about 25 km (15 miles) from the site. The other controlling earthquake represents a potential repeat of the 1886 Charleston earthquake. Selection of these controlling earthquakes for design basis spectra has not changed significantly in over 20 years. However, the assumed maximum earthquake moment and magnitude estimates have increased in the more recent assessments of the 1886 Charleston earthquake. In addition, the assumed distance to a repeat of the 1886 Charleston-type earthquake has slightly decreased.

Until the late 1980s, investigations performed for the NRC focused on the uniqueness of the location of the Charleston earthquake, due to a lack of knowledge of a positive causative structure at Charleston. At issue was the possibility of a rupture on any one of the numerous northeast-trending basement faults located throughout the eastern seaboard. Further, there were no obvious geomorphic expressions that might suggest large repeated faulting.

Evidence that defines the Charleston Seismic Zone (CSZ) is as follows (WSRS 2000a):

- The detailed analyses of isoseismals following the 1886 Charleston earthquake.
- Instrumental locations and focal mechanisms of seismicity defining the 50-km long Woodstock fault lineament, which closely parallels the north-northeast trending Dutton isoseismals
- The remote-sensed 2.5-meter high, 25-km long lineament that also parallels the Woodstock fault.

Paleoliquefaction investigations along the Georgia, North and South Carolina coasts have identified and dated multiple episodes of paleoliquefaction that have constrained the latitude of the episodes. Crater frequency and width are greatest in the Charleston area, and decrease in frequency and width with increased distance along the coast, away from Charleston. This evidence led the NRC in 1992 to its position that a repeat of the Charleston earthquake was

assumed to be restricted to the Charleston, Middleton Place region. NRC guidance for the nearby VEGP commercial nuclear power plant has, therefore, been based on an assumed recurrence of the 1886 Charleston earthquake in the Summerville-Charleston area (WSRC2000a). Sporadic and apparently random low level seismicity is characteristic of the Coastal Plain and Piedmont geologic provinces (excepting clusters of seismicity in Bowman and Middleton Place). Regulatory guidance has prescribed a design basis local event to occur at a random location within a specified radius of the site (WSRC 2000a).

3.4.3.2 WSRC PC-3 And PC-4 Site-Wide Design Spectra

The site-wide design spectra fully implement DOE-STD-1023-95. DOE-STD-1023-95 specifies a broadened mean-based UHS representing a specified annual probability of exceedance for a systems, structures and components (SSCs) performance category and a historical earthquake deterministic spectrum that ensures breadth of the UHS. For the SRS, the deterministic spectrum is represented by a repeat of the 1886 Charleston earthquake. The development of the SRS design basis spectra uses a statistical methodology to verify that a mean-based response is achieved at the soil free surface.

The current PC-3 and PC-4 sitewide spectra are based on the WSRC analysis developed in 1997 (WSRC 1997) and incorporates variability in soil properties and soil column thickness. Following the development of PC3 and PC4 design basis spectra, the Defense Nuclear Facilities Safety Board (DNFSB) had several interactions with the DOE and WSRC on seismic design spectra. As a result, additional conservatisms were applied to the PC3 spectral shape at high and intermediate frequencies (Gutierrez 1999). The shape change was incorporated in the Site Engineering Standard. The shape change, illustrated in Figure 3-6, increased the low-frequency (0.1-0.5 Hz) portion of the PC-3 spectrum and also increased intermediate frequencies (1.6-13 Hz) of the design basis spectrum.

PC-3 spectra is applicable to DOE essential facilities, as described in DOE-STD-1020 (DOE 1996). Conversely, PC-4 spectra is reserved for more critical facilities such as nuclear reactors. The design spectra were intended for simple response analysis of SSCs and are not appropriate for soil-structure interaction analysis or geotechnical assessments. The current design basis spectra for the PC3 and PC4 design events are given in Figures 3-6 and 3-7, respectively.

3.4.4. Selection of MFFF Site Design Earthquake Motion

To select a design basis spectra for the MFFF site, DCS performed calculation "Selection of Extreme Natural Phenomena for MOX Design:" (DCS 2000e). Appendix A to NUREG-1718 requires that design of the MFFF ensure that high consequence events are highly unlikely to occur. Sufficient design margin must be available in the design of SSCs for the physical natural phenomena effects of earthquake loading. The calculation presents how the design earthquake (DE) for the extreme natural phenomena earthquake was established for the MFFF site. The performance goal that was achieved is based on the approach recommended in DOE-STD-1020-94, Change Notice #1, January 1996 (DOE 1996).

The PC-3 and PC-4 seismic events and response spectra for SRS is discussed in Section 3.4.3. The PC-3 is applicable to the MFFF site and can be used to establish the DE for the MFFF site. The PC-3 is applicable for essential facilities and conversely, the PC-4 is reserved for more critical facilities, such as nuclear reactors (DOE 1996).

To achieve target performance goals, the surface response for seismic events between the PC-3 and PC-4 events were evaluated. A Reg. Guide 1.60 spectrum, scaled to 0.20g, which lies between the PC-3 and PC-4 spectra, was analyzed in DCS 2000e. Figure 3-8 presents the Reg. Guide 1.60 spectrum anchored to 0.20g and along with the pc-3 and PC-4 spectrum. DCS 2000e demonstrates that this Reg. Guide 1.60 spectra anchored to 20g horizontal acceleration, satisfies the required performance goal and is acceptable to the probability of exceedance required for the MFFF design. On this basis, the Reg. Guide 1.60 spectra, anchored to 0.20g horizontal acceleration, has conservatively been selected as the horizontal design spectra for the MFFF. Site. The 0.20g Reg. Guide 1.60 spectra is also the same spectrum used for design of the Vogtle Nuclear Power Station, located directly across the Savannah River from the SRS. This Reg. Guide 1.60 response spectra, scaled to 0.20g is intended for simple response analysis of SSCs and is not appropriate for soil-structure interaction analysis or geotechnical assessments.

The horizontal surface peak ground acceleration (PGA) at the MFFF site for th DE has been established at 0.20g (DCS 2000e). The one-dimensional free-field site response analyses for a 0.20g surface response at the MFFF site is presented in Section 6.2 of this report. The results of liquefaction and post-earthquake settlement analyses are presented in Section 8.1 and 8.2, respectively, of the report.

4. GEOTECHNICAL EXPLORATION AND TESTING PROGRAM

4.1 APPROACH

The Geotechnical Exploration and Testing Program for the MFFF site was developed utilizing existing subsurface information that was available for the F-Area and the MFFF site. The topography for the MFFF site and the results of previous geotechnical investigations at and near the MFFF were used to define tentative critical structure locations for the MFFF (MOX Fuel Fabrication Building and Emergency Diesel Generator Building). These previous geotechnical investigations in the F-Area provided valuable insight to conditions that would be encountered at the MFFF site.

A detailed exploration program, consisting of exploration borings with standard penetration tests (SPT) and cone penetration test soundings (CPT), was developed to define subsurface conditions at and in the vicinity of proposed MFFF building locations. Major emphasis of the exploration program was to adequately define subsurface conditions at the location of critical structures. Figure 4-1 presents the location of previous site exploration borings and CPT test holes and the site specific MFFF exploration program completed in 2000. Based on the results of these exploration programs, the MFFF and Emergency Diesel Generator buildings have been located to avoid being directly located over extensive soft zones, as shown on Figure 4-1.

The original exploration program consisted of thirteen (13) exploration borings and thirty-seven (37) CPT soundings. The CPT program was extended to sixty-three (63) soundings after thick soft zones were encountered in the eastern portion of the MFFF site at the original building locations. The critical structures had to be relocated to avoid thick soft zones. The original soil boring locations were also adjusted to provide coverage at the present MOX and EDG building location and remained at a total of 13. Five dilatometer test holes (DMT holes) were performed at representative locations near CPT soundings and exploration borings to evaluate insitu stress conditions and to collect insitu data for correlation with the CPT, exploration boring and laboratory test results.

The primary purpose of the exploration borings was to obtain SPT results for correlation with CPT results and to collect representative soil samples for laboratory testing. The exploration borings were also drilled to a depth adequate to define the contact between the upper soil units at the site and the Congaree Formation. The Congaree Formation is an established geologic marker bed in the F-Area and at the SRS. The exploration borings were located to provide representative subsurface soil sampling across the MFFF site and to provide a positive definition of the geologic contact with the Congaree Formation.

CPTs were located to establish a continuous and representative profile of the upper soil stratigraphy and to collect in situ information for engineering evaluation of the site. CPTs have been used extensively and successfully at SRS and the F-Area in recent years to define subsurface conditions for engineering and groundwater evaluations. The extensive use of the CPT at SRS provides an extensive database for correlation with the MFFF site program.

All CPTs at the MFFF site measured tip resistance, pore water pressure, and sleeve resistance. Fourteen (14) of the CPT's were seismic cones and included measurement of compression (P) and shear (S) wave velocities at depth intervals of 5 feet. Seventeen (17) of the CPTs included measurement of electrical resistively for the full depth pushed. Pore water pressure dissipation testing was performed at selected locations to define the groundwater conditions. The CPT holes within the MFFF-Area were pushed to cone refusal, which occurred at some locations before reaching the Congaree Formation.

The CPT soundings were located to provide representative coverage across the MFFF site and to provide detailed subsurface information at the location of critical structures. Isolated soft zones at depth are known to be present in the F-Area from previous investigations, therefore, the CPT soundings for the MFFF site were spaced closer at critical structure locations than generally used for standard geotechnical programs. The CPT spacing was tightened to define the extent any soft zones or loose zones identified in the vicinity of critical structures. The management of the field geotechnical program included adding additional CPT or exploration holes, when soft zones were encountered so that their limits could be mapped. The CPT thin wall sampler was also utilized to obtain soil samples in identified softer layers at depth, when directed by the DCS Field Engineer.

All exploration borings and CPT and DMT soundings were drilled and grouted in compliance with established SRS procedures to prevent cross flow or contamination from the upper site aquifer system to the Congaree aquifer system. All holes were grouted after completion to seal any penetration into the Congaree aquifer system as required by SRS procedures.

The soil laboratory testing program was developed to establish a representative database for classification, static and dynamic testing of the soils engineering units defined at the MFFF site. The testing program was designed to provide adequate classification of engineering units encountered at the MFFF site for correlation with available test data for the F-Area and other relevant stratigraphic units at SRS. The laboratory testing program was designed to be adequate for correlation with CPT and SPT results and for establishing geotechnical design criteria that is required for the analysis of MFFF structures.

Samples of cuttings from exploration borings and all soil samples were tested by SRS Health Physics to verify whether radiological contamination was present at the site. For this extensive exploration and sampling program across the MFFF site, no radiological contamination was identified in any samples tested. No samples were removed from SRS until cleared by SRS Health Physics.

4.2 METHODOLOGY

The Geotechnical Exploration and Testing Program was conducted in accordance with MOX Project Procedure PP 9-19 Rev. 0, Geotechnical Exploration and Testing DCS 2000a. All field exploration programs were conducted under the engineering oversight of DCS Field Engineers, as outlined in DCS (2000a). The DCS Field Engineer performed engineering oversight and field documentation for all test boring and sampling activities, including preparation of the Log of Borings, selection of soil sampling type and location, and sample handling, packaging, storage

and shipment. The DCS Field Engineer also performed engineering oversight and field documentation for field activities associated with conducting the CPT soundings and DMT testing.

The DCS Field Engineer also coordinated with WSRC Health Physics for radiological testing of cuttings from exploration borings and samples collected for laboratory testing. No samples were removed from SRS until cleared of radiological contamination by WSRC Health Physics.

Miller Drilling Company, Inc. (Miller) performed the exploration borings and soil sampling in accordance with Specification for Geotechnical Test Borings and Sampling, DCS01-WRS-DS-SPE-G-00002-A (DCS 2000b). All Quality related technical field activities for soil exploration and sampling were performed by the DCS Field Engineer, in accordance with DCS 2000a. The drop hammer weight for SPT testing was certified under the DCS Quality Program.

The CPT program was performed by Applied Research Associates, Inc. (ARA) in accordance with Specification for Cone Penetration Testing of Soil, DCS01-WRS-DS-SPE-G-00001-A (DCS 2000c). ARA implemented their Quality Assurance Program requirements for all work performed per this specification. The DCS Field Engineer provided engineering oversight of the CPT program in accordance with DCS 2000a.

The soil testing program was performed by LAWGibbs Group, Inc. (LAW) in accordance with Specification for Laboratory Testing of Soils, DCS01-WRS-DS-SPE-G-00003-A (DCS 2000d). Selection of samples for laboratory testing and definition of the soils testing program was prepared by DCS geotechnical engineers, under the supervision of the DCS Lead Geotechnical Engineer in accordance with DCS 2000a.

4.3 GEOTECHNICAL EXPLORATION PROGRAM

4.3.1 General

The geotechnical exploration program for the MFFF site consisted primarily of cone penetration testing (CPT) and soil exploration borings. The CPT program began on May 31, 2000, approximately two weeks before the soil borings, to allow use of the CPT data to revise boring locations, as needed. The soil boring exploration program was completed on July 22, 2000 and the CPT program was completed on July 24, 2000. In addition, the results of site geotechnical programs previously performed by others (Raymond, 1973; Geomatrix, 1998; WSRC, 1999a) adjacent to and on the MFFF site were also used to evaluate subsurface soil and groundwater conditions. The locations of soil borings and CPT holes advanced by DCS, as well as those by others that were used to help characterize the MFFF subsurface soils are shown on Figure 4-1. In addition to CPT and soil borings, dilatometer testing and seismic downhole testing were performed at the site. The following sections describe work performed as part of the geotechnical exploration program.

4.3.2 Exploration Boring Program

Thirteen soil exploration borings were drilled at the MFFF site to depths ranging from approximately 131 feet to 181 feet below the present site grade. All of the borings were advanced using a CME-75 truck-mounted drill rig and the mud-rotary drilling method. Ten of the holes were drilled with a diameter of six inches, and three were drilled with a diameter of eight inches. The larger diameter holes were drilled for BH-2, BH-5 and BH-10 to facilitate the installation six-inch-diameter PVC casing for downhole geophysical testing as discussed in Section 4.3.5. The main purpose of the borings was to correlate the results of the CPT soundings, obtain soil samples for laboratory testing and identify the Warley Hill Formation ("green clay" GC) and the top of the Congaree Formation (CG). A complete description of the boring program and the Logs of Borings are presented in Attachment 1.

4.3.3 Cone Penetration Tests (CPT)

Sixty-three (63) CPT soundings were advanced to depths ranging from 64 feet to 140 feet below present site grade. All soundings were conducted with piezocones (P-CPT), 15 soundings were conducted with a combined seismic cone (S-CPT), and 20 soundings were conducted with a combined resistivity cone (R-CPT). The target depth of the CPT soundings was the top of the Congaree Formation. However, this depth was achieved in only a few soundings due to reaching the push capacity of the truck (60,000 pounds) before reaching the target depth. The main purpose of the CPT soundings was to obtain a continuous profile of the subsurface conditions at several locations across the MFFF site. In this regard, the CPT is a particularly useful method for identifying soft zones. A thin-walled sampler was also used in conjunction with the CPT rig to obtain samples of potential soft zone materials for laboratory classification testing.

The initial CPT program consisted of 36 soundings, but was increased as a result of the need to further delineate identified soft zones and to investigate the relocated sites for the MFFF and emergency diesel generator buildings. The details of the CPT program conducted at the MFFF site, along with test results, are presented in Attachment 2.

4.3.4 Dilatometer Tests

Dilatometer testing (DMT) was performed in an attempt to provide additional insitu test data to correlate with the CPT, SPT and laboratory test results. DMT was performed adjacent to CPT-10, CPT-15, CPT-23 and CPT-29. The DMT holes are labeled DMT-10, DMT-15, DMT-23 and DMT-29, to indicate the respective CPT correlation hole.

High push pressures, generally greater than 500 psi, were required to advance the dilatometer vane into the predominately subsurface sandy soils. Analysis of the DMT results indicates disturbance around the vane during the testing, which is considered to a result of the high push pressures in the sand. Calculated modulus values from the DMT data were considerably lower than indicated by the adjacent CPT and SPT test results. The DMT test results were not considered representative of the subsurface encountered in the other exploration holes and were not used in further analysis. The data from the DMT holes is presented in Attachment 2.

4.3.5 Seismic Downhole Testing

Seismic downhole testing was performed in three of the soil borings (BH-2, BH-5 and BH-10) drilled at the MFFF site. The primary purpose of this testing was to obtain shear wave velocity data for correlation with the results of the S-CPT data. Also compression wave velocity data was obtained in each test hole. The seismic downhole testing was conducted from September 26 through September 28, 2000, after all CPT and drilling activities had finished in order to have a relatively "quiet" site.

The boreholes in which the testing was conducted were each located adjacent to a S-CPT sounding in order to obtain presumably direct correlations between the downhole survey and S-CPT data. The boreholes were cased with 6-inch-diameter PVC plastic pipe, and the annular space between the outside of the casing and the borehole wall was grouted with lean cement grout. Based on the results of the seismic downhole testing (discussed in Section 6.2.1), it appears that a solid coupling of grout between the casings and boreholes was not achieved in the lower sections of BH-5 and throughout BH-2. A solid coupling is important between the casing and soil. Loose soil along the casing can result in significant attenuation of seismic energy being measured. This lack of coupling between the soil and casing resulted in a reduced quality of data being obtained in BH-5 and BH-2. A detailed description of the seismic downhole testing conducted at the MFFF site, along with test results, are presented in Attachment 3.

4.4 LABORATORY TESTING PROGRAM

Laboratory testing was conducted on soil samples collected during the geotechnical field investigation. The purpose of the testing was to determine the index and engineering properties of the site soils pertinent to the geotechnical analyses and design for the MFFF facilities. The results of the testing program were also correlated with published laboratory test data from the APSF site, the F-Area Northeast Expansion and F-Area to help identify the major soil units in the shallow (<200 feet) stratigraphy. The soil properties tested include:

- moisture content;
- wet and dry density;
- particle size;
- plasticity;
- consolidation characteristics;
- shear strength parameters; and
- dynamic behavior (shear modulus reduction and damping).

Samples for laboratory testing were selected based on a review of the soil boring logs and CPT results in order to obtain information most representative of the soils encountered. Revisions to the testing program were made as necessary, based on observations made in the laboratory. The

results of the laboratory testing are discussed in Section 6 of this report. A description of the laboratory testing program and test results, are presented in Attachment 4.

5. SUBSURFACE SOIL CONDITIONS

5.1 ENGINEERING STRATIGRAPHIC UNITS

5.1.1 General

The subsurface soil conditions at the MFFF site were characterized based on information obtained from the field geotechnical exploration program. Material characterization included field logging and extensive laboratory index and engineering property testing. Groundwater conditions were estimated from CPT pore pressure dissipation test results and a review of monitoring well data from locations near the MFFF site. The MFFF subsurface characterizations were compared to published subsurface data from F-Area in general, and the adjacent Actinide Packaging and Storage Facility (APSF) and Northeast Expansion area geotechnical investigations (WSRC 1996, 1999a; Geomatrix 1998) as a check for reasonableness and to help identify potentially anomalous soil conditions at the MFFF site.

Nine (9) geotechnical subsurface cross sections through the MFFF site were developed to illustrate the subsurface stratigraphy to a depth of approximately 130 to 150 feet below existing grade. The locations of the subsurface cross sections are shown on Figure 5-1, and the cross sections are shown in Figures 5-2 through 5-10. For comparison to previous geotechnical investigations adjacent to the MFFF site, Cross Section 4-4 from the F-Area Northeast Expansion Report (WSRC 1999a) is presented in Figure 5-11. This cross section is located along the southern portion of the MFFF site, just south of the MFFF building location.

5.1.2 Subsurface Stratigraphy

The subsurface stratigraphy at the MFFF site was determined mainly from CPT measurements including tip resistance, sleeve friction, pore pressure and shear wave velocity. This information was correlated with data from adjacent geotechnical borings, where available. The basis for the stratigraphic subdivisions described in the following sections was developed by WSRC (1996, 1999a). WSRC developed an "engineering stratigraphy" nomenclature to distinguish the soil layers from the geologic formations in which they lie. To maintain consistency with previously published SRS data, the alphanumeric system used at SRS to describe the engineering stratigraphy is maintained in this report. The correlation currently used at SRS for identifying engineering stratigraphy units relative to their respective geologic units is presented in Table 5-1. A detailed discussion of the geologic units is presented in Section 3.1.

In general, the subsurface stratigraphy of the MFFF site is consistent with the conditions found at the APSF site, located immediately south of the MFFF site, and the F-Area Northeast Expansion Area, located about 150 yards southeast. In addition, the average material properties for each of the stratigraphic layers discussed below correlate quite well with those averages found at the APSF and F-Area Northeast Expansion sites. A summary of the material properties for the MFFF subsurface soils, with comparisons to published averages for F-Area, APSF and F-Area Northeast Expansion soils, is shown in Table 5-2. Table 5-2 shows that layer thickness and engineering unit index properties for the subsurface units at the MFFF site are reasonably consistent with those found at the F-Area sites.

5.1.2.1 TR1 and TR1A

The TR1 and TR1A layers are most probably part of the Altamaha Formation, sometimes referred to as the "Upland Unit." In general, these soils consist of red, purple and brown poorly sorted sands, clayey sands and silty sands in a medium dense to dense state. The base of the Altamaha is characterized by an irregular erosional surface, and ranges from zero to about 70 feet thick at the SRS. The TR1 layer is typically found at El 260 or higher at the MFFF site, often contains some fine gravel, and is less fine-grained than the underlying TR1A. The TR1 layer is characterized by moderate CPT tip resistances and relatively high friction ratios while the underlying TR1A layer is generally less dense. TR1 ranges in thickness from about 10 feet in the western portion of the MFFF site, tapering to near zero thickness in the eastern third. TR1A ranges in thickness from about 10 to 20 feet. The top of TR1 ranges in elevation from approximately El 265 to El 280. The average top of layer of TR1A at the MFFF site is about El 267.

5.1.2.2 TR2A and TR2B

The TR2A and TR2B layers are used to differentiate the Tobacco Road Formation. Sediments of the Tobacco Road Formation were deposited in low energy, shallow marine transitional environments such as tidal flats. As a result, these sediments exhibit a relatively distinct laminated structure consisting of red, brown and purple, medium dense to dense, poorly sorted sands and clayey sands. The boundary between TR1A and TR2A is often identified by an increase in CPT tip resistance and notably lower sleeve friction values, resulting in substantially lower friction ratios. Although TR2A and TR2B have very similar material properties, TR2B is typically identified by an increase in CPT tip resistance. TR2A ranges in thickness from near 30 feet near the center of the MFFF site, to about five feet near the eastern edge of the site. The thickness of TR2B ranges from about 32 feet near the center of the site to approximately 15 feet at the eastern edge of the site. The average top of layer of TR2A at the site is about El 255. The average top of TR2B is about El 235.

5.1.2.3 TR3/4 and DB1/3

The TR3/4 and DB1/3 layers are part of the Dry Branch Formation. The TR3/4 layer consists primarily of stiff clay and sandy clay inter-bedded with loose to medium dense clayey sands and sandy silts. The fine-grained fraction (minus No. 200 sieve) of the TR3/4 soils are moderately to highly plastic. The upper boundary of the TR3/4 layer is defined by a significant decrease in CPT tip resistance and an increase in friction ratio and pore pressure, relative to the TR2B layer. The TR3/4 layer appears to be continuous across the MFFF site and ranges in thickness from about three to 10 feet. The average top of the layer for TR3/4 is approximately El 212.

The DB1/3 layer corresponds to the Irwinton Sand member, and consists mainly of silty sands and poorly graded sands, with widely interspersed thin sandy clay and clayey sand layers. The sands are generally medium dense, with widely interspersed pockets of loose and dense to very dense material. The fines within this layer are notably more plastic than those reported in the DB1/3 layer at the APSF and F-Area Northeast Expansion sites, but closer to the average values reported for F-Area (WSRC, 1996, 1999a). DB1/3 is characterized by variable, but generally high CPT tip resistances and low friction ratios, and near hydrostatic pore pressures. The DB1/3

layer at the site ranges in thickness from about 18 to 30 feet, and has an average top of layer elevation of approximately El 205.

Isolated soft silty clay zones, generally less than 2 to 3 feet in thickness, are occasionally encountered in the TR3/4 and DB1/3 units. These soft material zones are defined by a SPT N value less than 5 or a CPT tip resistance less than 15 tsf. These soft material zones, when encountered, are limited in vertical and lateral extent. The location and extent of soft material zones found in the vicinity of the MFFF buildings is conservatively estimated on Figures 5-2 and 5-5. These soft material zones are not related to the "soft zones" encountered in the DB4/5, ST1 and ST2 units, as discussed in Section 5.2.

5.1.2.4 DB4/5, ST1 and ST2

The DB4/5, ST1 and ST2 layers are part of the Santee/Tinker Formation, generally regarded as the most complex geologic unit in the shallow subsurface of F-Area (WSRC, 1999a). Soils in this formation are characterized by highly variable material properties. The Santee/Tinker Formation is generally distinguished from the overlying Dry Branch Formation by CPT tip resistances and friction ratios exhibiting a pronounced saw tooth profile, with large variations over small vertical intervals. This pattern is consistent with lenses of clayey and silty sands interfingered with more resistant, silica-cemented sediments and less resistant, calcareous sediments.

The soils of the DB4/5 layer consist mainly of loose to medium dense, medium to highly plastic clayey and silty sands. In general, this layer is more clayey and silty than the underlying ST1 layer. The DB4/5 layer typically exhibits moderate to low CPT tip resistances and moderate friction ratios. The DB4/5 layer has been extensively characterized within the APSF site because of the presence of several "soft zones" (defined at SRS as zones having a CPT tip resistance of 15 tsf or less, or an SPT N-value of 5 or less). Some soft zones were also identified in the DB4/5 layer at the MFFF site, as discussed in Section 5.1.3, below. At the MFFF site, the DB4/5 layer ranges in thickness from about five to 13 feet, and has an average top of layer elevation of about El 183.

The ST1 and ST2 layers mainly consist of silty sands and poorly sorted sands. In general, the ST1 layer is dense to very dense, while the underlying ST2 layer is loose to medium dense. The ST1 is characterized by markedly higher CPT tip resistances and sleeve friction than either the DB4/5 or ST2 layers. At the MFFF site, the ST1 layer ranges in thickness from about 15 to 21 feet, with average top elevation of about El 174. Based on the limited number of CPT soundings that penetrated the ST2 layer, the thickness ranges from about 5 to 15 feet, with the top elevation averaging about El 157.

5.1.2.5 GC

The term "green clay" is an informal stratigraphic name at SRS for the medium dense to dense green, brown and gray clayey sands, silty sands, sandy silts and stiff to very stiff sandy clays that mark the Warley Hill Formation at the base of the Santee/Tinker Formation. The GC layer is locally continuous across F-Area, and is typically used at SRS to define the lower boundary of the shallow stratigraphy. Only a few of the CPTs at the MFFF site fully penetrated the GC layer

due to its depth. All of the exploration borings fully penetrated the GC layer. Based on this limited information, the GC layer at the site ranges in thickness from about three to 10 feet, and has an average top elevation of about El 146. The GC layer is considered to be continuous across the MFFF site.

The GC unit also contains some isolated soft silty clay zones that are generally less than 2 to 3 feet in thickness. These soft material zones are defined by a SPT N value less than 5 or a CPT tip resistance less than 15 tsf. These soft material zones, when encountered, are limited in vertical and lateral extent. The location and extent of soft material zones found in the vicinity of the MFFF buildings is conservatively estimated on Figure 5-5. These soft material zones are not related to the "soft zones" encountered in the DB4/5, ST1 and ST2 units, as discussed in Section 5.2.

5.1.2.6 Congaree to Bedrock

The Congaree Formation at the MFFF site is identified in all exploration borings as a very dense sand. Once the Congaree Formation is encountered at the MFFF site, SPT refusal (N>100 blows/foot) and CPT refusal (tip resistance >400tsf) is quickly achieved, as shown on the geotechnical cross sections, Figures 5-2 through 5-10. Not all CPTs were able to penetrate to the Congaree because they reached practical refusal in dense sand layers above the Congaree.

Once the Congaree Formation is encountered at SRS, very dense to hard soils units are encountered down to the basement bedrock. The geologic formations below the Congaree are discussed in Section 3.1.1. Since these geologic units have high strength and low compressibility, they will not contribute to any significant settlement at the MFFF buildings. These geologic units, however, are considered in the dynamic evaluation of the MFFF site. The Congree Formation is considered to be continuous across the MFFF site and has an average top at about EL 141.

The bedrock at the MFFF site location is Paleozoic crystalline bedrock at about El -585 feet, (WSRC, 1996).

5.2 SOFT ZONES

Soft zones at the MFFF site are defined using the same criteria applied to the APSF site and F-Area Northeast Expansion (WSRC, 1999a, 1999b). That is, zones having SPT N-values ≤ 5 or CPT tip resistance of ≤ 15 tsf. It is important to note that the generally accepted formation mechanism for soft zones (i.e., dissolutioning of carbonate-rich, clastic sediments) suggests that they are restricted mainly to the lower Dry Branch (DB4/5) and Santee/Tinker Formations (ST1 and ST2). Section 3.2 presents a discussion of the origin and characteristics of soft zones encountered at SRS.

At the MFFF site, some isolated zones of soft clayey materials with low SPT N-values or CPT tip resistances were identified within the TR3/4, DB1/3 and GC layers. These soft material zones are also discussed in Sections 5.1.2.5 and 5.1.2.6 and are very limited in lateral and vertical extent. These zones not considered to have the same origin as "soft zones," but will be

referred as soft materials. These soft materials are pockets of softer or less dense, relatively plastic clays or clayey sands and are consistent with other references cited at SRS.

Using the above definitions, soft zones encountered during the site exploration program had their limits defined by adding additional CPT holes and borings to determine their lateral extent. The exploration hole spacing, in the vicinity identified soft zone, was generally on the order of 90 feet or less. The locations of borings and CPTs with soft zones identified during the field investigation are shown in plan view in Figure 5-1, and in section in Figures 5-2 through 5-10. The extent of soft zones shown on Figures 5-2 through 5-10 has been conservatively mapped as to thickness and lateral extent, based on exploration spacing. The conservatively assumed thickness and lateral extent has been used in analysis to assess their affect to the MOX and EDG buildings in later Sections. The soft zone intervals identified at the MFFF site are summarized in Table 5-3. The presence of numerous and somewhat laterally continuous soft zones within the eastern portion of the MFFF site resulted in relocating the MOX and Emergency Diesel Generator buildings further west within the MFFF site boundary to avoid placing these structures directly over any significant soft zones (present in adjacent exploration holes and soft zones thickness >4feet).

Only three soft zone "hits" were recorded within the relocated structure footprints and these were determined to be limited in lateral extent and ≤4 feet thick by comparing results from near by exploration holes.. These are located in CPT-50 between El 180.9 and 184.6, BH-11 between El 139 to 143, and BH-13 between 180 to 182. No soft zones were identified in one boring and four CPTs located around CPT-50. CPT-58 and boring C-3, adjacent to BH-11 did not indicate any soft zones. Also the area identified as a soft zone in BH-11 appears to be primarily in the GC layer and clayey in properties. No soft zones were found in CPT-48 and CPT-49 near BH-13. On this basis, the soft zones identified beneath the MOX building appear to be limited in thickness to ≤4 feet and their lateral extent appears to be limited and confined, based on the lack of soft zones in adjacent exploration holes.

Soft zones were encountered within 100 feet of the MFFF building footprints in four CPT soundings and one boring as shown in Figure 5-1. Two soft zone intervals were detected in both CPT-45 and CPT-46. These upper zones were identified at about El 182 with thicknesses of approximately 3.1 and 5.1 feet, respectively. The lower zones occur at about El 150 with thicknesses of approximately 3.4 and 6.7 feet, respectively. In addition, an earlier geotechnical investigation (Raymond, 1973) of the site area identified a soft zone from about El 143 to El 151 in a boring located approximately 60 feet south of the proposed location of the diesel generator building (B9 on Figure 5-1). However, no soft zones were detected in three CPT soundings and one boring located between the generator building and boring B9. A relatively thick soft zone was identified between El 185.3 and 173.9 in CPT-61, located near the southwest corner of the MFFF building. However, no soft zones were found in the nearby CPT soundings, or in two adjacent borings conducted as part of an earlier investigation (Raymond, 1967). The potential effects of the identified soft zones on the MFFF buildings are discussed in Sections 7.0 and 8.0 of this report.

No unusual soft zone conditions were identified at the MFFF site, and in particular, no voids were encountered. The soft zones identified within or near the critical MFFF and emergency diesel generator buildings are at depths of about 90 feet or more, and are considered to be isolated with limited lateral extent, based on the overall spacing and layout for borings and CPTs.

5.2.1 Soft Zone Confirmatory Exploration Program-Fall 2001

A soil exploration program that will be conducted in Fall 2001 to confirm that there are no detrimental soft zones that were missed in the initial program. The exploration program conducted in 2000 is presented in Section 4. Due to the slopes of the spoil fill at the MFFF site, exploration equipment could not access desired exploration locations during the initial field program. Also SCPT equipment was not available when the exploration was expanded to the relocated MOX and EDG building locations. Due to the spoil fill slopes, some of the boring and CPT spacing beneath the MOX and EDG buildings is presently considered to widely spaced to specifically state that there are no soft zones that could be detrimental to these structures.

A bench will be cut into the slopes of the fill so that exploration rigs can access the locations necessary to tighten the exploration program and confirm the absence of soft zones that could potentially impact the present critical building locations. Approximately 16 CPT will be located on the bench, 6 CPTs on the north and east toe of the slope, and 10 CPTs on top of the fill to tighten previous exploration spacing. The final exploration spacing beneath the MOX and DEDG building areas will be approximately 100 feet or less on center. The main purpose of the exploration program is to use exploration borings and CPTs to penetrate through the Santee units. It is anticipated that predrilling of the fill and upper units may be required at some locations so that the CPTs can achieve full penetration through the Santee units. The exploration program will be adjusted, as required in the field, to adequately define any soft zones and achieve desired program results.

Approximately 5 exploration borings, SPT and Shelby tube sampling will be used to augment the CPT holes and to obtain samples for laboratory testing. The CPT thin wall sampler will also be used to obtain soft zone samples where appropriate. Additional seismic profiles will be obtained during this exploration program by the use of S-CPTs and two additional cased holes for seismic downhole testing. Soil sampling and laboratory testing will be performed but generally limited to soft zones or loose soil zones for further material characterization.

The results of the Fall 2001 geotechnical program will be reviewed to the analysis and conclusions presented in this report. Analysis will be revised, as required, based on the results of the Fall 2001 geotechnical program. The results of the geotechnical program and any revised conclusions and recommendations will be presented in a revision to this Geotechnical Report. The methodology presented in Section 4.2 for the Geotechnical Program conducted in 2000, will be followed for the Fall 2001 exploration program.

5.3 GROUNDWATER CONDITIONS

Section 1.4.2 of WSRC (2000a) provides a detailed discussion for the groundwater hydrology at SRS and the F-Area. This section presents a summary of groundwater hydrology for the MFFF site.

5.3.1 Groundwater Aquifers

The groundwater conditions at the MFFF site have the same characteristics as the F-Area. Groundwater in the shallow, intermediate, and deep aquifers at the MFFF site flows in different directions, depending on the depths of the streams that cut the aquifers. The Upper Three Runs Aquifer Unit is the shallow groundwater, unconfined aquifer at the MFFF site and discharges into Upper Three Runs Creek to the north. The Upper Three Runs Aquifer Unit occurs between the groundwater table and the Gorden Confining Unit (Warley Hill Formation and GC) and includes all strata above. The Gorden aquifer underlies the Three Runs aquifer at the MFFF Site and flows horizontally toward the Savannah River. Underlying deeper aquifer units at the MFFF site flow southeast toward the coast (WSRC 2000a).

Groundwater flow at the MFFF site area is vertical and lateral. In the Upper Three Runs and Gorden aquifers, flow moves downward until its movement is obstructed by impermeable material. This was illustrated in CPTs located near the existing sedimentation basins at the south portion of the site. Water held in the sediment basins infiltrates into the Upper Three Runs aquifer, causing local groundwater mounding at these locations. The GC layer provides a confining layer between the Upper Three Runs and Gorden Aquifers. Groundwater in the Upper Three Runs aquifer flows laterally to the north across the MFFF site, as shown on Figure 5-12.

Operating under a different set of physical conditions, groundwater in the intermediate and deep aquifers flows mostly horizontally. At the F-Area and MFFF site, flow from deeper aquifers moves upward due to higher water pressure below the confining unit between the upper and lower aquifer systems. This upward movement helps to protect the lower aquifers from any contaminants that might be present in the F-Area in the shallow Upper Three Runs aquifer.

The depth to groundwater at the MFFF site area varies from approximately elevation 200-feet to 210 feet and is found at a depth of over 60 feet below existing ground level. At the location of the MOX and EDG buildings, the groundwater level was encountered at approximately El 200. The groundwater levels shown on Cross Sections 1, 2, 3 and 4 (Figures 5-2, 5-3 and 5-4) were developed based on pore dissipation tests performed in the respective CPTs during the exploration program in June and July 2000. These groundwater levels support groundwater contours presented for the Upper Three Runs aquifer in WSRC (2000a and 2000b). At the time of the field exploration program, SRS had been experiencing drought conditions. Long term groundwater monitoring in the F-Area indicates that the groundwater level can fluctuate as much a 10 feet seasonally (WSRC 1999a).

The design groundwater level at the location of the MOX and EDG buildings has been conservatively established at El 210, which is approximately 60 feet below planned final grade for the site.

5.3.2 Groundwater Quality

Groundwater quality in F-Area and MFFF site is not significantly different from that for SRS as a whole. It is abundant, usually soft, slightly acidic, and low in dissolved solids. High dissolved iron concentrations occur in some aquifers.

WSRC (2000b) provides a comprehensive discussion of groundwater contamination plumes in the F-Area and covers the MFFF site. Also WSRC (1995) for the Old F-Area Seepage Basin defines the soil and groundwater contamination from past disposal practice into the seepage basin. The Old F-Area Seepage Basin is located just northwest of the MFFF site area, as shown on Figure 5-1. The contaminated soil zone within the Old F-Area Seepage Basin was remediated in 2000.

The contamination plume and direction of migration from the Old F-Area Seepage Basin is defined in WSRC (1995). The contamination plume from the Old F-Area Seepage Basin migrates to the north, away from the MFFF building area and is located at depth within the groundwater level. This contamination plume is known to pass under the northwest corner of the MFFF site within the boundary area located northwest of the transmission corridor.

WSRC (2000b) characterized groundwater conditions in and around the F-Area and delineated contamination plumes migrating from the mixed waste burial area, southeast of F-Area. This report indicates that there are no known soil or groundwater contamination plumes that will pass beneath the MFFF building areas. The lack of subsurface site radiological contamination was confirmed with the recent comprehensive geotechnical investigations conducted during 2000 at the MFFF site. Radiological testing was performed for drill cuttings and all samples obtained from the exploration activities. No radioactive contamination was encountered during this program in the Upper Three Runs or Gorden Aquifers, which are the upper aquifers at the MFFF site. WSRC (1995 and 2000a) indicate that the identified groundwater contamination plumes in the F-Area should not pass beneath the MFFF building areas, since the buildings are located up gradient of the direction of known plume migrations.

Any contamination plumes, which could potentially pass near the MFFF building locations in the future, would be located at depth (approximately 60 feet) within the groundwater level. The anticipated construction, site preparation, and development for the MFFF facilities will be confined within the geologic units well above the Upper Three Runs Aquifer units. The planned construction and operation activities for the MFFF will not have any adverse affect to the existing aquifer systems beneath the MFFF site area.

6. ENGINEERING PROPERTIES

6.1 STATIC PROPERTIES

6.1.1 General

The engineering properties for static analysis have been developed using the results of field and laboratory tests. Field insitu test measurements consisted of SPT N values, SCPTU data (shear wave velocities, tip resistance, friction ratio and pore pressure), RCPTU data (soil resistively, tip resistance, friction ratio and pore pressure), and CPTU data (tip resistance, friction ratio, and pore pressure. Dilatometer testing was performed at select locations adjacent to CPT test holes. However, the results of the dilatometer testing were not considered representative of in situ soil conditions and have not been used to develop engineering properties. Downhole seismic testing was also performed to obtain shear and compressional wave velocities. The details of the geotechnical exploration programs are discussed in Section 4.3.

Laboratory tests were performed on representative samples obtained for the various engineering units to establish physical and static engineering properties for the soils. Laboratory testing consisted of testing for moisture content, wet and dry density, particle size analysis, plasticity, consolidation characteristics, and shear strength. The details for the laboratory testing program are discussed in Section 4.4.

The field and laboratory test results were combined to create a data base for each engineering unit. The results of testing for the MFFF site were compared to results published for the: F-Area Northeast Expansion Report (WSRC 1999a); Significance of Soft Zone Sediments at the Savannah River Site (WSRC 1999b); APSF Additional Geotechnical Investigation Program Plan (WSRC 1998); Final Geotechnical Report Actinide Packaging Storage Facility (APSF) (Geomatrix 1998) and F-Area Geotechnical Characterization Report (WSRC 1996);.

6.1.2 CPT and SPT Field Test Results

The results of the CPT and SPT testing from the field exploration program have been summarized by engineering unit. The results are presented as an average of properties for each engineering unit. The averages SPT N values were determined using the 13 exploration borings. Any N values for soft layers were incorporated into the average N value determination.

The tip resistance, friction ratio and pore pressure results were obtained using all 63 CPTs performed on the site. The CPTs tip resistance has been corrected using the net area concept as discussed in Attachment 2. The corrected tip resistance is referred to as q_{cor} (same as q_t in Attachment 2). The CPTs did not always penetrate to, the full depth of the GC, as can be seen on Figures 5-2 through 5-10. The CPT tip resistance presented in these figures was also used to identify the locations of soft zones and soft materials. The shear wave velocity profile was determined from the 15 SCPTU test performed at the MFFF site.

The results are presented on Table 6-1 (DCS 2001a) for each engineering unit and are compared with average values available for the F-Area and NE Expansion reports. Data comparisons were

also made where data was available for the APSF area. The average SPT and CPT test values for the MFFF site compare well with those obtained from the other nearby F-Area studies. The N values and q_{cor} for the MFFF site trend slightly lower to approximately the same as values defined for the same engineering unit in the F-Area. The shear wave velocities determined for each engineering unit are approximately the same as those determined in the other studies. With the high number of exploration holes used for the MFFF site, the N values, q_{cor} and shear wave velocities for the MFFF site are considered representative for each engineering unit and are recommended for use in engineering analysis. No unusual site conditions were encountered at the MFFF site and the subsurface conditions are similar those anticipated, based on results from the adjacent geotechnical studies.

6.1.3 Classification and Physical Property Test Results

Classification and physical property testing consisted of Atterberg limits, water content, grain size analysis, hydrometer analysis, and density determinations. The averages determined for each engineering unit are presented on Table 6-2 (DCS 2001a).

The liquid limit (LL) and plasticity index (PI) averages for the fines content at MFFF site are slightly higher than the results listed in the APSF and NE Expansion reports. The water content, dry density and percent sand content averages are approximately the same as reported in other geotechnical reports. As shown by the averages, the percent sand content is generally greater than 60 percent. The sandy nature of the subsurface soil units is also confirmed by the CPT results, which indicate a low friction ratio (f_s) and a low pore pressure during testing (refer to Table 6-1). Even though the percentage of fines is low in the samples, the fine content in each engineering unit generally classifies as a CL or CH.

The classification and physical test result averages presented for each engineering unit are considered representative of the subsurface engineering units for the MFFF site. The values are considered appropriate to use for engineering analysis and evaluation of each engineering unit.

6.1.4 Strength Test Results

Triaxial shear tests were performed at nine representative test locations in the subsurface soils, which represent five engineering units. All of the samples tested were clayey sand and the results presented are only considered representative of the clayey sands that would be encountered in these respective engineering unit. The samples obtained for testing were also representative of the softer and lower density materials encountered in the engineering unit. Representative undisturbed samples of the cleaner, denser sand could not be effectively obtained during the exploration program.

The results are presented in Table 6-3 (DCS 2001a) and should only be used to evaluate the softer, lower density and more clayey zones within the engineering units. The test results for the MFFF site compare favorably with test results reported in the F-Area and APSF reports for the TR3/4 and DB4/5 engineering units. The shear strength results presented for the TR3/4 and DB4/5 units are considered appropriate for these two softer units. Other strength values presented in Table 6-3 are not considered representative of the average strength of the

engineering unit. The N values and q_{cor} results presented in Table 6-1 are considered more appropriate to estimate the in situ strength of the sandy subsurface materials and have been used for the geotechnical analysis.

6.1.5 Consolidation Test Results

Consolidation tests were performed to obtain representative consolidation properties for each of the engineering units. Again, sampling was generally limited to the more clayey and softer materials within the engineering unit. The average results for the consolidation tests performed in each engineering unit are presented in Table 6-4 (DCS 2001 b).

The results from the MFFF site show close correlation with results from the F-Area and APSF results for void ratio C_c and C_r. The variation seen in the average test results between the areas is to be expected. Disturbance during sampling and sample preparation for testing of the sandy soils is reflected in the shape of the consolidation curves. The P_c values are considered suspect due to the sandy nature of the site soils and observed disturbance affects in the consolidation test curves. The C_r values in Table 6-4 are considered appropriate for use in settlement analysis and are appropriate for each engineering unit (DCS 2001b).

6.1.5 Soft Zone Engineering Properties

Engineering properties of the soft zones at SRS have been studied extensively over several years and have been summarized in WSRC (1999b). The analysis of soft zones and soft materials at the MFFF site (DCS 2001c) indicates that the measured index and mechanical properties of soft zone soils for SRS are consistent. The soft zones encountered at the MFFF site were defined using the criteria presented in WSRC (1996b), as described in Section 5.2. In the MFFF site exploration program, soft zones encountered at the site are consistent with those described for the F-Area and APSF (WSRC 1999a and WSRC 1999b). On this basis, the recommendations presented in WSRC (1999b) for soft zone engineering properties were used to develop properties for the MFFF site. The recommended soft zone properties to be used for the WFFF site are as follows:

- Total Unit Weight 100 pcf
- Dry Unit Weight 80 pcf
- Shear Wave Velocity 250 fps
- Poisson's Ratio 0.47
- Effective Friction Angle 5°
- Dilation Angle 0°
- Compression Ratio ($C_c/1+e_0$) 0.24
- Best Estimate value of OCR 0.7

6.1.6 Engineered Structural Fill

An engineered structural fill will be utilized beneath the MOX and EDG buildings, as described in Section 7.2. The fill will be designed as a high strength fill to provide for high quality and

uniform bearing material for foundations and to better distribute anticipated settlement to the structures. The actual structural fill gradation and material requirements have not been established at this time. Based on subsurface conditions ad the MOX and EDG buildings, it is anticipated that the structural fill will be on the order of 10 feet beneath the main MOX building and 5 feet thick beneath the sublevel of the MOX building and EDG building. These structural fill thickness have been assumed for analysis at this time. Engineering properties to be assumed for design at this time are:

• Durable crushed rock material	
• Well graded gravel	
• Maximum particle size	1.5 to 2 inches
• Fines content	10 percent or less
• Fines	non plastic
• Dry density	133 pcf
• Total unit weight	145 pcf
• Angle of internal friction ϕ'	45°
• Cohesion	0

6.2 DYNAMIC PROPERTIES

6.2.1 Shear Wave Velocities

Field measurements of in situ shear wave velocity were conducted at the MFFF site during the geotechnical field investigations and are discussed in Section 4. This data was used to develop an idealized profile of low-strain shear wave velocity at the site, suitable for input into various engineering analyses as discussed in Sections 6.2 and 8. Shear wave data was collected using S-CPT and seismic downhole techniques to a depth of approximately 130 feet ("Green Clay" layer) as discussed in the following sections.

Shear wave velocities for deeper soils were based on data from the APSF site and other areas within the SRS (Geomatrix, 1997b). The bedrock within F-Area is reported to be crystalline in nature and very strong with an estimated shear wave velocity of 11,000 fps (WSRC, 1996). The idealized geologic profile for the MFFF site is presented on Table 6-5 and shown on Figure 6-1. The idealized low-strain shear wave velocity profile for the MFFF site is shown on Figure 6-2. (DCS 2001d) The upper 130 feet of the idealized geological profile (above the congree) was developed using MFFF site specific data and the lower profile (below the GC) was based on Geomatrix 1997b.

6.2.1.1 Seismic CPT Data (S-CPT)

Of the 63 CPT soundings performed at the MFFF site, 15 were S-CPTs. The location of the S-CPTs are shown on Figure 5-1. The results of the shear wave velocity tests from the S-CPTs are presented in Attachment 2. After identifying the various soil layers on each S-CPT trace, the

shear wave velocity for each layer was taken as the mean of all CPT velocities for that layer. Since the S-CPTs extend only as deep as the "green clay" (GC) layer, this methodology was adopted to determine the idealized shear wave velocity profile from the ground surface to the GC layer. The statistical summary of shear wave velocity data computed from the S-CPT sounding data is presented in Table 6-6. The interval velocities for each S-CPT compared to the idealized profile (down to the GC layer) area shown on Figure 6-3 (DCS 2001d). As discussed in Section 5.2.1, additional S-CPT soundings are planned within the MOX and EDG building areas in Fall 2001.

Below the GC layer, the geologic conditions at the MFF site are considered to be the same as described in Section 3.1.1. The deep shear wave velocity profile used for the MFFF building area is the same as that used by Geomatrix 1997 for the APSF site, located approximately 800 feet south of the MFFF building area.

6.2.1.2 Seismic Downhole Survey Data

Seismic downhole surveys were conducted in three boreholes at the MFFF site as discussed in Section 4. The results of the surveys are presented in Attachment 3. As discussed previously, the downhole survey data is considered questionable because of apparently poor coupling between the soil and borehole casings. While relatively good agreement was achieved with the adjacent S-CPT soundings in the upper 50 to 60 feet of BH-5 (S-CPT-19) and BH-10 (S-CPT-34), there was a significant divergence between the data at greater depths. In BH-2, there was no discernible correlation of data with S-CPT-11. Because of the lack of data correlation with S-CPT, and the poor qualities of the boreholes casing grouting the downhole survey data was only used in the MFFF analyses, when it was considered representative of adjacent S-CPT data..

6.2.1.3 Maximum Shear Modulus

The maximum shear modulus (G_{max}) of the soil is used to calculate strain-compatible shear wave velocities as discussed in Section 6.2.2.3. The maximum shear modulus represents the stiffness of the soil at very small strain levels, and is computed as:

$$G_{max} = \rho V_s^2$$

where: ρ = mass density of the soil (slug or pcf/(ft/sec^2)) and V_s = shear wave velocity (ft/sec)

For the site response analyses discussed in this report, G_{max} was calculated using the above equation.

6.2.1.4 Shear Modulus and Damping Variations with Strain

As strains increase in a soil mass, the soil behavior becomes progressively more non-linear with a reduction in shear modulus and an increase in material damping. The reduction in shear modulus is typically normalized with respect to G_{max} and expressed as a normalized (G/G_{max}) modulus reduction versus strain relationship. Both modulus reduction and damping ratio variations with strain are dependent upon material type.

Based on the results of an extensive study of SRS soils, WSRC (1996a has developed a set of recommended modulus reduction and damping curves for the major SRS soil units. Tables 6-7 and 6-8 present the recommended values, which are also shown graphically in Figures 6-4 and 6-5.

Cyclic triaxial and resonant column testing was conducted on a limited number of soil samples from the MFFF site to evaluate the applicability of the WSRC-recommended modulus reduction and damping curves to the site soils. The modulus reduction and damping ratio relationships resulting from these tests are shown in Figures 6-6 through 6-9, along with the WSRC-recommended curves for the same soil units (DCS 2001). From the figures, it is apparent that the modulus reduction values for the MFFF site soils compare quite well with the SRS-recommended values. However, the MFFF damping ratios are considerably higher than those recommended by SRS. Based on evidence presented by Stokoe et. al. (WSRC, 1996a) from an analysis of numerous dynamic laboratory test results, the over-estimation of damping is likely due to the effects of excitation frequency at small strains that occur when using the resonant column test (DCS 2001e). Based on the good agreement between the lab and SRS modulus reduction values, the SRS-recommended modulus reduction and damping values were used in the free-field site response analysis conducted for the MFFF site, as discussed in Section 6.2.2.

6.2.2 One-Dimensional Free-Field Site Response Analyses

6.2.2.1 General

One-dimensional free-field site response analyses were performed for the MFFF site to estimate the response characteristics of the soil column and the dynamic soil properties representative for the small to moderate cyclic strains generated during the design-level earthquake. Three sets of site response analyses were performed as described below:

- In the first, an idealized soil column and associated material properties were developed as described in Section 6.2.2.3. This set of data was analyzed using a modified PC-3 time history as described in Section 6.2.2.2. The strain-compatible dynamic properties discussed in Section 6.2.2.6 were developed from this set of analyses.
- The second set consisted of performing an analysis for each of the 15 S-CPTs using the modified PC-3 time history. The purpose of the second set of analyses was to evaluate the variability of the individual S-CPTs to the composite site response using the idealized soil column and material properties.
- The third set of analyses was the same as the second, but utilized the 1886 Charleston earthquake for the control motion. The purpose of computing the site response utilizing the 1886 Charleston earthquake was to compare liquefaction potential

The strain-compatible dynamic soil properties computed from the first analysis were input into the dynamic soil-structure interaction analyses of the critical MFFF structures. The cyclic stress ratios computed from all three sets of response analyses were input into the liquefaction analyses performed for the MFFF site, as described in Section 8.

6.2.2.2 Control Ground Motion

6.2.2.2.1 Modified PC-3 Motion

This motion is based on the SRS PC-3 uniform hazard rock design response spectrum, and is used as the design basis motion to establish the required bedrock motion necessary to achieve the design PGA of 0.20g at the MFFF site per Section 3.4.9. The spectrum-compatible acceleration time history for this design spectrum was developed by WSRC 1998. The SRS PC-3 rock motion has a peak ground acceleration (PGA) of 0.11 g, and a probability of exceedence of 5 x 10^{-4} (WSRC, 2000). For use as the design basis motion in the site response analyses, the PC-3 rock motion was increased by a factor of 1.25 (PC-3+), yielding a PGA of 0.14 g with an estimated probability of exceedence of 3.2 x 10^{-4} (WSRC, 2000c). The response spectrum for the PC-3+ rock is shown on Figure 6-10 (DCS 2001d).

6.2.2.2.2 1886 Charleston Motion

The second control motion is the 1886 Charleston earthquake (50th percentile) attenuated to rock at the APSF site. The response of the MFFF site to the 1886 Charleston earthquake was conducted to evaluate the liquefaction potential associated with a large, distant event. The spectrum-compatible acceleration time history for this motion was also developed by WSRC (1998), and has a PGA of about 0.05 g. The response spectrum for the 1886 Charleston motion is shown on Figure 6-10 (DCS 2001f).

6.2.2.3 Idealized Soil Column and Soil Properties

The stratigraphy within the MFFF site is reasonably uniform and nearly horizontal, as indicated by the cross-sections through the site presented in Figures 5-2 through 5-10. The idealized horizontally layered soil profile developed for the site response analysis is based on the results of 27 CPTs located within or near the MFFF building and diesel generator building footprints. The CPT data are discussed further in Section 6.2.1.1. The shallow idealized subsurface profile based on CPT results extends to the base the GC layer at about El 135.

As discussed in Section 5, the shallow (above El 135) subsurface conditions at the MFFF site have been established to be similar and consistent with the shallow subsurface geologic conditions reported in other F-Area geotechnical studies (DCS 2001d). Therefore, it is assumed that the engineering properties and shear wave velocities established for the deep subsurface geologic profile within the F-Area are also representative of the MFFF site. The subsurface profile and material properties below El 135 assumed for the APSF site response analyses (Geomatrix 1997) are considered appropriate for use at the MFFF site. The APSF deep soil profile was based on the confirmatory drilling study performed in the central portion of the SRS (Geomatrix 1997, WSRC 1996a). The deep soil profile, assumed to be representative of the MFFF site is presented on Figure 6-1 (DCS 2001d).

Low-strain shear wave velocities developed from the MFFF geotechnical investigations were assigned to the soils down to the Green Clay layer (GC). Shear wave velocities assigned to the deeper materials are the same as those used in the APSF site response analyses (Geomatrix, 1997). The low-strain shear wave velocity profile used for the site response analyses is shown in

Table 6-9 and Figures 6-2 and 6-3. Also shown in Table 6-9 are the total unit weights, compression wave velocities and Poisson's ratios assumed for the analyses (DCS 2001d).

According to the recommendations of ASCE (1998) for safety related nuclear structures, the strain compatible dynamic soil properties used in SSI analyses should be best estimate, lower-bound and upper-bound values. To obtain upper- and lower-bound values of shear wave velocity per the ASCE recommendations, the upper- and lower-bound dynamic shear moduli are assumed to be equal to 1/1.5 and 1.5 times the best-estimate values, respectively. The upper- and lower-bound low-strain shear wave velocity profiles were calculated by calculating maximum shear modulus values from the best estimate shear wave velocities, then applying the 1.5 and 1/1.5 factors to the "best estimate" shear moduli to obtain upper- and lower-bound values. The upper- and lower-bound Vs profiles, shown in Table 6-9 and Figures 6-2 and 6-3, adequately envelope the range of low-strain shear wave velocities measured, and are considered appropriately conservative (DCS 2001e).

The shear modulus reduction and damping ratio versus shear strain relationships used in the analyses are discussed in Section 6.2.1.4, and presented in Tables 6-7 and 6-8 and in Figures 6-4 and 6-5 (DCAS 2001e).

6.2.2.4 Soil Column and Material Properties for Individual S-CPTs

The soil column for each S-CPT down to the GC layer was defined based on the layer picks for each CPT. Not all S-CPTs fully penetrated down to the GC layer, stopping within either the ST1 or ST2 layers. In these cases, the thickness of the un-penetrated layers was taken to be the average layer thickness based on the results of all 63 CPT soundings, similar to the idealized soil column.

The measured interval shear wave velocities from each S-CPT sounding were input directly into the response analyses. In intervals where no velocity data was collected, the average velocity (based on the results of the 15 S-CPTs) for that engineering unit was used. The total unit weights used in the analyses are the average unit weights used in the analysis of the idealized soil column, and are shown in Table 6-9 (DCS 2001d).

6.2.2.5 Response Analyses

The site response analyses were performed using the computer program PROSHAKE, which is essentially a Windows version of the DOS program SHAKE91 (Idriss and Sun, 1992). The control motions described in Section 6.2.2.2 are defined in the analyses at bedrock at the base of the soil column.

6.2.2.5.1 PC-3-Based Control Motion

The surface response spectrum of the PC-3 control motion increased by a factor of 1.25 (PC-3+) for the idealized soil column and best estimate dynamic soil properties, is shown on Figure 6-11. Also shown on this figure are the response spectra for the 15 S-CPT soil columns and the NRC Reg. Guide 1.60 surface spectrum anchored to 0.20 g. Figure 6-12 presents the average response spectrum from the 15 S-CPTs compared to the spectrum for the idealized soil column and best

estimate soil properties. Based on the results presented, the response of the idealized soil column is very close to the average of the 15 S-CPTs (DCS 2001g).

As shown in the figures, the Reg. Guide 1.60 spectrum envelopes the spectrum generated by the PC-3+ in the frequency ranges that are typically of structural interest. As can be seen by these results, the PC-3+ bedrock time history produces a surface PGA of 0.20g and a surface spectrum that correlates well to the Reg. Guide 1.60 surface spectrum. The MFFF PC-3+, therefore, satisfies the requirement for a bedrock time history that can be used for dynamic analysis at the MFFF site, as specified in Section 3.4.4 (DCS 2001g).

6.2.2.5.2 1886 Charleston Control Motion

The responses of the idealized soil column (with best estimate soil properties) and each of the 15 S-CPT soil columns to the 1886 Charleston bedrock motion attenuated to the SRS were also computed. The surface response spectra for the soil columns, along with the Reg. Guide 1.60 surface spectrum, is shown in Figure 6-13. Figure 6-14 presents the average response spectrum from the 15 S-CPTs compared to the spectrum for the idealized soil column and best estimate soil properties. Again, the response of the idealized soil column is very close to the average of the 15 S-CPTS (DCS 2001g).

6.2.2.6 Strain-Compatible Dynamic Soil Properties

Profiles of the best estimate, upper- and lower-bound strain-compatible shear-wave velocity and damping ratio for the PC-3+ are shown in Figures 6-15 and 6-16, as well as in Table 6. These values were computed using the idealized soil column and material properties, and are recommended as appropriate for use in the dynamic soil-structure interaction (SSI) analyses of the MFFF critical structures (DCS 2001g).

6.2.2.7 Cyclic Stress Ratio Profiles

6.2.2.7.1 PC-3-Based Control Motion

As part of the input for the liquefaction analysis of the MFFF site (Section 8.1), a best estimate profile of cyclic stress ratio (CSR) was computed for the PC-3+ based control motion, and is shown in Figure 6-17 and Table 6-10. This profile was developed using the idealized soil column and material properties discussed in Section 6.2.2.3. Also shown in Figure 6-17 are the CSR profiles computed for each of the 15 S-CPTs. It can be seen that the CSR profile for the average soil column compares well with the profiles for the S-CPTs (DCS 2001g).

6.2.2.7.2 1886 Charleston Control Motion

Profiles of CSR for the idealized soil column (with best estimate soil properties) and 15 S-CPTs were also computed utilizing the 1886 Charleston control motion, and are shown in Figure 6-18. Again, the CSR profile computed from the idealized soil column compares well with the individual S-CPT profiles.(DCS 2201g)

7. ENGINEERING EVALUATION

7.1 FOUNDATION DESIGN CRITERIA

The MOX and EDG buildings are QL-1 IROFS structures and will be supported by mat foundations. The main floor level of the MOX building is established at EL 273 msl. The EDG building main floor level is established as El 271 msl. This report and the geotechnical design calculations will be based on MFFF site grade elevations. The MFFF building has basically two foundation floor levels that will be considered for analysis and these will be discussed as the main level at El 273 and a sublevel at El 255.5. The sublevel is for the Aqueous Polishing Area. The mat foundations for the MFFF building will be four (4) feet thick. The mat foundation for the emergency diesel generator building will be two (2) feet thick. The location of these structures is shown on Figure 4-1.

Preliminary foundation design loads (TBV) have been provided for the MOX and EDG buildings and are presented on Table 7-1. The load combination assumed are indicated on the Table 7-1.

7.2 FOUNDATION PREPARATION

The MOX building is a reinforced concrete structure area with a maximum height of approximately 67 ft and a foundation base area approximately 403 ft by 296 ft. The EDG building is also a reinforced concrete structure with a height of approximately 26 ft and a foundation base area approximately 143 ft by 44 ft. The main floor level for these two buildings will be established at a shallow depth, approximately 2.5 ft to 3.5 ft below final grade of El 272. Both of the structures also have very high edge bearing pressures for static and seismic loads.

A review of the subsurface conditions at the MOX and EDG building locations indicates that there is considerable variability in soil strength, in the soils near planned foundation grade, as illustrated by the CPT tip resistance and boring N values on Figures 5-2 through 5-5. Based on the subsurface conditions that exist below planned foundation grades for the MOX and EDG buildings, it has been determined that these variable upper soils will be removed and replaced with a high strength structural fill. The structural fill will provide a firm foundation bearing material for the high static and dynamic loads and will also provide for distribution of potential settlement.

At the present time it is anticipated that a 10 ft of engineered structural fill will be placed beneath the main foundation level for the MOX building. Approximately five (5) ft of engineered structural fill will be placed below the EDG building and sublevel area of the MOX building. Typical fill placement for these structures is shown on Figures 5-2 through 5-5. Preliminary engineering requirements for the engineered structural fill are presented in Section 6.1.6.

The engineered structural fill will extend at least 10ft out from the edge of the mat foundations and will slope outward, down to the bottom of the fill at a slope at least one horizontal to one vertical. The engineered structural fill will be compacted to a density of at least 95 percent of the maximum density a determined by ASTM D-1557. Excavated site materials can be utilized as backfill adjacent to the engineered structural fill and adjacent to the foundations. The site soils

will be placed either as common backfill or as structural fill. Common fill will be placed to at least 90 percent and structural fill will be placed at 95 percent of the maximum density as determined by ASTM-1557.

The subsurface soils will be excavated to planned final grade for engineered structural placement and proof rolled. Prior to placement of the engineered structural fill, any soft areas will be moisture conditioned and recompacted to structural fill requirements or excavated and replaced with engineered structural fill.

7.3 BEARING CAPACITY

Bearing capacity analysis was performed based on the placement of the engineered fill, as described in Section 7.2 and engineering properties presented in Section 6.1.6.(DCS 2001h)

The factor of safety for bearing capacity was determined based on the design loads presented on Table 7-1. The ultimate bearing capacity for the engineered structural fill and the underlying subsurface engineering units was evaluated to determine foundation stability for a deep shear failure. The standard bearing capacity equation used to the evaluate mat foundations is:

$$q_{ult} = cN_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + 1/2\gamma_b BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \qquad \text{DCS (2001h)}$$

The ultimate bearing capacity equation takes into account the size of the mat foundation and the depth of foundation embedment. The ultimate bearing was determined for the main level mat foundations for the MFFF and diesel generator buildings and the sublevel mat foundation for the MFFF building. Maximum edge bearing and average bearing pressures were analyzed for both static and dynamic load conditions. Appropriate bearing capacity factors were utilized per the reference. The factor of safety against the bearing failure is determined by:

$$FS = q_{u\,(net)}/q_b\,(\text{bearing load}) \qquad (\text{DCS2001h})$$

Based on the results ultimate bearing capacity analyses, the engineered structural fill and underlying soil units provide a very high ultimate bearing capacity against failure. The actual bearing loads presented on Table 7-1 have a factor of safety much greater than three (3) for a all load cases, including both static and dynamic. The critical foundation load condition that limits the allowable foundation bearing capacity will be settlement.

7.4 SETTLEMENT

7.4.1 General

Recent settlement analyses at SRS and at the F-Area have been performed by conventional settlement analysis methods using consolidation test results of C_c, C_r and preconsolidation pressure, and by FLAC, a computer software program.(WSRC 1999a, WSCR 1999b, Geomatrix 1998, and WSRC 1996). Discussion of settlement analyses and settlement monitoring results are discussed in these reports.

As stated above, settlement analysis at SRS using conventional consolidation test results generally over estimates settlement. Accurately predicting over consolidation ratios (OCR) is extremely difficult since sample disturbance is prominent with the sandy nature of the subsurface soils. Very few consolidation tests provide realistic preconsolidation pressures, and this was confirmed by the testing performed for this study. (Refer to Section 6.1.5) Settlement analysis using only the recompression index C_r still overestimates actual settlements measured at SRS but generally provides settlement estimates that are closer to the measured settlements at completed structures. The isolated soft zone found at depth do not appear to have much affect on static settlements and this is contributed to arching of the overlying stiffer layers (WSRC 1995a, WSRC 1999b).

Settlement for the MOX and EDG buildings were determined based on removal of unsuitable subsurface soils and replacement with approximately 10 ft or 5 ft of MOX engineered structural fill, as discussed in Section 7.2.. The properties for the structural fill and the subsurface engineering units are presented in Section 6.1.

Two methods of settlement analysis were performed to provide a detailed evaluation of estimated settlements for the MOX and EDG buildings. A conventional settlement analysis was performed using the recompression indices (c_r) determined for the subsurface engineering units. A FLAC analysis was also used to provide a more detailed settlement analysis and deformation profile for the MOX and EDG buildings. Conservative parameters for soft zones and soft materials zones were incorporated into the FLAC models to evaluate their affect on settlement at the structures. The settlement analysis performed using the c_r values was used to confirm the reliability of settlement results obtained from the FLAC model.

The settlement analyses assumes that the existing soil fill at the MFFF site will be removed prior to construction of the MOX and EDG buildings and that rebound will have occurred from this fill loading. No consideration for this fill loading has been assumed in the settlement analyses since any settlement from the fill was considered primarily recompression.

7.4.2 Settlement Determined By Recompression Indices

A conventional settlement analysis was performed using only the c_r values presented for each engineering unit in Section 6.1. The physical properties for the soils in each engineering unit used in the analysis are also the same as presented in Section 6.1. Settlement analysis was performed for the center and at the edge of the mat for the MOX building. The subsurface conditions used for this settlement analysis are typical to the subsurface profiles at CPT-54 and CPT-55, as shown on Figure 5-2.

The profile used for the mat evaluation assumed 10 ft of structural fill beneath the main level mat. The Boussinesq approach to define the vertical distribution of stresses at the center of the mat and at the center edge of the mat did not take into account the sublevel shown on Section 1. The soft material zone shown on the profile for CPT-54 was not evaluated separately since in is averaged into the properties for the engineering unit. The results of this analysis have also been used to evaluate settlement results obtained from the FLAC analysis.

DCS Calculation Number DCS01-WRS-DS-CAL-G-00019-A, "Determination of Consolidation Settlement and Recompression Indices" (DCS 2001b) provides the details for this settlement analysis. The results of the settlement analysis indicate that the estimated settlement for the main level mat in the MOX building is:

Settlement at Edge of mat	1.4 inches
Settlement at center mat	2.5 inches

These settlement results are considered to be conservative based on the assumptions used for the analysis and the discussion presented in Section 7.4.1.

7.4.2 FLAC Settlement Analysis

A settlement analysis was performed for the MOX and EDG buildings using FLAC A.D. DCS Calculation Number DCSOQ-WRS-DS-CAL-G-OOO17-A (DCS 2001 i)provides the details of this analysis.

Detailed subsurface models were made beneath the MOX and EDG buildings for this settlement analysis. The geotechnical sections presented on Figures 5-2, 5-3 and 5-5 were used to develop these detailed model sections identified as Section 1, Section 2 and Section 4, respectively. The model section developed from the section in Figure 5-3 includes the emergency diesel generator building. The model section developed from the section in Figure 5-5 includes the sublevel (Aqueous Polishing Area) of the MOX building. The model sections extended at least 150 ft outside the edge of the buildings and to a depth of 200 feet, well into the very dense Congree unit, to minimize model boundary effects. All soft zones and soft material locations are incorporated into the model sections, as shown on the figures. The FLAC models have the same engineering layering as shown on their respective geotechnical cross-section.

Soil properties for the model sections were determined based on the properties presented in Section 6.1. Parameters used for the Best Estimate and Lower Estimate fill engineering unit properties are presented an Figures 7-2 and 7-3. The shear wave velocities (V_s) and poissons ratio (ν) are presented in Tables 7-2 and 7-3 for each engineering unit and were used to determine the shear modulus (G) and bulk modulus (K) for input into the model. The shear modulus G for input into the model has been reduced to 15 percent of G_{max} to account for the strain loading expected for settlement and seismic loading. (G_{max}) is determined from the shear wave velocity using:

$$G = .15\ G_{max} = .15\ \rho V_s$$

Bulk modulus K was determined from G and ν using:

$$K = \frac{2G(1+\nu)}{3(1-2\nu)}$$

Soft zone properties were established using engineering properties for a very soft material.(DCS 2001c) The soft zone properties were also used for the soft material zones identified on the cross sections. The soft zones properties were considered to be lower bound material properties and were not varied in the FLAC analysis runs.

FLAC analysis was performed for all three model sections using the a Best Estimate shear wave velocity and Lower Bound shear wave velocity presented in Tables 7-2 and 7-3. Section 4 is considered the most sensitive section to differential settlement since it contains the sublevel and main level foundations for the MFFF building.

The results of the settlement displacements from the FLAC analysis for Sections 1, 2, and 3 are presented on Tables 7-4, 7-5 and 7-6, respectively. The displacement contour results for the foundation and subsurface are shown graphically on Figures 7-1 through 7-9.

The results of the conventional settlement analysis performed in Section 7.4.2 is consistent with the FLAC Best Estimate settlement results presented Tables 7-4, 7-5, and 7-6. Since the conventional settlement analysis is still considered to be a conservative estimate, the Best Estimate settlement results are considered to be the most appropriate values for evaluating the MOX and EDG building structure performance. The similarities between the conventional settlement analysis and the FLAC analysis provide confirmation that the input parameters being used in FLAC are appropriate and conservative. This modeled behavior is consistent with the settlement observed for structures and tanks at SRS. (WSRC 1999b).

The Best Estimate settlement results are recommended to evaluate MOX and EDG structures for total and differential settlement effects during structure design.

7.5 SOFT ZONE SETTLEMENT

The influence of the soft zones can be seen by comparing Figures 7-2, 7-3, 7-5, 7-6, 7-8, and 7-9. The deformation contours presented in these figures support engineering judgment that the upper, stiff engineering units redistribute the settlement and stresses from lower layers so that localized differential settlements are not observed at the foundation level. The contours do provide an indication that the soft zones may appear to contribute to additional surface settlement directly above the zone. This difference is so slight that it is hardly detectable at the scale the model sections are drawn. The settlements from soft zones at depth do not result in adverse differential settlement at the foundation grade level. The soft zones identified beneath and adjacent to the MOX and EDG buildings will not have any adverse affect to these structures.

8. STABILITY OF SUBSURFACE MATERIALS

8.1 LIQUEFACTION

8.1.1 Methodology

8.1.1.1 Overall Approach

The liquefaction potential of the MFFF site within the proximity of the MOX and EDG buildings was evaluated using the Cyclic Stress Approach as described in the Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (NCEER, 1997). The approach as described by NCEER is well suited for utilizing both SPT N-values and CPT tip resistances for estimating the cyclic resistance of soils.

The Cyclic Stress Approach characterizes earthquake loading by the amplitude of an equivalent uniform cyclic stress, and liquefaction resistance by the amplitude of the uniform cyclic stress required to produce liquefaction in the same number of cycles. The liquefaction potential is then evaluated by comparing the earthquake demand (cyclic stress ratio or CSR) with the liquefaction resistance (cyclic resistance ratio or CRR) throughout the soil profile. If the CSR is close to, or greater than the CRR, liquefaction is possible. For the MFFF analysis, full liquefaction is considered to have been triggered when the factor of safety against liquefaction (FS_L = CRR/SCR) is 1.1 or less. For factors of safety of 1.1 to 1.4, it is possible for excess pore pressures to build up in the soil, thus causing a reduction in strength and stiffness, which may result in soil settlement.

It should be noted that the Cyclic Stress Approach does not take into account the cohesiveness of a soil. It is entirely possible that a cyclic stress analysis would predict that a soil deposit is liquefiable, when such an occurrence would be highly unlikely or impossible due to the cohesive nature of the soil. In the MFFF analysis, the liquefaction potential of cohesive soils was evaluated using the following criteria:

- cohesive soils having a fines content (-No. 200 material) greater than 30 percent and fines that either classify as clays or have a Plasticity Index greater than 30 percent are not generally considered liquefiable; and

- soils with a clay content greater than 15 percent and a liquid limit greater than 35 percent, occurring at a natural water content lower than 90 percent of the Liquid Limit are considered non-liquefiable. (Wang, 1979)

8.1.1.2 Seismic Demand

The CSRs for the MFFF site were computed utilizing two bedrock control motions and an idealized soil column for the MOX building area, as well as soil columns for each of the 15 S-CPTs conducted at the site. The computation of the CSRs is described in Section 6.2. The PC-3 motion increased by a factor of 1.25 represents a near-field event and is conservatively assumed to have a M_w of 6.0. The 1886 Charleston motion represents a larger, more distant event with a

M_w of 7.3 (WSRC, 2000a). A groundwater depth of 60 feet (El 210 feet) was used to compute all of the CSR profiles.(DCS 2001g) This ground water depth is the MFFF site design level and is considered conservative. The groundwater level across the MFFF site presently averages over 70 feet deep. Section 5.3.1 discusses the groundwater conditions at the MFFF site in detail.

The purpose of computing CSR profiles for both an idealized soil column and the 15 S-CPTs is to compare the computed liquefaction potential (and resulting settlement, if any) for the generalized subsurface condition to those for specific, known points. All of the S-CPTs except two (S-CPT-8 and S-CPT-13) are located east of the MFFF building area.

Before using the CSRs to compute factors of safety against liquefaction, the CSR must be normalized for earthquake magnitude. For the M_w 6.0 PC-3-based motion, a magnitude scaling factor (MSF) of 2.0 was used to compute the normalized CSRs. For the M_w 7.3 motion, a MSF of 1.1 was utilized. These MSFs represent the middle of the range of MSFs recommended by the NCEER workshop (NCEER, 1997) and are considered conservative for the MFFF site.

8.1.1.3 Determination of Soil Capacity

8.1.1.3.1 Cone Penetration Tests

The results of 37 CPT soundings located across the MFFF site were used to evaluate the liquefaction potential in the area of the MOX and EDG buildings. Of the 37 CPTs used, 24 are located within or very close to the proposed structure footprints of the MOX and EDG buildings, and the remaining 13 are located to the east. The CRR was estimated directly from CPT data utilizing the methodology outlined by Robertson and Wride (1997) in the NCEER workshop. A key parameter in the CPT liquefaction analysis is the correction of tip resistance for fines content. A relatively good correlation between the CPT-estimated fines contents and laboratory data at the MFFF site was obtained using site-specific equations developed for use at SRS (WSRC, 2000c). The CRR profile below the groundwater table was computed for each of the 37 CPTs, without regard to fines content or cohesiveness of the soils.(DCS 2001j and 2001k)

8.1.1.3.2 Standard Penetration Tests

The results of 11 exploratory borings located across the MFFF site were also used to evaluate liquefaction potential. Six of the borings are located within or near the proposed structure footprints for the MOX and EDG buildings. The remaining five are located east of the footprints. In most cases, the borings are located immediately adjacent to a CPT sounding. CRRs were computed directly from the corrected and normalized standard penetration test blowcounts obtained in the borings.

8.1.2 Results

8.1.2.2 Cone Penetration Tests

8.1.2.2.1 Idealized Soil Column and PC-3+ based Control Motion

As discussed in Sections 6.2.2.1 and 8.1.1.2, CSRs were generated for two control motions and utilizing both an idealized soil column and soil columns from individual S-CPTs. The FS_Ls for the 24 CPTs located within or near the proposed structure footprints of the MOX and EDG buildings, computed with CSRs from the idealized soil column and PC-3-based control motion, are presented in Figures 8-1 through 8-24. The FS_L appear clipped because the factors of safety are plotted only to the nearest tenth. None of the 24 CPTs evaluated under these conditions yield liquefaction factors of safety below 1.5, indicating that neither liquefaction nor excess pore pressure buildup will occur.

8.1.2.2.2 Soil Columns by S-CPT and PC-3+ based Control Motion

The FS_Ls for the 15 S-CPTs, computed with CSRs calculated by individual S-CPT and the PC-3-based control motion, are presented in Figures 8-25 through 8-39. All FS_Ls are essentially 1.5 or greater. S-CPT-8 and S-CPT-13 each contain one interval of less than one foot with a FS_L of 1.4. (DCS 2001 L)

8.1.2.2.3 Soil Columns by S-CPT and 1886 Charleston Motion

The FS_Ls for the 15 S-CPTs, computed with CSRs calculated by individual S-CPT and the 1886 Charleston control motion, are presented in Figures 8-40 through 8-54. Of the 15 S-CPTs, five have minimum FS_Ls of 1.5 or greater. Of the remaining 10, five have FS_Ls of at least 1.4, and five have FS_Ls of at least 1.3. In all cases, the FS_Ls below 1.5 occurred mainly within the DB 4/5 and ST2 Formations, although some also occurred within siltier zones of the "Green Clay" layer.(DCS 2001 L)

8.1.2.3 Standard Penetration Tests

8.1.2.3.1 Idealized Soil Column and PC-3+ based Control Motion

The FS_Ls based on SPT data from the six borings located within or near the proposed structure footprints of the MOX and diesel generator buildings, computed with CSRs from the idealized soil column and PC-3-based control motion, are presented in Figures 8-2, 8-8, 8-21 and 8-25 through 8-27. As can be seen from the figures, four individual SPT blowcounts in three borings indicate factors of safety of less than 1.5 as discussed below: (DCS 2001 j)

<u>BH-11</u>

There were two SPT blowcounts yielding factors of safety of less than 1.5 in boring BH-11. The first one was at El 205 with a $(N_1)_{60}$ of 4, and a FS_L of 1.2. This sample was taken from within the DB1/3 layer, about five feet below the assumed groundwater level. The sample had a fines content (-No. 200 material) of 12 percent and was classified as a SP-SM. While this material may not fully liquefy, it could experience significant strength loss due to excess pore pressure generation.

The second sample was from El 142 and was identified as coming from the "Green Clay" layer. The sample had a $(N_1)_{60}$ of 2 and a FS_L of 1.3. The fines content was 65 percent (14 percent clay-size) and the material classified as an ML (LL = 39, PI = 11). The "Green Clay" layer at the SRS is not generally considered liquefiable. In both cases, the material within five feet above and below the samples had blowcounts resulting in factors of safety of greater than 1.5.

BH-12

There was one sample at El 188 with a $(N_1)_{60}$ of 4, resulting in a FS_L of 1.1. This sample was from within the DB4/5 layer, and classified as a SM with a fines content of 23 percent (20 percent clay-size). The sample was highly plastic (LL = 109, PI = 42), and combined with a clay fraction of 20 percent, liquefaction of this material is highly unlikely. The samples within five feet above and below this sample had factors of safety greater than 2.5.

BH-13

There was one sample at El 182 with a $(N_1)_{60}$ of 3 resulting in a FS_L of 1.1. This sample was from the DB4/5 layer. The sample was classified in the field as a ML, however, no laboratory tests were conducted on this sample. This material is assumed to be fully liquefiable. The samples within five feet above and below this sample had a FS_L greater than 6.

8.1.2.3.2 Soil Columns by S-CPT and PC-3+ based Control Motion

The FS_Ls based on SPT data from five borings located adjacent to S-CPTs, computed with CSRs from the idealized soil column and PC-3-based control motion are presented in Figures 8-32, 8-35, 8-36, 8-39 and 8-40. As shown, five SPT blowcounts in two borings indicate factors of safety of less than 1.5 as discussed below: (DCS 2001 l)

BH-2

There was one sample at about El 191 with a $(N_1)_{60}$ of 4 resulting in a FS_L of 1.3. This sample is from the DB4/5 layer, and classified as a SM with a fines content of 16 percent. The samples within five feet above and below this sample had factors of safety greater than 4.

BH-5

There were four SPT blowcounts yielding factors of safety of less than 1.5 in boring BH-5. The first one was at El 192 with a $(N_1)_{60}$ of 4, and a FS_L of 1.0. This sample was taken from within the DB1/3 layer, and classified as an SM with a fines content of 9 percent. This material is assumed to be fully liquefiable. The blowcounts within five feet above and below this sample were greater than 30.

The second sample was from El 182 and was taken from the DB 4/5 Formation. The sample had a $(N_1)_{60}$ value of 3 and a FS_L of 1.2. The fines content was 16 percent (11 percent clay) and the material classified as an SC (LL = 60, PI = 33). The highly plastic nature of the fines probably contributed to the low blowcount, and would probably limit the liquefaction potential of this

material. The material within five feet above and below the samples had blowcounts resulting in a FS_L of greater than 3.

The third and forth samples had a $(N_1)_{60}$ value of 4 and a FS_L of 1.3, and were collected from within the ST2 Formation. The third sample classified as an SC (LL = 61, PI = 31) with a clay content of 21 percent and a natural moisture content of 35 percent. Based on Wang's (1979) criteria, this sample is considered non-liquefiable. The forth sample classified as a CH and is non-liquefiable.

8.1.2.3.3 Soil Columns by S-CPT and 1886 Charleston Motion

The FS_Ls based on SPT data from seven borings located adjacent to S-CPTs, computed with CSRs from the idealized soil column and 1886 Charleston control motion are presented in Figures 8-47, 8-50, 8-51, 8-54 and 8-55. (DCS 2001 l) As shown, six SPT blowcounts in three borings indicate factors of safety of less than 1.5. The first five samples are the same and had FS_Ls less than 1.5 utilizing the PC-3-based motion described above. Of the potentially liquefiable samples, the one in BH-2 has a computed FS_L of 1.1, and two in BH-5 have FS_Ls of 0.9 and 1.1. These materials are assumed to be fully liquefiable, but have materials within five feet above and below with FS_Ls well above 2.

The sixth sample was collected from BH-9 within the ST2 Formation, and had a $(N_1)_{60}$ value of 7 and FS_L of 1.4. This sample was classified in the field as a SM and has an estimated fines content of 13 percent based on the adjacent CPT sounding. The samples within five feet above and below this sample had a FS_L of at least 2.8.

8.1.3 Conclusions

The results of the liquefaction analyses discussed above indicate that the liquefaction potential of the MFFF site is quite low. Using the PC-3 control motion increased by a factor of 1.25, (-PC-3+) all of the CPT-based factors of safety are at least 1.5, suggesting that not only is liquefaction not triggered, but that generation of excess pore pressures is unlikely. This result is verified utilizing both the idealized soil column, and by individual S-CPT results. The SPT-based results assuming the PC-3-based control motion also indicate very low liquefaction potential, with only two samples out of 11 borings indicating that liquefaction would be triggered. And only five samples indicating possible generation of excess pore pressures.

The results of the CPT-based analyses utilizing a control motion based on the 1886 Charleston earthquake indicate that while some generation of excess pore pressures may occur, triggering of liquefaction will not occur. The SPT-based analyses indicate that liquefaction may be triggered in very limited zones (three samples out of five borings). Overall, it appears that a larger, more distant event like the 1886 Charleston earthquake will control liquefaction potential at the MFFF site. However, widespread liquefaction is not predicted to occur. All liquefaction that may potentially occur will be in isolated zones at depth that are surrounded by non-liquefiable materials.

It should also be noted that the effect of soil aging at the MFFF site was neglected to be conservative in our analysis. The Cyclic Stress Approach was developed for evaluating liquefaction potential in Holocene soils, and does not account for the effects of aging. It is well documented that most liquefaction risk is associated with Holocene-age deposits. The potentially liquefiable soils (Tobacco Road, Dry Branch and Santee Formations) at the SRS site are Tertiary-aged, and have been found to have significantly higher cyclic shear strengths than similar soils of the Holocene period (WSRC, 1995).

Another conservatism in the analyses is that in the case of the CPT-based results, the cohesiveness of the soils was neglected. The results of laboratory tests on site soils indicate a significant presence of plastic, fine-grained material. As discussed above, several SPT samples that had $(N_1)_{60}$ values indicating potential liquefaction or excess pore pressure generation are considered non-liquefiable due to the quantity and plasticity of fine-grained soil.

8.2 POST-EARTHQUAKE DYNAMIC SETTLEMENT

The post-earthquake dynamic settlement of the potentially liquefiable layers was estimated using the Ishihara/Yoshimine (1990) method. In this method, the post-liquefaction volumetric strain is computed as a function of the factor of safety against liquefaction and soil density. The settlement of a layer is computed by multiplying the volumetric strain by the layer thickness. With this method, some settlement, however small, is assumed to occur up to a factor of safety against liquefaction of 2.0. It is generally accepted that excess pore pressure generation is not significant above a factor of safety of 1.5.

Post-earthquake settlements were calculated for both the idealized soil column and S-CPT soil columns, and utilizing both the PC-3+ and 1886 Charleston control motions. The settlement from the SPT blow counts and CPT tip resistance was computed for those layers judged to be potentially liquefiable. The settlement from the CPT values was computed for all CPT tip resistances with an estimated fines content of 30 percent or less, without considering the cohesiveness of the soil. For the SPT samples, the thickness of the potentially liquefiable layer is assumed to extend halfway between the sample in question, and the next samples above and below that indicate little or no liquefaction potential.

8.2.1 Correction for settlement in Tinker/Santee Unit

As discussed in earlier Sections, not all of the SPTs penetrated through the Tinker/Santee Formation, engineering units ST-1 and ST-2. All of the exploration boring penetrated through the ST-1 and ST-2 units.

To account for potential post-earthquake settlement that might occur below CPTs, which achieved penetration resistance prior to the ST-1 and/or ST-2 units, the average post-earthquake settlement of these engineering units was determined. The calculated average liquefaction induced volumetric strain and the calculated average thickness of units ST-1 and ST-1 were established. The average post earthquake settlement was then determined for each unit to establish an estimated post-earthquake settlement that needed to be added to the CPT analysis, for CPTs did not penetrate the ST-1 and/or ST-2 units. This estimated post-earthquake settlement in the ST-1 and ST-2 units was determined for both the PC-3+ and Charleston events.

Material properties at the MFFF site were also taken into account for the analysis of these two units. The results of this analysis are presented in Table 8-1. (DCS 2001 m)

The results of the analysis for estimated post-earthquake induced settlements for units ST-1 and ST-2 were added to the post-earthquake settlement determined for CPTS, which stopped above the ST-1 and/or ST-2 layers.

8.2.2 Results

The total computed settlements based on CPT and SPT N value blow counts are summarized in Tables 8-2 through 8-4. These settlement values incorporate estimated post-earthquake settlement for ST-1 and/or ST-2, when appropriate to the CPT. The Tables also indicates which CPT's required this settlement adjustment. As shown on Table 8-2, the computed post-earthquake settlement utilizing the idealized soil column and PC-3+ ranges from nil to 0.8 inches, with an overall average of 0.5 inches for 24 CPTs and six borings. Utilizing the soil columns for the 15 S-CPTs and seven adjacent borings for the PC-3+ and Charleston motion, the computed settlement ranges from nil to 1.7 inches, with an average of 0.4 inches. The 1886 Charleston control motion for the 15-S-CPTs and seven borings, has settlements ranging from nil to 1.9 inches, with an average of 0.9 inches.(DCS 2001j, 2001k, and 2001h)

8.2.3 Conclusions

Conservatively assuming that some post-earthquake dynamic settlement is possible up to a FS_L of 2.0, an average of 0.5 inches of potential settlement is expected to occur as a result of the PC-3+ earthquake. The conservative estimate of post earthquake settlement for the 1886 Charleston event for a FS_L of 2.0 will have an average of approximately one inch and is the controlling event. The average potential post-earthquake settlement is predicted to increase to approximately one inch across the MFFF site assuming an earthquake similar to the 1886 Charleston event.

The post earthquake dynamic settlement estimated for the Charleston event for each CPT and boring in Table 8-2 and 8-3 has been plotted at the respective CPT and boring location on Figure 8-58. This recorded settlement represents post-earthquake dynamic settlement that may occur in loose or soft strata below the groundwater level, at a depth of 60 feet or more. These values do not represent total or differential settlements that would occur at the ground surface or foundation level.

TR2B, TR2A and the structural fill provide for a significant stiff soil layer (over 40 feet thick) between the deep potentially liquefiable zone and the mat foundations for the MOX and EDG buildings (refer to Figures 5-2 through 5-5). These stiff soil units will successfully redistribute these estimated post earthquake dynamic settlements such that no abrupt differential settlement will occur at the foundation base elevation. The post earthquake dynamic settlements will not result in any adverse settlement to the MOX or EDG buildings. This distribution of settlement at depth to the MOX and EDG foundation levels has been demonstrated by the FLAC analysis in Section 7.4.2. The effects of settlement distribution from soft zones would be analogous to post-earthquake settlement from potential liquefiable pockets at similar depths. The post earthquake settlement analysis indicates that post earthquake dynamic settlement does not create any subsurface stability problems

8.3 SOFT ZONE SETTLEMENT

Soft zone conditions identified at the MFFF site are discussed in Section 5.2. Any soft zones identified within the influence of critical structure foundation systems have been evaluated for both static and dynamic loading conditions. Settlement from static loading is discussed in Section 6.1.5 and does not result in any adverse total or differential settlement. The soft zones were evaluated in the liquefaction and post earthquake dynamic settlement evaluation. The results are discussed in Sections 8.1 and 8.2. The soft zones are isolated with respect to static and dynamic loads and, therefore, are considered stable and are not anticipated to cause any adverse affect to the foundations for the MOX and EDG buildings. The soft zones do not present any subsurface stability problems since they are limited in lateral and vertical extent and are surrounded by competent, non-liqueficiable materials..

8.4 FAULTING

Studies at SRS have indicated that identified faults at or near SRS are not capable, therefore, faults do not present a subsurface stability problem for the MFFF site. Refer to Section 3.1.2 for more detail.

8.5 SLOPE STABILITY

There are no permanent slopes on the MFFF site, therefore, there are no slope instabilities.

9. CONCLUSIONS

The MFFF Site Geotechnical Program and analyses performed indicate that this site is suitable for design and construction of the MOX and EDG buildings. The subsurface conditions encountered at the MFFF site demonstrate consistency with subsurface conditions found throughout the adjacent F-Area. No unusual subsurface conditions have been identified. All soft zones and loose soil deposits identified at depth are consistent with those identified in the adjacent F-Area. All of the soft zones and loose soil zones identified at the MFFF site appear to be deep, isolated and have a limited lateral extent. No unusual soft conditions were found at the location of the MOX and EDG buildings.

The exploration and testing program performed for the site is considered adequate to define subsurface conditions and to establish the geotechnical design criteria required for the MFFF. All subsurface conditions identified during this geotechnical program indicate that the underlying geology at the MFFF site is consistent with conditions described in WSRC 2000a. The regional and SRS specific hydrology, geology and seismology descriptions presented in Sections 1.4.2, 1.4.4, and 1.4.4 of WSRC (2000a), respectively, are applicable for use at the MFFF site.

An assessment of the subsurface conditions indicates that an engineered structural fill is required beneath the MFFF and emergency diesel generator buildings. An evaluation of the variable subsurface soils at foundation grade indicate that approximately 5 to 10 ft of unsuitable materials will require removal. Approximately 10 ft of engineered structural fill will be required beneath the MOX building. The EDG building will required approximately 5 feet of structured fill. This structural fill will be sufficient to adequate support for the high edge bearing pressure anticipated during seismic loading. Also this structural fill will provide differential settlement control to the structure foundations. Settlement analysis included soft zones and the results indicate that the MOX and EDG buildings will not experience any detrimental differential settlements. The total and differential settlements estimated are considered within the limits that these structures can tolerate. Further structural analysis will be performed during design to accommodate these anticipated static and dynamic settlements.

Dynamic analysis indicates that there are no detrimental liquefaction or dynamic settlements that will result from the design level earthquakes, such as the PC-3+ and 1886 Charleston event.

The present location for the MFFF and emergency diesel generator buildings is considered appropriate for design and construction of the planned facilities.

10. REFERENCES

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TABLES

TABLE 5-1

CORRELATION OF ENGINEERING AND GEOLOGIC STRATIGRAPHY UNITS FOR MFFF SITE

Engineering Unit	Geologic Unit
TR1 and TR1A	"Upland Unit" Formation
TR2A and TR2B	Tobacco Road Formation
TR3/4 and DB1/3	Dry Branch Formation
DB4/5. ST1 and ST2	Tinker/Santee Formation
GC Layer	Warley Hill Formation
CG Layer	Congaree Formation

TABLE 5-2
AVERAGE SOIL INDEX PROPERTIES FOR MFFF, APSF, AND F-AREA NORTHEAST EXPANSION

	ENGR UNIT	AVE. THICK.[1] (ft)	AVE TOP EL.[1] (MSL)	AVE SPT N VALUE	AVE q_{cor} (tsf)	AVE PI (%)	LIQUID LIMIT (%)	MOISTURE CONTENT (%)	AVE - NO. 200 (%)
F-Area	TR1 [2]	25		25	91	17	38	15	33
NE Exp.		13	291	31	95	23	48	18	34
APSF		16	289	33	142	11	30	16	25
MOX FFF		9	276	17	92				
F-Area	TR1A [2]	19	278	25	120	14	36	19	30
NE Exp.		16.0	278	31	103	20	35	19	30
APSF		14	273	27	68	22	46	20	37
MOX FFF		12	267	18	103	16	39	14	30
F-Area	TR2A	25	261	28	147	10	33	17	17
NE Exp.		26	262	37	146	9	28	17	14
APSF		27	260	34	136	10	33	21	16
MOX FFF		20	255	22	129	25	50	16	17
F-Area	TR2B	19	233	36	201	18	41	22	19
NE Exp.		23	236	39	164	12	24	18	10
APSF		22	233	38	154	NP	NP	24	11
MOX FFF		23	235	28	140			16	10
F-Area	TR3/4	10	213	18	55	58	96	51	64
NE Exp.		7	213	27	73	19	54	34	36
APSF		8	211	19	37	19	54	42	34
MOX FFF		7	212	18	41	47	82	35	37
F-Area	DB1/3	28	204	33	172	19	44	27	14
NE Exp.		28	206	37	194	16	11	25	11
APSF		28	203	50	166	NP	NP	27	9
MOX FFF		22	205	30	120	39	69	29	14
F-Area	DB4/5	7	175	28	61	15	48	39	22
NE Exp.		6	178	29	67	11	45	36	20
APSF		7	175	21	52	11	45	38	21
MOX FFF		9	183	19	35	35	67	37	26
F-Area	ST1	19	167	47	131	18	40	29	29
NE Exp.		20	172	43	138	14	23	30	19
APSF			168	46	137	25	49	30	18
MOX FFF		17	174	47	200			26	9
F-Area	ST2								
NE Exp.		11	152						
APSF									
MOX FFF		11	157	20	44	27	53	33	32
F-Area	GC	7	138	21	58	47	83	32	39
NE Exp.		7	141	39	97	27	42	32	33
APSF		9	143	49	79	30	57	28	52
MOX FFF		5	146	32	63	30	55	29	47
F-Area	CG								
NE Exp.			134						
APSF			134						
MOX FFF			141	89	213	27	50	24	14

[1] NE expansion values include APSF data.
[2] Surface effects have not been accounted for.

TABLE 5-3

SOFT ZONE SUMMARY

CPT No.	Top Elev.	Bottom Elev.	Approx. Thick. (ft)
2	166.3	158.0	8.3
10	147.3	145.1	2.2
12	185.5	181.2	4.3
18	174.0	171.6	2.3
26	156.9	147.9	9.0
30	152.7	148.4	4.3
32	184.4	179.5	4.9
37	157.5	144.6	12.9
	180.7	177.7	2.9
38	182.2	177.3	4.9
39	158.8	152.8	6.0
	151.3	146.8	4.6
45	187.2	182.1	5.1
	151.0	147.6	3.4
46	182.8	179.7	3.1
	155.9	149.2	6.7
50	184.6	180.9	3.8
55	190.1	186.3	3.8
61	185.3	173.9	11.3
Boring No.	Top Elev.	Bottom Elev.	Approx. Thick. (ft)
BH-3	156	153.5	2.5
BH-5	193	191	2
	183	181	2
BH-6	184	182	2
	161	157	4
BH-13	182	180	2

1. In CPT soundings, a soft zone is defined as a zone with a CPT corrected tip stress of less than 15 tsf over a continuous interval of at least two feet.
2. In boreholes, a soft zone is defined as a zone with SPT N-value of 5 or less over a continuous interval of at least two feet.
3. If two or more soft zones are separated by less than one foot of firmer material, they are treated as one layer.
4. Soft Zones fall within the lower Santee/Tinker (ST2) or lower Dry Branch (DB4/5) formations.

TABLE 6-1
AVERAGE CPT AND SPT TEST RESULTS

	ENGR UNIT	AVE TOP EL.[1] (MSL)	AVE SPT N VALUE	AVE q_{cor}/N VALUE	AVE f_s (tsf)	AVE q_{cor} (tsf)	AVE R_f [2] (%)	AVE Pore Press. (tsf)	SHEAR WAVE VELOCITY (ft/sec)
F-Area	TR1 [3]		25	3.6		91			
NE Exp.		291	31	3.1		95	3.0		1544
APSF		289	33	4.3		142	2.0		1637
MOX FFF		276	17	5.4	1.6	92	1.6	0.39	1541
F-Area	TR1A [3]	278	25	4.8		120			
NE Exp.		278	31	3.3		103	3.0		1454
APSF		273	27	2.5		68	4.0		1464
MOX FFF		267	18	5.7	2.4	103	2.5	0.64	1476
F-Area	TR2A	261	28	5.3		147			
NE Exp.		262	37	3.9		146	1.0		1257
APSF		260	34	4.0		136	1.0		1284
MOX FFF		255	22	5.9	1.3	129	1.1	0.09	1324
F-Area	TR2B	233	36	5.6		201			
NE Exp.		236	39	4.2		164	1.0		1165
APSF		233	38	4.1		154	1.0		1215
MOX FFF		235	28	5.0	1.1	140	0.9	0.10	1253
F-Area	TR3/4	213	18	3.1		55			
NE Exp.		213	27	2.7		73	2.0		1056
APSF		211	19	1.9		37	2.0		1020
MOX FFF		212	18	2.4	1.0	41	3.0	3.08	1016
F-Area	DB1/3	204	33	5.2		172			
NE Exp.		206	37	5.2		194	1.0		1176
APSF		203	50	3.3		166	1.0		1197
MOX FFF		205	30	4.0	0.9	120	1.1	0.36	1126
F-Area	DB4/5	175	28	2.2		61			
NE Exp.		178	29	2.3		67	2.0		1180
APSF		175	21	2.5		52	2.0		1231
MOX FFF		183	19	1.8	1.4	35	2.5	5.25	1104
F-Area	ST1[4]	167	47	2.8		131			
NE Exp.		172	43	3.2		138	1.0		1273
APSF		168	46	3.0		137	1.0		1223
MOX FFF		174	47	4.2	2.0	200	1.2	1.39	1129
F-Area	ST2[4]								
NE Exp.		152							
APSF									
MOX FFF		157	20	2.2	0.8	44	2.0	9.18	1068
F-Area	GC	138	21	2.8		58			
NE Exp.		141	39	2.5		97	2.0		1319
APSF		143	49	1.6		79	2.0		1160
MOX FFF		146	32	2.0	1.3	63	2.1	14.21	1217
F-Area	CG								
NE Exp.		134							
APSF		134							
MOX FFF		141	89	2.4	2.0	213	1.5	6.65	

[1] NE Expansion values include APSF data.
[2] Friction Ratio = sleeve(f_s)/q_{COR} ratio.
[3] Surface effects have not been accounted for.
[4] The Northeast Expansion report does not separate ST1 and ST2.

TABLE 6-2
AVERAGE CLASSIFICATION AND PHYSICAL TEST RESULTS

	ENGR UNIT	AVE. THICK.[1] (ft)	AVE PI (%)	LIQUID LIMIT (%)	FINES CLASS.	MOISTURE CONTENT (%)	AVE % SAND (%)	AVE - NO. 200 (%)	AVE % SILT (%)	AVE % CLAY (%)	D_{50} Mean Grain Size (mm)	WET DENSITY (pcf)	DRY DENSITY (pcf)
F-Area	TR1 [2]	25	17	38		15	67	33		18.3	0.23	122	106
NE Exp.		13	23	48		18	66	34					
APSF		16	11	30		16	75	25					
MOX FFF		9									-		
F-Area	TR1A [2]	19	14	36		19	70	30		32.5	0.12	114	96
NE Exp.		16.0	20	35		19	70	30					
APSF		14	22	46		20	63	37				123	101
MOX FFF		12	16	39	CL	14	70	30	17.9	23.0	0.15		
F-Area	TR2A	25	10	33		17	83	17		10.1	0.27	122	101
NE Exp.		26	9	28		17	86	14					
APSF		27	10	33		21	84	16					
MOX FFF		20	25	50	CL-CH	16	83	17	3.9	14.4	0.33	120	100
F-Area	TR2B	19	18	41		22	81	19		8.3	0.36	123	99
NE Exp.		23	12	24		18	90	10					
APSF		22	NP	NP		24	89	11				124	102
MOX FFF		23				16	90	10			0.37	118	103
F-Area	TR3/4	10	58	96		51	36	64		39.6		108	76
NE Exp.		7	19	54		34	64	36					
APSF		8	19	54		42	66	34				115	89
MOX FFF		7	47	82	CH	35	63	37	8.6	28.7	0.23	115	89
F-Area	DB1/3	28	19	44		27	86	14		12.1	0.34	124	99
NE Exp.		28	16	11		25	89	11					
APSF		28	NP	NP		27	91	9				122	98
MOX FFF		22	39	69	CH	29	86	14	5	15.6	0.37	119	95
F-Area	DB4/5	7	15	48		39	78	22		20.1	0.29	118	86
NE Exp.		6	11	45		36	80	20					
APSF		7	11	45		38	79	21				115	87
MOX FFF		9	35	67	CH	37	74	26	6.6	19.4	0.23	108	78
F-Area	ST1[3]	19	18	40		29	71	29		23.9	0.22	116	87
NE Exp.		20	14	23		30	81	19					
APSF			25	49		30	82	18					
MOX FFF		17				26	90	9	4.9	13.6	0.21	113	83
F-Area	ST2[3]												
NE Exp.		11											
APSF													
MOX FFF		11	27	53	CH	33	68	32	15.2	18.7	0.14	113	85
F-Area	GC	7	47	83		32	61	39		2.8	0.11	121	92
NE Exp.		7	27	42		32	67	33					
APSF		9	30	57		28	48	52					
MOX FFF		5	30	55	CH	29	53	47	28.5	24.6	0.08	114	91
F-Area	CG												
NE Exp.													
APSF													
MOX FFF			27	50	CH	24	85	14			0.35		

[1] NE Expansion values include APSF data.
[2] Surface effects have not been accounted for.
[3] The Northeast Expansion report does not separate ST1 and ST2.

TABLE 6-3
AVERAGE STRENGTH TEST RESULTS

	ENGR UNIT	AVE. THICK.[1] (ft)	AVE TOP EL.[1] (MSL)	AVE Triaxial C (ksf)	AVE Triaxial ϕ (degree)	AVE Triaxial C' (ksf)	AVE Triaxial ϕ' (degree)
F-Area	TR1 [2]	25					
NE Exp.		13	291				
APSF		16	289				
MOX FFF		9	276				
F-Area	TR1A [2]	19	278				32
NE Exp.		16.0	278	0	28		33
APSF		14	273				
MOX FFF		12	267				
F-Area	TR2A	25	261				
NE Exp.		26	262				
APSF		27	260				
MOX FFF		20	255	0.6	28	0	33
F-Area	TR2B	19	233				
NE Exp.		23	236				
APSF		22	233				
MOX FFF		23	235			0	36
F-Area	TR3/4	10	213	0.75	13	0	30
NE Exp.		7	213				
APSF		8	211	0.9	12	0	29
MOX FFF		7	212	1.5	12	0	33
F-Area	DB1/3	28	204				
NE Exp.		28	206				
APSF		28	203				
MOX FFF		22	205				
F-Area	DB4/5	7	175		17		26
NE Exp.		6	178				
APSF		7	175				39
MOX FFF		9	183	0.2	11	0	29
F-Area	ST1[3]	19	167				
NE Exp.		20	172				
APSF			168				
MOX FFF		17	174				
F-Area	ST2[3]						
NE Exp.		11	152				
APSF							
MOX FFF		11	157	1.1	9	1	24
F-Area	GC	7	138				
NE Exp.		7	141				
APSF		9	143				
MOX FFF		5	146				
F-Area	CG						
NE Exp.			134				
APSF			134				
MOX FFF			141				

[1] NE Expansion values include APSF data.
[2] Surface effects have not been accounted for.
[3] The Northeast Expansion report does not separate ST1 and ST2.

TABLE 6-4
AVERAGE CONSOLIDATION TEST RESULTS

	ENGR UNIT	AVE. THICK.[1] (ft)	AVE TOP EL.[1] (MSL)	VOID RATIO	C_c	C_r	P_c (ksf)
F-Area	TR1 [2]	25		0.68	0.12	0.011	6.2
NE Exp.		13	291				
APSF		16	289				
MOX FFF		9	276				
F-Area	TR1A [2]	19	278	0.74	0.08	0.009	6.3
NE Exp.		16.0	278				
APSF		14	273	0.66	0.12	0.100	
MOX FFF		12	267				
F-Area	TR2A	25	261	0.69	0.07	0.011	5.0
NE Exp.		26	262				
APSF		27	260				
MOX FFF		20	255	0.65	0.12	0.011	8.0
F-Area	TR2B	19	233	0.80	0.05		12.0
NE Exp.		23	236				
APSF		22	233				
MOX FFF		23	235	0.64			
F-Area	TR3/4	10	213	1.39	0.85	0.138	14.8
NE Exp.		7	213				
APSF		8	211	0.89	0.28	0.16	
MOX FFF		7	212	0.94	0.21	0.021	5.8
F-Area	DB1/3	28	204	0.75	0.27	0.109	13.9
NE Exp.		28	206				
APSF		28	203				
MOX FFF		22	205	0.83	0.10	0.011	4.0
F-Area	DB4/5	7	175	1.04	0.55	0.053	10.7
NE Exp.		6	178				
APSF		7	175	1.03	0.25	0.009	
MOX FFF		9	183	1.19	0.45	0.035	7.9
F-Area	ST1[3]	19	167	0.98	0.31	0.042	14.8
NE Exp.		20	172				
APSF			168				
MOX FFF		17	174	0.96	0.15	0.017	9.1
F-Area	ST2[3]						
NE Exp.		11	152				
APSF							
MOX FFF		11	157	0.99	0.28	0.024	8.4
F-Area	GC	7	138	0.83	0.31	0.035	11.5
NE Exp.		7	141				
APSF		9	143				
MOX FFF		5	146	0.87	0.21	0.045	9.6
F-Area	CG						
NE Exp.			134				
APSF			134				
MOX FFF			141				

[1] NE Expansion values include APSF data.
[2] Surface effects have not been accounted for.
[3] The Northeast Expansion report does not separate ST1 and ST2.

TABLE 6-5
IDEALIZED GEOLOGIC PROFILE

Formation	Sub-unit I.D.	Elevation (ft, msl)
Tobacco Road	TR1A	270
Tobacco Road	TR2A	260
Tobacco Road	TR2B	236
Tobacco Road	TR3/4	211
Dry Branch	DB1/3	204
Dry Branch	DB4/5	182
Santee	ST1	172
Santee	ST2	152
Warley Hill "Green Clay"	GC	140
Congaree	CG	135
Fourmile	FM	85
Steel Creek	SC1	-2
Steel Creek	SC2	-50
Black Creek	BC1	-120
Black Creek	BC2	-280
Black Creek	BC3	-310
Middendorf	MD1	-380
Middendorf	MD2	-438
Middendorf	MD3	-496
Cape Fear	CF	-534
Basement	BM	-595

TABLE 6-6

STATISTICAL SUMMARY OF S-CPT SHEAR WAVE VELOCITY DATA

Soil Type	Average Vs (fps)	No. of Data Points	Min	Max	Median	Std Dev	Ave. + 1 Std Dev	Ave. - 1 Std Dev
TR1	1541	3	1312	1672	1641	199	1741	1342
TR1A	1476	10	1107	2019	1460	285	1761	1191
TR2A	1324	44	1053	1896	1263	187	1511	1137
TR2B	1253	65	911	3489	1209	333	1586	921
TR3/4	1016	19	857	1375	988	132	1148	883
DB1/3	1126	63	848	1941	1109	164	1290	963
DB4/5	1104	19	890	1309	1095	120	1225	984
ST1	1129	44	927	1406	1118	125	1254	1005
ST2	1068	25	775	1438	1043	175	1243	893
GC	1217	6	927	1471	1279	216	1433	1001
CG	no data							

TABLE 6-7

SHEAR MODULUS REDUCTION VALUES FOR USE AT MFFF SITE[1]

Soil Type	Stiff Upland Sands	Tobacco Road	Snapp Sands	Dry Branch/ Santee	Shallow Sand	Shallow Clay	Deep Sand	Deep Clay
Depth Range (ft)	0-30	30-70	280-300	70-250	<300	<300	>300	>300
ε_r[2]	0.021	0.044	0.044	0.077	0.066	0.148	0.111	0.23
Strain (ε)								
0.0001	0.99526	0.99773	0.99773	0.9987	0.99849	0.99932	0.9991	0.99957
0.0002	0.99057	0.99548	0.99548	0.99741	0.99698	0.99865	0.9982	0.99913
0.0003	0.98592	0.99323	0.99323	0.99612	0.99548	0.99798	0.9973	0.9987
0.0005	0.97674	0.98876	0.98876	0.99355	0.99248	0.99663	0.99552	0.99783
0.001	0.95455	0.97778	0.97778	0.98718	0.98507	0.99329	0.99107	0.99567
0.002	0.91304	0.95652	0.95652	0.97468	0.97059	0.98667	0.9823	0.99138
0.003	0.875	0.93617	0.93617	0.9625	0.95652	0.98013	0.97368	0.98712
0.005	0.80769	0.89796	0.89796	0.93902	0.92958	0.96732	0.9569	0.97872
0.01	0.67742	0.81481	0.81481	0.88506	0.86842	0.93671	0.91736	0.95833
0.02	0.5122	0.6875	0.6875	0.79381	0.76744	0.88095	0.84733	0.92
0.03	0.41176	0.59459	0.59459	0.71963	0.6875	0.83146	0.78723	0.88462
0.05	0.29577	0.46809	0.46809	0.6063	0.56897	0.74747	0.68944	0.82143
0.1	0.17355	0.30556	0.30556	0.43503	0.39759	0.59677	0.52607	0.69697
0.2	0.09502	0.18033	0.18033	0.27798	0.24812	0.42529	0.35691	0.53488
0.3	0.06542	0.12791	0.12791	0.20424	0.18033	0.33036	0.27007	0.43396
0.5	0.04031	0.08088	0.08088	0.13345	0.11661	0.2284	0.18167	0.31507
1	0.02057	0.04215	0.04215	0.07149	0.06191	0.12892	0.09991	0.18699
2	0.01039	0.02153	0.02153	0.03707	0.03195	0.0689	0.05258	0.10314
3	0.00695	0.01445	0.01445	0.02502	0.02153	0.04701	0.03568	0.07121
5	0.00418	0.00872	0.00872	0.01517	0.01303	0.02875	0.02172	0.04398
10	0.0021	0.00438	0.00438	0.00764	0.00656	0.01458	0.01098	0.02248

[1] From WSRC (1996a)
[2] ε_r is the reference strain in the relationship $G/G_{max} = 1/(1+\varepsilon/\varepsilon_r)$

TABLE 6-8

DAMPING RATIOS RECOMMENDED FOR USE AT MFFF SITE[1]

Soil Type	Stiff Upland Sands	Tobacco Road	Snapp Sands	Dry Branch/Santee	Shallow Sand	Shallow Clay	Deep Sand	Deep Clay
Depth Range (ft)	0-30	30-70	280-300	70-250	<300	<300	>300	>300
Strain (%)								
0.0001	1.059	0.625	0.625	0.825	0.674	1.296	0.489	0.992
0.0002	1.103	0.647	0.647	0.835	0.687	1.292	0.497	0.99
0.0003	1.151	0.67	0.67	0.846	0.702	1.293	0.505	0.991
0.0005	1.248	0.717	0.717	0.871	0.733	1.3	0.524	0.995
0.001	1.493	0.835	0.835	0.936	0.811	1.326	0.57	1.103
0.002	1.973	1.07	1.07	1.07	0.97	1.389	0.665	1.054
0.003	2.434	1.3	1.3	1.205	1.127	1.456	0.759	1.097
0.005	3.302	1.747	1.747	1.47	1.435	1.594	0.945	1.186
0.01	5.201	2.79	2.79	2.108	2.171	1.938	1.398	1.41
0.02	8.165	4.605	4.605	3.281	3.505	2.603	2.251	1.851
0.03	10.407	6.139	6.139	4.336	4.686	3.233	3.039	2.276
0.05	13.639	8.614	8.614	6.162	6.692	4.392	4.453	3.08
0.1	18.317	12.799	12.799	9.605	10.363	6.82	7.289	4.856
0.2		17.425	17.425	13.951	14.825	10.356	11.179	7.671
0.3				16.683		12.844	13.799	9.833
0.5						16.317	17.21	12.955

[1] From WSRC (1996a)

TABLE 6-9

PROFILES OF LOW-STRAIN SHEAR WAVE VELOCITY, COMPRESSION WAVE VELOCITY, DENSITY AND POISSON'S RATIO FOR THE MFFF SITE

			Shear Wave Velocity (fps)				
Top Elevation (ft)	Depth (ft)	Unit Weight (pcf)	Best Estimate	Upper Bound	Lower Bound	Compression Wave Velocity (fps)	Poisson's Ratio
270	0	114	1476	1808	1205	3003	0.34
260	10	120	1324	1621	1081	2244	0.25
236	34	118	1253	1534	1023	2361	0.31
211	59	114	1016	1244	830	4153	0.47
204	66	119	1126	1379	919	5061	0.47
182	88	110	1104	1352	902	4610	0.47
172	98	121	1129	1383	922	5325	0.47
152	118	113	1068	1308	872	3631	0.47
140	130	121	1217	1490	994	4922	0.47
135	135	125	1364	1670	1114	5733	0.47
85	185	125	1467	1796	1198	6162	0.47
-2	272	125	1818	2226	1485	7641	0.47
-50	320	130	2124	2601	1734	8928	0.47
-120	390	125	2230	2731	1821	9373	0.47
-280	550	130	2712	3321	2215	11399	0.47
-310	580	130	2457	3009	2006	10327	0.47
-380	650	135	2460	3013	2009	10340	0.47
-438	708	135	2741	3357	2238	11521	0.47
-496	766	135	2980	3649	2433	12525	0.47
-534	804	135	2744	3360	2241	11534	0.47
-595	865	150	11000	11000	11000	14000	0.26

TABLE 6-10
STRAIN-COMPATIBLE SHEAR WAVE VELOCITIES, DAMPING RATIOS, AND CYCLIC STRESS RATIO FOR MFFF SITE

Top Elevation (ft)	Strain-Compatible Shear Wave Velocity (fps)			Damping Ratio (%)			Cyclic Stress Ratio
	Best Estimate	Lower Bound	Upper Bound	Best Estimate	Lower Bound	Upper Bound	Best Estimate
270	1429.8	1165.4	1754.9	0.74	0.77	0.69	0.1309
265	1411.9	1146.9	1740.4	1.01	1.11	0.87	0.1305
260	1275.7	1026.1	1576.7	1.41	1.74	1.2	0.1295
253.5	1250.5	1005.6	1560.6	1.85	2.18	1.42	0.1280
250	1234.4	992.4	1541.0	2.13	2.45	1.71	0.1263
245	1219.1	979.9	1522.6	2.39	2.71	1.97	0.1240
240	1207.9	962.8	1509.3	2.58	3.13	2.16	0.1218
236	1104.4	865.1	1394.1	3.18	4.13	2.49	0.1196
231	1083.9	847.5	1384.9	3.62	4.58	2.63	0.1171
226	1065.1	833.5	1377.3	4.02	4.92	2.74	0.1150
221	1048.2	822.9	1365.9	4.37	5.18	2.93	0.1129
216	1034.0	815.2	1349.7	4.66	5.37	3.22	0.1102
211	913.8	739.1	1141.7	2.99	3.20	2.56	0.1073
208	912.1	735.8	1139.2	3.03	3.31	2.61	0.1076
204	977.9	778.8	1242.2	3.79	4.28	3.02	0.1074
196	972.8	767.8	1233.9	3.90	4.64	3.16	0.1057
188	969.0	758.5	1226.5	3.98	4.95	3.29	0.1039
182	897.5	688.1	1135.9	4.33	5.92	3.67	0.1023
177	895.7	683.7	1133.1	4.38	6.06	3.72	0.1006
172	977.7	755.2	1233.2	4.02	5.35	3.42	0.0971
162	975.8	749.0	1226.9	4.06	5.55	3.53	0.0929
152	855.0	647.4	1092.1	5.07	6.83	4.07	0.0926
146	849.4	648.6	1090.4	5.23	6.79	4.10	0.0922
140	1060.3	840.8	1337.5	3.90	4.53	3.27	0.0917
135	1241.1	994.0	1566.6	3.33	3.74	2.66	0.0823
85	1299.7	1038.0	1701.9	3.78	4.23	2.27	0.0849
65	1287.5	1034.5	1701.6	3.97	4.30	2.27	0.0861
59	1268.2	1044.9	1665.8	4.28	4.10	2.77	0.0843
-2	1804.8	1475.2	2228.1	1.56	1.55	1.38	0.0693
-50	2127.8	1735.1	2612.9	1.30	1.33	1.25	0.0562
-120	2234.0	1823.1	2745.9	1.36	1.38	1.29	0.0478
-280	2733.4	2236.2	3384.8	1.18	1.14	0.94	0.0475
-310	2459.8	2013.1	3045.9	1.32	1.27	1.08	0.0462
-380	2565.2	2305.8	3153.0	1.33	1.31	1.25	0.0387
-438	2813.6	2295.7	3472.3	1.19	1.20	1.21	0.0393
-496	3067.7	2504.4	3783.4	1.13	1.13	0.97	0.0389
-534	2809.3	2293.6	3464.4	1.24	1.25	1.09	0.0381
-595	11000	11000	11000	0.01	0.01	0.01	0.0366

TABLE 7 – 1

FOUNDATION DESIGN CRITERIA

MOX AND EMERGENCY DIESEL GENERATOR BUILDINGS

ITEM	MFFF Building Main Level	Aqueous Polishing Sublevel	Emergency Diesel Generator Bldg
1. Mat thickness (FT)	4	4	2
2. Top of Mat EL. (FT)	273	255.5	271
3. Bottom of Mat EL (FT)	269	251.5	269
4. Minimum Structural Fill Thickness (FT)	10	5	5
5. Approximate Bottom Fill EL. (FT)	259	246.5	264
6. Static Design Load (PL + LL)			
Max. Edge Bearing Pressure (PSF)	7,200	7,800	3,600
Avg. Bearing Pressure	4,600	6,100	2,000
7. Dynamic Design Load (Seismic)			
Max. Edge Bearing Pressure (PSF)	10,900	13,200	6,900
Ave. Bearing Pressure	6,800	9,100	3,100

Table 7-2. Best Estimate of Material Properties for FLAC Analysis

Layer	Shear Wave Velocity Vs (fps)	Poisson's Ratio ν	Bulk Modulus K (ksf)	Shear Modulus G (ksf)	Effective Friction Angle ϕ' (degrees)	Dilation Angle ϕ_d (degrees)	Soil Density γ_m (lb/ft^3)	Soil Density ρ (slugs/ft^3)
10' Structural Fill	1600	0.30	3230	1491	45	15	145	3.88
TR1/TR1A	1476	0.34	3230	1157	32	14	114	3.54
TR2A	1324	0.25	1633	980	32	13	120	3.73
TR2B1	1253	0.31	1983	863	31	13	118	3.66
TR2B2	1253	0.47	14096	863	31	13	118	3.66
TR3/4	1016	0.47	8954	548	30	13	114	3.54
DB1/3	1126	0.47	11480	703	30	13	119	3.70
DB4/5	1104	0.47	10201	625	26	11	110	3.42
ST1	1129	0.47	11735	718	31	13	121	3.76
ST2	1068	0.47	13636	600	30	13	113	3.51
GC	1217	0.47	13636	835	28	12	121	3.76
CG	1364	0.47	17695	1083	35	15	125	3.88
Soft Zone	250	0.47	476	29	5	0	100	3.11

Table 7-3. Lower Bound of Material Properties for FLAC Analysis

Layer	Shear Wave Velocity Vs (fps)	Poisson's Ratio ν	Bulk Modulus K (ksf)	Shear Modulus G (ksf)	Effective Friction Angle ϕ' (degrees)	Dilation Angle ϕ_d (degrees)	Soil Density γ_m (lb/ft^3)	Soil Density ρ (slugs/ft^3)
10' Structural Fill	1600	0.30	3230	1491	45	15	145	3.88
TR1/TR1A	1205	0.34	2153	771	32	14	114	3.54
TR2A	1081	0.25	1501	653	32	13	120	3.73
TR2B1	1023	0.36	1863	575	31	13	118	3.66
TR2B2	1023	0.47	9396	575	31	13	118	3.66
TR3/4	830	0.47	5975	366	30	13	114	3.54
DB1/3	919	0.47	7647	468	30	13	119	3.70
DB4/5	902	0.47	6810	417	26	11	110	3.42
ST1	922	0.47	7826	479	31	13	121	3.76
ST2	872	0.47	6538	400	30	13	113	3.51
GC	994	0.47	9096	557	28	12	121	3.76
CG	1144	0.47	11803	723	35	15	124	3.88
Soft Zone	250	0.47	476	29	5	0	100	3.11

NOTE:
Structural Fill will be a high strength engineered crushed rock fill which as $\gamma_m \geq 140$ pcf, $V_s \geq 1600$ fps and a ν=0.30.

Table 7-4. Settlement Results for the MOX Building at Geotechnical Cross-Section 1

Case Analyzed	Figure No.	Total Settlement Right Edge (in)	Total Settlement Centerline (in)	Total Settlement Left Edge (in)	Max Differential Settlement MOX From CL to Edge (in)	Max Total Settlement Values from FLAC Model (in)
Best Estimate	7-1, 7-2	1.5	2.1	1.1	0.9	2.1
Lower Bound	7-1, 7-3	2.1	2.8	1.5	1.3	2.9

Table 7-5. Settlement Results for the MOX and Emergency Diesel Generator Buildings at Geotechnical Cross-Section 2

Case Analyzed	Figure No.	Total Settlement Right Edge MOX (in)	Total Settlement Centerline MOX (in)	Total Settlement Left Edge MOX (in)	Max Differential Settlement CL to edge MOX (in)	Total Settlement Centerline EDG (in)	Max Differential Settlement EDG (in)	Max Total Settlement Values from FLAC Model (in)
Best Estimate	7-4, 7-5	1.4	2.0	1.2	0.8	0.8	0.1	2.0
Lower Bound	7-4, 7-6	1.5	2.1	1.3	0.8	0.9	0.1	2.2

Table 7-6. Settlement Results for the MOX Building at Geotechnical Cross-Section 4

Case Analyzed	Figure No.	Total Settlement Right Edge MOX (in)	Total Settlement Centerline MOX (in)	Total Settlement Left Edge MOX (in)	Max Differential Settlement CL to Edge MOX (in)	Total Settlement Centerline of Sublevel (in)	Max Differential Settlement Sublevel (in)	Max Total Settlement Values from FLAC Model (in)
Best Estimate	7-7, 7-8	1.4	2.2	1.5	0.7	0.6	0.9	2.2
Lower Bound	7-7, 7-9	1.9	2.9	1.9	1.0	0.7	1.2	2.9

NOTES FOR TABLES:

1. Analysis assumes 10 ft of structural fill beneath main MOX building.
2. Analysis assumed 5 ft of structural fill beneath the Aqueous Polishing Area (sublevel) of the MOX building and beneath the Emergency Diesel Generator (EDG) building.

TABLE 8-1
ESTIMATED POST-EARTHQUAKE SETTLEMENT IN ST-1 AND ST-2

Engineering unit	ST-1	ST-2
Average thickness beneath and adjacent to MFFF building	20 ft	11 ft
Average volumetric strain from seismic CPTs for Charleston Earthquake	0.00035	0.00316
Average volumetric strain from seismic CPTs for PC3+ Earthquake	0.00010	0.00118
Liquefaction induced settlement of ST-1 from Charleston Earthquake	0.08 inch	0.42 inch
Liquefaction induced settlement of ST-1 from PC3+ Earthquake	0.03 inch	0.16 inch

TABLE 8-2
SUMMARY OF ESTIMATED POST-EARTHQUAKE SETTLEMENT FOR PC3+ BASED CONTROL MOTION AND CHARLESTON CONTROL MOTION (CSRs FROM MFFF IDEALIZED SOIL COLUMN)

CPT/Boring No.	Total Depth (ft)	PC3+ Settlement (in)	Charleston Settlement (in)
BH-1	150	0.2	0.4
BH-4	181	0.0	0.0
BH-8	153	0.0	0.0
BH-11	170	0.6	1.6
BH-12	154	0.1	0.1
BH-13	155	0.6	1.8
CPT-7	114	0.6*	1.1*
CPT-8	140	0.4	1.1
CPT-13	167	0.4*	1.1*
CPT-14	142	0.5	1.3
CPT-21	139	0.7*	1.4*
CPT-22	153	0.7*	1.6*
CPT-27	129	0.8*	1.4*
CPT-28	150	0.3	0.9
CPT-47	116	0.4*	0.8*
CPT-48	111	0.6*	1.0*
CPT-49	123	0.4*	0.9*
CPT-50	134	0.5*	1.0*
CPT-51	139	0.7*	1.3*
CPT-52	120	0.5*	1.0*
CPT-53	125	0.5*	1.1*
CPT-54	123	0.6*	1.2*
CPT-55	137	0.7*	1.4*
CPT-56	120	0.4*	0.9*
CPT-57	129	0.5*	1.1*
CPT-58	122	0.4*	0.9*
CPT-59	126	0.6*	1.2*
CPT-60	141	0.7*	1.4*
CPT-63	119	0.5*	1.1*
CPT-64	141	0.4	1.0
Average	**137**	**0.5**	**1.0**

*CPT sounding refusal in ST-1/ST-2 Units. Average site Unit thickness of ST1 and ST2 was used to compute remainder of settlement in these layers.

TABLE 8-3
SUMMARY OF ESTIMATED POST EARTHQUAKE SETTLEMENT (CSRs BY INDIVIDUAL S-CPTs)

CPT/Boring No.	Total Depth (ft)	PC3+ Settlement (in)	Charleston Settlement (in)
BH-1	150	0.1	0.4
BH-2	138	0.3	0.5
BH-5	158	1.7	1.9
BH-7	153	0.1	0.3
BH-8	153	0.0	0.0
BH-9	138	0.0	0.2
BH-10	153	0.0	0.0
CPT-1	104	0.2	0.9
CPT-3	109	0.4	1.3
CPT-5	125	0.7	1.4
CPT-8	140	0.6	1.3
CPT-11	106	0.2	0.8
CPT-13	157	0.7*	1.3*
CPT-16	122	0.4	0.8
CPT-19	117	0.5*	0.9*
CPT-23	123	0.5*	1.1*
CPT-26	127	0.3	1.4
CPT-28	150	0.4	0.7
CPT-31	126	0.3	0.8
CPT-34	146	0.5	1.0
CPT-35	144	0.6*	1.4*
CPT-37	135	0.3	0.6
Average	**135**	**0.4**	**0.9**

*CPT sounding refusal in ST-1/ST-2 Units. Average site Unit thickness of ST-1 and ST-2 was used to compute remainder of settlement in these layers.

TABLE 8-4
COMPARISON OF ESTIMATED POST EARTHQUAKE SETTLEMENTS FROM CSRs BETWEEN CPTs/BORINGS OF MFFF IDEALIZED SOIL COLUMN AND INDIVIDUAL SCPTs/BORINGS

		MFFF Idealized Soil Column		Individual Seismic-CPTs	
CPT/Boring No.	Total Depth (ft)	PC3+ Settlement (in)	Charleston Settlement (in)	PC3+ Settlement (in)	Charleston Settlement (in)
BH-1	150	0.2	0.4	0.1	0.4
BH-8	153	0.0	0.0	0.0	0.0
CPT-8	140	0.4	1.1	0.6	1.3
CPT-13*	167	0.4	1.1	0.7	1.3
CPT-28	150	0.3	0.9	0.4	0.7

*CPT sounding refusal in ST-1/ST-2 Units. Average site Unit thickness of ST-1 and ST-2 was used to compute remainder of settlement.

FIGURES

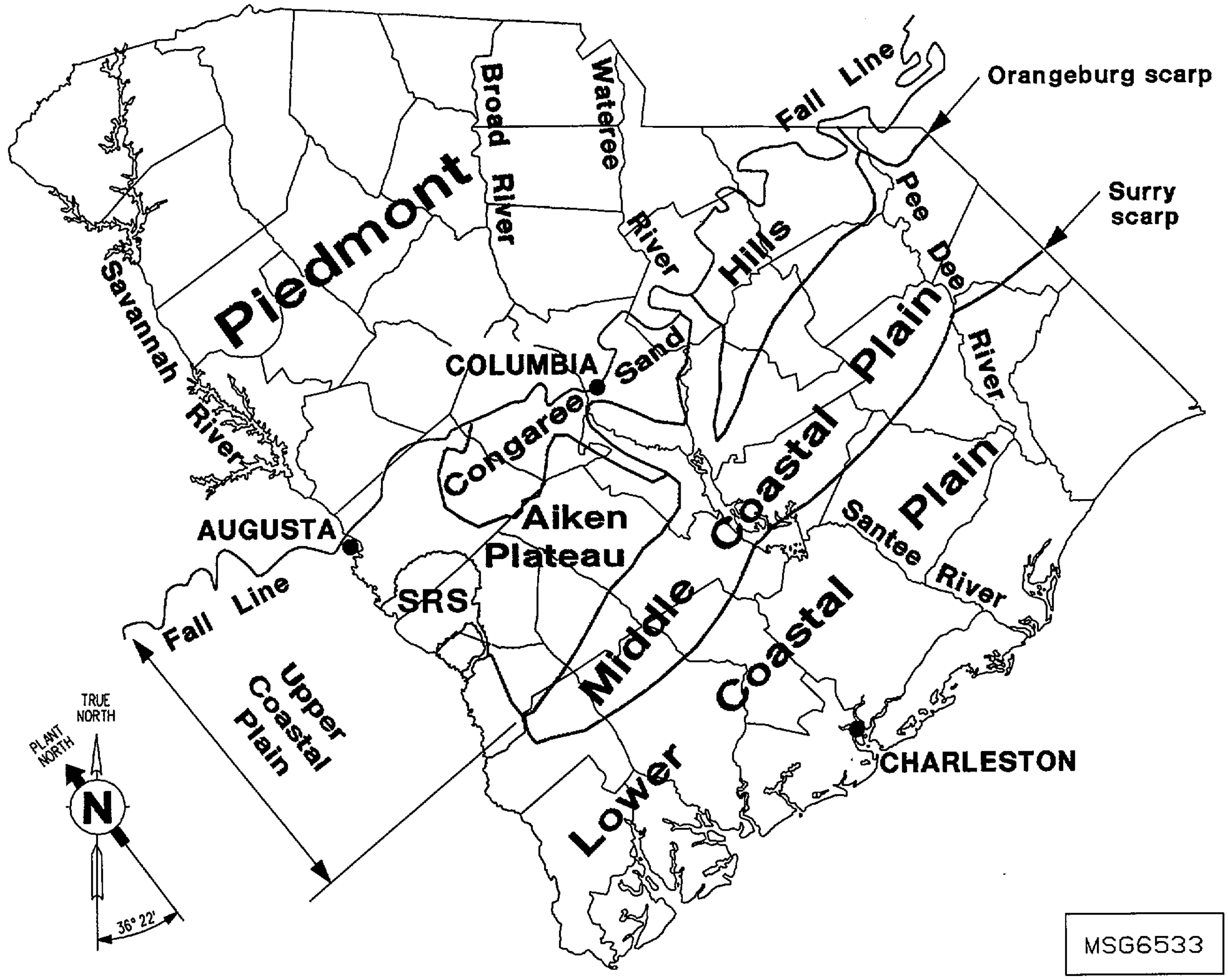


Figure 3-1. Physiography of the SRS Area (WSRC 2000a).

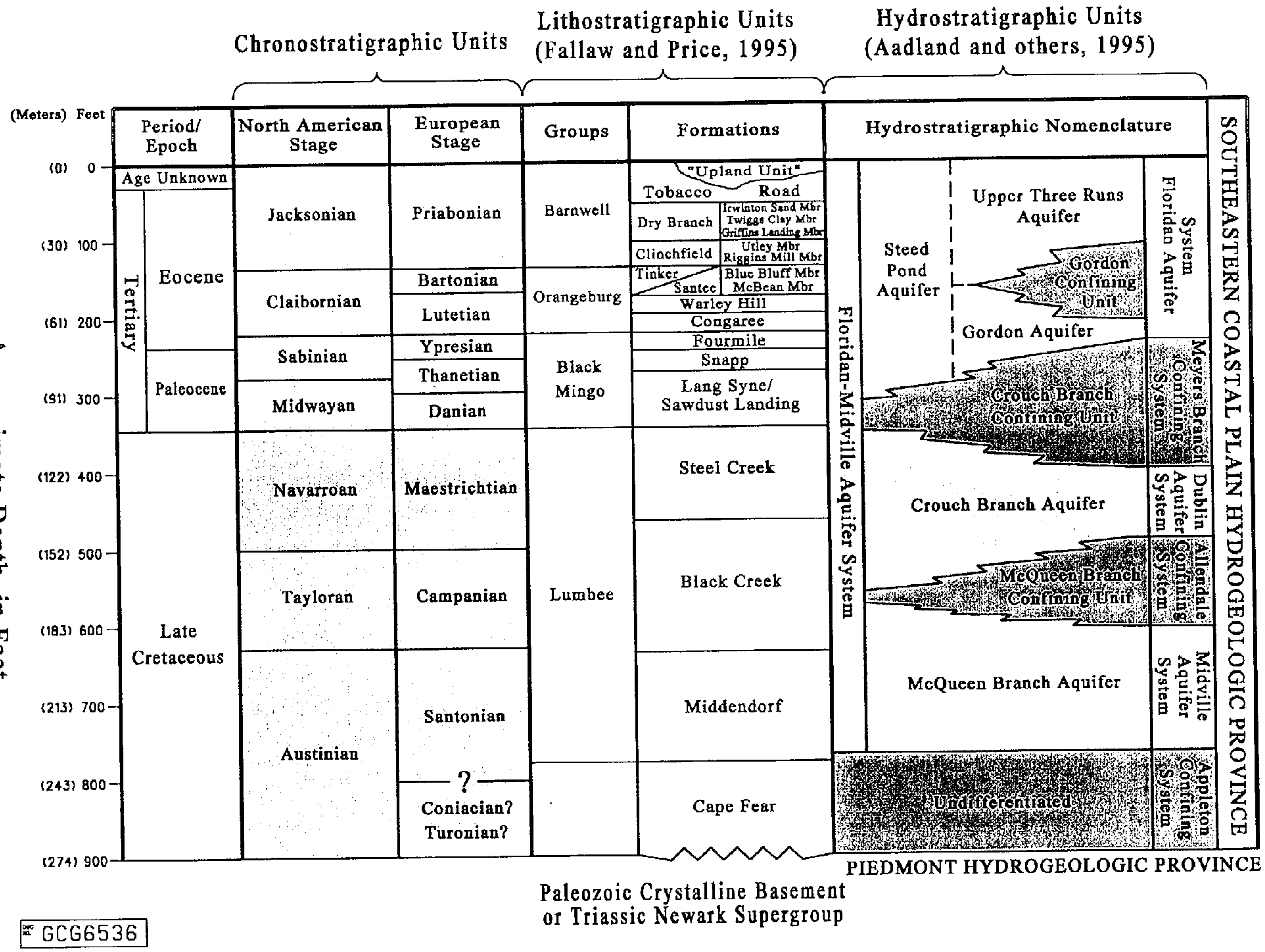


Figure 3-2. Comparison of Chronostratigraphic, Lithostratigraphic, and Hydrostratigraphic Units in the SRS Region (WSRC 2000a).

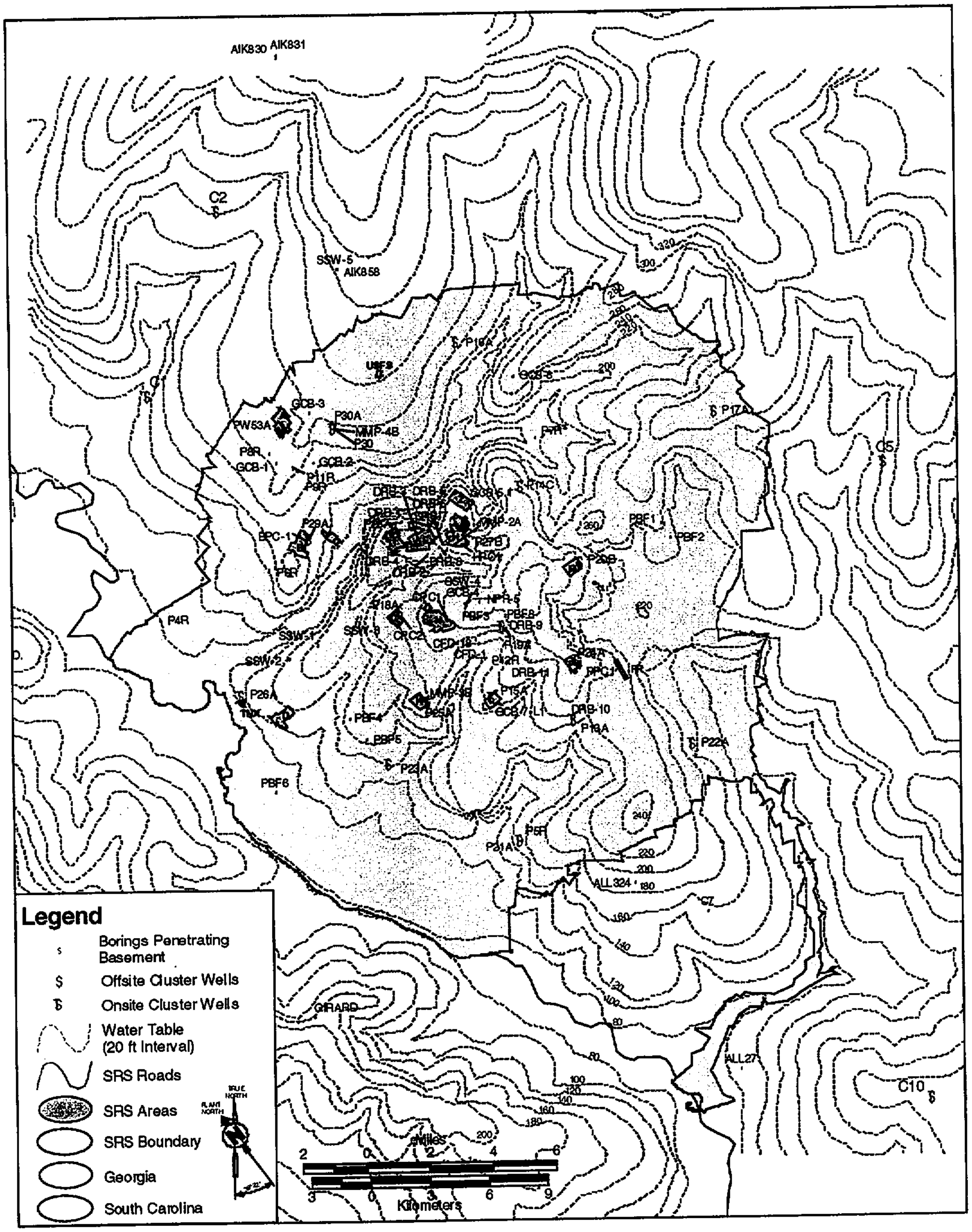


Figure 3-3. Location of Type and Reference Wells for Geologic Units at SRS (WSRC 2000a).

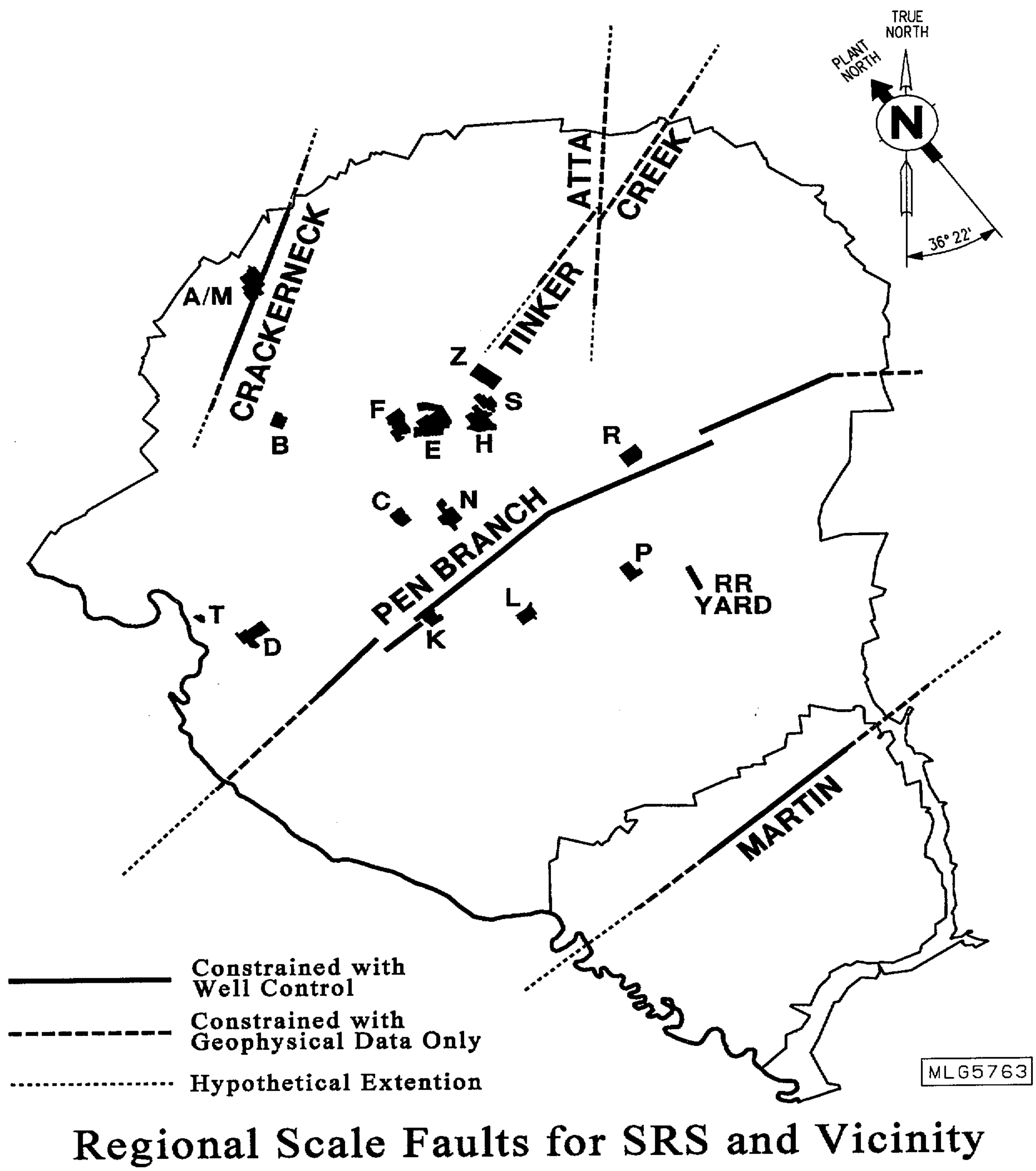


Figure 3-4. Regional Scale Faults for SRS and Vicinity (WSRS 2000a).

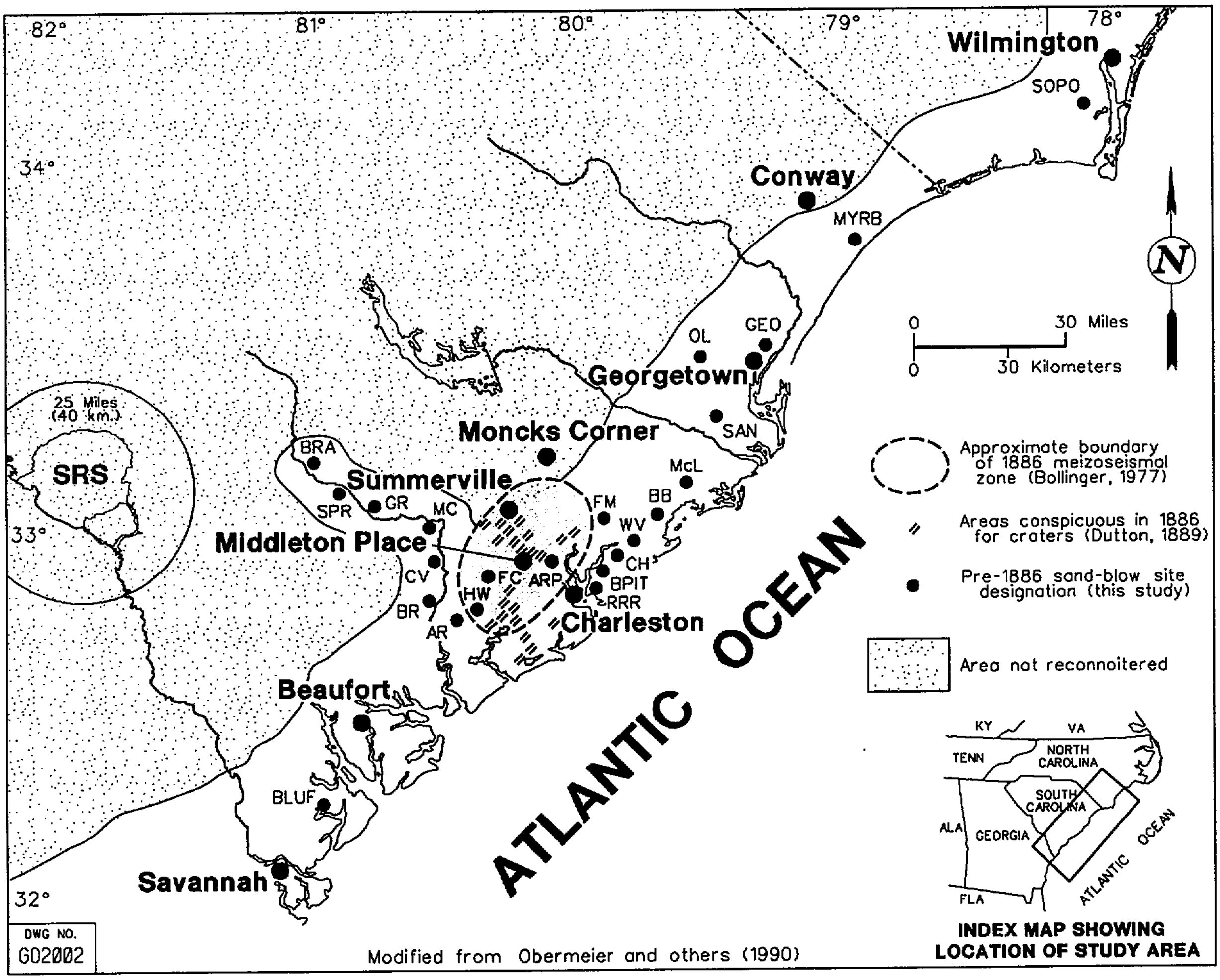


Figure 3-5. Location of sand blows (WSRC 2000a).

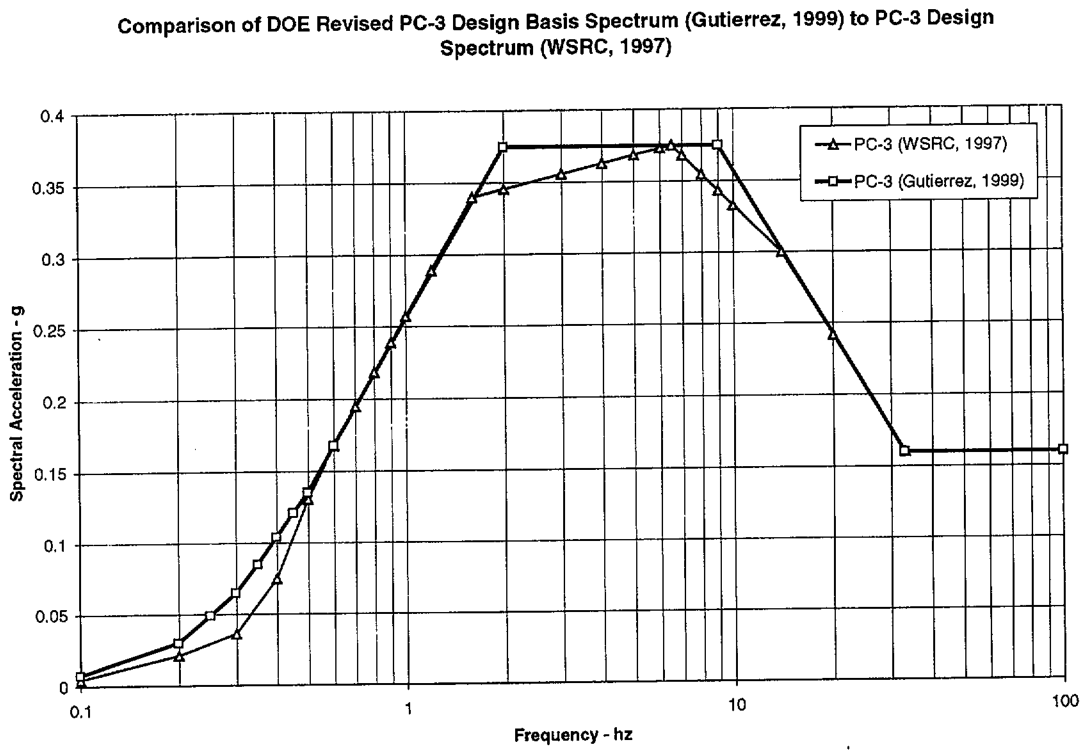


Figure 3-6. Revised SRS PC-3 5% damped design response spectrum (Gutierrez, 1999).

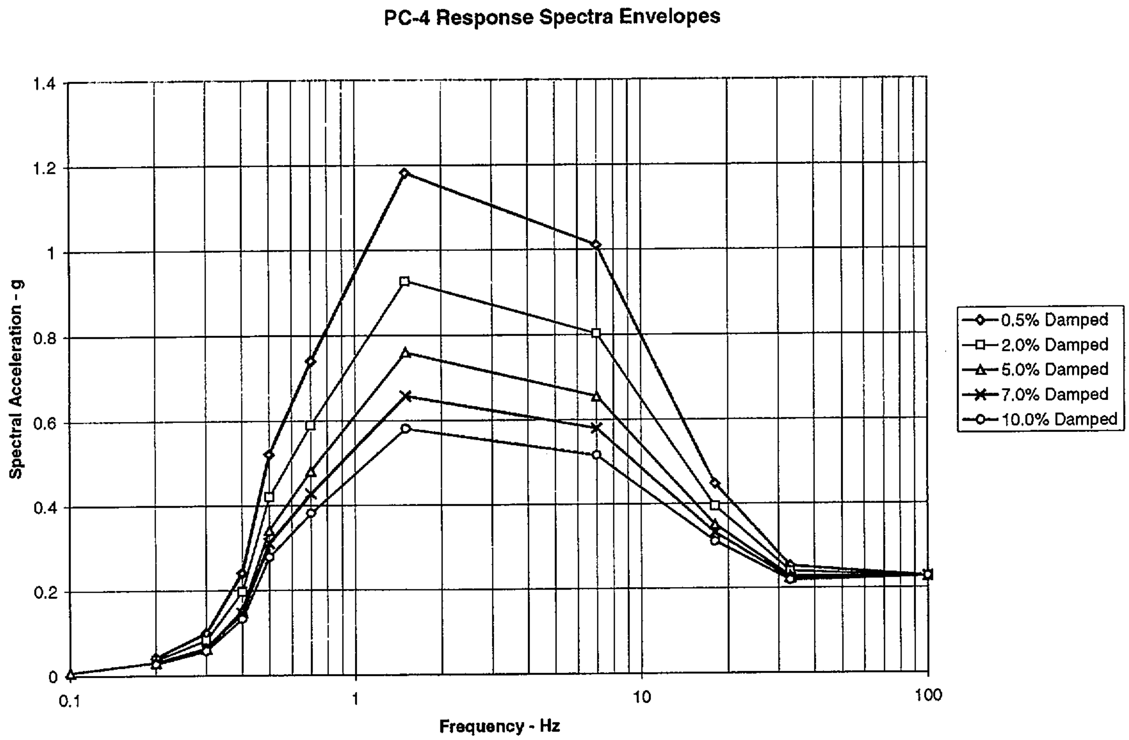


Figure 3-7. PC-4 Response spectra envelopes (WSRC, 1997).

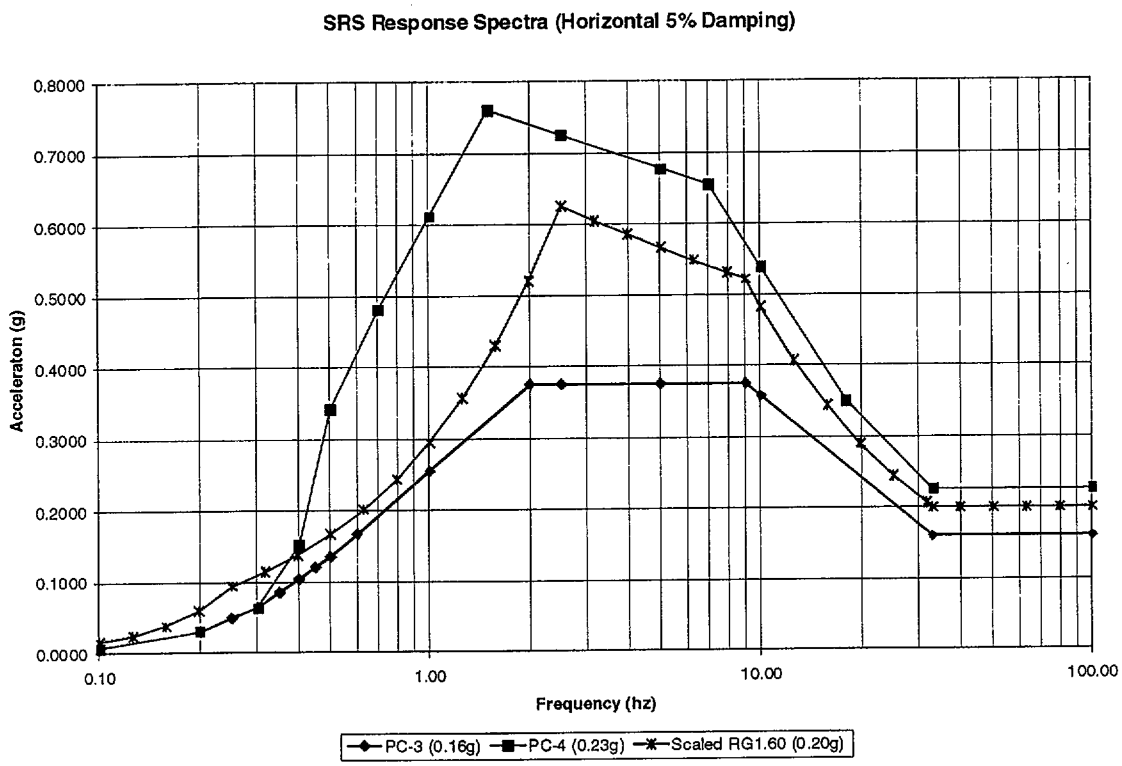
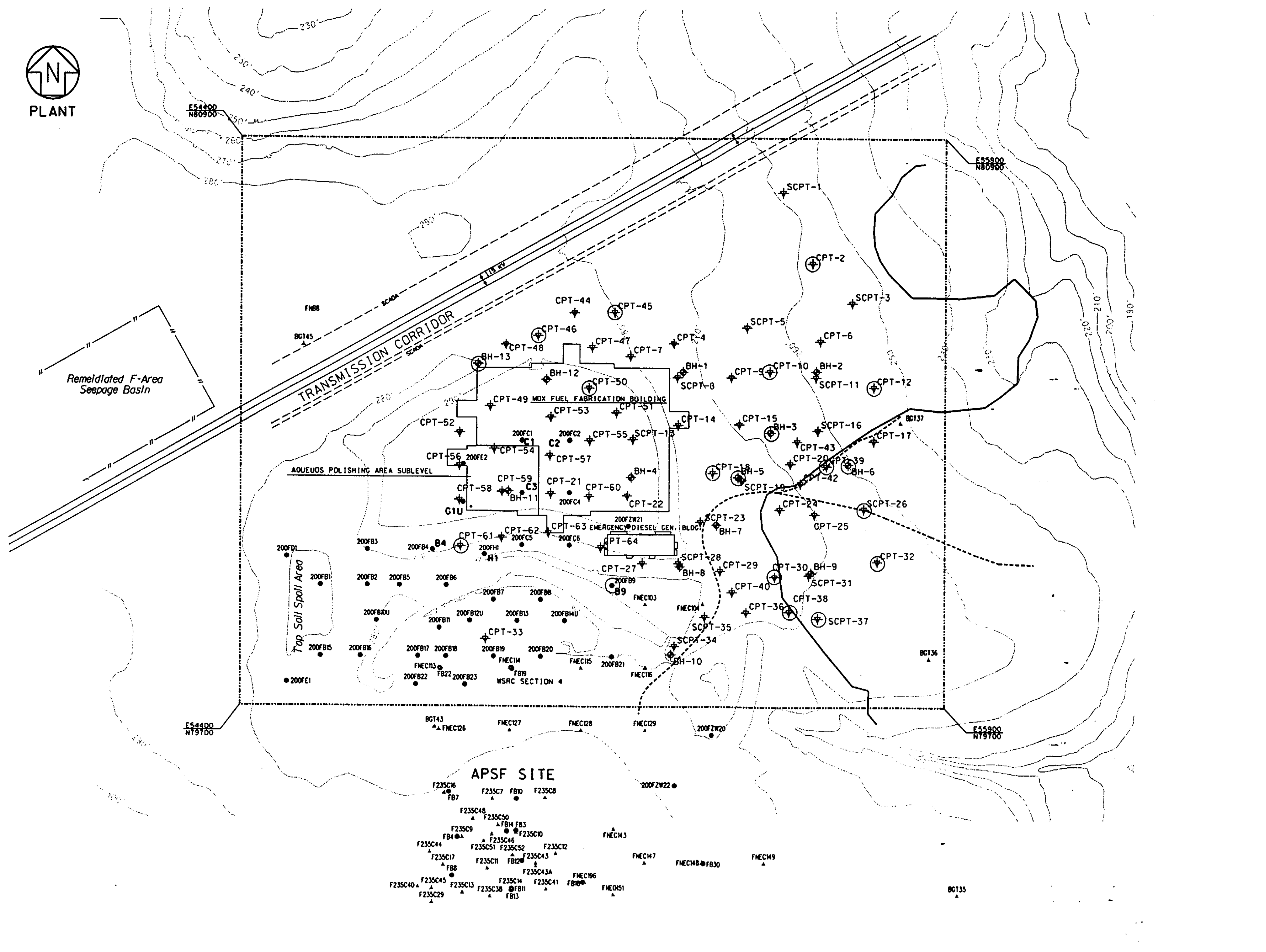


Figure 3-8. Comparison of 0.2g RG 1.60 Spectrum to PC-3 and PC-4 (DCS 2000e)



LEGEND:

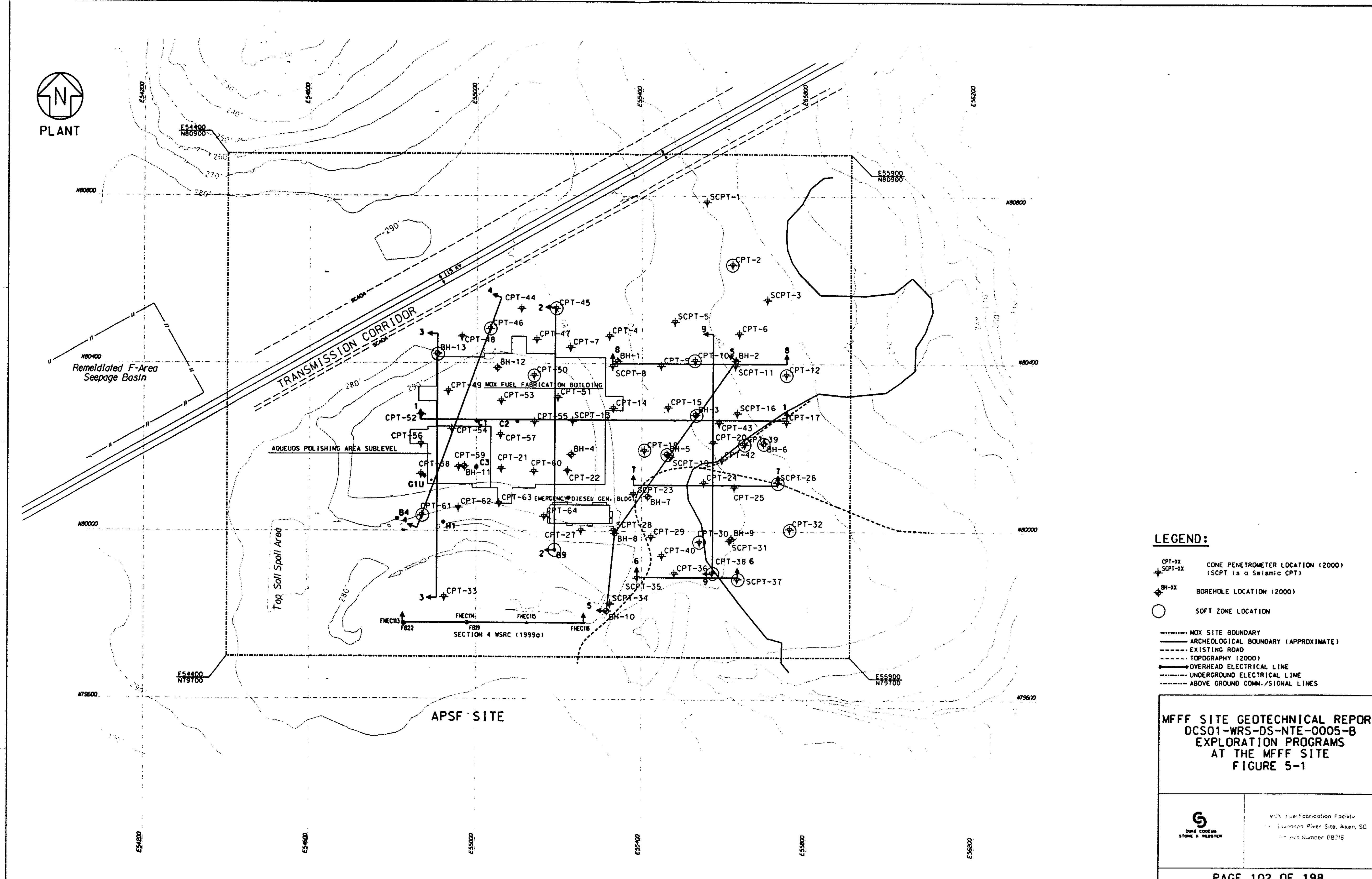
▲ FNECXXX	WSRC F-AREA EXPANSION REPORT CPT'S (1999a)
● FBXX	WSRC F-AREA EXPANSION REPORT BORINGS (1999a)
● BX, CX, GXU	1973 MUESER, RUTLEDGE SERIES BORINGS
CPT-XX SCPT-XX	CONE PENETROMETER LOCATION (2000) (SCPT is a Seismic CPT)
BH-XX	BOREHOLE LOCATION (2000)
○	SOFT ZONE LOCATION

MOX SITE BOUNDARY
ARCHEOLOGICAL BOUNDARY (APPROXIMATE)
EXISTING ROAD
TOPOGRAPHY (2000)
OVERHEAD ELECTRICAL LINE
UNDERGROUND ELECTRICAL LINE
ABOVE GROUND COMM./SIGNAL LINES

MFFF SITE GEOTECHNICAL REPORT
DCS01-WRS-DS-NTE-0005-B
EXPLORATION PROGRAMS
AT THE MFFF SITE
FIGURE 4-1

DUKE COGEMA STONE & WEBSTER

plantfig4-1.dgn Final Geot. Report

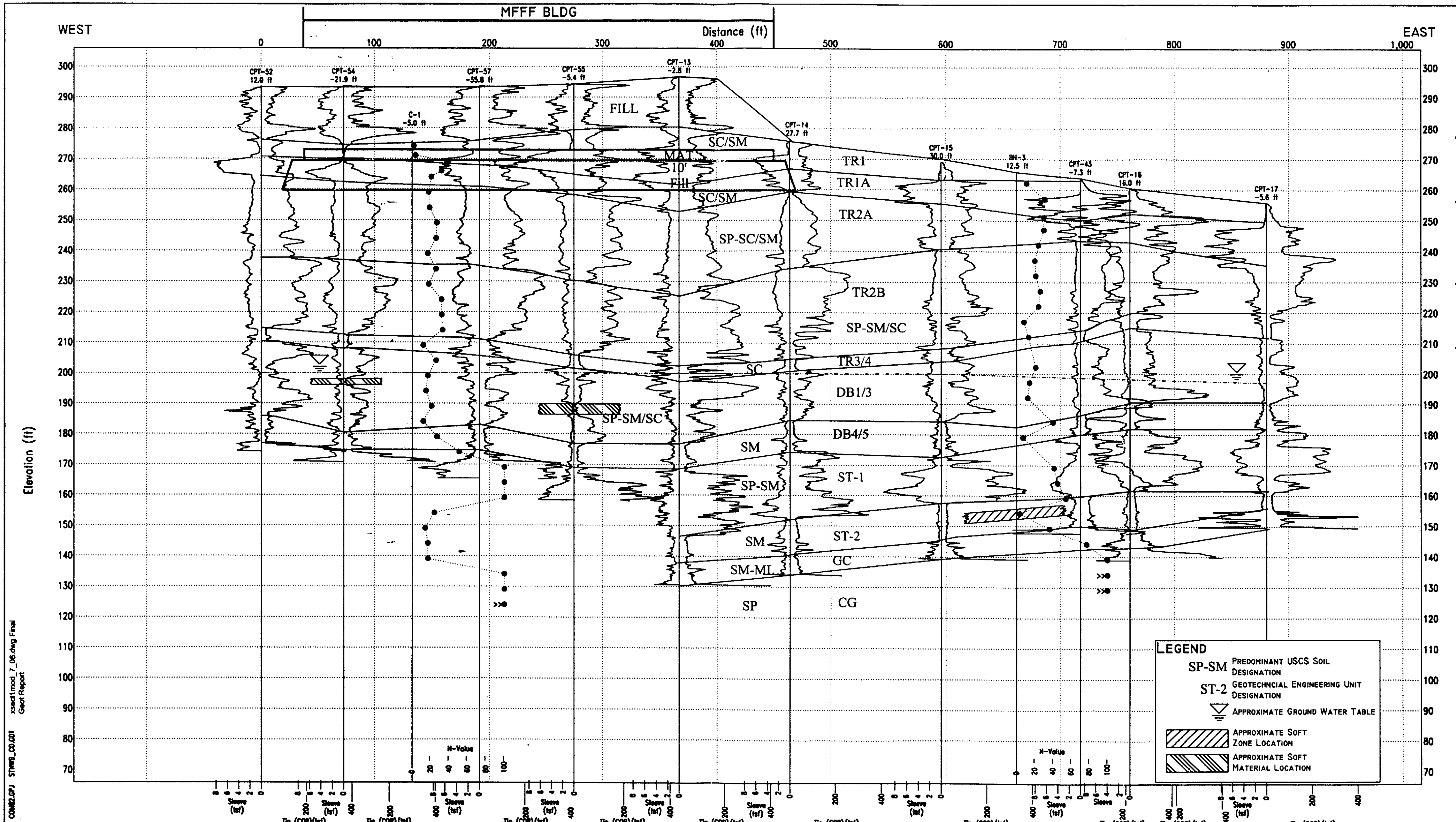


MFFF SITE GEOTECHNICAL REPORT
DCS01-WRS-DS-NTE-0005-B
EXPLORATION PROGRAMS
AT THE MFFF SITE
FIGURE 5-1

DUKE COGEMA STONE & WEBSTER

MOX Fuel Fabrication Facility
Savannah River Site, Aiken, SC
Project Number DB716

fig4-1.5-1.dgn Final Geot. Report



DUKE COGEMA STONE & WEBSTER

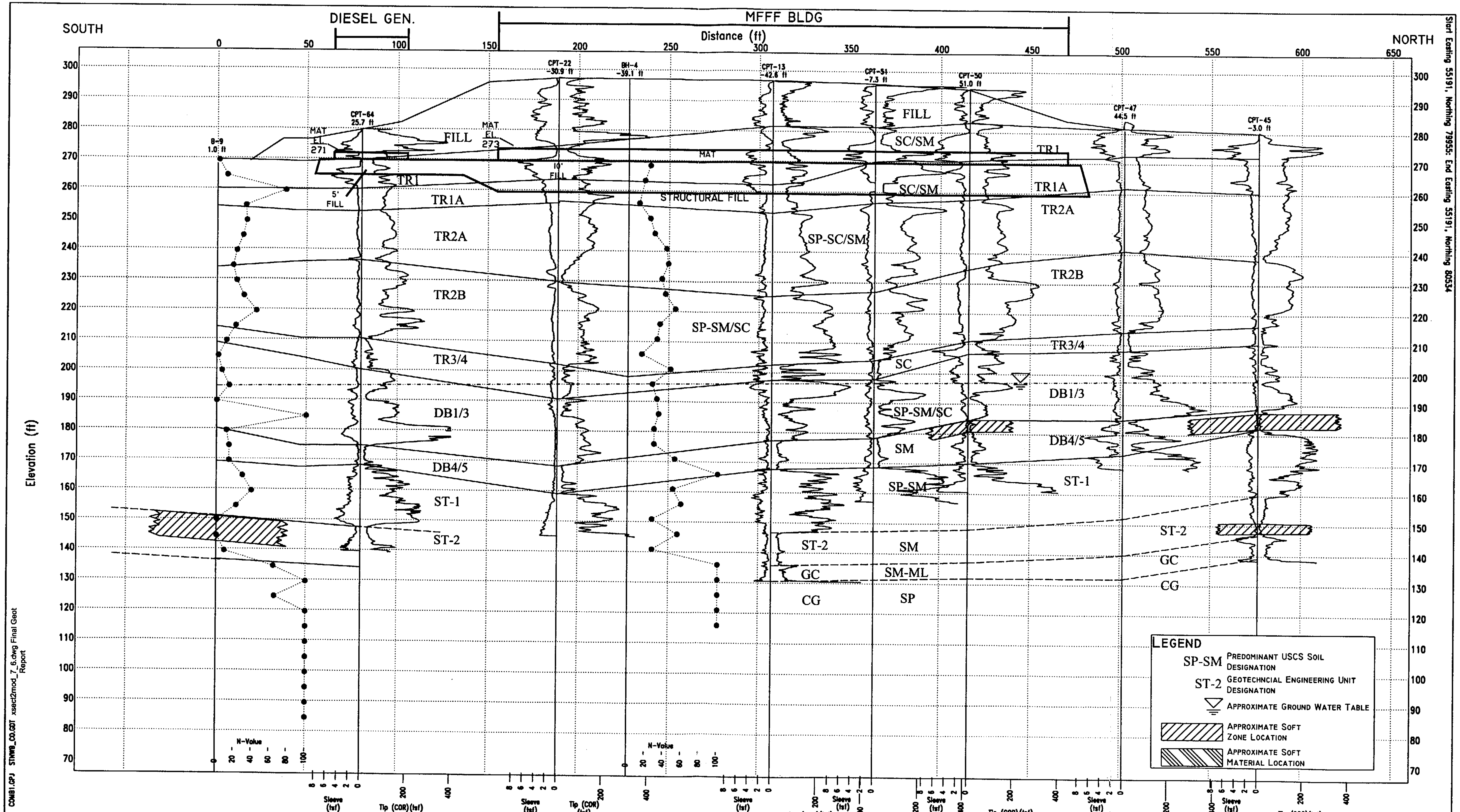
MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

NOTES:
1. Subsurface stratigraphy shown represents a reasonable judgement of the conditions expected based on the data shown. Some variation from these conditions must be expected.
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000).
3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity.
4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.

GEOTECHNICAL CROSS SECTION I

NOT TO SCALE

FIGURE 5-2



DUKE COGEMA STONE & WEBSTER

MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

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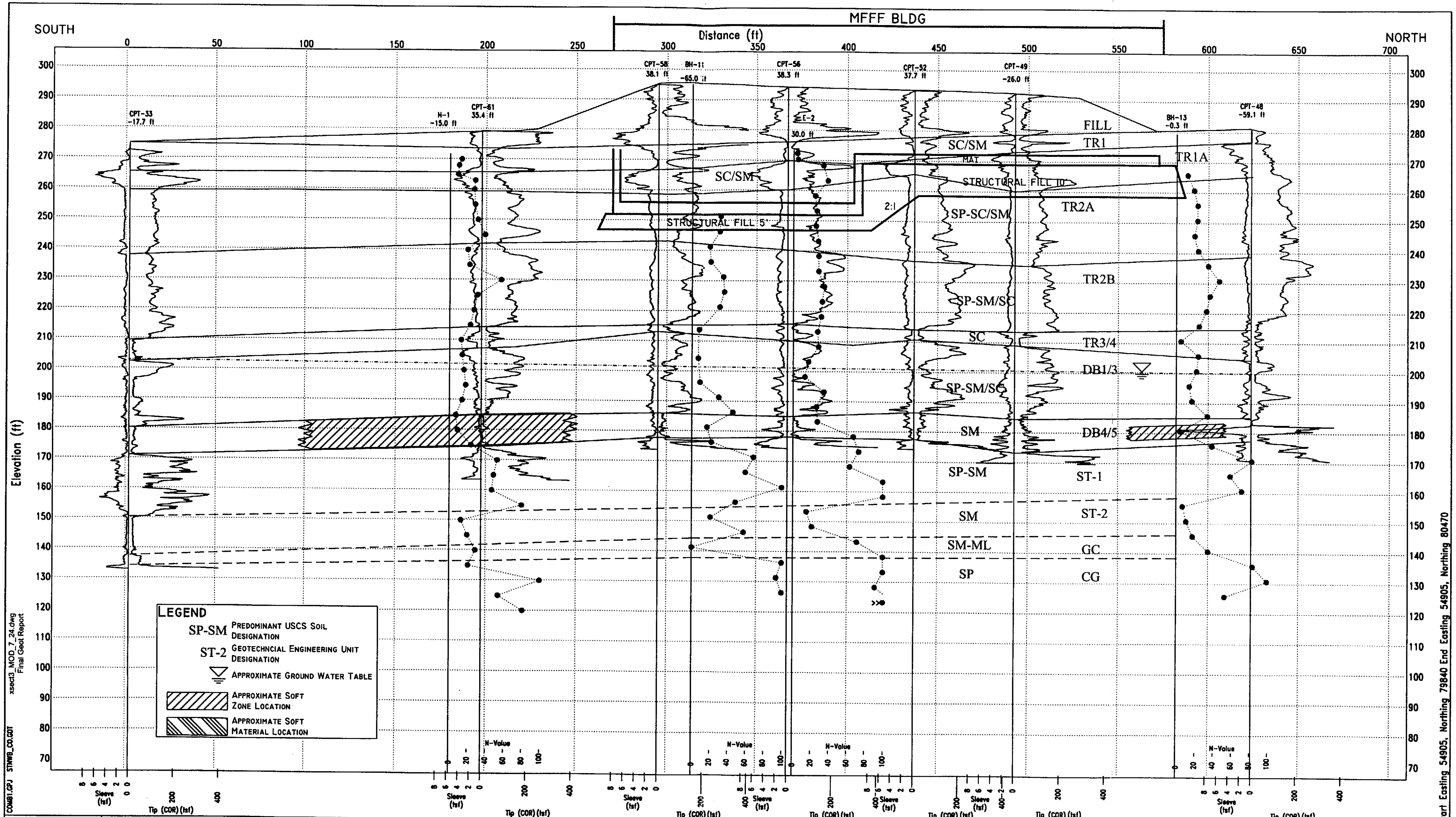
GEOTECHNICAL CROSS SECTION 2

NOT TO SCALE

FIGURE 5-3



DUKE COGEMA STONE & WEBSTER

MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

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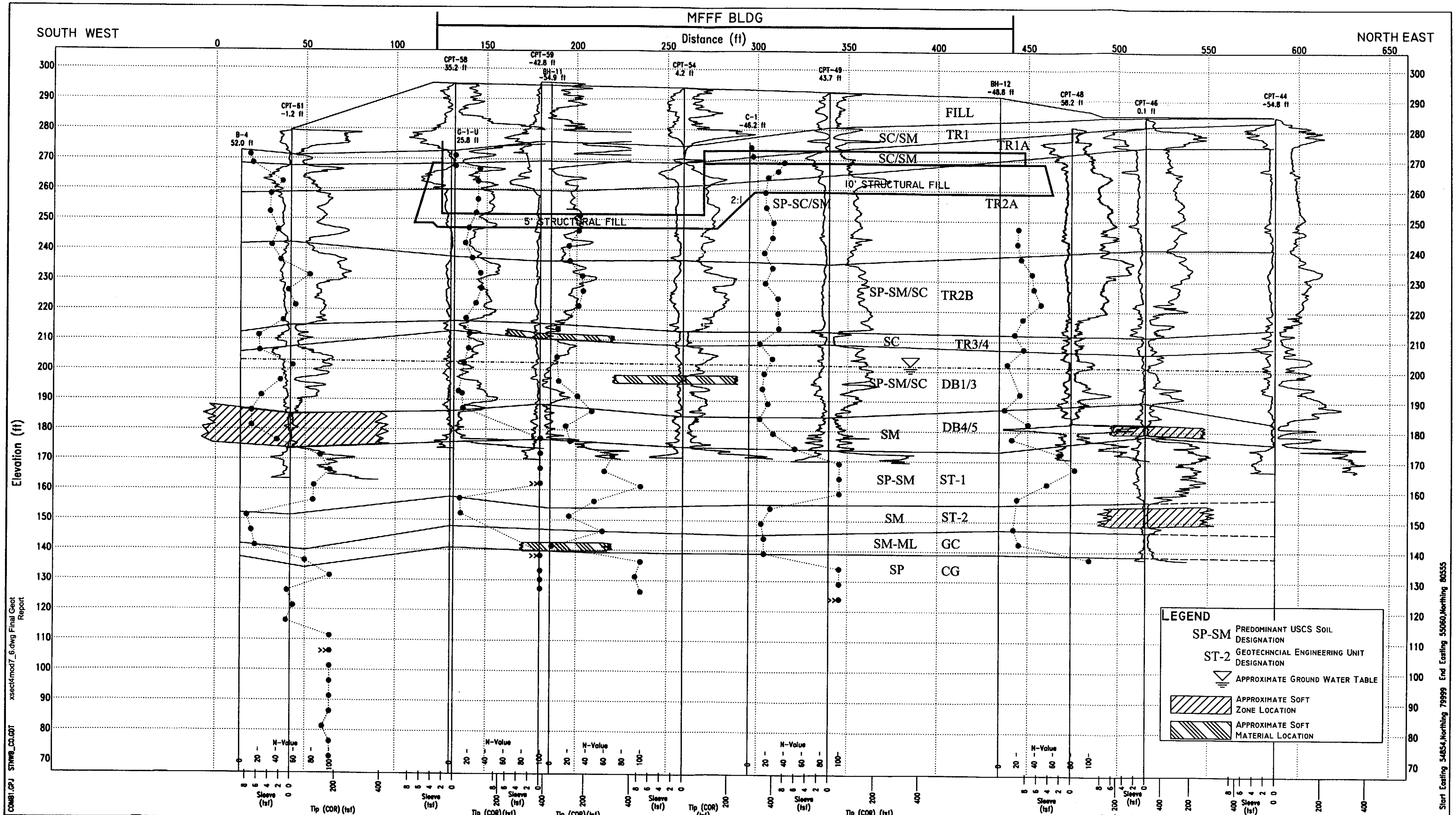
GEOTECHNICAL CROSS SECTION 3

NOT TO SCALE

FIGURE 5-4



DUKE COGEMA STONE & WEBSTER

MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

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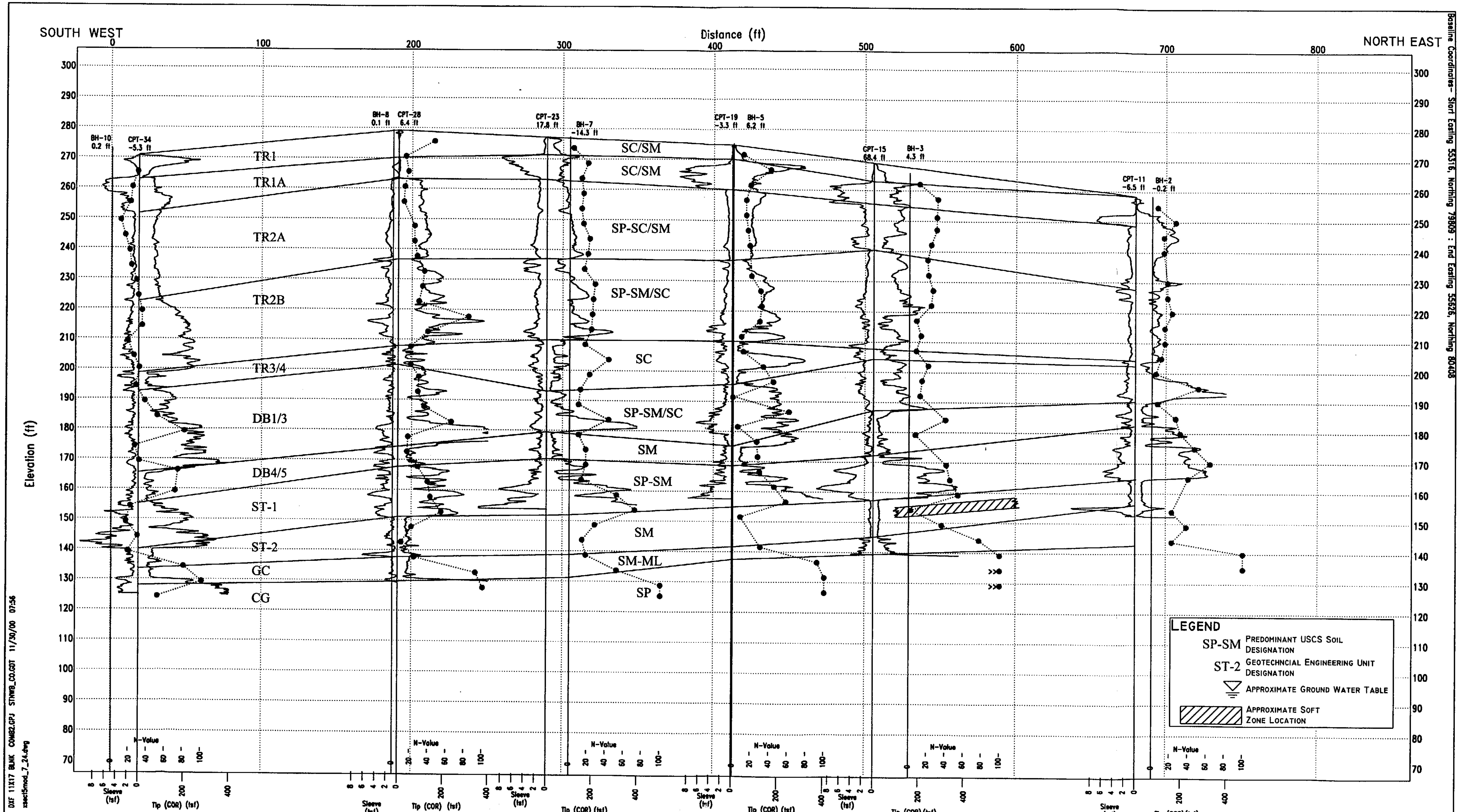
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000).
3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity.
4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.

GEOTECHNICAL CROSS SECTION 4

NOT TO SCALE

FIGURE 5-5



DUKE COGEMA STONE & WEBSTER

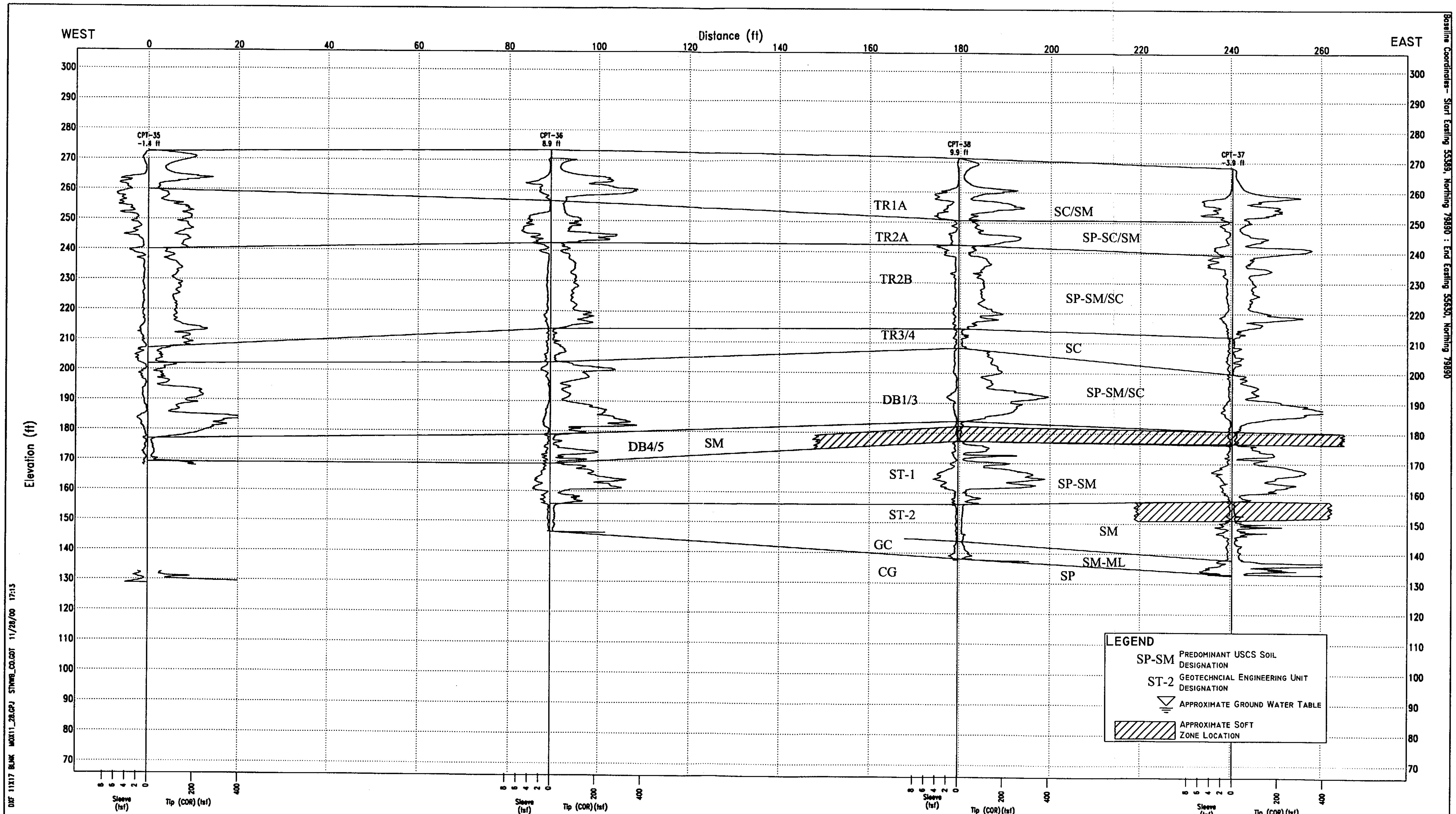
MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

NOTES:
1. Subsurface stratigraphy shown represents a reasonable judgement of the conditions expected based on the data shown. Some variation from these conditions must be expected.
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000).
3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity.
4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.

GEOTECHNICAL CROSS SECTION 5

NOT TO SCALE

FIGURE 5-6



DUKE COGEMA
STONE & WEBSTER

MOX FUEL FABRICATION FACILITY
DOE SAVANNAH RIVER SITE
JOB # 08716

NOTES:
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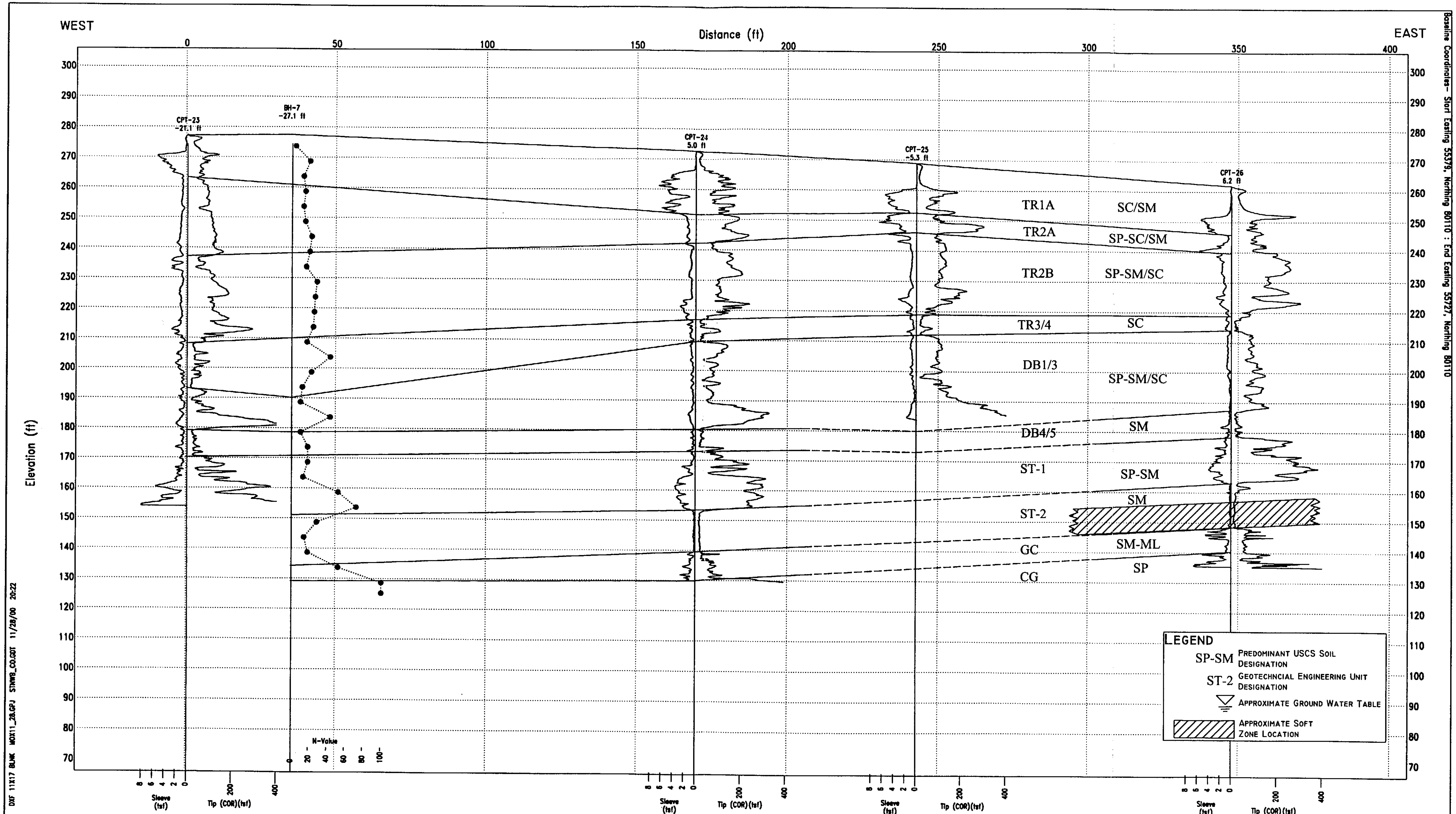
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000).
3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity.
4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.

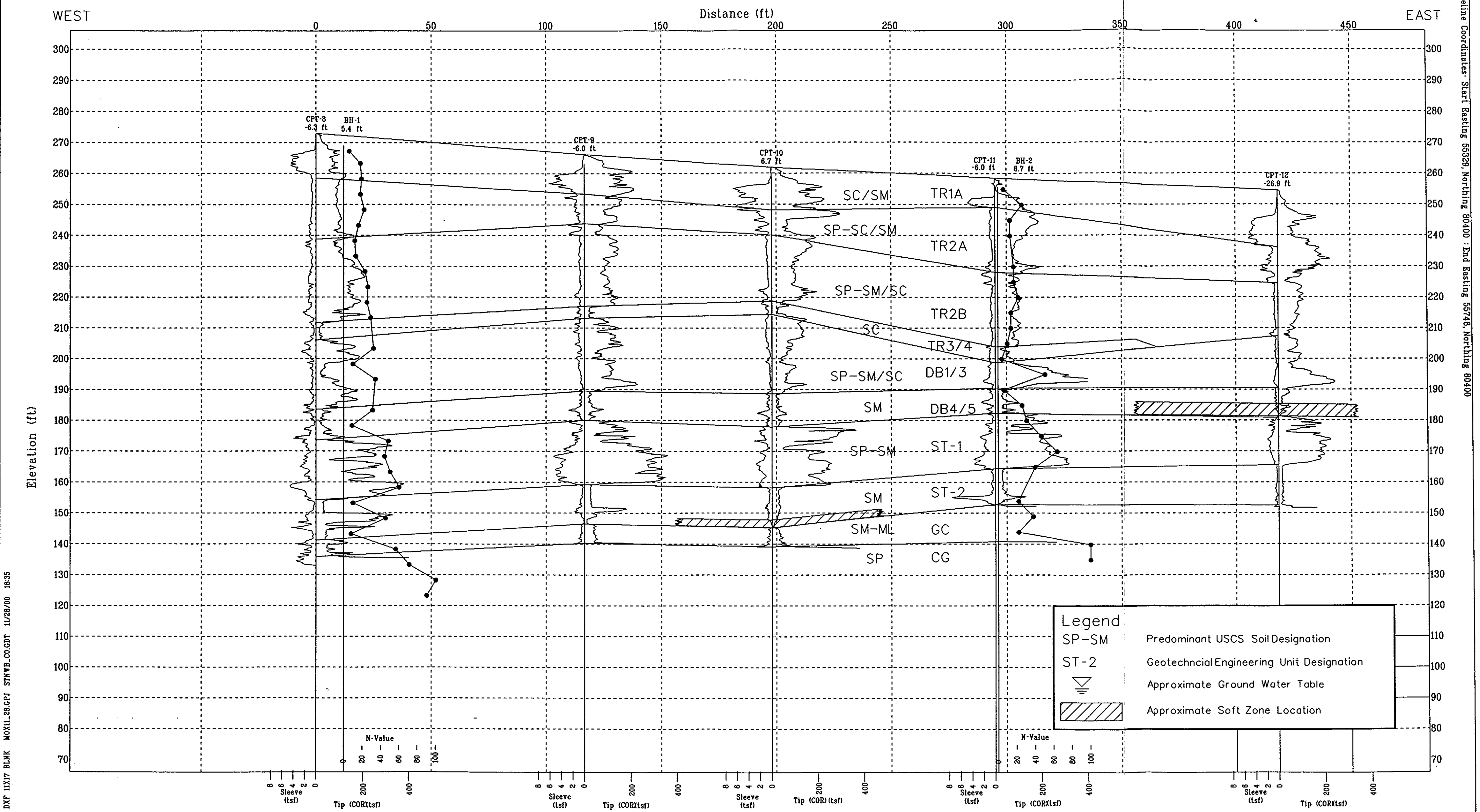
GEOTECHNICAL CROSS SECTION 6

NOT TO SCALE

FIGURE 5-7



DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility DOE Savannah River Site Job # 08716	NOTES: 1. Subsurface stratigraphy shown represents a reasonable judgement of the conditions expected based on the data shown. Some variation from these conditions must be expected. 2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000). 3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity. 4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.	GEOTECHNICAL CROSS SECTION 7 NOT TO SCALE FIGURE 5-8



DUKE COGEMA STONE & WEBSTER

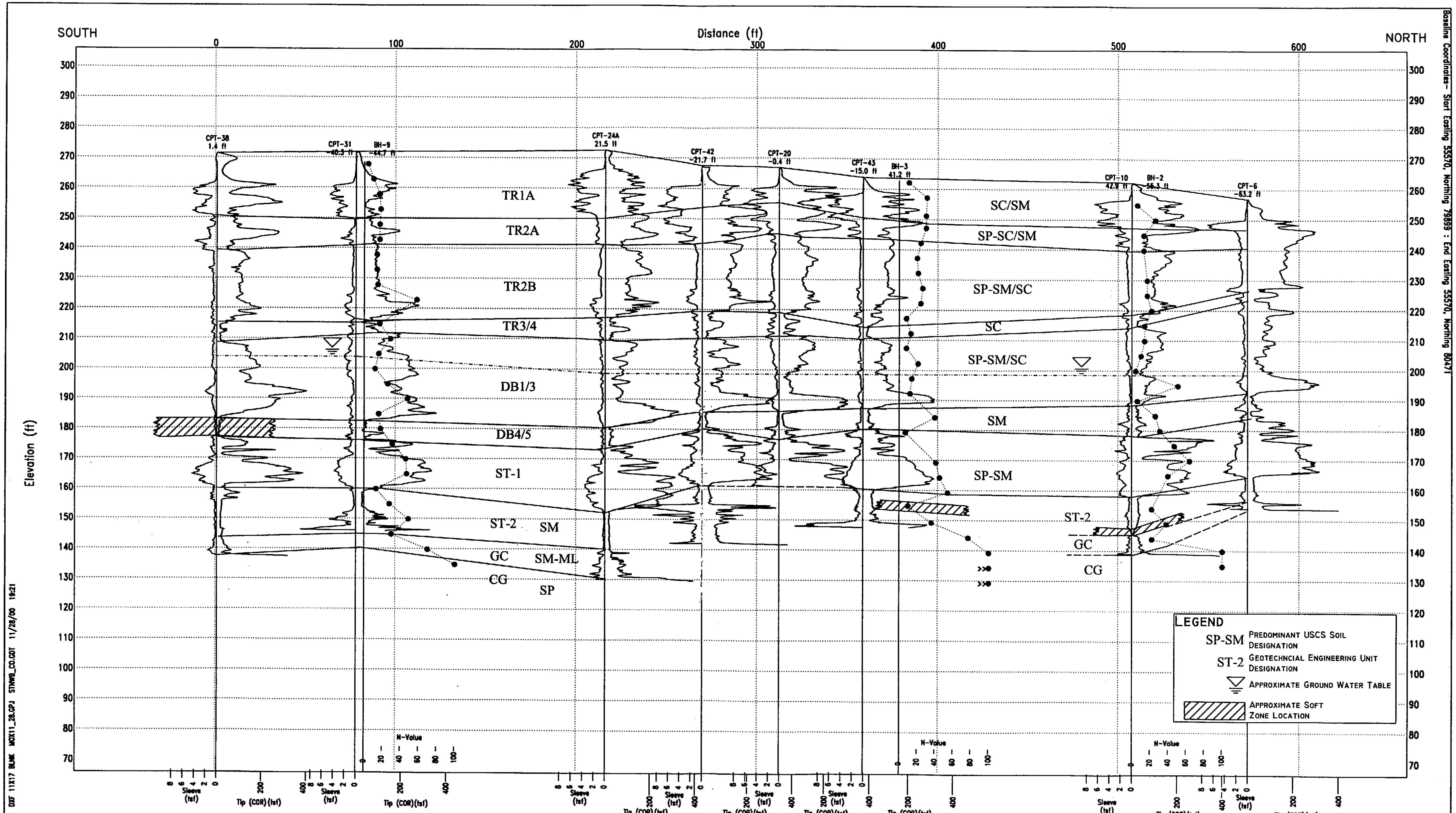
MOX Fuel Fabrication Facility
DOE Savannah River Site
Job # 08716

NOTES:
1. Subsurface stratigrahpy shown represents a resonable judgement of the contitions expected based on the data shown. Some variation from these conditions must be expected.
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000)
3. Some boring and CPT soundings used to develop this cross section are not shown to the maintain clarity.
4. "B- and C-' series borings from 1973 Foundation investigatio for Delta Program, Musser, Rutledge Consulting Engineers

GEOTECHNICAL CROSS SECTION 8

NOT TO SCALE

FIGURE 5-9



DUKE COGEMA STONE & WEBSTER

MOX FUEL FABRICATION FACILITY
DOE SAVANNAH RIVER SITE
JOB # 08716

NOTES:
1. Subsurface stratigraphy shown represents a reasonable judgement of the conditions expected based on the data shown. Some variation from these conditions must be expected.
2. The water table elevation shown represents approximate conditions at the time of the site investigation (June/July 2000).
3. Some borings and CPT soundings used to develop this cross section are not shown to maintain clarity.
4. "B- and C-" series borings from 1973 Foundation Investigation for Delta Program, Mueser, Rutledge Consulting Engineers.

GEOTECHNICAL CROSS SECTION 9

NOT TO SCALE

FIGURE 5-10

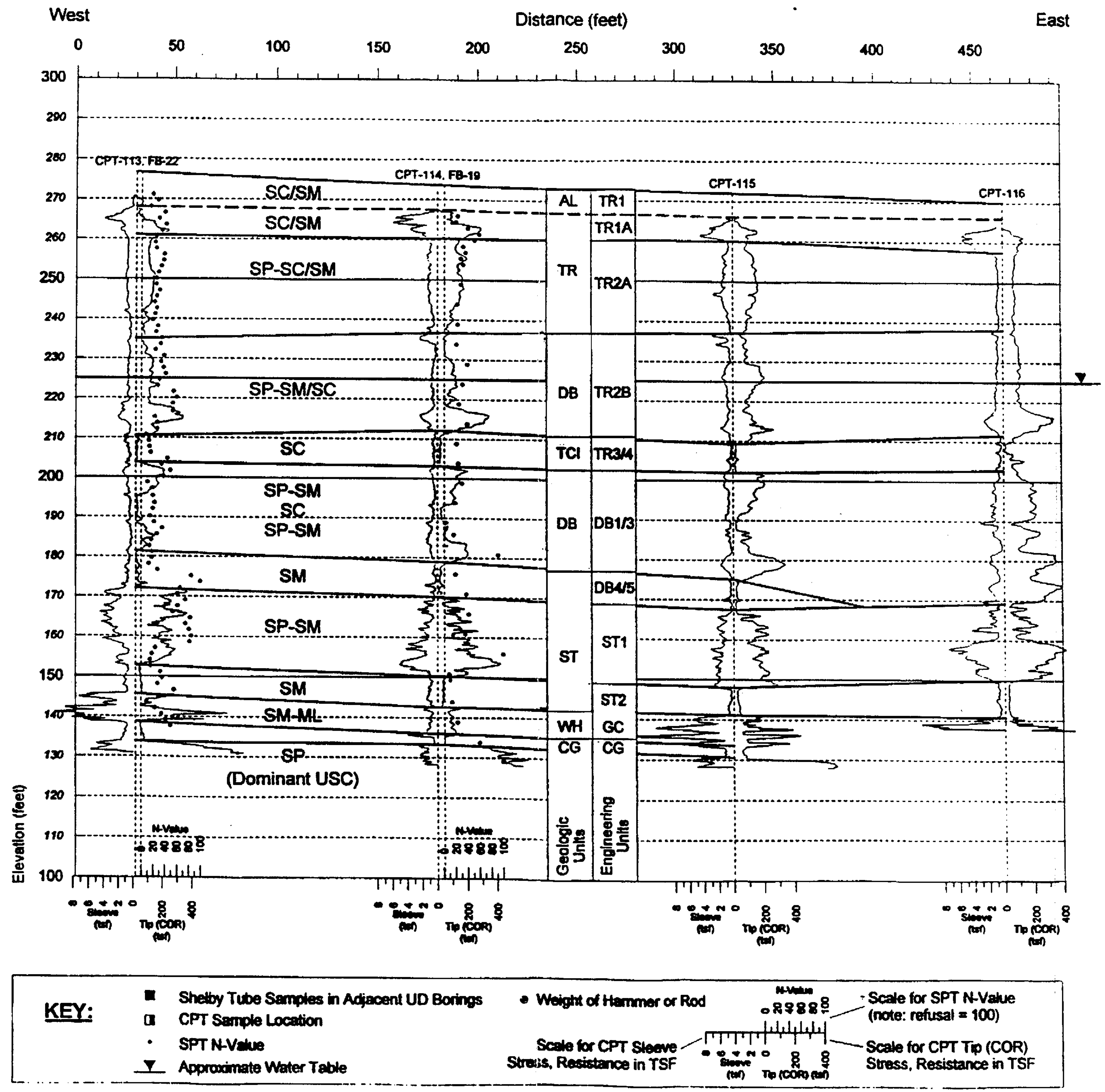


Figure 5-11. Geologic Cross Section 4 (WSRC 1999a)

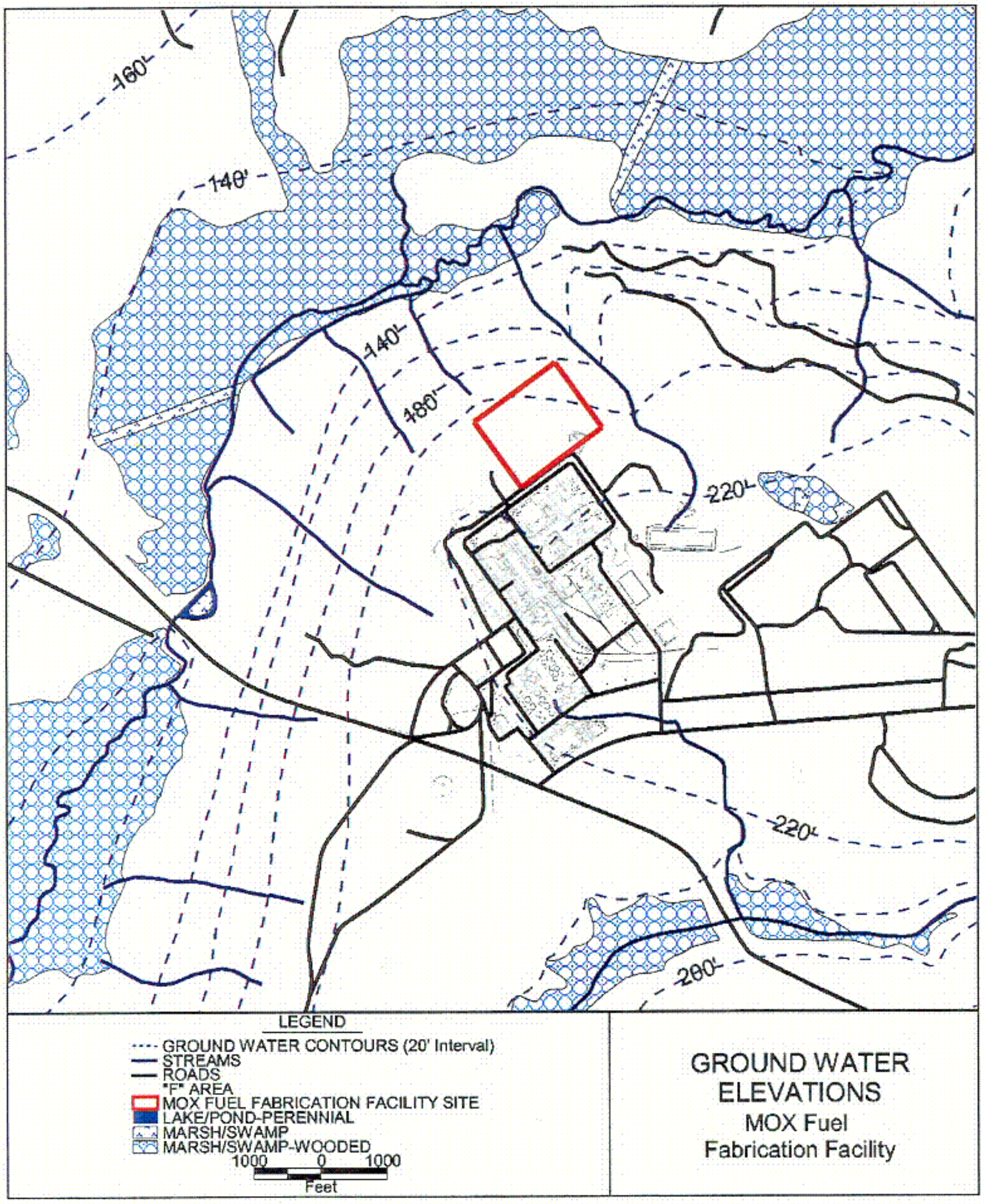


Figure 5-12. Groundwater Elevations in F Area

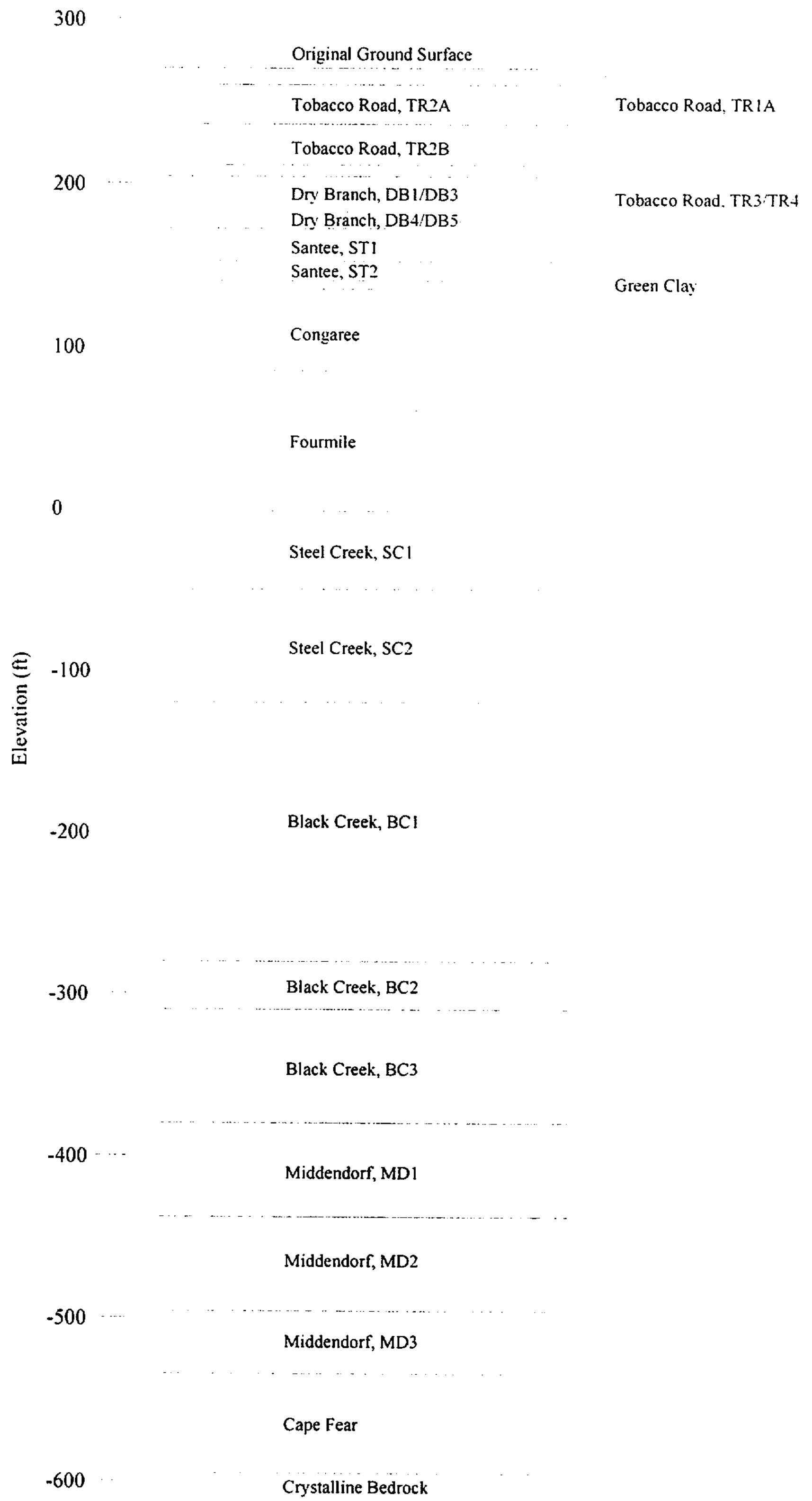
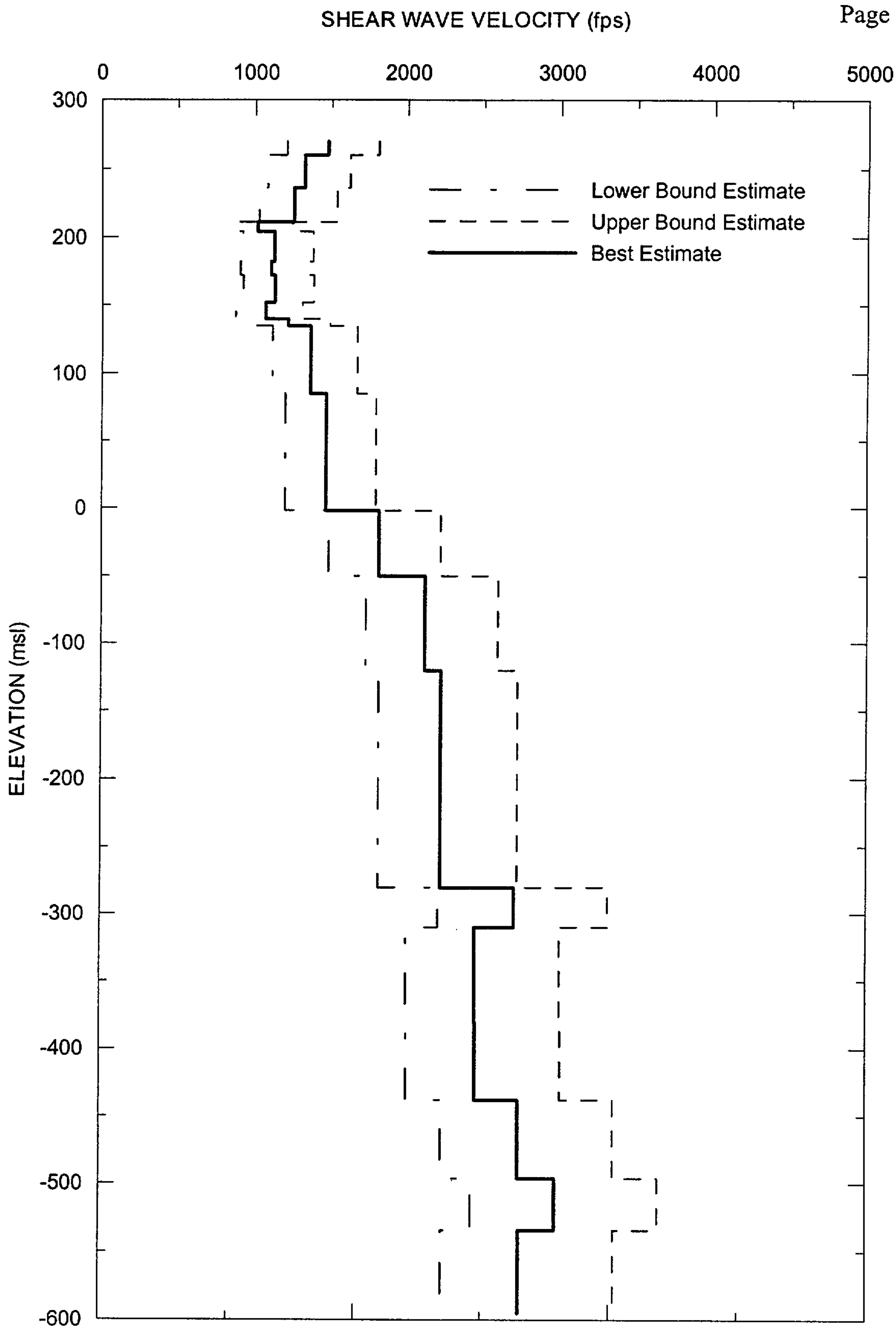


Figure 6-1 Idealized Soil Profile beneath MOX FFF



Note: Bedrock velocity (11,000 fps) not shown

Fig 6-1-B.grf w/Shear_Wave_Values.xls by RCC [8/8/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Idealized Profile of Low-Strain Shear Wave Velocity at MFFF Site	Figure 6-2

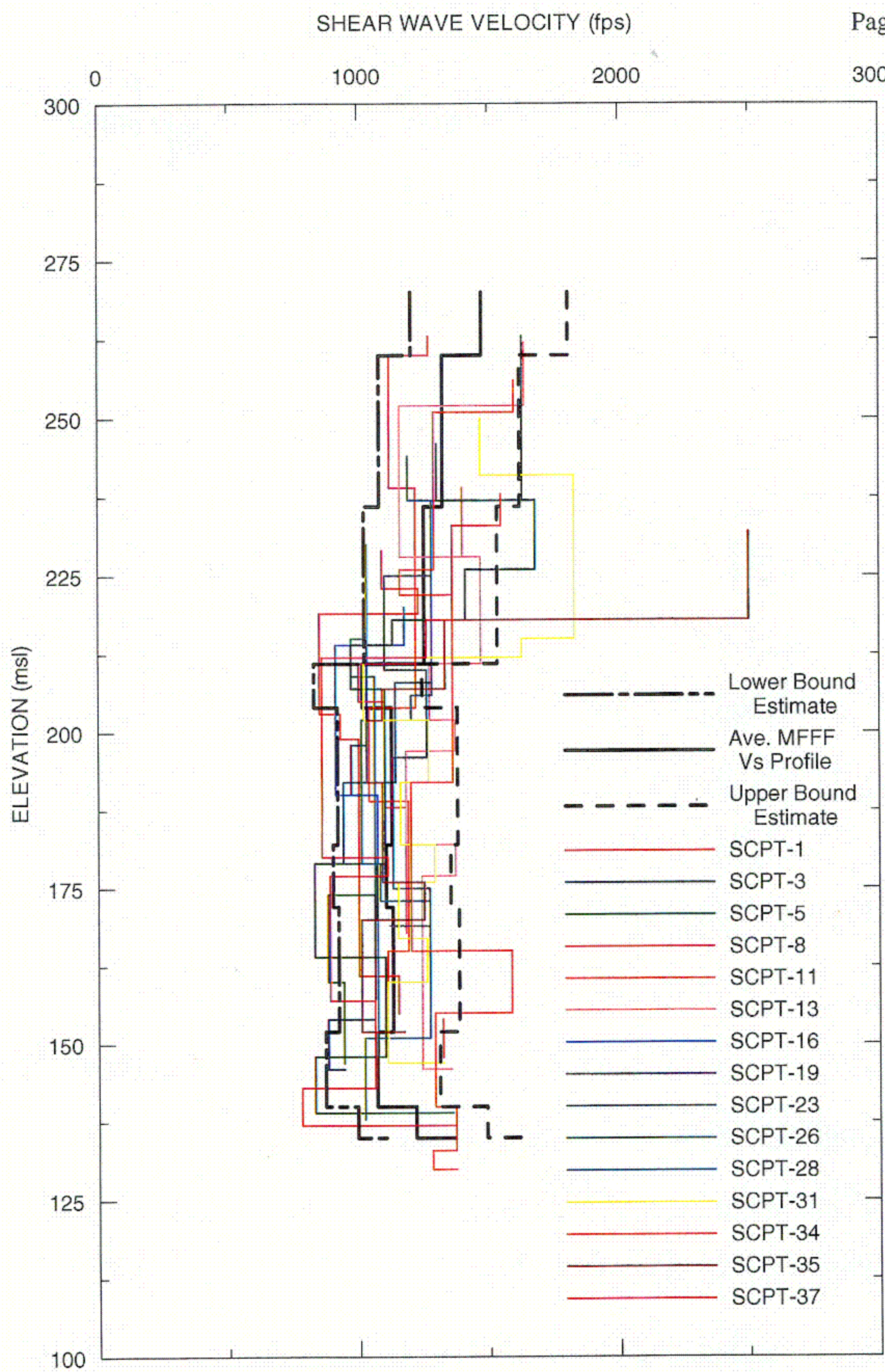


Fig 6-2-B.grf w/Shear_Wave_Values.xls 8/8/01

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Low Strain Shear Wave Velocity by SCPT	Figure 6-3

C-2

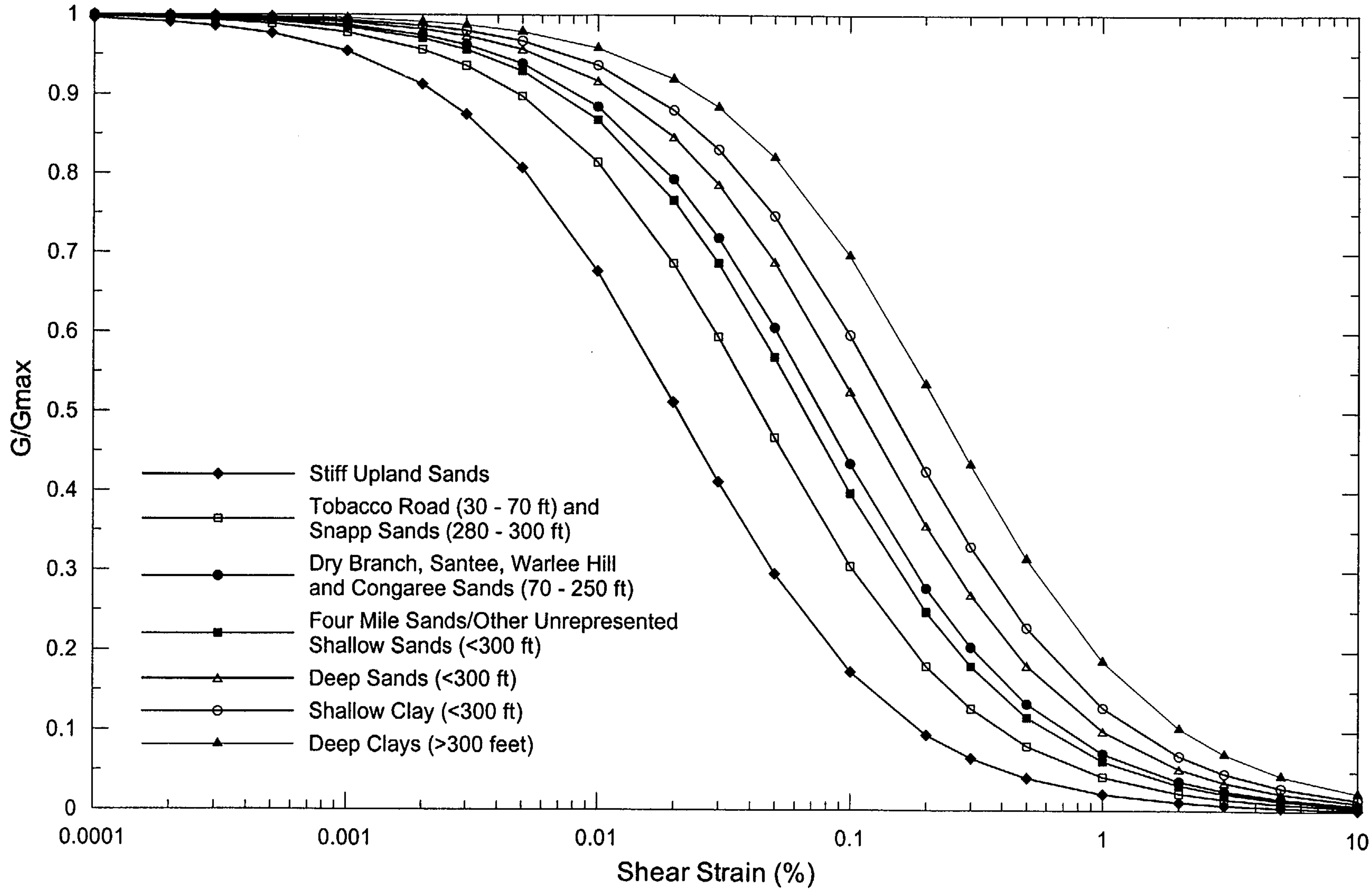


Fig 6-3-B.grf w/Stokoe_G-D_curves.xls by fjw [8/8/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH**	SRS-Recommended Shear Modulus Reduction Curves	Figure 6-4
	J.O. 08716		

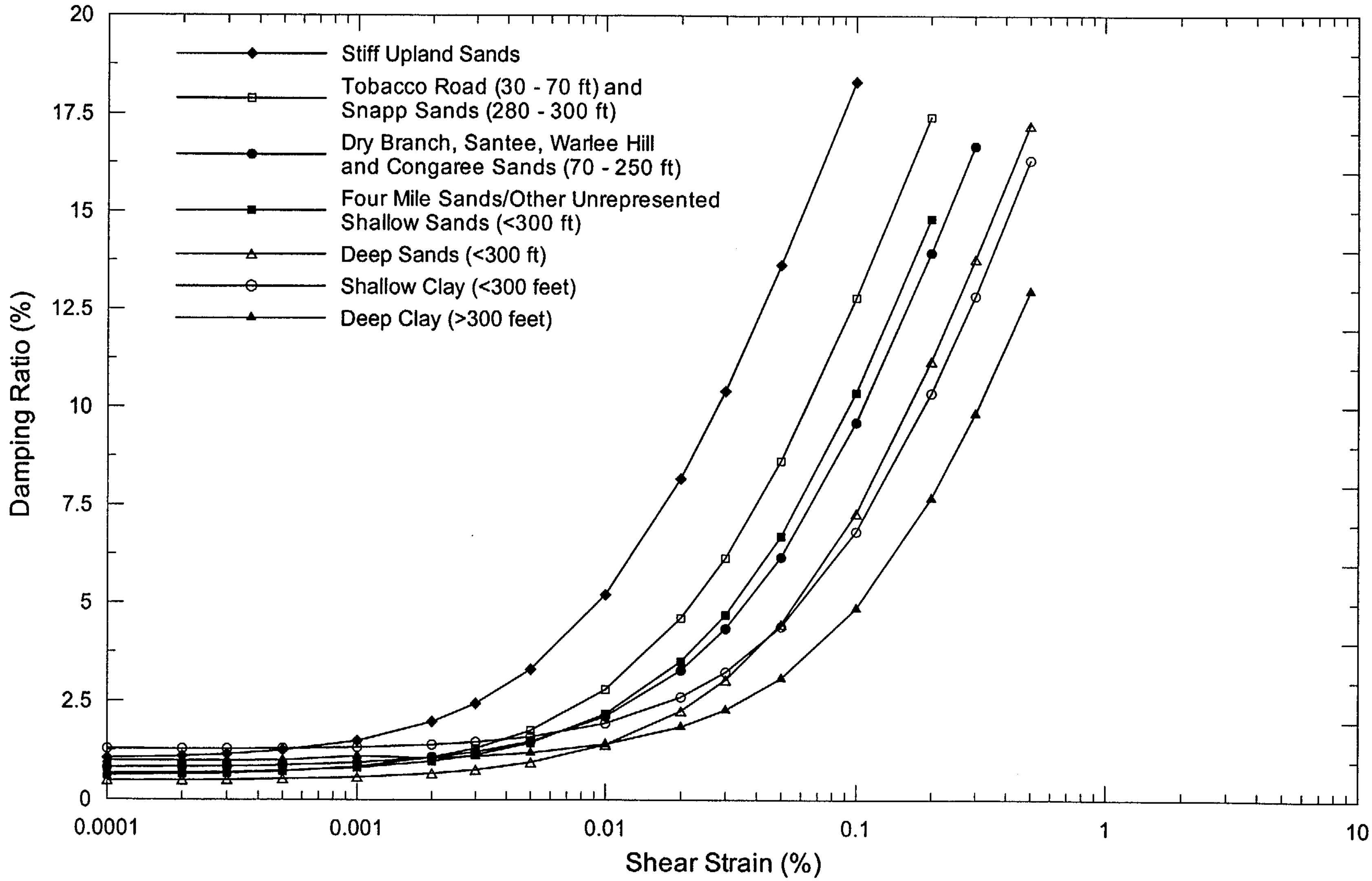


Fig 6-4-B.grf w/Stokoe_G-D_curves.xls by jjt [8/8/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	SRS-Recommended Damping Ratio Curves	Figure 6-5

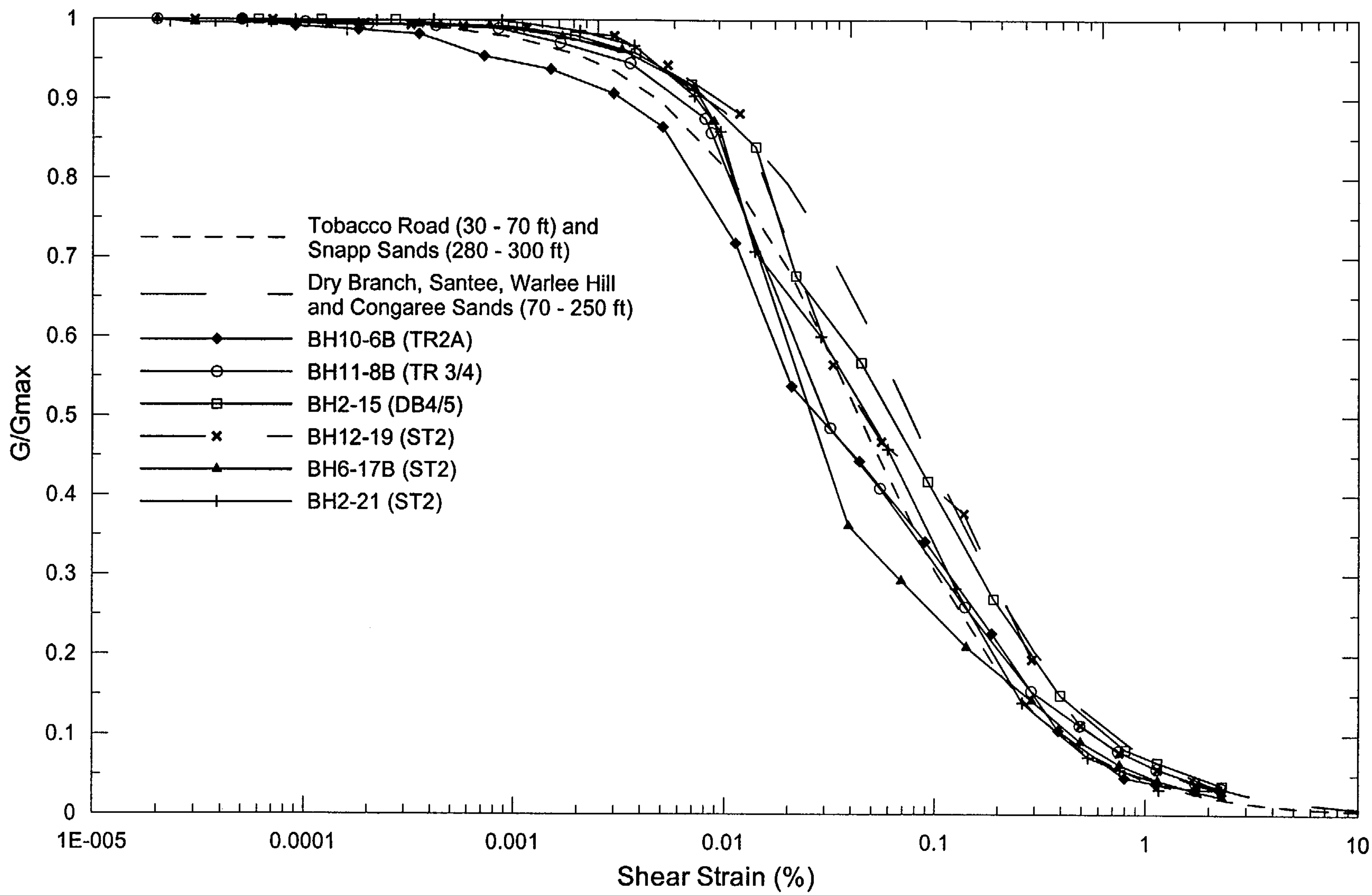
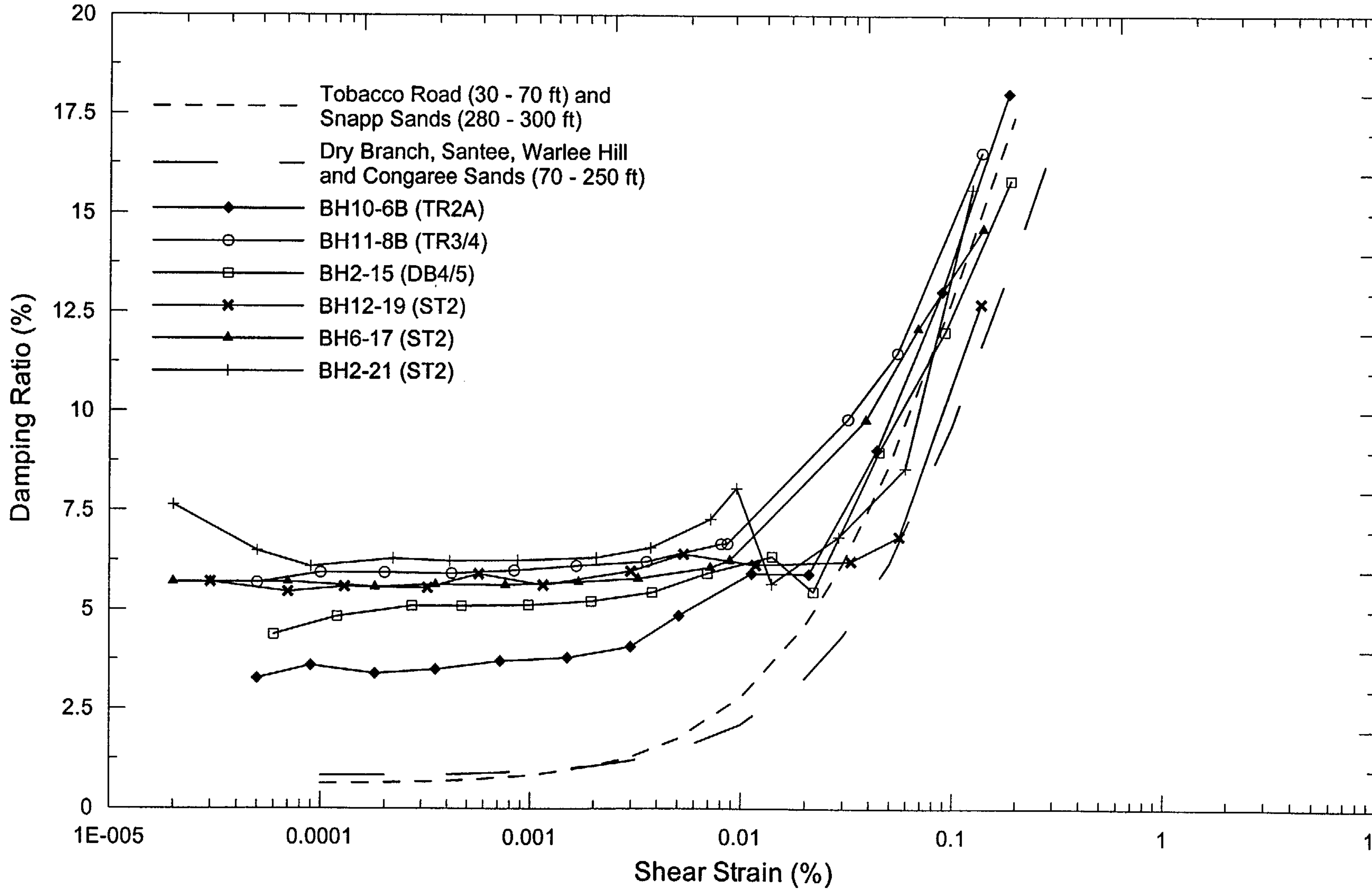


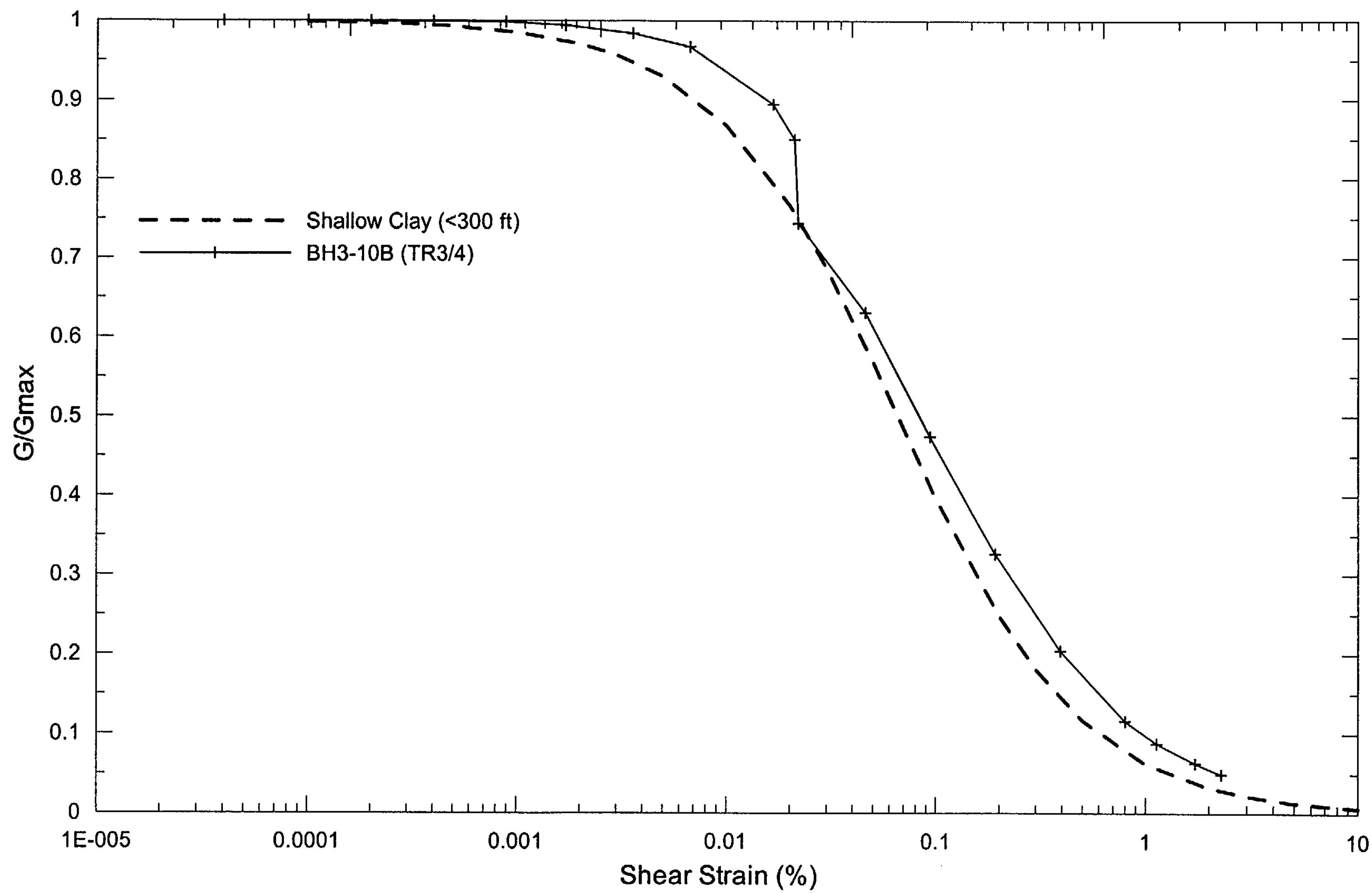
Fig 6-5-B.grf w/MOX_G-D_Curves & Stokoe_G-D_curves.xls by fjw [8/8/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	MFFF Site-Specific and SRS-Recommended Shear Modulus Reduction Curves for Sands	Figure 6-6



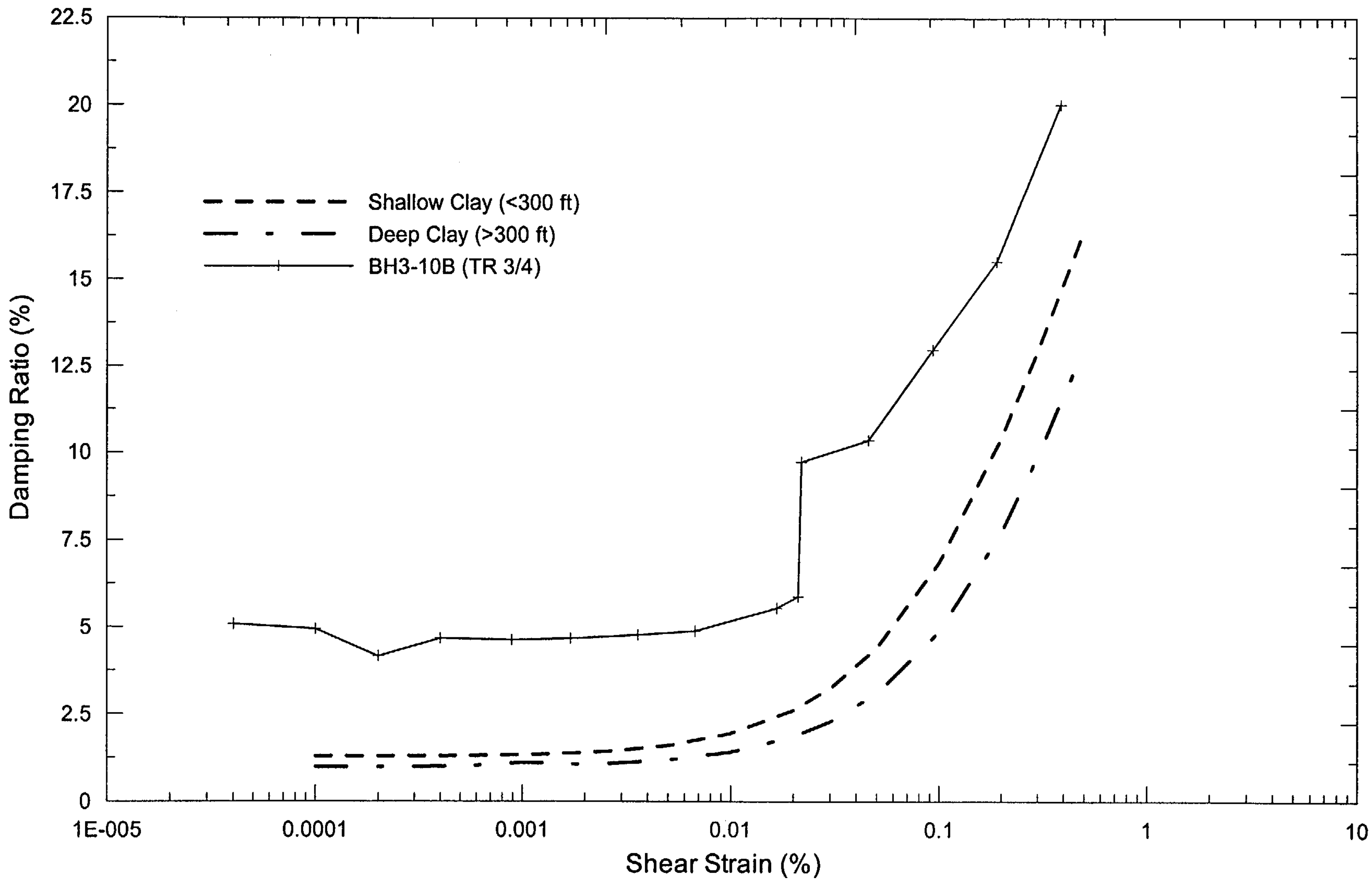
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	MFFF Site-Specific and SRS-Recommended Damping Ratio Curves for Sands	Figure 6-7

6-6-B.grf w/MOX_G-D_Curves & Stokoe_G-D_curves.xls by jjt [8/8/01]



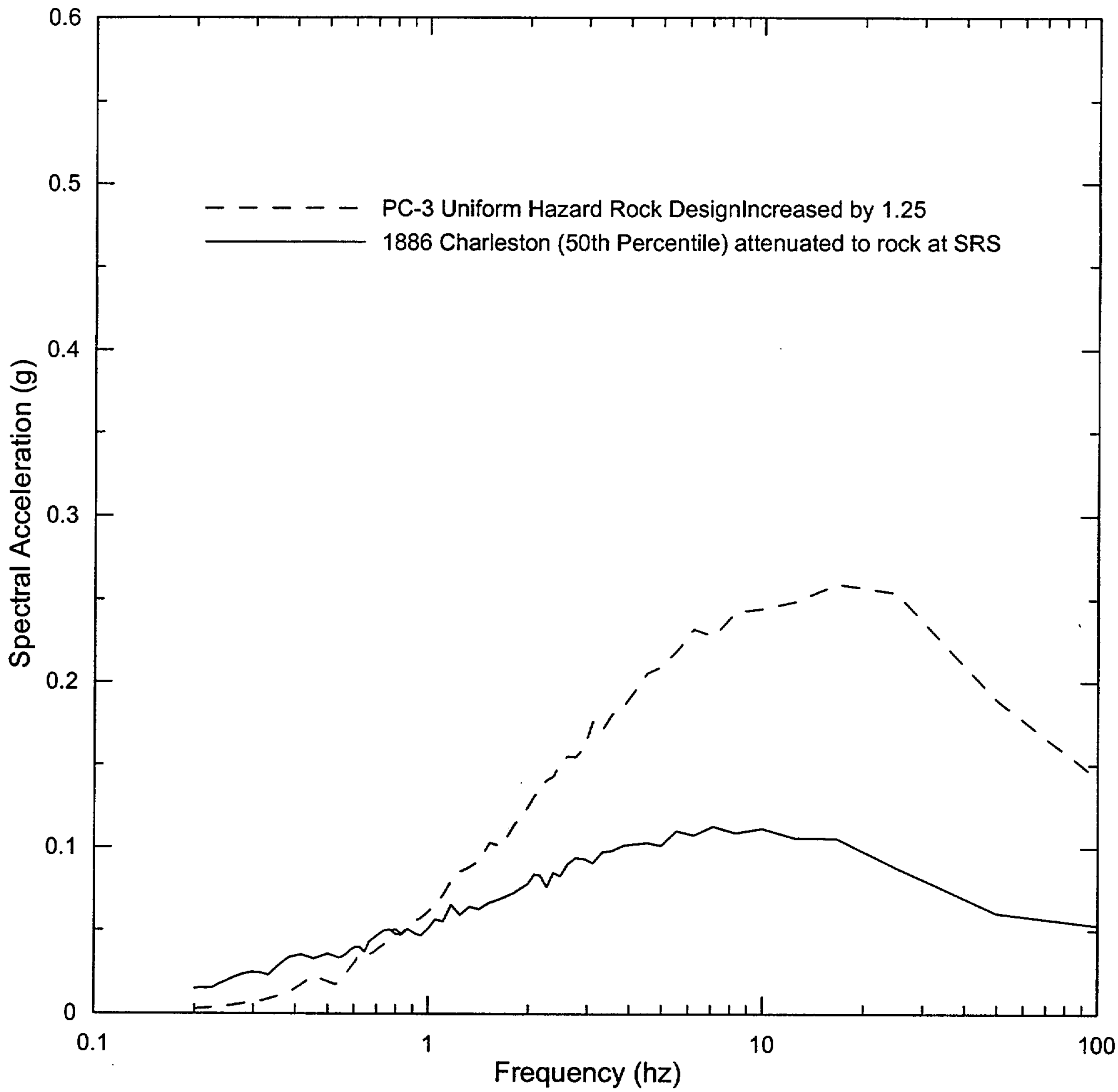
6-7-B.grf w/MOX_G-D_Curves & Stokoe_G-D_curves.xls by fjw [8/8/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH**	MFFF Site-Specific and SRS-Recommended Shear Modulus Reduction Curves for Clays	Figure 6-8
	J.O. 08716		



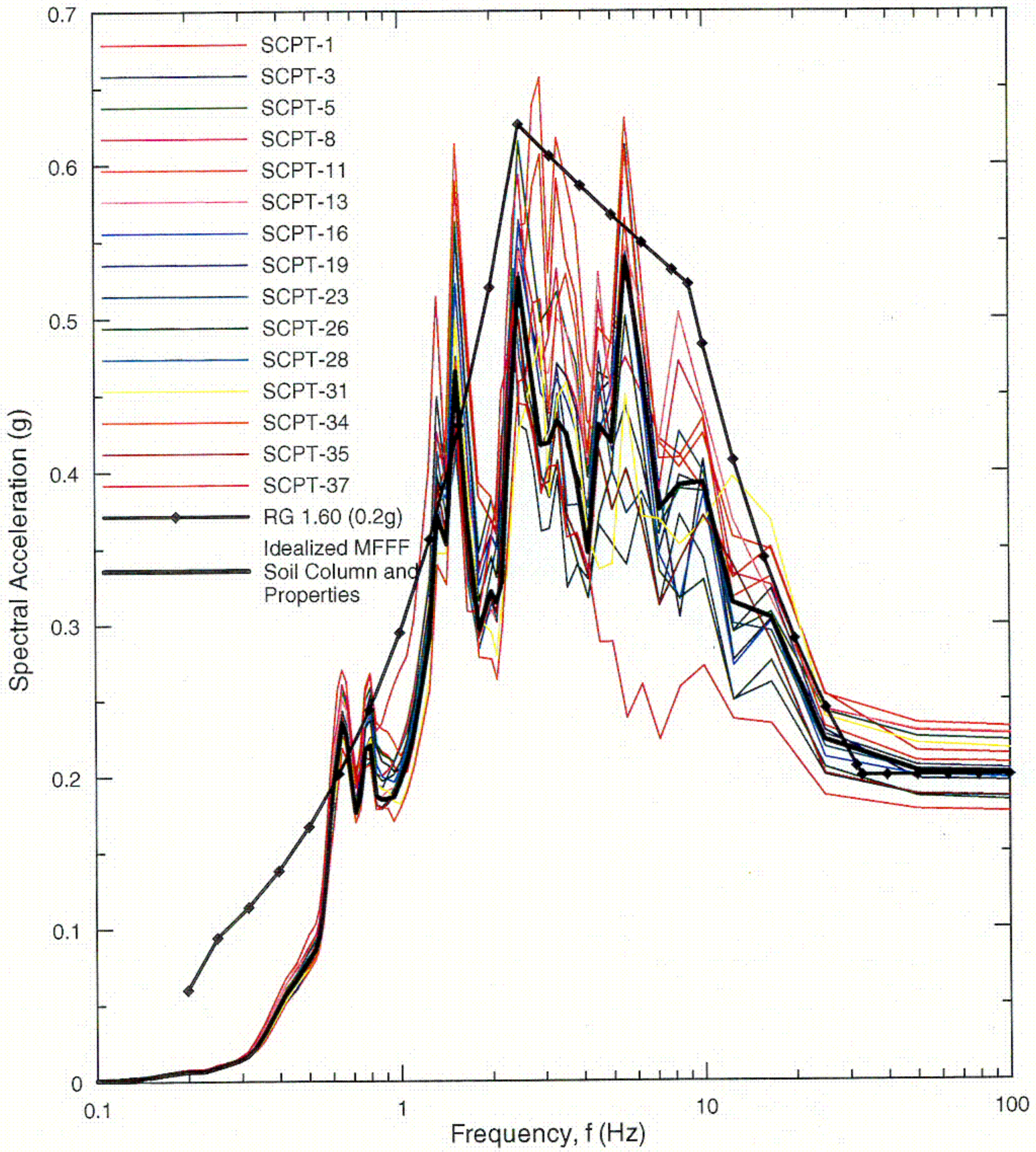
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	MFFF Site-Specific and SRS-Recommended Damping Ratio Curves for Clays	Figure 6-9

6-8-B.grf w/MOX_G-D_Curves & Stokoe_G-D_curves.xls byfjw [8/8/01]



SAvF_P-C.grf w/PC3-ALL & CHASC-ALL..xls by FJW 8/8/01

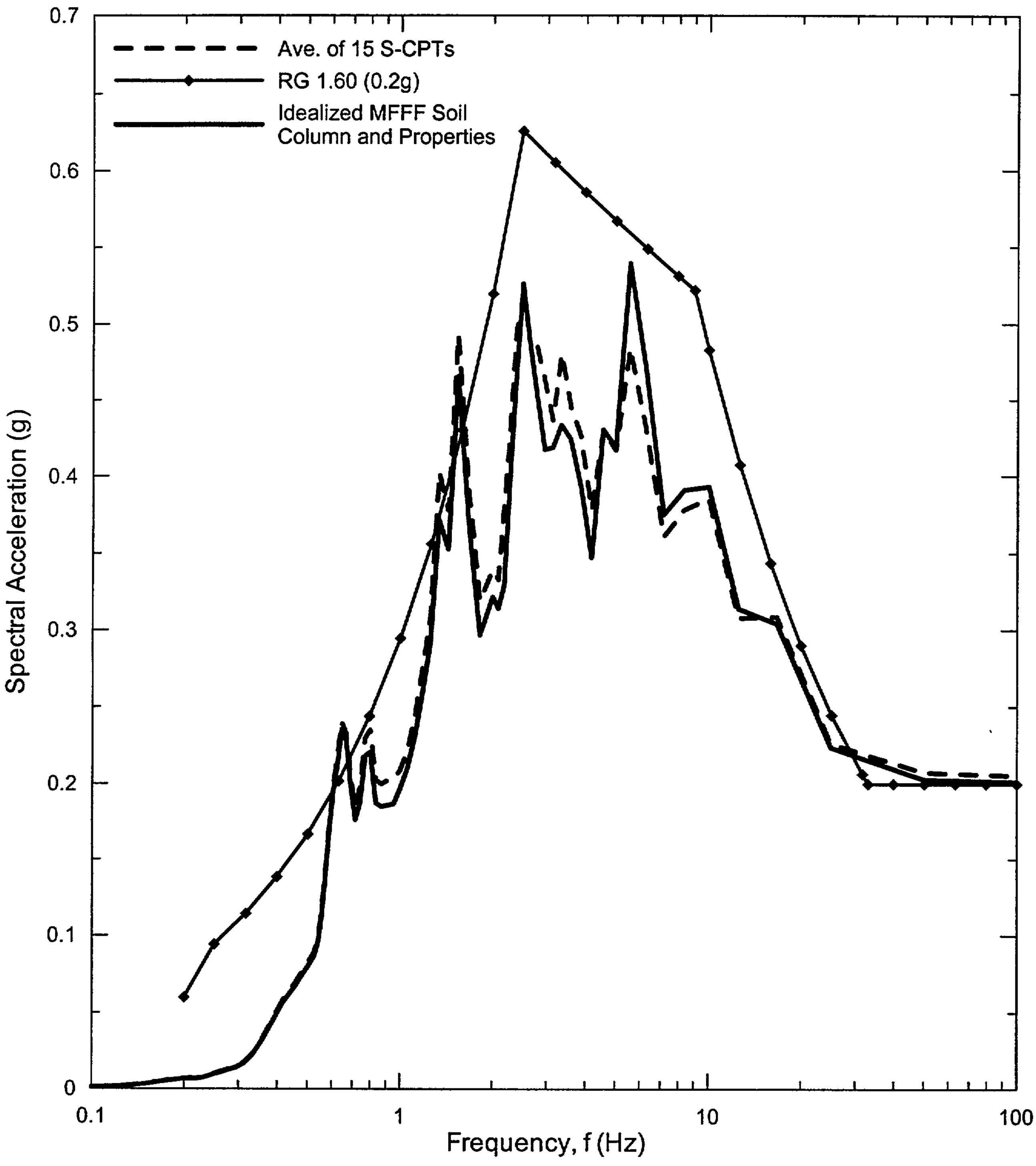
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Response Spectra (Damped 5%) used in MFFF Site Response Analyses	Figure 6-10



SAvT_comp-1-P.GRF w/PC3ALL.xls by JJT [08/08/01]

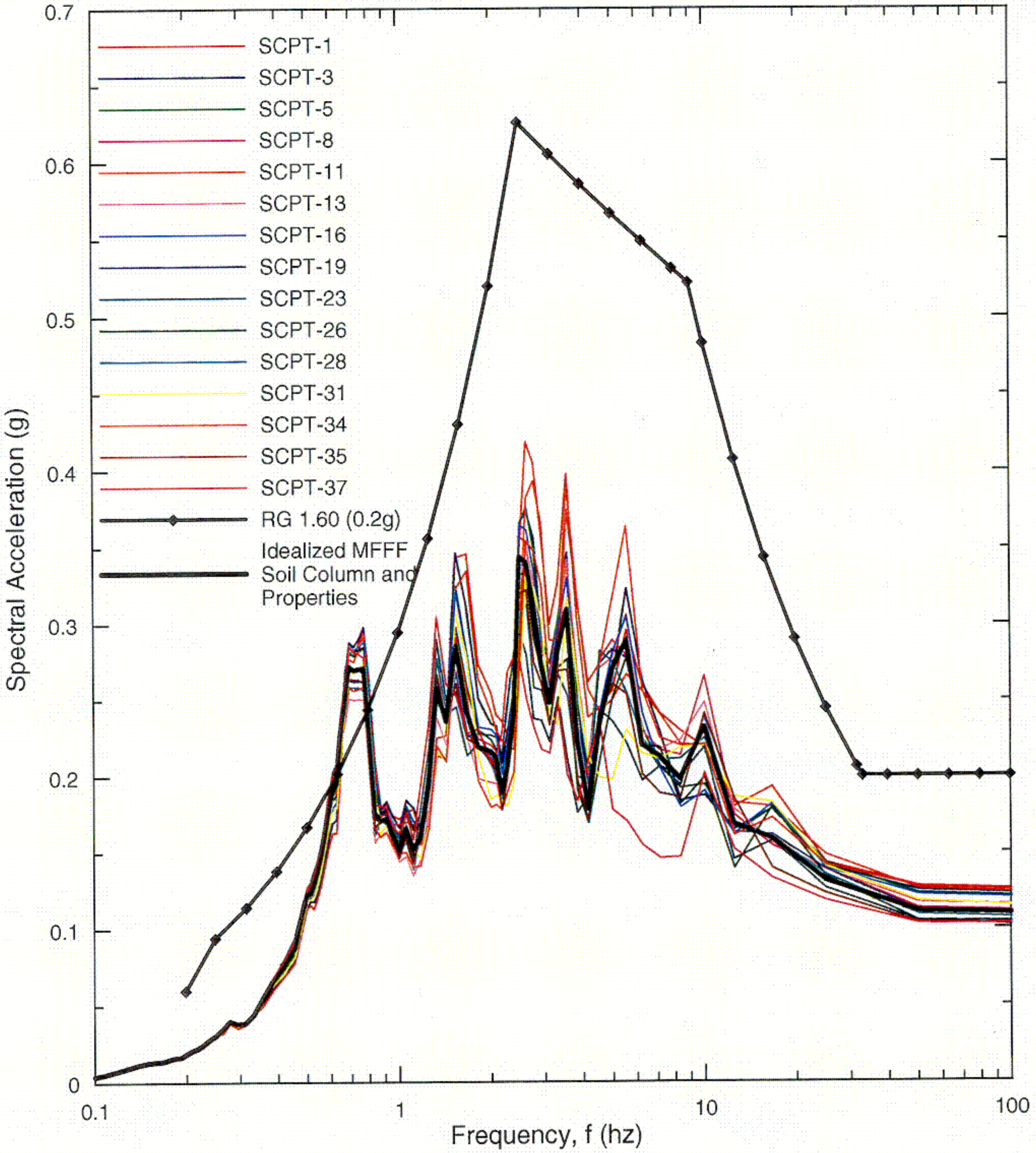
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Comparison of Surface Response Spectra (5% Damped) for 15 SCPTs PC-3 Motion Scaled up 1.25	Figure 6-11

C-4



SAvT_comp-2-P.GRF w/PC3ALL.xls by JJT [08/08/01]

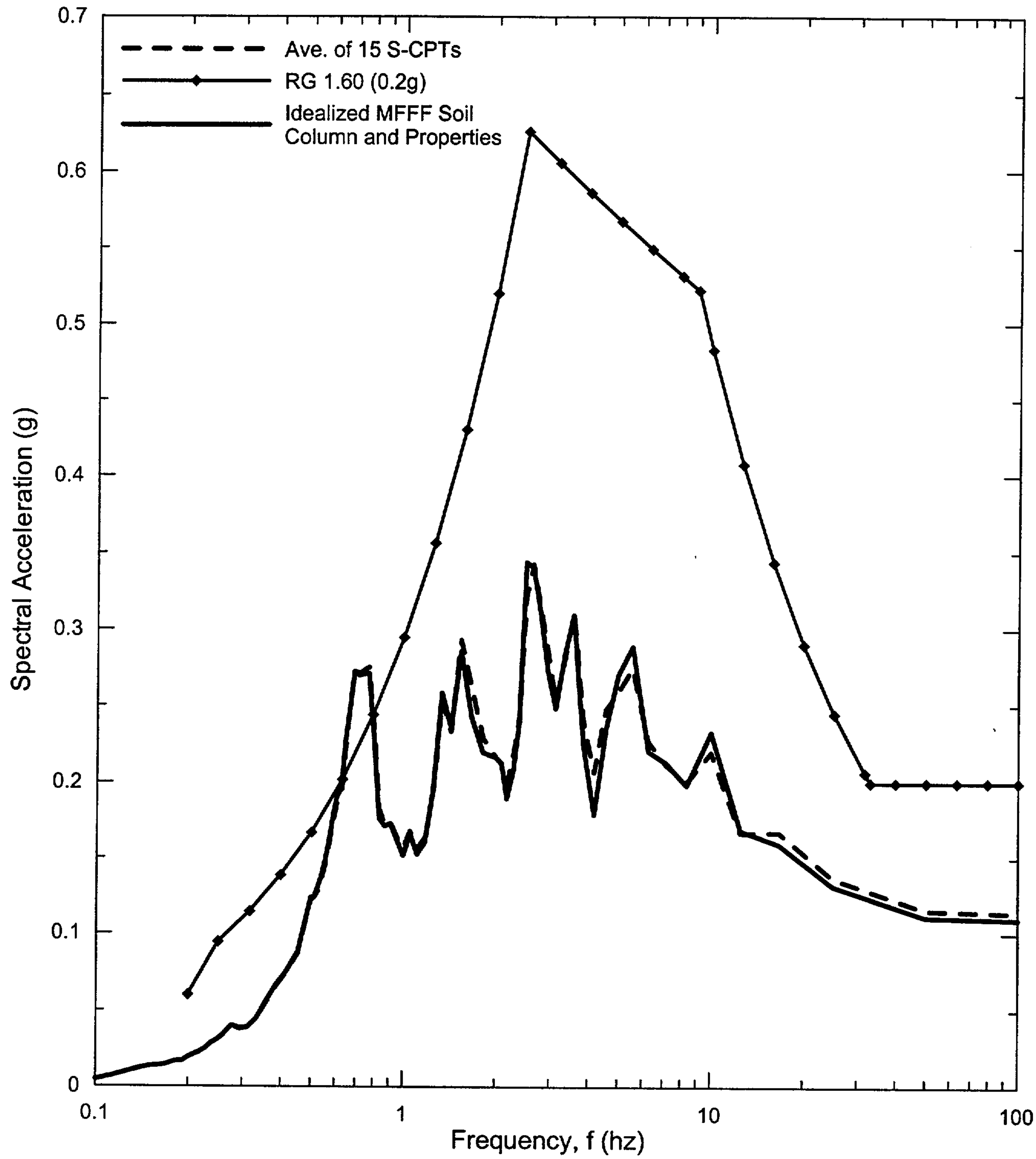
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Comparison of Surface Response Spectra (5% Damped) for Ave. of 15 SCPTs PC-3 Motion Scaled up 1.25	Figure 6-12



SAvT_comp-1C.GRF w/CHASC-ALL .xls by FJW [08/08/01]

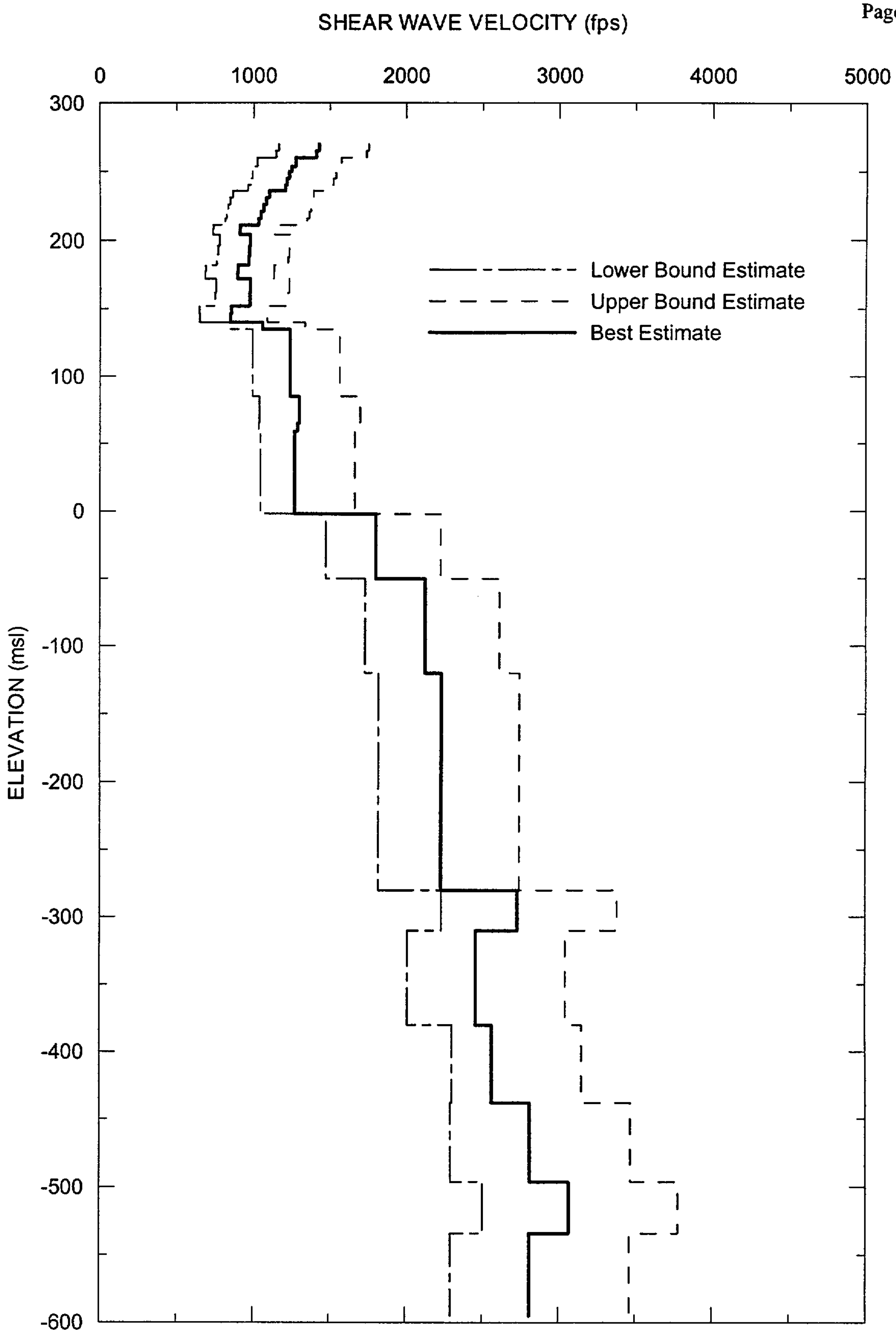
DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Comparison of Surface Response Spectra (5% Damped) for 15 SCPTs 1886 Charleston 50th Percentile Motion	Figure 6-13

L-5



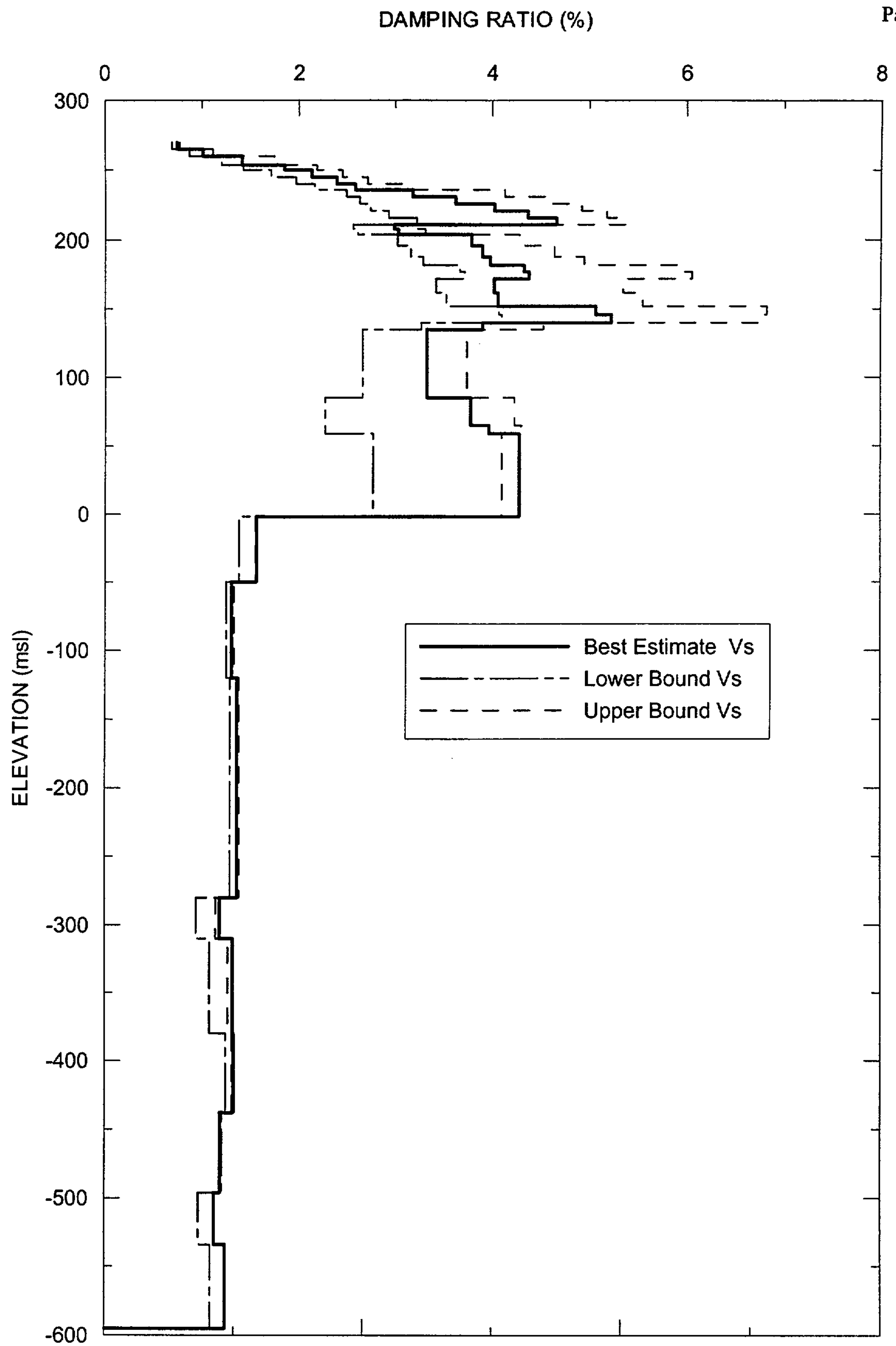
SAvT_comp-2-C.GRF w/CHASC-ALL .xls by FJW [08/08/01]

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Comparison of Surface Response Spectra (5% Damped) for Ave. of 15 SCPTs 1886 Charleston 50th Percentile Motion	Figure 6-14



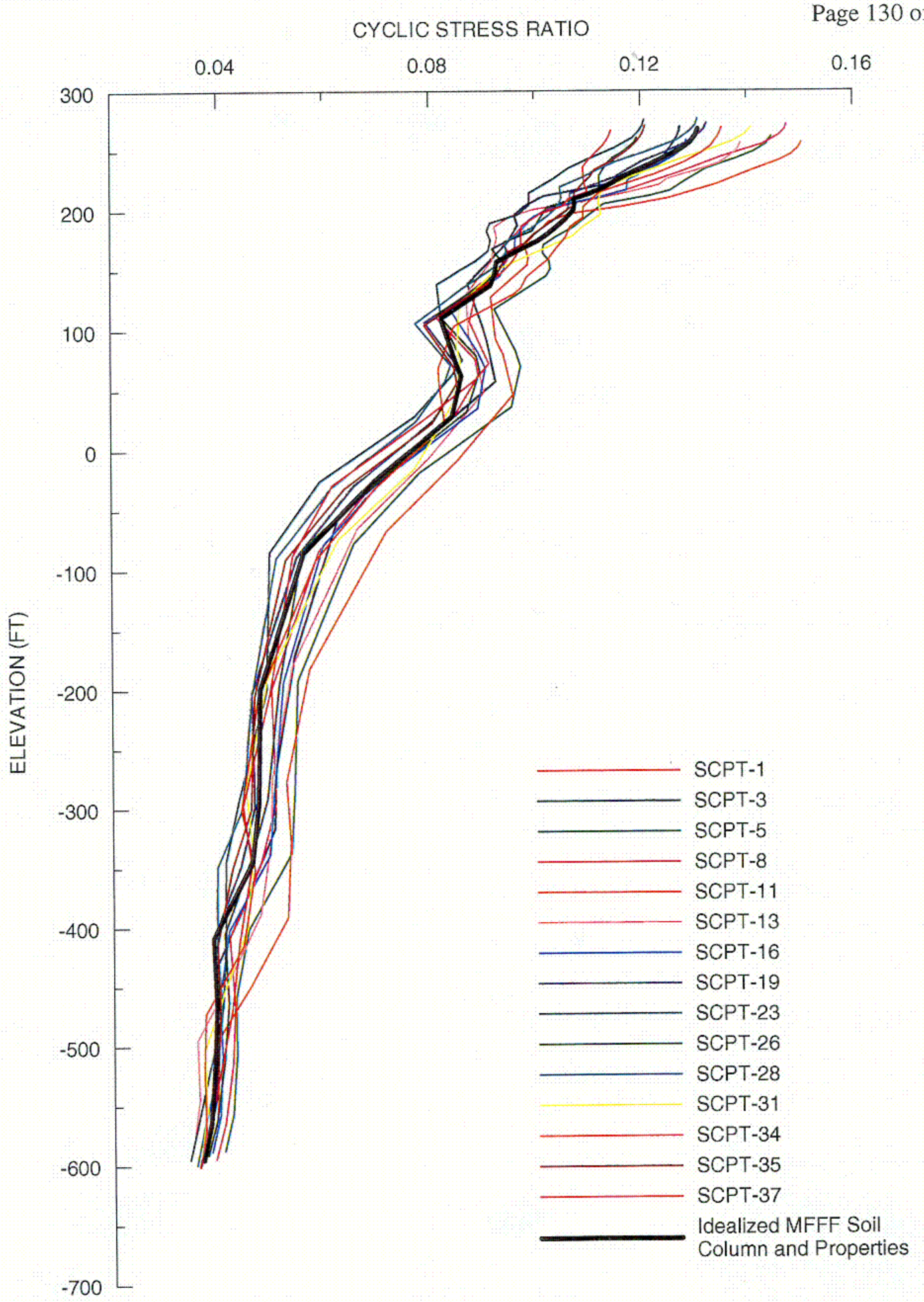
Strain-Comp_Vs-Depth.GRFw/Shear_Wave_Values.xls by RCC [8/8/01]

J.O. No. 08716	MOX Fuel Fabrication Facility **Duke Cogema Stone & Webster**	Profiles of Strain-Compatible Shear Wave Velocities for Idealized MFFF Soil Column	Figure 6-15
Stone & Webster			



\SHAKE\Damp_Values.GRFw/Damp_Values.xls by RCC [8/8/01]

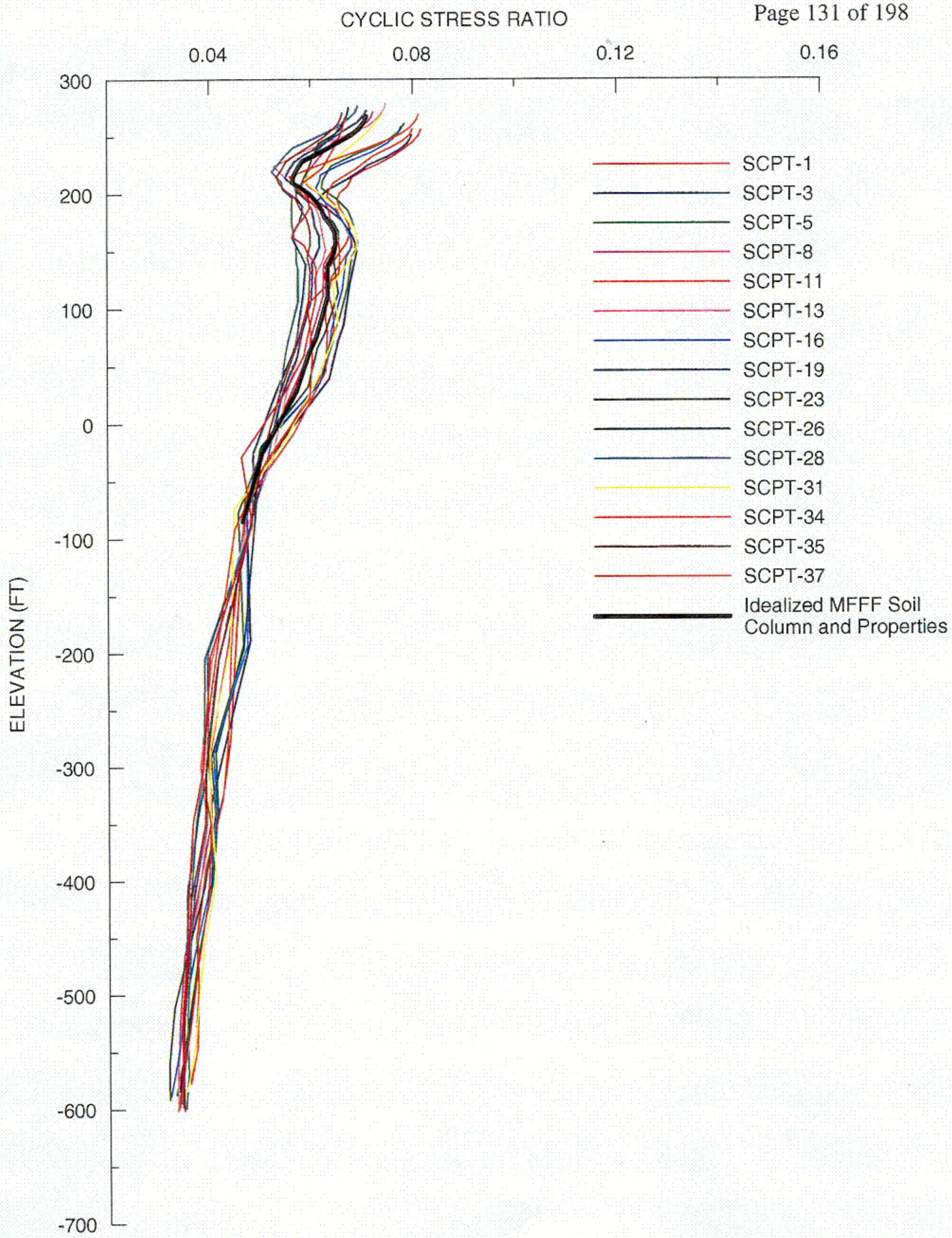
J.O. No. 08716	MOX Fuel Fabrication Facility **Duke Cogema Stone & Webster**	Profiles of Damping Ratio for Idealized MFFF Soil Column	Figure 6-16
Stone & Webster			



CSRvDpt-P.grf w\PC3-ALL.xls by RCC 8/8/01

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Cyclic Stress Ratio vs Depth for 15 SCPTs PC-3 Rock Motion Scaled Up By 1.25	Figure 6-17

C-6



CSRvDpt-C.grf w/CHASC-ALL.xls by fjw 8/8/01

DUKE COGEMA STONE & WEBSTER	MOX Fuel Fabrication Facility **US Department of Energy CH** J.O. 08716	Cyclic Stress Ratio vs Depth for 15 SCPTs 1886 Charleston 50th Percentile	Figure 6-18

C-7

Figure 7-1. MFFF Site - Section 1 Difference Between Load and No Load Cases

FLAC (Version 4.00)

LEGEND

11-Jul-01 8:34
step 10
-1.506E+01 <x< 2.861E+02
-1.001E+02 <y< 2.011E+02

NO LOAD CASE - 10' STRUCTURAL FILL

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1785 ft (2.14")
BEST ESTIMATE PROPERTIES

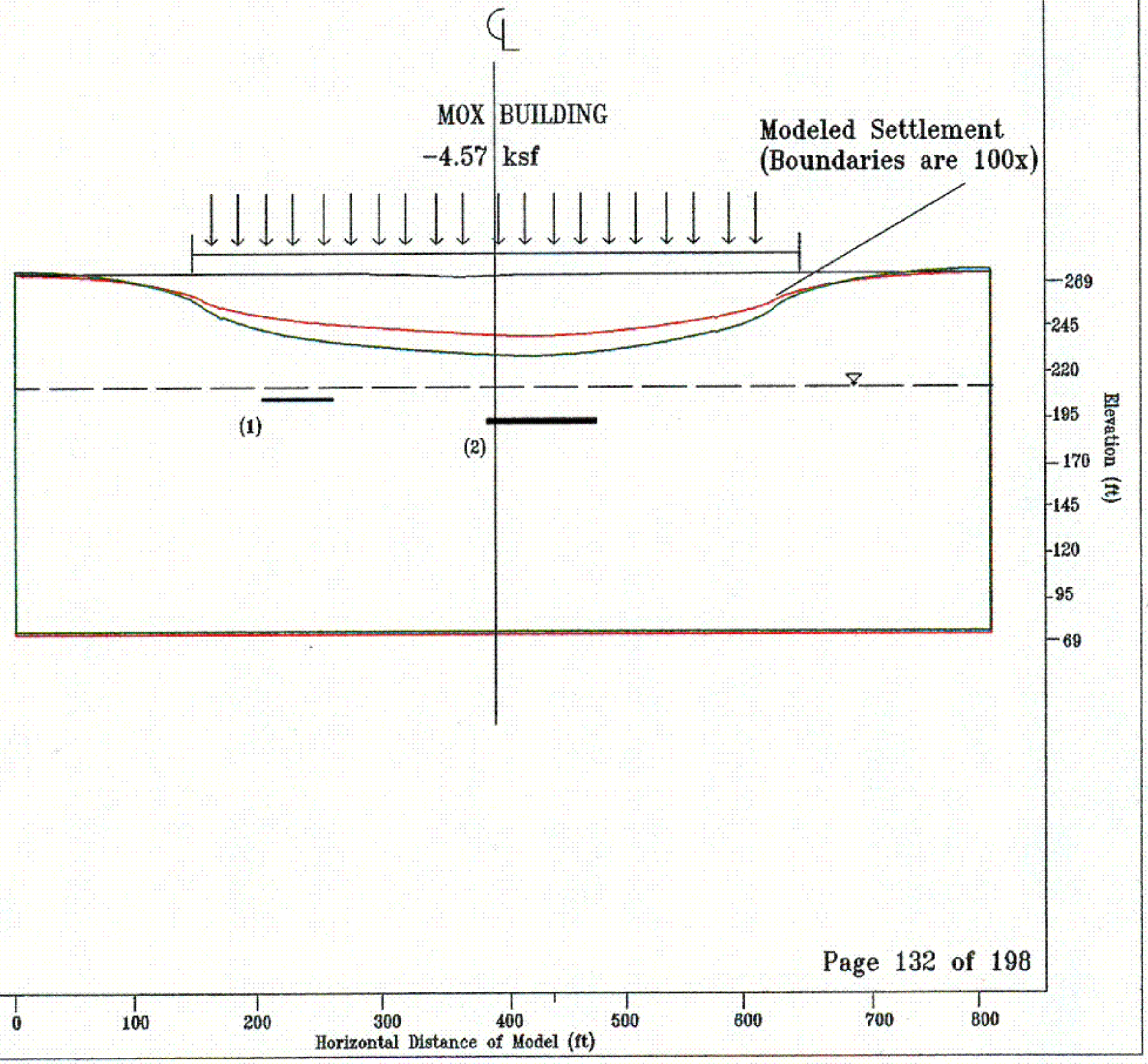
Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.2411 ft (2.89")
LOWER BOUND PROPERTIES

Soft Material Location

Water Table at El. 210'

Duke Cogema Stone & Webster

FILENAME:
figure7-1.dwg



L-8

Figure 7-2. MFFF Site - Section 1 Settlement Analysis BEST ESTIMATE VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

12-Jul-01 7:21
step 10
-1.506E+01 <x< 2.861E+02
-1.001E+02 <y< 2.011E+02

Y-displacement contours (ft)
-1.75E-01
-1.50E-01
-1.25E-01
-1.00E-01
-7.50E-02
-5.00E-02
-2.50E-02
0.00E+00

Contour interval= 2.50E-02 (ft)

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1785 ft (2.14")

Soft Material Location

Water Table at El. 210'

Duke Cogema Stone & Webster

FILENAME:
figure7-2.dwg

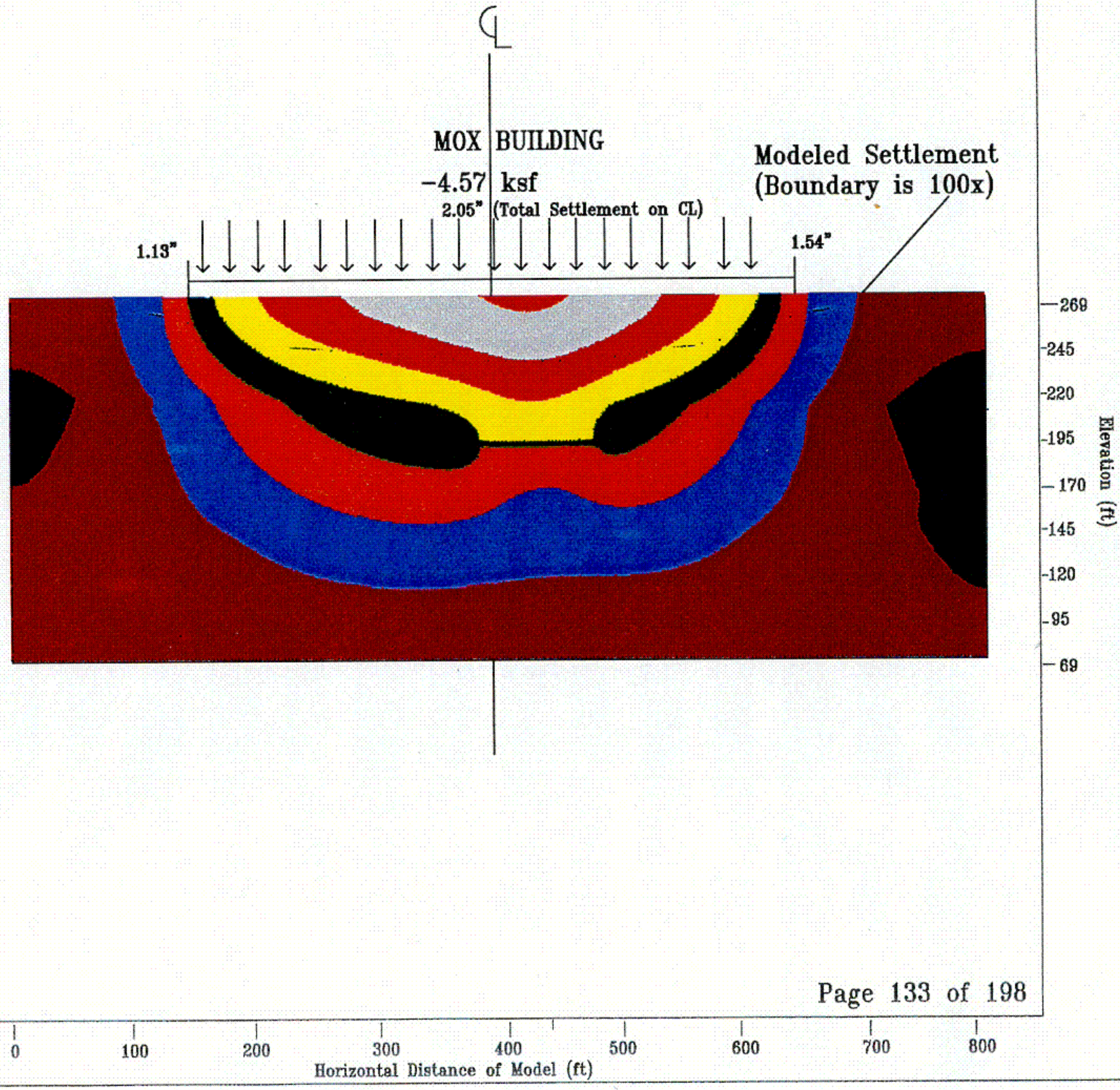


Figure 7-3. MFFF Site - Section 1 Settlement Analysis LOWER BOUND VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

13-Jul-01 8:53
step 10
-1.506E+01 <x< 2.861E+02
-1.001E+02 <y< 2.011E+02

Y-displacement contours (ft)
-2.00E-01
-1.50E-01
-1.00E-01
-5.00E-02
0.00E+00

Contour interval= 5.00E-02 (ft)

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.2411 ft (2.89")

Soft Material Location

Water Table at El. 210'

C-10

Duke Cogema Stone & Webster

FILENAME:
figure7-3.dwg

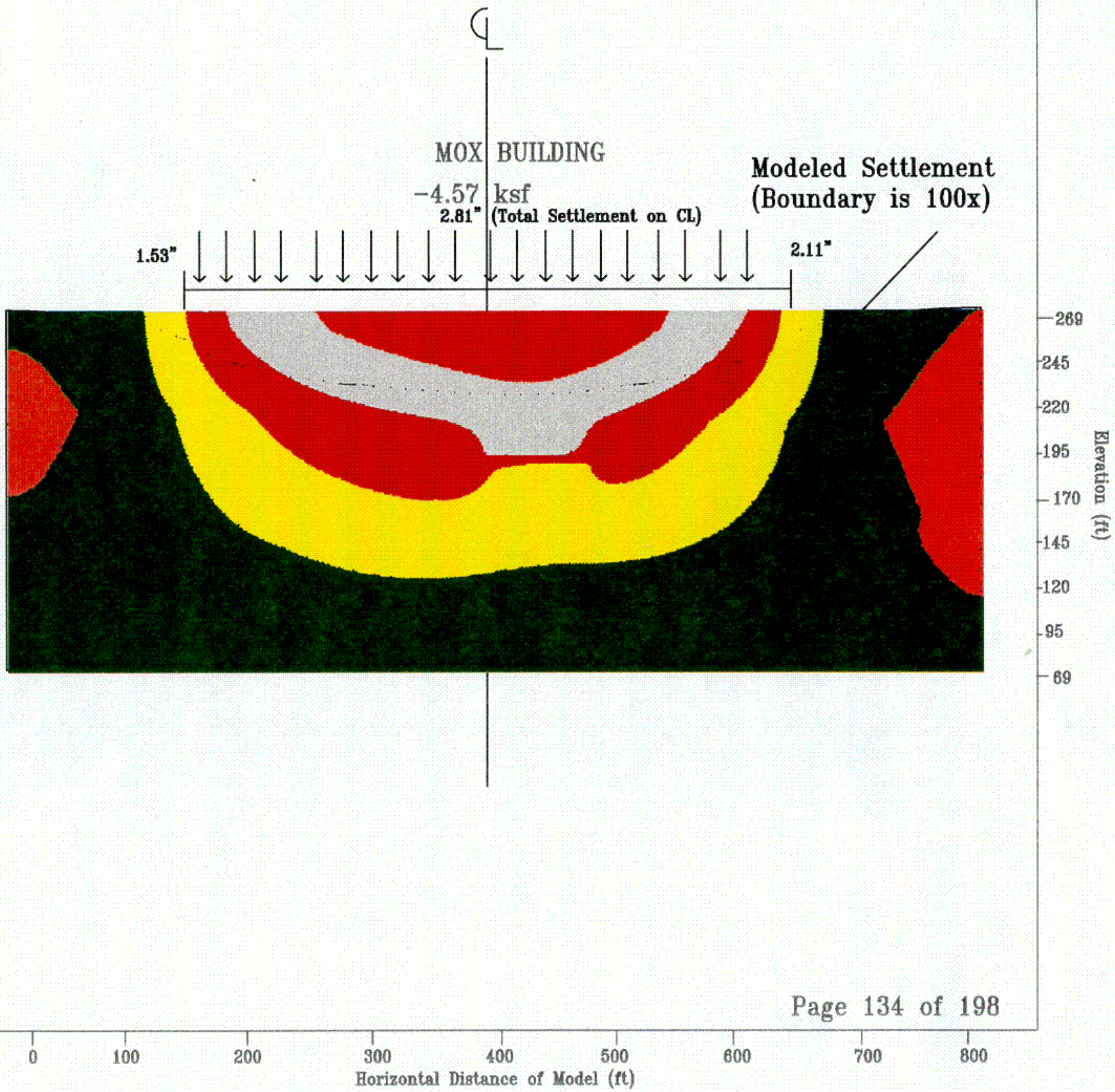


Figure 7-4. MFFF Site - Section 2 Difference Between Load and No Load Cases, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

14-May-01 11:57
step 37
-1.489E+01 <x< 2.829E+02
-9.839E+01 <y< 1.994E+02

NO LOAD - 10' STRUCTURAL FILL

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1696 ft (2.04")
w/10' FILL - MFFF Building
w/5' FILL - Diesel Generator
BEST ESTIMATE VALUES

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1783 ft (2.15")
w/10' FILL - MFFF Building
w/5' FILL - Diesel Generator
LOWER BOUND VALUES

Softzone Location

Water Table at El. 210'

Duke Cogema Stone & Webster

FILENAME:
figure7-4.dwg

C-11

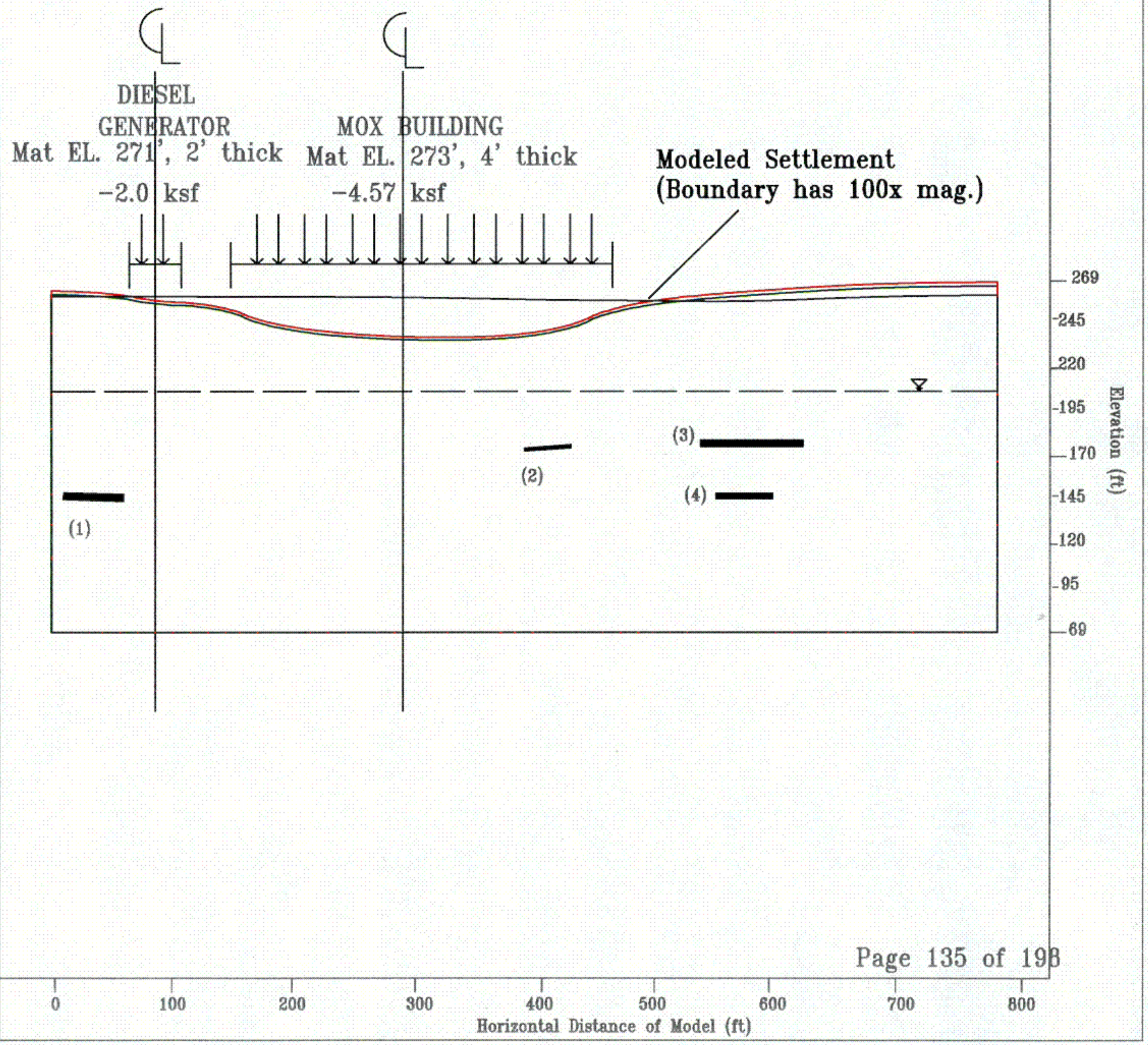


Figure 7-5. MFFF Site - Section 2 Settlement Analysis BEST ESTIMATE VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

11-Jul-01 19:06
step 37
-1.489E+01 <x< 2.829E+02
-9.838E+01 <y< 1.994E+02

Y-displacement contours (ft)
-1.50E-01
-1.25E-01
-1.00E-01
-7.50E-02
-5.00E-02
-2.50E-02
0.00E+00

Contour interval= 2.50E-02 (ft)

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1696 ft (2.04")
BEST ESTIMATE VALUES

Softzone Location

Water Table at El. 210'

Duke Cogema Stone & Webster

FILENAME:
figure7-5.dwg

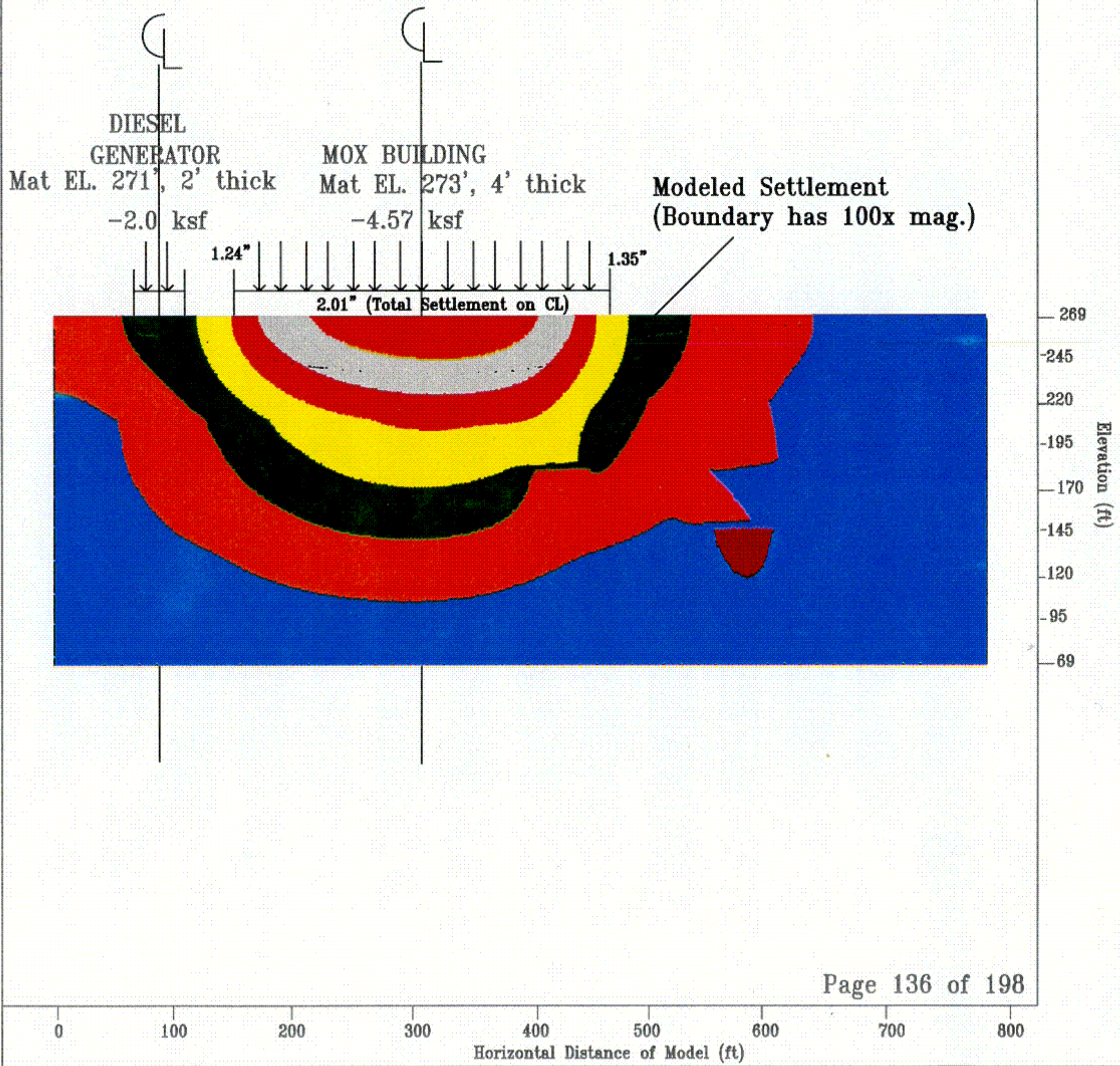


Figure 7-6. MFFF Site - Section 2 Settlement Analysis LOWER BOUND VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

16-Jul-01 17:41
step 33
-1.489E+01 <x< 2.829E+02
-9.838E+01 <y< 1.994E+02

Y-displacement contours (ft)
-1.75E-01
-1.50E-01
-1.25E-01
-1.00E-01
-7.50E-02
-5.00E-02
-2.50E-02
0.00E+00

Contour interval= 2.50E-02 (ft)

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1783 ft (2.15")
LOWER BOUND VALUES

Softzone Location

Water Table at El. 210'

G-13

FILENAME:
figure7-6.dwg

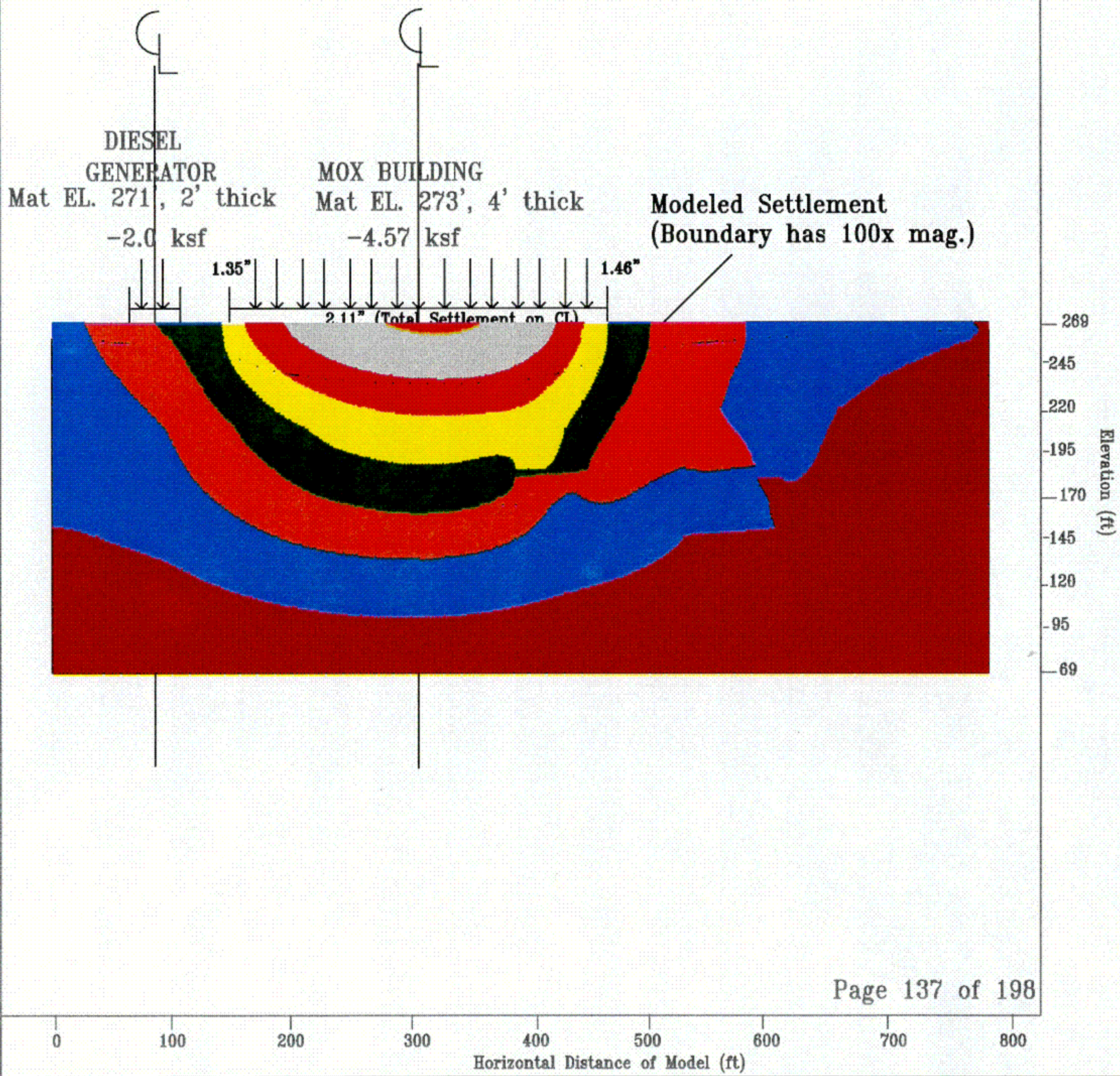


Figure 7-7. MFFF Site - Section 4 Difference Between Load and No Load Cases

FLAC (Version 4.00)

LEGEND

20-Jul-01 18:20
step 46
-1.394E+01 <x< 2.649E+02
-8.896E+01 <y< 1.899E+02

NO LOAD - 10' STRUCTURAL FILL

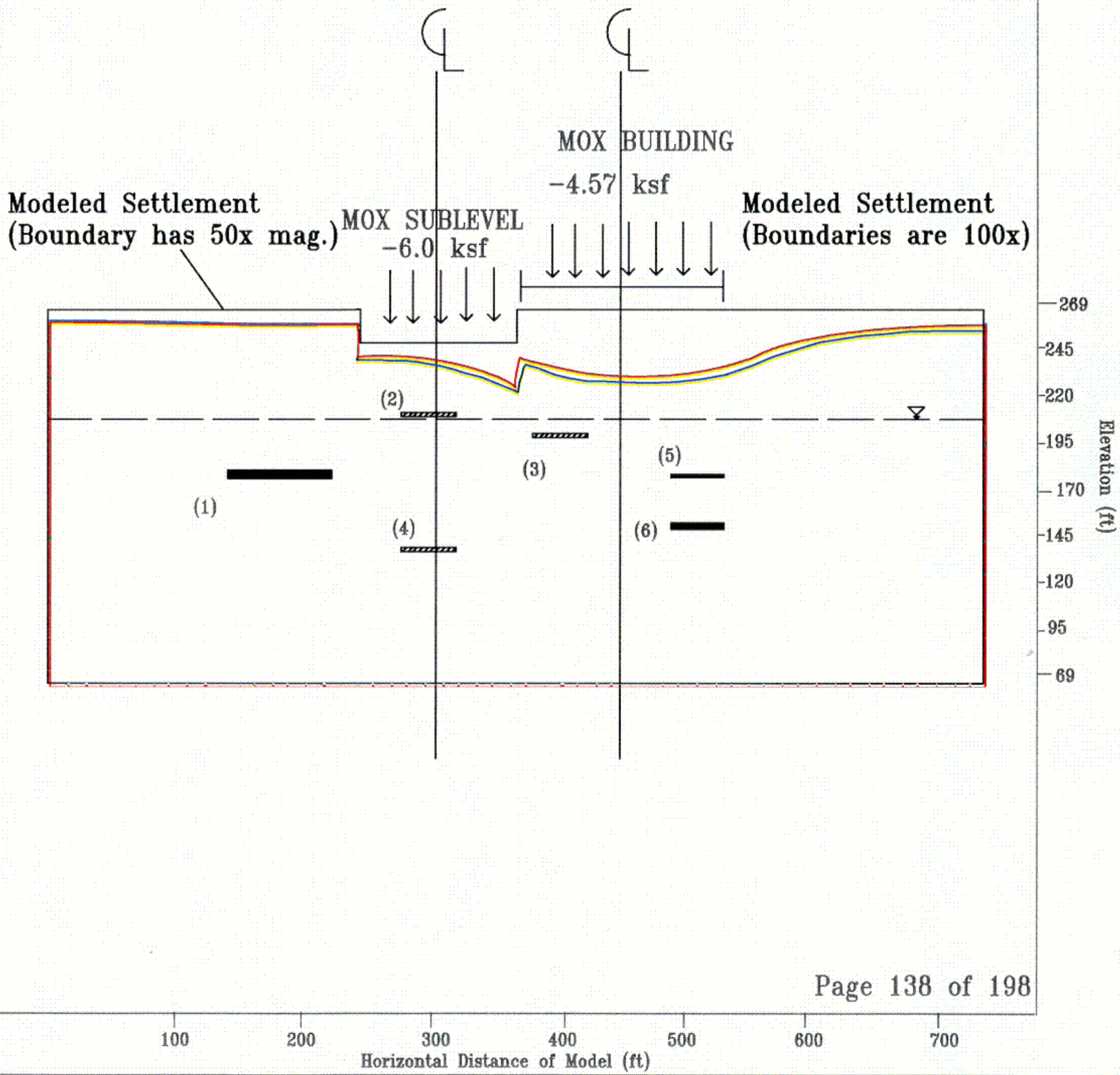
Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1794 ft (2.15")
BEST ESTIMATE VALUES

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.2442 ft (2.93")
LOWER BOUND VALUES

Water Table at El. 210'
Softzone Location
Soft Material Location

Duke Cogema Stone & Webster

FILENAME:
figure7-7.dwg



C-14

Figure 7-8. MFFF Site - Section 4 Settlement Analysis BEST ESTIMATE VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

11-Jul-01 15:26
step 40
-1.394E+01 <x< 2.649E+02
-8.896E+01 <y< 1.899E+02

Y-displacement contours (ft)
-1.75E-01
-1.50E-01
-1.25E-01
-1.00E-01
-7.50E-02
-5.00E-02
-2.50E-02
0.00E+00

Contour interval= 2.50E-02 (ft)

Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.1794 ft (2.15")

Water Table at El. 210'
Softzone Location
Soft Material Location

Duke Cogema Stone & Webster

FILENAME:
figure7-8.dwg

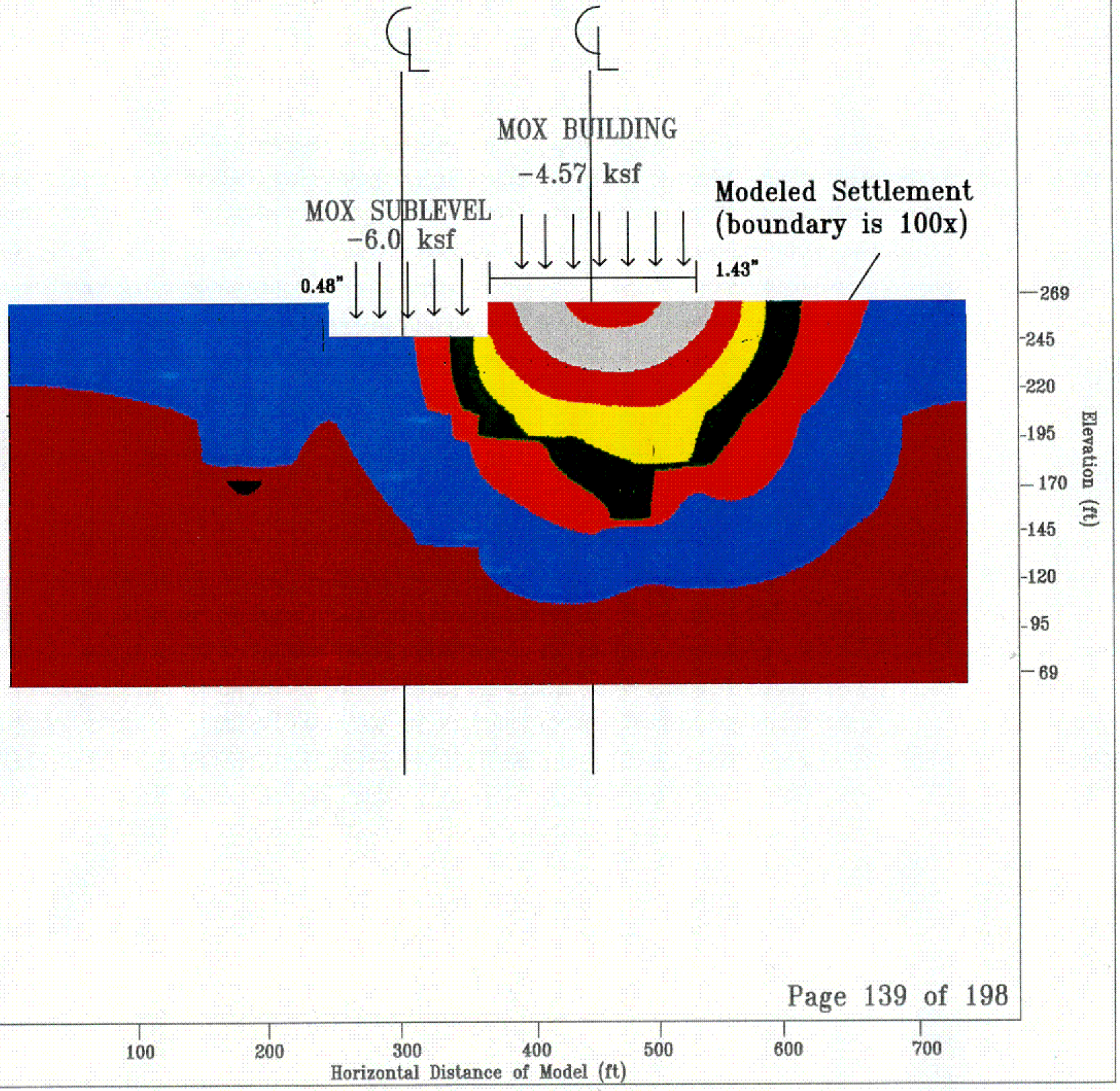


Figure 7-9. MFFF Site - Section 4 Settlement Analysis LOWER BOUND VALUES, 10' Structural Fill

FLAC (Version 4.00)

LEGEND

20-Jul-01 18:20
step 46
-1.394E+01 <x< 2.649E+02
-8.897E+01 <y< 1.899E+02

Y-displacement contours (ft)
-2.25E-01
-2.00E-01
-1.75E-01
-1.50E-01
-1.25E-01
-1.00E-01
-7.50E-02
-5.00E-02
-2.50E-02
0.00E+00

Contour interval= 2.50E-02 (ft)
Exaggerated Boundary Disp.
Magnification = 100 Times
Max Disp = 0.2442 ft (2.93")
▽ — — Water Table at El. 210'
Softzone Location
Soft Material Location

Duke Cogema Stone & Webster

FILENAME:
figure7-9.dwg

C-116