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NUREG-1923

Safety Evaluation Report for an Early Site Permit (ESP) at the Vogtle Electric Generating Plant (VEGP) ESP Site

Office of New Reactors

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ABSTRACT

This safety evaluation report[1] (SER) documents the U.S. Nuclear Regulatory Commission (NRC) staff's technical review of the site safety analysis report (SSAR) and emergency planning information included in the early site permit (ESP) application submitted by Southern Nuclear Operating Company (SNC or the applicant), for the Vogtle Electric Generating Plant (Vogtle or VEGP) site. The SER also documents the NRC staff's technical review of the limited work authorization (LWA) activities for which SNC has requested approval.

By letter dated August 14, 2006, SNC submitted an ESP application for the VEGP site in accordance with Subpart A, "Early Site Permits," of Title 10 of the Code of Federal Regulations (10 CFR) Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants." The VEGP site is located in Burke County, Georgia, approximately 26 miles southeast of Augusta, Georgia. In its application, SNC seeks an ESP that could support a future application to construct and operate additional nuclear power reactors at the ESP site with a total nuclear generating capacity of up to 6800 megawatts thermal (MWt). The proposed ESP Units 3 and 4 would be built on the VEGP site adjacent to and west of two existing nuclear power reactors operated by SNC.

By letter dated August 16, 2007, SNC also submitted an LWA request in accordance with 10 CFR 52.17(c). The activities that SNC requested under its LWA are limited to placement of engineering backfill, retaining walls, lean concrete backfill, mudmats, and waterproof membrane.

This SER presents the results of the staff's review of information submitted in conjunction with the ESP and LWA application. The staff has identified in Appendix A to this SER, certain site-related items that will need to be addressed at the combined license (COL) or construction permit (CP) stage, should the applicant desire to construct one or more new nuclear reactors on the VEGP site. The staff determined that these items do not affect the staff's regulatory findings at the ESP or LWA stage and are, for reasons specified in Section 1.7 of the SER, more appropriately addressed at later stages in the licensing process. Appendix A to this SER also identifies the proposed permit conditions, site characteristics, bounding parameters, and inspections, tests, analyses and acceptance criteria (ITAAC) that the staff recommends the Commission impose, should an ESP and an LWA be issued to the applicant.

1 This SER documents the NRC staff's position on all safety issues associated with the early site permit application and limited work authorization request. This SER has undergone a final review by the Advisory Committee on Reactor Safeguards (ACRS), and the results of the ACRS review are in a final letter report provided by the ACRS. This report is included as Appendix E to this SER.

CONTENTS

In accordance with U.S. Nuclear Regulatory Commission Review Standard (RS)-002, "Processing Applications for Early Site Permits," the chapter and section layout of this safety evaluation report is consistent with the format of (1) NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," (2) Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants," and (3) the applicant's site safety analysis report. Numerous sections and chapters in the NUREG-0800 are not within the scope of or addressed in an Early Site Permit (ESP) or Limited Work Authorization (LWA) Request proceeding. The reader will therefore note "missing" chapter and section numbers in this document. The subjects of chapters and section in NUREG-0800 not addressed herein will be addressed, as appropriate and applicable, in other regulatory actions (design certifications, construction permit, or combined license) for a reactor or reactors that might be constructed on the Vogtle ESP site.

FIGURES

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EXECUTIVE SUMMARY

The regulations at 10 CFR Part 52 contain requirements for licensing new nuclear power plants.[2] These regulations include the NRC's requirements for early site permits (ESP), design certification, and combined license (COL) applications. The ESP process (10 CFR Part 52, Subpart A) is intended to address and resolve site-related issues. The design certification process (10 CFR Part 52, Subpart B, "Standard Design Certifications") provides a means for a vendor to obtain NRC certification of a particular reactor design. Finally, the COL process (10 CFR Part 52, Subpart C, "Combined Licenses") allows an applicant to seek authorization to construct and operate a new nuclear power plant. A COL may reference an ESP, a certified design, both, or neither. A COL applicant referencing an ESP or certified design must resolve any licensing issues that were not resolved as part of the referenced ESP or design certification proceeding before the NRC issues that COL. In addition, an applicant may request a limited work authorization (LWA) for approval of a limited set of construction activities in accordance with 10 CFR 50.10(d). Pursuant to 10 CFR 50.10(d)(3), an LWA request must contain the design and construction information otherwise required by the Commission's rules and regulations to be submitted for a combined license, but limited to those portions of the facility that are within the scope of the LWA. Pursuant to 10 CFR 50.10(d)(2), this request may come from an ESP applicant, and pursuant to 10 CFR 52.17(c), an ESP applicant may request that an LWA be issued in conjunction with the ESP.

This SER describes the results of a review by the NRC staff of both an ESP application and an associated LWA request submitted by Southern Nuclear Operating Company (SNC, or the applicant) for the Vogtle Electric Generating Plant (VEGP) site. The staff's review was to determine the applicant's compliance with the requirements of Subpart A of 10 CFR Part 52 as well as the applicable LWA requirements under 10 CFR Part 50. The SER serves to identify the staff's conclusions with respect to the ESP and LWA safety review and to identify items that would need to be addressed by a future COL applicant referencing a Vogtle ESP.

The NRC regulations also contain requirements for an applicant to submit an environmental report pursuant to 10 CFR Part 51, "Environmental Protection Regulations for Domestic Licensing and Related Regulatory Functions." The NRC reviews the environmental report as part of the Agency's responsibilities under the National Environmental Policy Act of 1969, as amended. The NRC presents the results of that review in a final environmental impact statement (FEIS), which is a report separate from this SER. The staff's FEIS, NUREG-1872, "Final Environmental Impact Statement for an Early Site Permit (ESP) at the Vogtle Electric Generating Plant Site," for the ESP application and LWA request was issued in August 2008, and can be accessed through the agencywide documents access and management system (ADAMS) at ML082260190.

By letter dated August 14, 2006, SNC, acting on behalf of itself and Georgia Power Company (GPC), Oglethorpe Power Corporation (an electric membership corporation), Municipal Electric Authority of Georgia, and the City of Dalton, Georgia, an incorporated municipality in the State of Georgia acting by and through its Board of Water, Light and Sinking Fund Commissioners,

2 Applicants may also choose to seek a CP and operating license in accordance with 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," instead of using the 10 CFR Part 52 process.

submitted an ESP application (ADAMS Accession No. ML062290246)[3] for the VEGP site. The VEGP site is located on a coastal plain bluff on the southwest side of the Savannah River in eastern Burke County, Georgia. The site is approximately 26 miles southeast of Augusta, Georgia and 100 miles northwest of Savannah, Georgia. Directly across from the site, on the eastern side of the Savannah River, is the U.S. Department of Energy's (DOE's) Savannah River Site in Barnwell County, South Carolina. The proposed ESP Units 3 and 4 would be built on the VEGP site adjacent to two existing nuclear power reactors, Vogtle, Units 1 and 2, operated by SNC.

By letter dated August 16, 2007, SNC and its affiliates also submitted an LWA request in accordance with 10 CFR 52.17(c). The activities that SNC requested under its LWA are limited to placement of engineering backfill, retaining walls, lean concrete backfill, mudmats, and a waterproof membrane.

In accordance with 10 CFR Part 52, the VEGP application includes: (1) a description of the site and nearby areas that could affect or be affected by a nuclear power plant(s) located at the site; (2) a safety assessment of the site on which the facility would be located, including an analysis and evaluation of the major structures, systems, and components (SSC) of the facility that bear significantly on the acceptability of the site; (3) complete and integrated emergency plans; and (4) a safety assessment of the construction activities requested under the LWA. The application describes how the site, and the requested construction activities under the LWA, complies with the applicable requirements of 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," 10 CFR Part 52 and the siting criteria of 10 CFR Part 100, "Reactor Site Criteria."[4]

The SER presents the conclusions of the staff's review of the ESP application and associated LWA request. The staff has reviewed the information provided by the applicant to resolve the open items identified in the SER with open items for the VEGP ESP, issued on August 30, 2007 (ML071581032). In addition, the staff has reviewed the information provided by the applicant in response to requests for additional information (RAI) pertaining to both the ESP application and the LWA request. In Section 1.5 of this SER, the staff provides a brief summary of the process used to resolve these items; specific details on the resolution for each open item are presented in the corresponding sections of this report.

The staff identified, in Appendix A to this SER, the proposed permit conditions that it will recommend the Commission impose, if an ESP is issued to the applicant. Appendix A also

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3 ADAMS (Agencywide Documents Access and Management System) is the NRC's information system that provides access to all image and text documents that the NRC has made public since November 1, 1999, as well as bibliographic records (some with abstracts and full text) that the NRC made public before November 1999. Documents available to the public may be accessed via the Internet at http://www.nrc.gov/reading-rm/adams/web-based.html. Documents may also be viewed by visiting the NRC's Public Document Room at One White Flint North, 11555 Rockville Pike, Rockville, Maryland. Telephone assistance for using web-based ADAMS is available at (800) 397-4209 between 8:30 a.m. and 4:15 p.m., eastern time, Monday through Friday, except Federal holidays. The staff is also making this SER available on the NRC's new reactor licensing public web site at http://www.nrc.gov/reactors/new-reactors/esp/vogtle.html.

4 The applicant has also submitted information intended to partially address some of the general design criteria (GDC) in Appendix A, "General Design Criteria for Nuclear Power Plants," to 10 CFR Part 50. Only GDC 2, "Design Bases for Protection Against Natural Phenomena," applies to an ESP application, and it does so only to the extent necessary to determine the safe-shutdown earthquake (SSE) and the seismically induced flood. The staff has explicitly addressed partial compliance with GDC 2, in accordance with 10 CFR 52.17(a)(1) and 10 CFR 50.34(a)(12), only in connection with the applicant's analysis of the SSE and the seismically induced flood. Otherwise, an ESP applicant need not demonstrate compliance with the GDC. The staff has included a statement to this effect in those sections of the SER that do not relate to the SSE or the seismically induced flood. Nonetheless, this SER describes the staff's evaluation of information submitted by the applicant to address GDC 2 with respect to the ESP application. Furthermore, with the applicant's submission of the LWA request, the staff also considered the application's compliance with GDC 1, "Quality Standards and Records," with respect to safety-related structures being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed.

includes a list of COL action items or certain site-related items that will need to be addressed at the COL or CP stage, if the applicant desires to construct one or more new nuclear reactors on the VEGP site and references the Vogtle ESP in its application. The staff determined that these items are not required for the staff to make its regulatory findings on the ESP or LWA and are, for reasons specified in Section 1.6, more appropriately addressed at a later stage in the licensing process. In addition, Appendix A lists the site characteristics, bounding parameters, and the inspections, tests, analyses, and acceptance criteria (ITAAC) that the staff recommends the Commission impose, should an ESP and an LWA be issued to the applicant.

Inspections conducted by the NRC have verified, where appropriate, the conclusions in this SER. The inspections focused on selected information in the ESP application and its references. The SER identifies applicable inspection reports as reference documents.

The NRC's Advisory Committee on Reactor Safeguards (ACRS) also reviewed the bases for the conclusions in this report. The ACRS independently reviewed those aspects of the application that concern safety, as well as the SER, and provided the results of its review to the Commission in an interim report dated November 20, 2007, and in a final report dated December 22, 2008. Appendix E includes a copy of the report by the ACRS on the final safety evaluation report, as required by 10 CFR 52.23, "Referral to the ACRS."

ABBREVIATIONS

ACI	American Concrete Institute
ACRS	Advisory Committee on Reactor Safeguards
ADAMS	Agencywide Documents Access and Management System
ADL	administrative decision line
AF	amplification functions
AFCCC	Air Force Combat Climatology Center
ALARA	as low as reasonably achievable
ALI	annual limits on intake
ANS	American Nuclear Society
ANSI	American National Standards Institute
ANSS	Advanced National Seismic System
ARC	American Red Cross
AREOP	Annual Radiological Environmental Operating Report
ASB	Auxiliary Shield Building
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating and Air-Conditioning Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing and Materials
ATWS	anticipated transients without scram
BBM	Blue Bluff Marl
bpf	blows per foot
BE	best estimate
Bechtel	Bechtel Power Corporation
BLWM	Bureau of Land and Waste Management
BOP	Behavioral Observation Program
BRH	Bureau of Radiological Health
CADD	computer-aided design and drafting
CAR	Corrective Action Reports
C/D	capacity over demand
CDE	committed dose equivalent
CEUS	Central and Eastern United States
cfps	cubic feet per second
CFR	Code of Federal Regulations
CIS	Containment Internal Structure
COL	Combined Operating License
CP	construction permit
cpm	counts per minute
CPT	(seismic) cone penetration test
CR	condition report
CRR	cyclic resistance ratio
Cs	cesium
CSDRS	Certified Design Response Spectra
CSR	cyclic stress ratio
CU	consolidated undrained
CVSZ	Central Virginia Seismic Zone

D	distance
DAC	derived air concentrations
DBA	design-basis accident
Dbar	mean distance
DC	design certification
DCD	design certification document
DEIS	Draft Environmental Impact Statement
DEM	digital elevation model
DF	design factor
DFCS	Department of Family and Children Services
DG	Draft Regulatory Guide
DHEC	Department of Environmental Control
DHS	Department of Homeland Security
DNR	Department of Natural Resources
DOE	Department of Energy
DOE-SR	Department of Energy, Savannah River Site
DOT	Department of Transportation
DQ	deposition factors
DS	document services
E	elastic modulus
EAB	exclusion area boundary
EAL	emergency action levels
EAS	emergency alert system
ECFS	East Coast Fault System
ECL	emergency classification levels
ECMA	East Coast Magnetic Anomaly
EF	Enhanced Fujita
EIP	emergency implementing procedures
El.	elevation
EMA	Emergency Management Agency
EMS	emergency medical services
ENC	Emergency News Center
ENN	Emergency Notification Network
ENS	emergency notification system
ENS	emergency operations center
EOC	emergency operations facility
EOF	emergency operations facility
EOP	emergency operating procedures
EPA	Environmental Protection Agency
EPC	emergency preparedness coordinator
EPD	Environmental Protection Division
EPIP	emergency plan implementing procedures
EPRI	Electric Power Research Institute
EPZ	emergency planning zones
ER	Environmental Report
ERDS	emergency response data system
ERF	emergency response facility
ERO	emergency response organization
ESBWR	Economic Simplified Boiling Water Reactor
ESF	Emergency Support Function
ESP	Early Site Permit

EST	Earth Science Team
ETE	evacuation time estimate
ETML	elevated temperature material liquid
ETSZ	Eastern Tennessee Seismic Zone
ETV	Educational Television Network
EW	East, West
FA	felt area
FAA	Federal Aviation Administration
FDA	Food and Drug Administration
FEIS	final environmental impact statement
FEMA	Federal Emergency Management Agency
FEOC	Forward Emergency Operations Center
FERC	Federal Emergency Regulatory Commission
FIRS	foundation input response spectra
FNARS	Federal National Alert Radio System
FNF	fixed nuclear facility / facilities
FOSID	frequency of onset of inelastic deformation
fps	feet per second
FRC	Federal Response Center
FRERP	Federal Ragiological Emergency Response Plan
FRMAC	Federal Radiological Monitoring and Assessment Center
FS	factors of safety
FSAR	final safety analysis report
FSER	final safety evaluation report
ft	feet / foot
GA	Georgia
GA REP	Georgia Radiological Emergency Plan
GBU	Global Business Unit
GCSZ	Giles County Seismic Zone
GDC	general design criteria
Ge (Li)	lithium drifted germanium
GEMA	Georgia Emergency Management Agency
GEOP	Georgia Emergency Operations Plan
GET	general employee training
GIS	geographical information system
GL	Generic Letter
GMRS	ground motion response spectra
GPC	Georgia Power Company
h	hour
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEPA	high-efficient particulate air
HHS	Department of Health and Human Services
HMR	hydrometeorological Report
HP	health physics
HPN	Health Physics Netowrk
I	Iodine
IBR	incorporated by reference
IC	initiating condition
ICC	Intrastate Coordinating Channel
IEEE	Institute of Electrical and Electronic Engineers

IEM	Innovative Emergency Management, Inc.
in.	inch(es)
INPO	Institute of Nuclear Power Operators
IPCC	Intergovernmental Panel on Climate Change
IPZ	Ingestion Pathway Emergency Planning Zone
ITAAC	inspections, tests, analyses, and acceptance criteria
JFD	joint frequency distribution
JIC	joint information center
KI	potassium iodide
kPa	kilopascals
LB	lower bound
lbf/ft^2	pounds-force per square foot
LGR	local government radio
LLEA	local law enforcement agencies
LLNL	Lawrence Livermore National Laboratory
LOCA	loss-of-coolant accident
LPZ	low population zone
LWA	limited work authorization
LWR	light-water reactor
m	meter
M	moment magnitude
Mbar	mean magnitude
MbLg	body-wave local magnitude
M&TE	measuring and test equipment
m/s	meters per second
MACTEC	MACTEC Engineering and Consulting, Inc.
MAST	Military Assistance to Safety and Traffic
Mbar	mean magnitude
MEI	maximally exposed individual
MGD	million gallons a day
mGy	milliGray
mi	miles
MIDAS	Meteorological Information and Dispersion Assessment System
MLW	mean low water
ML	local magnitude
Mmax	largest maximum magnitude
MM	modified mercalli
MMI	modified mercalli intensity
MOA	Military Operation Area
MOU	memorandum of understanding
MOX	mixed oxide
MPA	methoxypropylamine
MPA	methoxypropylamine
mrad	milliard
mrem	millirem
MRO	Medical Review Officer
m/s	meters per second
MS	surface-wave magnitude
MSE	mechanically stabilized earth
msl	mean sea level
mSv	milliSieverts

MWt	megawatts thermal
mya	million years ago
NaI	sodium iodide
NAWAS	National Warning System
NCDC	National Climatic Data Center
ND	Nuclear Development
NDQAM	Nuclear Development Quality Assurance Manual
NEI	Nuclear Energy Institute
NGDC	National Geophysical Data Center
NHC	National Hurricane Center
NI	nuclear island
NIRMA	Nuclear Information and Records Management Association
NIST	National Institute of Standards and Technology
NMSZ	New Madrid Seismic Zone
NOAA	National Oceanic and Atmospheric Administration
NOAA-CSC	National Oceanic and Atmospheric Administration-Coastal Services Center
NQA	nuclear quality assurance
NQAM	Nuclear Quality Assurance Manual
NRC	Nuclear Regulatory Commission
NREES	Nuclear Response and Emergency Environmental Surveillance Section
NRP	National Response Plan
NS	North, South
NSSL	National Severe Storms Laboratory
NSSS	nuclear steam supply system
NUREG	NRC technical report (Nuclear Regulatory Commission)
NVLAP	National Voluntary Laboratory Accreditation Program
NWR	National Weather Radio
NWS	National Weather Service
NYAL	New York-Alabama Lineament
OBE	operating basis earthquake
OCA	owner-controlled area
OCGA	Official Code of Georgia Annotated
ODCM	Offsite Dose Calculation Manual
OHS	Office of Homeland Security
ORHMC	Oak Ridge Hospital of the Methodist Church
OSC	operational support center
OSID	onset of significant inelastic deformation
OWA	owner-controlled area
PA	protected area
PAG	protective action guideline
PAR	protective action recommendation
PCS	Passive containment cooling system (NRC defines passive containment system)
pcf	per cubic foot
PFT	performance frequency values
PGA	Peak Ground Acceleration
PI	plasticity index
PIO	public information officer
PMF	probable maximum flood
PMH	probable maximum hurricane

PMP	probable maximum precipitation
PMWP	probable maximum water precipitation
PNS	prompt notification system
PO	purchase order
PPM	parts per million
PQAM	Project Quality Assurance Manager
P-S	primary and secondary
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
PWR	pressurized-water reactor
QA	quality assurance
QAPD	Quality Assurance Program Description
QAPP	Quality Assurance Program Plan
RAI	Request for Additional Information
RAP	Radiological Assistance Program
RASCAL	Radiological Assessment System for Consequence Analysis
RCL	Record Control Log
RCTS	resonant column torsional shear
REI	Risk Engineering, Inc.
ReMi	refraction microtremor
REP	radiological emergency preparedness
RER	radiological emergency response
RERP	radiological emergency response plan
RG	Regulatory Guide
RIS	Regulatory Issue Summary
RMC	Radiation Management Consultants
RQD	Rock Quality Designations
RS	Review Standard
RWP	radiation work permit
SASSI	System for Analysis of Soil-Structure Interaction
SASW	Spectral Analysis of Surface Waves
SCDF	seismic core damage frequencies
SCDOT	South Carolina Department of Transportation
SCEMD	South Carolina Emergency Management Division
SCEOP	South Carolina Emergency Operations Plan
SCETV	South Carolina Educational Television Network
SCOL	Subsequent Combined Operating License
SCORERP	South Carolina Operational Radiological Emergency Response Plan
SCR	stable continental region
SCS	Southern Company Services, Inc.
SCTRERP	South Carolina Technical Radiological Emergency Response Plan
SCV	steel containment vessel
SEI	Structural Engineering Institute
SEN	sensitivity
SEOC	State Emergency Operations Center
SER	safety evaluation report
SERCC	Southeast Regional Climate Center
SERT	State Emergency Response Team
SEUSS	South Eastern United States Seismic Network
SL	severity level

SLED	South Carolina Law Enforcement Division
SMRAP	Southern Agreement for Mutual State Radiation Assistance Activation Procedure
SNC	Southern Nuclear Operating Company
SOC	State Operations Center
SOP	Standard Operating Procedure
SP	light gray sand
SPF	Standard Project Flood
SPT	Standard Penetration Test
SQAP	Software Quality Assurance Plan
Sr	strontium
SR	standard review
SRNL	Savannah River National Laboratory
SRP	Standard Review Plan
SRS	Savannah River Site
SSAR	site safety analysis report
SSC	structures, systems and components
SSE	safe-shutdown earthquake
SSHAC	Senior Seismic Hazard Advisory Committee
SSI	soil-structure-interaction
TAG	Technical Advisory Group
TEDE	total effective dose equivalent
TFI	technical facilitator/integrator
TI	Technical Integrator
TIP	Trial Implementation Project
TLD	thermoluminescent dosimeter
TNT	trinitrotoluene
TSC	technical support center
TtNUS	Tetra Tech, Inc.
TV	threshold value
UB	upper bound
UCSS	Updated Charleston Seismic Source
UFL	Upper Flammability Limit
UFSAR	undated final safety analysis report
UHRS	uniform hazard response spectrum
UHS	ultimate heat sink
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USCB	U. S. Census Bureau
USDA	U. S. Department of Agriculture
USGS	U. S. Geological Survey
UTM	Universal Transverse Mercator
UTS	Universal Transverse Mercator
UU	unconsolidated undrained
V/H	vertical-to-horizontal
VEGP	Vogtle Electric Generating Plant
VHF	very high frequency
VOAD	Voluntary Organizations Active in Disaster
Vs	shear wave velocity
WEC	Westinghouse Electric Company, LLC
WLA	William Lettis & Associates

WMA	Wildlife Management Area
WSRC	Washington Savannah River Company
WUS	Western United States
yd(s)	yard(s)
ZRA	zone of river anomalies

1.0 INTRODUCTION AND GENERAL DESCRIPTION

1.1 Introduction

By letter dated August 14, 2006, SNC, acting on behalf of itself and Georgia Power Company (GPC), Oglethorpe Power Corporation (an electric membership corporation), Municipal Electric Authority of Georgia, and the City of Dalton, Georgia, an incorporated municipality in the State of Georgia acting by and through its Board of Water, Light and Sinking Fund Commissioners, submitted an early site permit (ESP) application (ADAMS Accession No. ML062290246) for the Vogtle Electric Generating Plant (VEGP) site. The proposed site is located in eastern Burke County, GA, approximately 26 miles (mi) southeast of Augusta, GA, and approximately 100 mi northwest of Savannah, GA. The NRC docketed the application on September 19, 2006. Pursuant to Subpart A of 10 CFR Part 52, SNC requested an ESP with a permit duration of 20 years. On August 16, 2007, SNC submitted a limited work authorization (LWA) request for approval of construction activities including the placement of engineered backfill, retaining walls, lean concrete backfill, mudmats, and a waterproof membrane, in accordance with 10 CFR 52.17(c). Pursuant to 10 CFR 50.10(d)(3), an LWA request must contain the design and construction information otherwise required by the Commission's rules and regulations to be submitted for a combined license, but limited to those portions of the facility that are within the scope of the LWA.

The staff has completed its review of the information presented in the VEGP application concerning the site's meteorology, hydrology, geology, and seismology, as well as the potential hazards to a nuclear power plant that could result from manmade facilities and activities on or in the vicinity of the site. The staff also assessed the risks of potential accidents that could occur as a result of the operation of a nuclear plant(s) at the site and evaluated whether the site would support adequate physical security measures for a nuclear power plant(s). The staff evaluated whether the applicant's quality assurance measures were in accordance with the measures discussed in Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants," to 10 CFR Part 50. The staff reviewed the complete and integrated emergency plans that SNC would implement if a new reactor(s) is eventually constructed at the ESP site.

In addition, the staff reviewed the technical information presented in the VEGP application pertaining to the LWA activities being requested. Specifically, the staff reviewed the applicant's seismic design, seismic systems, and foundations, as they relate to the LWA activities being requested. The staff also evaluated the applicant's fitness for duty program in accordance with the requirements in 10 CFR Part 26.[5]

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5 As provided in Part 26, the entities that must comply with Part 26 requirements include "[e]arly site permit holders who have been issued a limited work authorization under § 50.10(e), if the limited work authorization authorizes the early site permit holder to install the foundations, including the placement of concrete, for safety- and security-related SSCs under the limited work authorization." 10 CFR 26.3(c)(5). The statement of considerations for Part 26 indicates that entities authorized by an LWA to perform "only the...placement of backfill" will not be required to comply with Part 26, but that entities who are authorized by an LWA "to perform installation of the foundation" for safety- and security-related SSCs will be required to comply. 73 FR 16966, 16998 (Mar. 31, 2008). The staff has determined that because of its implications for seismic safety, the placement of engineered backfill requested as part of the LWA for the Vogtle site represents an integral part of the foundation; accordingly, the staff considers placement of that backfill pursuant to the LWA to be "installation of the foundation" within the meaning of Part 26. Therefore, consistent with the text of the rule, the staff has determined that the applicant is required to comply with the requirements of Part 26 to establish a fitness for duty program.

The VEGP application includes the SSAR, which describes a safety assessment of the site, as required by 10 CFR 52.17, "Contents of Applications." The public may inspect copies of the ESP application in ADAMS under Accession No. ML081020073. The application is also available for public inspection at the NRC's Public Document Room at One White Flint North, 11555 Rockville Pike, Rockville, MD 20852, and at the Burke County Public Library, 130 Highway 24 South, Waynesboro, GA 30830.

This safety evaluation report (SER)[6] documents the staff's technical evaluation of the suitability of the proposed VEGP site for construction and operation of a nuclear power plant(s) falling within the design parameters that SNC specified in its application. It also documents the results of the staff's technical evaluation of the limited construction activities proposed under SNC's LWA request. The SER delineates the scope of the technical matters that the staff considered in evaluating the suitability of the site and the LWA request. NRC Review Standard (RS)-002, "Processing Applications for Early Site Permits," Attachment 2, provides guidance for the staff in conducting its review of the radiological safety and emergency planning aspects of a proposed nuclear power plant site. RS-002, Attachment 2, contains regulatory guidance based on NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants" (hereafter referred to as the SRP.) In addition to RS-002, the SRP provides the regulatory guidance applied by the staff in its review of the LWA request. The SRP reflects the staff's many years of experience in establishing and promulgating guidance to enhance the safety of nuclear facilities, as well as in performing safety assessments.

The applicant also filed an environmental report for the VEGP site in which it evaluated those matters relating to the environmental impact assessment that can be reasonably reviewed at this time. The staff discussed the results of its evaluation of the environmental report for the VEGP site in a final environmental impact statement (FEIS) issued in August 2008 (ML082260190). The applicant has also provided a site redress plan, in accordance with 10 CFR 52.17(c), in order to perform the LWA activities specifically requested in the application. The FEIS documents the staff's evaluation of the SNC site redress plan.

Appendix A to this SER contains the list of site characteristics, permit conditions, COL action items, and the bounding parameters, and inspections, tests, analyses and acceptance criteria (ITAAC) that the staff recommends the Commission include in any ESP and LWA that might be issued for the proposed site. Appendix B to the SER is a chronology of the principal actions and correspondence related to the staff's review of the ESP and LWA application for the VEGP site. Appendix C lists the references for this SER, Appendix D lists the principal contributors to this report, and Appendix E includes a copy of the report by the ACRS.

1.2 General Site Description

Proposed ESP Units 3 and 4 are planned to be built on the VEGP site. The VEGP site, which spans 3,169 acres, is located on a coastal plain bluff on the southwest side of the Savannah River in eastern Burke County. The site is approximately 15 miles east-northeast of Waynesboro, GA, 26 miles southeast of Augusta, GA, and it is also approximately 100 miles from Savannah, GA. Directly east of the site, across the Savannah River, is the U.S Department of Energy's (DOE) Savannah River Site.

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6 This SER documents the NRC staff's position on all safety issues associated with the early site permit application and limited work authorization request. This SER has undergone a final review by the Advisory Committee on Reactor Safeguards (ACRS), and the results of the ACRS review are in a final letter report provided by the ACRS. This report is included as Appendix E to this SER.

Numerous small towns exist within 50 miles of the site. U.S. Interstate Highway No. I-20 (I-20), a major interstate highway, crosses the northern portion of the 50-mile radius. The site can be accessed through U.S. Route 25; Georgia State Routes 23, 24, 56, and 80; and New River Road. A navigation channel is authorized on the Savannah River from the Port of Savannah to Augusta, GA, and a railroad spur connects the site to the Norfolk Southern Savannah-to-Augusta track. The applicant's SSAR Figures 1-1 and 1-2 show the site location and the area within a 6-mile and 50-mile radius. Section 2.1 of this SER discusses the site location in more detail.

With regard to the existing development of the site, the VEGP site currently has two Westinghouse pressurized water reactors (PWRs), rated at 3,625.6 Mwt. Also on the site are their supporting structures, which include two natural-draft cooling towers (one per unit), associated pumping and discharge structures, water treatment building, switchyard, and training center. Plant Wilson, a six-unit, oil-fueled combustion turbine facility, is also located on the VEGP site, east of Units 1 and 2. The applicant's SSAR Figure 1-3 shows the current VEGP site plan.

With regard to the proposed development of the site, the new plant footprint selected for proposed Units 3 and 4 is adjacent to the west side of the VEGP Units 1 and 2. The footprint is shown on the applicant's SSAR Figure 1-4.

The applicant has referenced the Westinghouse AP1000 certified reactor design for both the ESP application and the LWA request. The applicant's SSAR Section 1.3 identifies the design parameters, site characteristics, and site interface values used in the development of the application. The design parameters are based on the addition of two Westinghouse AP1000 units, to be designated Vogtle Units 3 and 4. The AP1000 has a thermal power rating of 3,400 MWt and a net electrical output of 1,117 megawatts electric. While the staff considered design parameters of the AP1000 certified design in order to make its ESP findings concerning site suitability, issuance of a Vogtle ESP does not constitute approval of future construction of the AP1000 certified design at the Vogtle site. If a CP or COL applicant references a Vogtle ESP in its application, the staff's CP or COL stage review would determine whether the reactor design that is ultimately selected by that applicant falls within the site characteristics and design parameters specified in the ESP. Likewise, while the LWA application references applicable design parameters of the AP1000 certified design, the staff's LWA review addresses only those aspects of the AP1000 design that are within the scope of that request.

1.3 Identification of Agents and Contractors

SNC, acting on behalf of itself and the owners of the VEGP site, is the applicant for the ESP and the LWA and has been the only participant in the review of the suitability of the VEGP site for a nuclear power plant. Bechtel Power Corporation (Bechtel) served as the principal contractor for the development of the SSAR portion of the ESP application and Tetra Tech NUS, Inc. (TtNUS), to assist with preparing the environmental report portion. Both Bechtel and TtNUS supplied personnel, systems, project management, and resources to work on an integrated team with SNC.

Several subcontractors also assisted in the development of SNC's ESP and LWA application. MACTEC Engineering and Consulting, Inc. performed geotechnical field investigations and laboratory testing in support of SSAR Section 2.5, "Geology, Seismology, and Geotechnical

Engineering." William Lettis & Associates, Inc. performed geologic mapping and characterized seismic sources in support of SSAR Section 2.5. Risk Engineering, Inc. performed probabilistic seismic hazard assessments (PSHA) and related sensitivity analyses in support of SSAR Section 2.5.

1.4 Summary of Principal Review Matters

This SER documents the NRC staff's technical evaluation of the VEGP site. The staff's evaluation included a technical review of the information and data the applicant submitted, with emphasis on the following principal matters:

- population density and land use characteristics of the site environs and the physical characteristics of the site, including meteorology, hydrology, geology, and seismology, to evaluate whether these characteristics were adequately described and appropriately considered in determining whether the site characteristics are in accordance with the Commission's siting criteria (10 CFR Part 100, Subpart B, "Evaluation Factors for Stationary Power Reactor Site Applications on or After January 10, 1997")

- potential hazards of man-made facilities and activities to a nuclear power plant(s) that might be constructed on the ESP site (e.g., mishaps involving storage of hazardous materials (toxic chemicals, explosives), transportation accidents (aircraft, marine traffic, railways, pipelines), and the existing nuclear power facility comprising the nearby VEGP units)

- potential capability of the site to support the construction and operation of a nuclear power plant(s) with design parameters falling within those specified in the application under the requirements of 10 CFR Parts 52 and 100

- suitability of the site for development of adequate physical security plans and measures for a nuclear power plant(s)

- proposed complete and integrated emergency plan, should an applicant for a construction permit (CP) or combined license (COL) referencing a Vogtle ESP decide to seek a license to construct and operate a nuclear power plant(s) on the ESP site; any significant impediments to the development of emergency plans for the VEGP site; and a description of contacts and arrangements made with Federal, State, and local government agencies with emergency planning responsibilities

- quality assurance measures SNC applied to the information submitted in support of the ESP application and safety assessment

- the acceptability of the applicant's proposed exclusion area and low-population zone (LPZ) under the dose consequence evaluation factors of 10 CFR 50.34(a)(1)

This SER also documents the NRC staff's technical evaluation of SNC's LWA request. The staff's evaluation included a technical review of the information and data the applicant submitted, with emphasis on the following principal matters:

- acceptability of the applicant's design properties related to the engineered backfill

- the acceptability of the applicant's mudmat and waterproof membrane design in accordance with 10 CFR 50.10(d)(3)
- quality assurance measures SNC applied to the information submitted in support of the LWA request, and will continue to apply when performing approved LWA activities
- A fitness for duty program developed, with respect to those limited construction activities requested in SNC's LWA application, to meet the applicable requirements contained in 10 CFR Part 26.

During its review, the staff held several meetings with representatives of SNC and its contractors and consultants to discuss various technical matters related to the staff's review of the VEGP site (refer to Appendix B to this SER) and LWA. The staff also visited the site to evaluate safety matters.

Appendix A to this SER includes a list of the site characteristics, bounding parameters, permit conditions, COL action items, and ITAAC that the staff recommends the Commission include in an ESP and LWA for the Vogtle site. The site characteristics are based on site investigation, exploration, analysis, and testing, performed by the applicant and are specific physical attributes of the site, whether natural or man-made. Bounding parameters set forth the postulated design parameters that provide design details to support the NRC staff's review. An explanation of COL action items, permit conditions, and ITAAC is provided below in sections 1.6, 1.7, and 1.8 respectively.

1.5 Summary of Open Items and Confirmatory Items

During its review of SNC's ESP application for the Vogtle site, the staff identified several issues that remained open at the time the SER with open items was issued on August 30, 2007. The staff considered an issue to be open if the applicant did not provide requested information and the staff did not know what would ultimately be included in the applicant's response. For tracking purposes, the staff assigned each of these issues a unique identifying number that indicated the section of this report describing it. The SER with open items was issued with 40 open items. Resolution of each open item is discussed in the SER section in which it appears. For example, Section 2.3 of this report discusses Open Item 2.3-1. As set forth in this report, all open items have been resolved.

During its review of SNC's LWA application for the Vogtle site, the staff also identified several issues for which it needed to obtain further information from the applicant. The staff relied on RAIs and site audits to resolve all outstanding issues. The staff's consideration of these RAIs, the applicant's responses to the RAIs, and the results of site audits are documented throughout this SER.

Previously, in the advanced SER, issued November 12, 2008, the staff identified confirmatory item 1.1-1, to verify that the applicant incorporated all of the necessary changes to which it had committed in RAI and open item responses. An item is identified as confirmatory if the staff and the applicant have agreed on a resolution of the particular item, but the resolution has not yet been formally documented.

The staff has completed its review of Revision 5 to the VEGP ESP application and LWA request, submitted December 23, 2008, and has verified that the applicant did incorporate those changes in Revision 5. Therefore, confirmatory item 1.1-1 is closed.

1.6 Summary of Combined License Action Items

The staff has also identified certain site-related items that will need to be addressed at the COL or CP stage if a COL or CP applicant desires to construct one or more new nuclear reactors on the VEGP site and references a Vogtle ESP. This report refers to these items as COL action items. The COL action items relate to issues that are outside the scope of this SER. The COL action items do not establish requirements; rather, they identify an acceptable set of information to be included in the site-specific portion of the safety analysis report submitted by a COL or CP applicant referencing the Vogtle ESP. An applicant for a COL or CP referencing a Vogtle ESP will need to address each of these items in its application. The applicant may deviate from or omit these items, provided that the COL or CP application identifies and justifies the deviation or omission. The staff determined that the COL action items are not required for the staff to make its regulatory findings on the ESP or LWA and are, for reasons specified in this report for each item, more appropriately addressed at a later stage in the licensing process.

At the time the SER with open items was issued, there were a total of 19 COL action items. As a result of the staff's review of the open item responses, and the supplemental information provided in the LWA request, the staff was able to close out several of the COL action items. In total, there are 5 COL action items remaining. This report highlights the closure of previously identified COL action items. It also highlights the existing and new COL action items proposed by the staff.

Appendix A to this SER includes a list of the COL action items to be addressed by a future COL or CP applicant referencing a Vogtle ESP. The staff identified COL action items in order to ensure that particular significant issues are tracked and considered during the COL or CP stage. The COL action items focus on matters that may be significant in any COL or CP application referencing the ESP and LWA for the Vogtle site, if one is issued. Usually, COL action items are not necessary for issues covered by permit conditions or explicitly covered by the bounding parameters. The list of COL action items is not exhaustive with respect to the information required to meet the requirements for a CP or COL.

1.7 Summary of Permit Conditions

The staff has identified certain permit conditions that it will recommend the Commission impose if an ESP is issued to the applicant. At the time the SER with open items was issued, there were 2 permit conditions identified. As a result of the staff's review of the responses to open items, and the supplemental information provided in the LWA request, the staff identified additional permit conditions and removed one pertaining to hydrology. In total, there are 9 permit conditions identified. This report highlights the closure of the permit condition related to hydrology. It also highlights the existing and new permit conditions proposed by the staff.

Appendix A to this SER summarizes these permit conditions. Each permit condition has been assigned a number based on the order which it appears in this SER. The staff has provided an explanation of each permit condition in the applicable section of this report. These permit conditions, or limitations on the ESP, are based on the provisions of 10 CFR 52.24, "Issuance of Early Site Permit."

1.8 Summary of Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC)

For the reasons explained in this report, an ESP application proposing complete and integrated emergency plans for review and approval should propose the inspections, tests, and analyses that the holder of a COL referencing the ESP shall perform, and the acceptance criteria that are necessary and sufficient to provide reasonable assurance that, if the inspections, tests, and analyses are performed and the acceptance criteria met, the facility has been constructed and will be operated in conformity with the emergency plans, the provisions of the Atomic Energy Act, and the Commission's rules and regulations.

Likewise, if a request for a limited work authorization (LWA) is to be issued in conjunction with an ESP, it should propose the inspections, tests, and analyses that the ESP holder authorized to conduct LWA activities shall perform, and the acceptance criteria that are necessary and sufficient to provide reasonable assurance that, if the inspections, tests, and analyses are performed and the acceptance criteria met, the approved construction activities will have been completed in conformity with the provisions of the Atomic Energy Act and the Commission's rules and regulations.

The staff has identified certain ITAAC that it will recommend the Commission impose with respect to an ESP and LWA issued to the applicant. At the time the SER with open items was issued, the staff had only reviewed and included ITAAC necessary for SNC's Emergency Plans. However, as a result of the staff's review of the supplemental information provided in the LWA request, the staff reviewed and approved additional ITAAC. This report highlights the applicant's proposed ITAAC and the staff's review and approval of them. In addition, Appendix A to this SER summarizes the ITAAC approved by the staff.

2.0 SITE CHARACTERISTICS

2.1 Geography and Demography

2.1.1 Site Location and Description

2.1.1.1 Introduction

This section provides details about the site location and site area description for the VEGP site. The proposed ESP Units 3 and 4 would be built on the VEGP site adjacent to existing VEGP Units 1 and 2. The 3169-acre VEGP site is located on a coastal plain bluff southwest of the Savannah River in eastern Burke County. The site exclusion area boundary (EAB) is bounded by River Road, Hancock Landing Road, and 1.7 miles of the Savannah River. The site is approximately 30 river-miles above the U.S. Highway 301 bridge and directly across the river from the U.S. Department of Energy (DOE) Savannah River Site (SRS), in Barnwell County, South Carolina. The VEGP site is approximately 15 miles northeast of Waynesboro, Georgia, and 26 miles southeast of Augusta, Georgia, which is the nearest population center (with more than 25,000 residents).

2.1.1.2 Regulatory Basis

The acceptance criteria for site location and description are based on meeting the relevant requirements of 10 CFR 52.17, "Contents of applications," and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the site location and area description:

- 10 CFR 52.17, as it relates to the applicant submitting information needed for evaluating factors involving the characteristics of the site environment, and describing the boundaries of the site and the proposed general location of each facility on the site.

- 10 CFR Part 100, Subpart B, as it relates to site acceptance being based on the consideration of factors relating to the proposed reactor design and the site characteristics.

Review Standard (RS)-002, "Processing Applications for Early Site Permits," Section 2.1.1, specifies that an applicant has submitted adequate information if it satisfies the following criteria:

- Highways, railroads, and waterways which traverse the exclusion area are sufficiently distant from planned or likely locations of structures of a nuclear power plant or plants of specified type that might be constructed on the proposed site so that routine use of these routes is not likely to interfere with normal plant operation.

- The site location, including the exclusion area and the proposed location of a nuclear power plant or plants of specified type that might be constructed on the proposed site, are described in sufficient detail to allow a determination (in Sections 2.1.2, 2.1.3, and 15.0 of RS-002) that 10 CFR Part 100, Subpart B is met.

In addition to identifying specific acceptable criteria to meet the relevant requirements, RS-002 indicates the NRC staff's review of the site location and description typically involves reviewing the following:

- reactor location with respect to (1) latitude and longitude, and the Universal Transverse Mercator (UTM) coordinates, (2) political subdivisions (i.e., counties, cities, states, or their respective agencies), and (3) prominent natural and manmade features of the area for use in independent evaluations of the exclusion area authority and control, the surrounding population, and nearby manmade hazards

- the site area map containing the reactor and associated principal plant structures to determine (1) the distance from the reactor to the boundary lines of the EAB and (2) the location, distance, and orientation of plant structures with respect to highways, railroads, and waterways that traverse or lie adjacent to the exclusion area to ensure that they are adequately described to permit analyses of the possible effects of plant accidents on these transportation routes.

2.1.1.3 Technical Evaluation

Following the procedures described in RS-002, Section 2.1.1, the NRC staff reviewed Section 2.1.1 of the SSAR in the VEGP application regarding the site location and site area description, as well as the information the applicant provided in response to the NRC staff's RAI 2.1.1-2 and 2.1.1-3.

The applicant provided the following information regarding the site location and site area description:

- the site boundary for the proposed VEGP Units 3 and 4 to be built on the proposed ESP site with respect to the existing VEGP Units 1 and 2

- the site layout for the proposed VEGP Units 3 and 4 to be built on the proposed ESP site

- the site location with respect to political subdivisions and prominent natural and manmade features of the area within the 6-mile LPZ and the 50-mile population zone

- the topography and characteristics of the land surrounding the proposed ESP site

- the commercial, industrial, institutional, recreational, and residential structures located within the site area

- the distance from the proposed ESP site to the nearest EAB, including the direction and distance

- the potential radioactive release points and their locations for the proposed units

- the distance of the proposed Units 3 and 4 to be built on the proposed ESP site from regional U.S. and State highways

The proposed Units 3 and 4 would be located within the existing VEGP site adjacent to existing Units 1 and 2. The ESP site boundary, as shown in Figure 1-4 of the SSAR, is the same as the site boundary for the existing VEGP Units 1 and 2. This figure depicts both the existing units and the proposed units in addition to the site boundary, exclusion area boundary (EAB), protected area (PA) for the proposed units, visitor's center, and Plant Wilson, a six-unit oil-fueled combustion turbine facility owned by Georgia Power Company (GPC), which is also located on the VEGP site.

The NRC staff has verified the following latitude and longitude and UTM coordinates of the proposed units, as provided in the SSAR:

UTM Coordinates	Latitude/Longitude Deg/Min/Sec
Unit 3: Zone 17 3,667,170 m N; 428,320 m E	33 08 27 N; 81 46 07 W
Unit 4: Zone 17 3,667,170 m N; 428,070 m E	33 08 27 N; 81 46 16 W

The EAB for the VEGP, Units 1 and 2 will also apply to the proposed ESP VEGP Units 3 and 4. There are no residents in this exclusion area. The site EAB is bounded by River Road, Hancock Landing Road, and 1.7 miles of the Savannah River. The property boundary encompasses the entire EAB and extends beyond River Road in some areas. The nearest point to the EAB is located approximately 3400 feet southwest of the proposed VEGP Units 3 and 4 power block area. The applicant established this EAB to meet the siting and evaluation factors in Subpart B of 10 CFR Part 100, as well as the radiation exposure criterion "as low as is reasonably achievable," defined in 10 CFR Part 50.

The 3,169-acre proposed ESP site is located on a coastal plain bluff southeast of the Savannah River in eastern Burke County. The VEGP site is situated within three major resource areas: (1) the Southern Piedmont, (2) Carolina and Georgia Sand Hills, and (3) the Coastal Plain. These characteristics are typical of land forms that resulted from historical marine sediment deposits in central and eastern Georgia. There are no mountains in the general area.

The proposed ESP site is approximately 15 miles east-northeast of Waynesboro, Georgia, and 26 miles southeast of Augusta, Georgia, the nearest population center having more than 25,000 residents. It is also about 100 miles from Savannah, Georgia, and 150 river-miles from the mouth of the Savannah River. Burke County includes five incorporated towns (1) Waynesboro, (2) Girard, (3) Keysville, (4) Midville, and (5) Sardis. Of these five towns, only the town of Girard is within 10 miles of the ESP site. Girard has a population of 227 residents, according to the 2000 census.

Based on the NRC staff's review of the general site area and the information collected from the local officials during the site visit, the applicant's information with regard to the site location and area description is adequate and acceptable because it satisfies the acceptance criteria specified in RS-002, Section 2.1.1.

First, although the site is accessible by River Road via U.S. Highway 25 and Georgia Routes 56, 80, 24, and 23, and a railroad spur connects the site to the Norfolk Southern Savannah-to-Augusta track, there are no highways, railroads, or waterways that traverse the proposed ESP site EAB. Accordingly, because there are no highways, railroads, and waterways that traverse the exclusion area, routine use of these routes is not likely to interfere with normal plant operations.

Second, based on the NRC staff's review of the general site area and the information collected from the local officials during the site visit, the applicant's information with regard to the site location and area description is adequate and acceptable to allow the NRC to evaluate whether the applicant met the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff has verified that the EAB distance is consistent with the distance the applicant used in its radiological consequence analyses described in Chapter 15 and in Chapter 13.3 of the SSAR. The applicant stated that all areas outside the EAB will be unrestricted in the context of 10 CFR Part 20, "Standards for Protection Against Radiation," and the gaseous effluent release limits, per guidelines provided in 10 CFR Part 50, for the proposed ESP units, would apply to the EAB. Further information regarding the site location and site description is provided in Sections 2.1.2, 2.1.3, and 11 of this SER.

2.1.1.4 Conclusion

As set forth above, the applicant provided and substantiated information concerning the site location and description of site area. The NRC staff has reviewed the information provided and, for the reasons given above, concludes that the applicant established site characteristics that meet the requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff further concludes that the applicant provided sufficient details about the site location and description of the site area to allow the NRC staff to evaluate, as documented in Sections 2.1.2, 2.1.3, 11, 13.3, and 15 of this SER, whether the applicant met the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Introduction

This section addresses the information concerning the legal authority to regulate any and all access and activity within the entire plant exclusion area for the proposed VEGP Units 3 and 4. Part 1, Chapter 3, of the SSAR provides general information pertaining to the owners/co-owners group. The applicant stated that GPC, for itself and as an agent for the other co-owners, has delegated complete authority to SNC to determine and regulate all activities within the designated exclusion area. "No Trespassing" signs are posted on the perimeter of the VEGP EAB on land and along the Savannah River, and indicate the actions to be taken in the event of emergency conditions at the plant.

2.1.2.2 Regulatory Basis

The acceptance criteria for exclusion area authority and control are based on meeting the relevant requirements of 10 CFR Part 100 with respect to the applicant's authority over the designated exclusion area.

- 10 CFR 100.3 states: Exclusion area means that area surrounding the reactor, in which the reactor licensee has the authority to determine all activities including exclusion or removal of personnel and property from the area. This area may be traversed by a highway, railroad, or waterway, provided these are not so close to the facility as to interfere with normal operations of the facility and provided appropriate and effective arrangements are made to control traffic on the highway, railroad, or waterway, in case of emergency, to protect the public health and safety. Residence within the exclusion area shall normally be prohibited. In any event, residents shall be subject to ready removal in case of necessity. Activities unrelated to operation of the reactor may be permitted in an exclusion area under appropriate limitations, provided that no significant hazards to the public health and safety will result.

As stated in RS-002, Section 2.1.2, specifies that an applicant has submitted adequate information if it satisfies the following criteria:

- The applicant demonstrates, prior to issuance of an ESP, that it has the authority within the exclusion area, as required by 10 CFR 100.3, or provides reasonable assurance that it will have such authority prior to start of construction of a proposed nuclear unit that might be located on the proposed ESP site.

- Activities unrelated to operation of a nuclear power plant or plants of specified type that might be constructed on the proposed site within the exclusion area are acceptable provided: (a) such activities, including accidents associated with such activities, represent no significant hazard to a nuclear power plant or plants of specified type that might be constructed on the proposed site, or are to be accommodated as part of the plant design basis at the COL stage. (See Section 2.2.3 of RS-002); (b) the applicant is aware of such activities and has made appropriate arrangements to evacuate persons engaged in such activities, in the event of an accident; and (c) there is reasonable assurance that persons engaged in such activities can be evacuated without receiving radiation doses in excess of the reference values of 10 CFR 50.34(a)(1).

RS-002, Section 2.1.2 also addresses review procedures that allow the NRC staff to determine whether the relevant requirements are met. This typically involves the NRC staff reviewing (1) the applicant's legal authority to determine all activities within the designated exclusion area, (2) the applicant's authority and control in excluding or removing personnel and property in the event of an emergency, and (3) proposed or permitted activities in the exclusion area which are unrelated to operation of the reactor to ensure that they do not result in a significant hazard to public health and safety.

2.1.2.3 Technical Evaluation

Following the procedures described in RS-002, Section 2.1.2, the NRC staff reviewed SSAR Chapter 2.1.2 of the VEGP ESP application regarding exclusion area authority and control, in addition to the applicant's responses to RAIs 2.1.2-1, 2.1.2-2, and 2.1.2-3.

In the SSAR Chapter 2.1.2, the applicant presented information concerning the following:

- complete legal authority to regulate any and all access and activity within the entire plant exclusion area

- identification of two facilities (the visitor's center and the GPC combustion turbine plant, Plant Wilson) within the EAB that have authorized activities unrelated to nuclear plant operations

- emergency planning, including arrangements for traffic control

Figure 1-4 of the SSAR depicts the boundary lines of the exclusion area for the proposed ESP site, which is the same as the EAB for the existing VEGP Units 1 and 2. The EAB is bounded by River Road, Hancock Landing Road, and 1.7 miles of the Savannah River. No state or county roads, railroads, or waterways traverse the VEGP exclusion area. The nearest point to the EAB is located approximately 3400 feet southwest of the proposed VEGP Units 3 and 4 ESP power block area.

The applicant stated that pursuant to the VEGP owner's agreement, GPC, for itself and as agent for the co-owners, has delegated to SNC (the applicant) complete authority to regulate any and all access and activity within the entire plant exclusion area. The applicant also stated that the perimeter of the VEGP EAB is adequately posted with "No Trespassing" signs on land and along the Savannah River, which indicate the actions to be taken in the event of emergency conditions at the plant. The applicant stated that it has complete authority to regulate any and all access and activity within the ESP EAB.

The NRC staff verified the applicant's description of exclusion area, the authority under which all activities within the exclusion area can be controlled, and the methods by which access and occupancy of the exclusion area can be controlled during normal operation and in the event of an emergency situation and concluded that the applicant has the required authority to control activities within the designated exclusion area.

The NRC staff verified for consistency the EAB the applicant considered for the radiological consequence evaluations in Chapters 15 and 13.3 of the SSAR.

The applicant stated that two facilities within the EAB have authorized activities unrelated to nuclear plant operations. These are the visitor's center and the GPC combustion turbine plant, Plant Wilson. The applicant also stated that the exclusion area outside the controlled area fence, including along the Savannah River, will be posted and closed to persons who have not received permission to enter the property.

The applicant stated that access to the visitor's center is controlled by security at the pavilion on the entrance road to the plant. Normally, only a few administrative personnel are located at the visitor's center, and the number of visitors at the center is minimal. In the event of emergency conditions at the plant, the emergency plan for the proposed Units 3 and 4 provides for notification of visitors to the center concerning the proper actions to be taken and evacuation instructions.

The applicant also stated that the VEGP staff control Plant Wilson, and locked gates limit access to the facility from New River Road. The emergency plan for the proposed Units 3 and 4 also provides for notification and evacuation of VEGP personnel at Plant Wilson. In addition, the applicant stated that SNC normally will not control passage or use of the Savannah River along the EAB. "No Trespassing" signs are posted near the river indicating the actions to be taken in the event of emergency conditions at the plant.

The NRC staff has evaluated and verified in Section 13.3 of this SER, the emergency plans and detailed information on the activities in the EAB as described above and in SSAR Chapter 13.3 to ensure that proper plans and procedures are in place. The NRC staff concludes that the specified activities unrelated to operation of a nuclear plant or plants that might be constructed on the proposed site within the exclusion area are acceptable.

2.1.2.4 Conclusion

As set forth above, the applicant appropriately described the exclusion area, the authority under which all activities within the exclusion area can be controlled, and the methods by which access and occupancy of the exclusion area can be controlled during normal operation and in the event of an emergency situation. In addition, the applicant has the required authority to control activities within the designated exclusion area, including the exclusion and removal of persons and property, and has established acceptable methods for control of the designated exclusion area. Therefore, the NRC staff concludes that the applicant's exclusion area is acceptable and meets the requirements of 10 CFR Part 100.

2.1.3 Population Distribution

2.1.3.1 Introduction

This section addresses the information provided by the applicant concerning the estimated population distribution surrounding the proposed ESP site up to a 50-mile radius, based on the year 2000 census. Data concerning the resident population distribution within the LPZ, the nearest population center, and population densities up to a 20-mile radius from the proposed site are provided by the applicant. The estimated transient population data out to 50 miles is also provided by the applicant. The cumulative population, including both the resident and transient population in 2000 within the LPZ, within 10 miles of the site, and within 50 miles from the center of the proposed ESP site is presented. The estimated population projections based on a 20-year (1980-2000) growth rate are also presented for the years 2010, 2020, 2030, 2040, and 2070. The established LPZ for the proposed Units 3 and 4 is the same as the LPZ for the existing VEGP, Units 1 and 2, falling within a 2-mile radius of the midpoint between the Units 1 and 2 containment buildings.

2.1.3.2 Regulatory Basis

The acceptance criteria for population distribution are based on the relevant requirements of 10 CFR 50.34, "Contents of Applications: Technical Information;" 10 CFR 52.17; and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the site location and area description:

- 10 CFR 52.17(a)(1)(ix) , insofar as it establishes the dose limits at the EAB and LPZ resulting from potential reactor accidents, as it relates to the requirements of 10 CFR 100.21(c).

- 10 CFR 52.17, insofar as it requires each applicant to provide a description of the existing and projected future population profile of the area surrounding the site.

- 10 CFR Part 100, insofar as it establishes the following requirements with respect to population.

    - 10 CFR 100.20(a), as it relates to population distribution and population density.

    - 10 CFR 100.21(a), which states that every site must have an exclusion area and an LPZ, as defined in 10 CFR 100.3.

- 10 CFR 100.21(b), which states that the population center distance, as defined in 10 CFR 100.3, must be at least one and one-third times the distance from the reactor to the outer boundary of the LPZ.

- 10 CFR 100.3, which defines exclusion area, LPZ, and population center distance.

RS-002, Section 2.1.3, specifies that an applicant has submitted adequate information if it satisfies the following criteria:

- Either there are no residents in the exclusion area, or if so, such residents are subject to ready removal, in case of necessity.

- The specified LPZ is acceptable if it is determined that appropriate protective measures could be taken on behalf of the enclosed populace in the event of a serious accident.

- The population center distance (as defined in 10 CFR 100.3) is at least one and one third times the distance from the reactor to the outer boundary of the LPZ.

- The population center distance is acceptable if there are no likely concentrations of greater than 25,000 people over the lifetime of a nuclear power plant or plants of specified type that might be constructed on the proposed site (plus the term of the ESP) closer than the distance designated by the applicant as the population center distance.

- The boundary of the population center shall be determined upon considerations of population distribution. Political boundaries are not controlling.

- The population data supplied by the applicant in the safety assessment are acceptable if (a) they contain population data for the latest census, projected year(s) of startup of a nuclear power plant or plants of specified type that might be constructed on the proposed site (such date or dates reflecting the term of the ESP) and projected year(s) of end of plant life; (b) they describe the methodology and sources used to obtain the population data, including the projections; (c) they include information on transient populations in the site vicinity; and (d) the population data in the site vicinity, including projections, are verified to be reasonable by other means such as U.S. Census publications, publications from State and local governments, and other independent projections.

- If the population density at the ESP stage exceeds the guidelines given in Position C.4 of Regulatory Guide (RG) 4.7 "General Site Suitability Criteria for Nuclear Power Stations," Revision 2, issued April 1998, special attention to the consideration of alternative sites with lower population densities is necessary. A site that exceeds the population density guidelines of Position C.4 of RG 4.7 can nevertheless be selected and approved if, on balance, it offers advantages compared with available alternative sites when all of the environmental, safety, and economic aspects of the proposed and alternative sites are considered.

Position C.4 of RG 4.7 states that, preferably, a reactor would be located so that, at the time of initial site approval and within about 5 years thereafter, the population density, including weighted transient population, averaged over any radial distance out to 20 miles (cumulative population at a distance divided by the circular area at that distance), does not exceed 500 persons per square mile.

In addition to identifying specific acceptance criteria to meet the relevant requirements, RS-002 also indicates the NRC staff review of population distribution typically involves reviewing the following:

- data about the population in the site vicinity
- the population in the exclusion area
- the LPZ to determine whether appropriate protective measures could be taken on behalf of the populace in that zone in the event of a serious accident
- the nearest boundary of the closest population center containing 25,000 or more residents to determine whether this boundary is at least one and one-third times the distance from the reactor to the outer boundary of the LPZ
- the population density in the site vicinity, including weighted transient population at the time of initial site approval and within 5 years thereafter, to determine whether it exceeds 500 persons per square mile averaged over any radial distance out to 20 miles

2.1.3.3 Technical Evaluation

Following the procedures described in RS-002, Section 2.1.3, the NRC staff reviewed SSAR Chapter 2.1.3 regarding population distribution, as well as the applicant's responses to RAIs 2.1.3-1 through 2.1.3-6.

The NRC staff notes that there are no residents in the exclusion area.

In SSAR Chapter 2.1.3, the applicant estimated and provided the population distribution surrounding the ESP site, up to a 50-mile radius, based on the 2000 census. In this section, the applicant provided the resident population distribution within the LPZ, the nearest population center, and population densities up to a 20-mile radius from the site.

The NRC staff reviewed the population data presented by the applicant in the SSAR, to determine whether the exclusion area, LPZ, and population center distance for the proposed ESP site comply with the requirements of 10 CFR Part 100 and the acceptance criteria described in Section 2.1.3.2 of this SER. The NRC staff also evaluated whether, consistent with Regulatory Position C.4 of RG 4.7, the applicant should consider alternative sites with lower population densities. The NRC staff also reviewed whether appropriate protective measures could be taken on behalf of the enclosed populace within the EPZ, which encompasses the LPZ, in the event of a serious accident.

The NRC staff obtained the 1980 and 2000 U.S. Census Bureau (USCB) population data for the 16 counties in Georgia and the 12 counties in South Carolina that are within a 50-mile radius of the center of the ESP site. By accounting the percentage of each county falling within the 50-mile radius, the NRC staff was able to estimate the 2000 population within the 50-mile radius. The NRC staff also estimated the 1980 population within a 50-mile radius using the same approach. As a confirmatory check, the NRC staff compared the applicant's

2000 population data against the NRC staff's estimated 2000 population data. The NRC staff found that the staff's estimate was within 2 percent of the data that the applicant presented in the SSAR.

The NRC staff also reviewed the projected population data provided by the applicant. The NRC staff reviewed information pertaining to the cumulative populations, including the weighted transient populations, for the years 2010, 2020, 2030, 2040, and 2070. The population projections have been verified for consistency with the population projections presented in Section 13.3 of this SER as part of emergency planning and preparedness. The NRC staff also made confirmatory population projection estimates using annualized growth rates calculated for each county within 50 miles of the site based on data from the USCB Web site. The NRC staff-estimated population projections are slightly higher than the applicant's estimated projections, which may be because of the NRC staff's application of growth rate on a county basis, rather than on a census-block basis within each county. Therefore, the NRC staff deems the applicant's methodology for estimating population projections appropriate, reasonable, and acceptable. If the NRC staff were to approve and issue an ESP in 2010 (assuming a combined operating license (COL) application is submitted at the end of the ESP-approved period of 20 years), with a projected startup of new units in 2030 and an operational period of 40 years, the projected year for end of plant life is 2070. Accordingly, the NRC staff finds that the applicant's projected population data set covers an appropriate number of years and is reasonable.

The NRC staff reviewed the applicant's transient population data. The transient population within a 10-mile radius includes 200 hunters and fishermen at recreational areas along the Savannah River. The transient population between 10 and 50 miles from the VEGP site includes workers at and occupants of colleges, schools, hospitals, a military base, and the SRS. In addition, the thousands of people who visit Augusta and the surrounding area annually during the week of the Masters Tournament and for other annual events are included. Based on this information, the NRC staff finds that the applicant's estimate of the transient population to be reasonable.

The applicant estimated and provided the cumulative population, including a transient population of 50 hunters and fishermen, in the LPZ. No towns, recreational facilities, hospitals, schools, prisons, or beaches are within the LPZ, and River Road is the only road within the LPZ. The applicant evaluated representative design-basis accidents (DBAs) in Chapter 15 of the SSAR, and the NRC staff independently verified the applicant's evaluation in Chapter 15 of this SER to demonstrate that the radiological consequences of design-basis reactor accidents at the proposed ESP site are within the dose limits set forth in 10 CFR 52.17(a)(1)(ix).

The distance to Augusta, Georgia, the nearest population center, is about 26 miles and is well in excess of 2.67 miles (one and one third times the distance of 2 miles from the reactor to the outer boundary of the LPZ). In addition, the applicant, as well as the NRC staff, did not identify any other population center closer than the population center distance, as identified above. Therefore, the NRC staff concludes that the proposed site meets the population center distance requirement, as defined in 10 CFR Part 100, Subpart B. The NRC staff has also determined and concluded, based on the projected cumulative resident and transient population within 10 miles of the site, during the lifetime of plant, that there is no likelihood of a future population center of 25,000 people or more within 2.7 miles of the ESP site.

The NRC staff evaluated the site against the criterion in Regulatory Position C.4 of RG 4.7, Revision 2, regarding whether it is necessary to consider alternative sites with lower population densities. The evaluation included the review and verification of whether the population densities in the vicinity of the proposed site, including the weighted transient population, projected at the time of initial site approval and 5 years thereafter, would exceed the criteria of 500 persons per square mile averaged over a radial distance of 20 miles (cumulative population at a distance divided by the area at that distance). The NRC staff has independently determined population density for the lifetime of the plant based on the NRC staff's confirmatory population projection estimates discussed earlier, and has found that the population densities for the proposed site would be well below this criterion. Therefore, the NRC staff concludes that the site conforms to Regulatory Position C.4 in RG 4.7, Revision 2. Based on the applicant's projected population data and population densities, assuming initial approval of the ESP in 2010, construction beginning at the end of the term of 20 years of the ESP approval, and a plant operating life of 40 years, the NRC staff finds that the site also meets the guidance of RS-002 regarding population densities over the lifetime of facilities that might be constructed on the site. Specifically, the population density over that period is not expected to exceed 500 persons per square mile averaged out to 20 miles from the site.

Based on the information provided by the applicant in SSAR Chapter 13.3, the applicant's response to RAI 2.1.3-3, and the NRC staff's conclusions discussed in Section 13.3 of this SER, the NRC staff finds that appropriate protective measures could be taken on behalf of the populace in the LPZ in the event of a serious accident. Therefore, the NRC staff finds the applicant's response to be satisfactory.

2.1.3.4 Conclusion

As set forth above, the applicant provided an acceptable description of current and projected population densities in and around the site. The NRC staff concludes that the population data provided are acceptable and meet the applicable requirements of 10 CFR Part 52 and 10 CFR Part 100, Subpart B. This conclusion is based on the applicant having provided an acceptable description and safety assessment of the site, which contain present and projected population densities that are within the guidelines of Regulatory Position C.4 of RG 4.7. In addition, the applicant properly specified the LPZ and population center distance. The NRC staff has reviewed and confirmed, by comparison with independently obtained population data, the applicant's estimates of the present and projected populations surrounding the site, including transients. The applicant also evaluated the radiological consequences of DBAs at the proposed site in SSAR Chapter 15 and provided reasonable assurance that appropriate protective measures can be taken within the LPZ to protect the population in the event of a radiological emergency.

2.2 Nearby Industrial, Transportation, and Military Facilities and Descriptions

2.2.1-2.2.2 Identification of Potential Hazards in Site Vicinity

2.2.1.1-2.2.2.1 Introduction

For its ESP application, the applicant provided information on the relative location and separation distance of the site from industrial, military, and transportation facilities and routes in its vicinity. Such facilities and routes include air, ground, and water traffic; pipelines; and fixed manufacturing, processing; and storage facilities. The purpose of the review is to verify that the applicant has submitted sufficient information concerning the presence and magnitude of potential external hazards, so that the reviews and evaluations described in Sections 2.2.3 and 3.5.1.6 can be performed. Section 2.2 of the SSAR covers information concerning the industrial, transportation, and military facilities in the vicinity of the proposed ESP site. The NRC staff prepared Sections 2.2.3 and 3.5.1.6 of this SER using information presented in SSAR, Section 2.2, in accordance with the procedures described in RS-002.

2.2.1.2- 2.2.2.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17, with respect to the requirement that the application contain information on the location and description of any nearby industrial, military, or transportation facilities and routes.

- 10 CFR 100.20(b), which requires that the nature and proximity of man-related hazards (e.g., airports, dams, transportation routes, military and chemical facilities) be evaluated to establish site parameters for use in determining whether a plant design can accommodate commonly occurring hazards, and whether the risk of other hazards is very low.

- 10 CFR 100.21(e), which requires that the potential hazards associated with nearby transportation routes, industrial, and military facilities be evaluated and site parameters established such that potential hazards from such routes and facilities will not pose undue risk to the type of facility proposed to be located at the site.

RS-002, Section 2.2.1-2.2.2, specifies that an applicant has submitted adequate information to meet the above requirements, if the submitted information satisfies the following criteria:

- data in the site safety assessment adequately describes the locations and distances of industrial, military, and transportation facilities in the vicinity of the plant, a nuclear power plant or plants of specified type that might be constructed on the proposed site, and are in agreement with data obtained from other sources, when available.
- descriptions of the nature and extent of activities conducted at the site and nearby facilities, including the products and materials likely to be processed, stored, used, or transported, are adequate to permit identification of possible hazards.

- sufficient statistical data with respect to hazardous materials are provided to establish a basis for evaluating the potential hazard to a nuclear power plant or plants of specified type that may be constructed on the proposed site.

2.2.1.3-2.2.2.3 Technical Evaluation

Following the procedures detailed in RS-002, Sections 2.2.1-2.2.2, the NRC staff evaluated the potential for man-made hazards in the vicinity of the proposed ESP site by reviewing

- information the applicant provided in Section 2.2.1-2.2.2 of the SSAR,
- information the NRC staff obtained during a visit to the proposed ESP site and its surrounding vicinity,
- other publicly available reference material, such as U.S. Geological Survey (USGS) topographic maps, geographic information system (GIS) information, road and railroad maps, and electric transmission lines and natural gas pipeline maps, and
- information the NRC staff collected independently from such sources as state and local authorities.

In SSAR Chapters 2.2.1 and 2.2.2, the applicant identified and described the following facilities and routes, within a 5-mile radius of the existing VEGP site, which may generate potential hazards or which may engage in potentially hazardous activities:

- Georgia State Highway 23,
- the CSX Railroad,
- Plant Wilson, a combustion turbine electrical plant owned by the GPC,
- the SRS,
- a coal-fired steam electrical plant operated by Washington Savannah River Company in the D-Area of the SRS,
- VEGP Units 1 and 2,
- the Chem-Nuclear Systems radioactive disposal site (18 miles east of the proposed site) in South Carolina, and
- the Unitech Service Group Nuclear Laundry Facility (21 miles east of the proposed site) in South Carolina.

The applicant included maps that show the locations of these facilities and routes (along with airways and military operations) in comparison to the proposed ESP site (SSAR Figures 2.2.2 and 2.2.3). The applicant presented descriptions of these facilities and routes in SSAR Chapter 2.2.2.

In SSAR Chapter 2.2.2.3, the applicant described the roads within a 5-mile radius of the site. Segments of Georgia State Highways 23, 80, and 56 Spur are located within a 5-mile radius. The nearest highway with commercial traffic is Georgia State Highway 23. State Highway 23 serves as a major link between Augusta and Savannah. The heaviest truck traffic along State Highway 23, near the proposed site, consists primarily of timber and wood products and materials. In SSAR Table 2.2-3, the applicant provided available statistical data on personal injury accidents on these roads between 1999 and 2003.

SSAR Chapter 2.2.2.4 states that the CSX Railroad in South Carolina is the nearest railroad with commercial traffic and is approximately 4.5 miles northeast of the VEGP site. The CSX Railroad runs through and services the SRS. The railroad carries a number of major chemical substances, including cyclohexane, anhydrous ammonia, carbon monoxide, molten sulfur, and elevated temperature material liquids (ETMLs).

(Two local Norfolk Southern rail lines exist in Burke County, operated by Norfolk Southern, one through Waynesboro and one through Midville. These rail lines are approximately 12 miles west of the VEGP site.)

Plant Wilson is located approximately 6000 feet east-southeast from the proposed VEGP, Units 3 and 4. This combustion turbine plant is a GPC electrical peaking power station. The plant consists of six combustion turbines with a total rated capacity of 351.6 MW. The storage capacity of the fuel oil storage tanks at Plant Wilson is 9,000,000 gallons.

The SRS borders the Savannah River for approximately 17 miles opposite the VEGP site. It occupies an approximately circular area 310 square miles (198, 344 acres), encompassing parts of Aiken, Barnwell, and Allendale Counties in South Carolina. The SRS is owned by DOE and operated by an integrated team led by the Washington Savannah River Company. The site is a closed Government reservation except for through traffic on South Carolina Highway 125 and the CSX railroad. The current and near-term operating SRS facilities are engaged in various activities. The SRS processes and stores nuclear materials in support of the national defense and the U.S. non-proliferation efforts. This site also develops and deploys technologies to improve the environment and treat nuclear and hazardous wastes left from the Cold War. Because the SRS facilities are distant (i.e., more than 17 miles) from the proposed units, they are not considered to pose a viable threat to the safe operation of the proposed units.

Washington Savannah River Company operates the 70 megawatt coal-fired steam and electrical plant in the D-Area of SRS. This plant has been in operation since 1952 and supplies steam and electricity to several facilities throughout the SRS.

Chem-Nuclear Systems developed, constructed, and currently operates the largest radioactive waste disposal site in the country, near Barnwell, South Carolina. In addition, Unitech Services Nuclear laundry facility is located in the Barnwell County Industrial Park and provides radiological laundry and respirator services. However, these facilities are not considered to be an external hazard to the proposed nuclear units because of their distance (18 and 21 miles, respectively) from the VEGP site.

The existing VEGP Units 1 and 2, are located about 3600 feet and 3900 feet respectively, west of the Savannah River. Besides the activities at Plant Wilson, the only other activities unrelated to plant operations that may occur within the exclusion area are those associated with the operation of the visitor's center. VEGP has made arrangements to control and, if necessary, evacuate the exclusion area in the event of an emergency.

In SSAR Chapter 2.2.2.1, the applicant referenced the "Burke County Comprehensive Plan: 2010, Part 1," which forecasts a relatively slow, stable population growth pattern for Burke County, indicative of the fact that nearby industries have not significantly grown. The applicant stated that currently no major development of industrial, military, or transportation facilities is projected to occur within a 25-mile radius of the VEGP site, except for the development of proposed VEGP Units 3 and 4.

The applicant also identified and described in SSAR, Chapter 2.2.2, the nature, extent, and location of any:

- mining activities,
- commercially-traversable waterways,
- airports,
- airways,
- military-operation areas and routes,
- natural gas or petroleum pipelines,
- military facilities, and
- storage tanks and chemicals found on the current VEGP site.

In SSAR Chapter 2.2.2.2, the applicant stated that no mining activities occur within 5 miles of the VEGP site.

SSAR Chapter 2.2.2.5 states that the footprint of the proposed VEGP Units 3 and 4 is located about 4850 feet southwest of the Savannah River. The small amount of water traffic on the Savannah River that does exist is primarily composed of barge-tug tows moving up and down the river channel out of the Port of Savannah. There are no locks or dams in the vicinity of the proposed plant site. In 2004, only 13 commercial vessels were recorded on the Savannah River below Augusta. Within this section of the river, a total of less than 500 tons of nonexplosive residual fuel oil was transported near or past the VEGP site. Except for the residual fuel oil, there were no flammable or potentially explosive materials transported on this portion of the Savannah River. However, in its response to the NRC staff's RAI dated March 16, 2007, the applicant stated that fuel oil is no longer transported by barge past the VEGP site, and the barge hazard has been eliminated from additional consideration. The proposed intake structure is located approximately 1800 feet upstream of the existing VEGP Units 1 and 2 intake structures.

In SSAR Chapter 2.2.2.6.1, the applicant addressed nearby airports. There are no airports within 10 miles of the VEGP site. The closest airport, Burke County Airport, is approximately 16 miles west-southwest of the site. The average number of operations (landings and takeoffs) is about 57 per week. The closest commercial airport is the Augusta Regional Airport at Bush Field, which is located approximately 17 miles north-northwest of the VEGP site. Based on Federal Aviation Administration (FAA) information, 17 aircraft are based on the field, of which 10 are single-engine airplanes, 4 are multi-engine airplanes, and 3 are jet-engine airplanes. The average number of operations is about 91 per day. Approach and departure paths at Bush Field are not aligned with the VEGP site, and no regular air traffic patterns for Bush Field extend into the airspace over the VEGP site.

A small, un-improved grass airstrip is located immediately north of the VEGP site (north of Hancock Landing Road and west of the Savannah River). At its closest point, the airstrip is about 1.4 miles from the power block of the proposed new units. This privately owned and operated airstrip has a 1650-foot runway oriented east-west. Therefore, the takeoffs and landings are tangential to the site and oriented away from the plant. No FAA information is available for this airstrip. Informal communication with the owner and operator revealed that the airstrip is for personal use, and the associated traffic consists only of small single-engine aircraft. In addition, there is a small helicopter landing pad on the VEGP site. This facility exists for corporate use and for use in case of an emergency. The traffic associated with both of these facilities is characterized as sporadic.

In Section 2.2.2.6.2 of the SSAR, the applicant addresses airways. The applicant stated that the centerline of Airway V185 is approximately 1.5 miles west of the VEGP site. Additionally, Airway V417 is about 12 miles northeast of the VEGP site, and Airway V70 is approximately 20 miles south of the VEGP site. Because of its close proximity to the VEGP site, SSAR Chapter 3.5.1.6 evaluates hazards from air traffic along the V185 airway.

Section 2.2.2.6.3 of the SSAR describes military air training routes. The west edge of the Pointsett Military Operation Area (MOA) is about 75 miles east-northeast of the VEGP site. The east edge of the Bulldog MOAs is about 11 miles west of the VEGP site. Military aircraft in the Bulldog MOA come mainly from Shaw Air Force Base (about 32 miles east of Columbia, South Carolina) and McEntire Air National Guard Station (about 13 miles east-southeast of Columbia). Among the military training air routes, VR97-1059 is located closest to the VEGP site. The distance between the centerline of VR97-1059 and the VEGP site is about 18 miles. The maximum route width of VR97-1059 is 20 nautical miles; therefore, the width on either side of the route centerline is assumed to be 10 nautical miles (11.5 miles). The VEGP site is located more than 6 miles from the edge of this training route. The total number of military aircraft using route VR97-1059 is approximately 833 per year.

In Section 2.2.2.7 of the SSAR, the applicant addressed the existence of natural gas and petroleum pipelines nearby the VEGP site. The applicant stated that there are three natural gas pipelines within 25 miles of the VEGP site (However, none are located within 10 miles of the VEGP site):

- Pipeline 1 is located approximately 21 miles northeast of the VEGP site.
- Pipeline 2 is located approximately 19 miles southwest of the VEGP site.
- Pipeline 3 is located approximately 20 miles northwest of the VEGP site.

Section 2.2.2.8 of the SSAR describes any existing nearby military facilities. The applicant stated that no military facilities are within 5 miles of the VEGP site.

Section 2.2.2.9 of the SSAR addresses the existence of any storage tanks and chemicals currently held on the VEGP site. The list of such chemicals can be found in the SSAR on Table 2.2.5.

Based on its review of the information provided by the applicant in SSAR Chapter 2.2.1-2.2.2, as supplemented by responses to the NRC staff's RAI 2.2.2-1 and 2.2.2-2, and the information discussed above, the NRC staff did not identify any potential source of additional hazards beyond those that the applicant has identified and described.

2.2.1.4-2.2.2.4 Conclusion

As set forth above, the applicant provided information in the SSAR regarding potential site hazards in accordance with RS-002, such that compliance with the requirements of 10 CFR 52.17, 10 CFR 100.20(b) and 10 CFR 100.21(e) can be evaluated. In the SSAR, the applicant identified the facilities and reviewed the nature and extent of activities involving potentially hazardous materials on or in the vicinity of the site and identified hazards that might pose undue risk to the proposed nuclear facility. Based on the information presented in the SSAR, as well as information the NRC staff obtained independently, the NRC concludes that all potential hazards and potentially hazardous activities on and in the vicinity of the site have been identified. These potential hazards and potentially hazardous activities have been reviewed and are discussed in Sections 2.2.3 and 3.5.1.6 of this safety evaluation report (SER).

2.2.3 Evaluation of Potential Accidents

2.2.3.1 Introduction

In this section of the SER, Section 2.2.3, the NRC staff documents its review and evaluation of potential accident sequences on and in the vicinity of the proposed ESP site, such as an explosion of a flammable substance or a release of a toxic chemical. The NRC staff reviews the applicant's probability analyses of potential accident sequences involving hazardous materials or activities on the proposed ESP site and its vicinity to determine that appropriate data and analytical models have been utilized and to ensure that the calculated risks associated with potential accident sequences are sufficiently low.

2.2.3.2 Regulatory Basis

The acceptance criteria for the evaluation of potential accidents are based on meeting the relevant requirements of 10 CFR 52.17, 10 CFR 100.20 and 10 CFR 100.21, as they relate to factors considered in site evaluation. These requirements stipulate that individual and societal risk of potential plant accident sequences must be low. The NRC staff considered the following regulatory requirements in evaluating the potentiality and consequences of accident sequences:

- 10 CFR 52.17, with respect to the requirement that the application contain information on the location and description of any nearby industrial, military, or transportation facilities and routes.

- 10 CFR 100.20(b), which states that the nature and proximity of man-related hazards (e.g., airports, dams, transportation routes, military and chemical facilities) be evaluated to establish site parameters for use in determining whether a plant design can accommodate commonly occurring hazards, and whether the risk of other hazards is very low.

- 10 CFR 100.21(e), which requires that the potential hazards associated with nearby transportation routes, industrial, and military facilities be evaluated and site parameters established such that potential hazards from such routes and facilities will not pose undue risk to the type of facility proposed to be located at the site.

RS-002, Section 2.2.3 specifies that an application meets the above requirements, if the application satisfies the following criteria:

- None of the identified potential accidents are design basis events. A design basis event is defined as an accident that has a probability of occurrence on the order of 10^{-7} per year (or greater) and the expected rate of radiological exposure, as a postulated consequence of the accident, is in excess of 10 CFR 100.21 exposure standards.

If any of the identified potential accidents are considered design basis events, a detailed analysis is required, for each of the accidents so categorized, of the effects of the accident on the plant's safety-related structured and components. Because of the difficulty of assigning accurate numerical values to the expected rate of unprecedented potential hazards, on the probabilistic order of 10^{-7}, the NRC staff employed its judgment as to the acceptability of the overall risk calculated for a potential accident.

To evaluate the information provided in SSAR 2.2.1-2.2.2 per the above acceptance criteria, applicant applied the NRC-endorsed analytical methodologies found in the following:

- RG 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," Revision 3, issued November 1978, which defines design basis events external to the nuclear plant as those accidents that have a probability of occurrence on the order of about 10^{-7} per year or greater.

- RG 1.78, "Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release," issued December 2001.

- RG 1.91, "Evaluation of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plant Sites," Revision 1, issued February 1978.

When independently assessing the applicant's analysis in SSAR Chapter 2.2.3, the NRC staff applied the same above-cited analytical methodologies.

2.2.3.3 Technical Evaluation

The NRC staff reviewed the information presented in SSAR Chapter 2.2.3 of the VEGP ESP application pertaining to potential accidents, as well as the applicant's responses to RAIs 2.2.3-1 through 2.2.3-16.

The applicant analyzed postulated accidents for various types, sources and locations:

- explosions and flammable vapor clouds
- release of hazardous chemicals
- fires
- radiological hazards

The applicant reviewed the existing analysis of potential hazards to VEGP Units 1 and 2 to determine its applicability to the proposed VEGP Units 3 and 4, in evaluating the postulated releases of flammable materials and toxic gases from transportation accidents or materials stored at industrial facilities within a 5-mile radius of the VEGP site. In addition, the applicant evaluated new chemicals identified for either VEGP Units 1 and 2, or VEGP Units 3 and 4, to determine their impact on the proposed VEGP Units 3 and 4. The NRC staff has reviewed the applicant's analyses and has made independent confirmatory checks and calculations to determine the applicant's conformance to the requirements and the applicant's reasonableness and approach in assessing these potential hazards.

2.2.3.3.1 Explosions and Flammable Vapor Clouds

Truck Traffic

The applicant analyzed the potential consequences of explosions postulated to occur on transportation routes near the proposed ESP site using the methodology given in RG 1.91. RG 1.91 details a method for determining distances from critical plant structures to a railway, highway, or navigable waterway beyond which any explosion that might occur on these transportation routes is not likely to have an adverse effect on plant operation or to prevent a safe shutdown. Under those conditions, a detailed review of the transport of explosives on those transportation routes would not be required. The RG 1.91 methodology is based on a level of peak positive incident over-pressure, below which no significant damage would be expected to plant structures. The NRC staff, in RG 1.91, conservatively chose 1 psi for this level. The calculation to determine the minimum safe distance at the chosen peak positive incident over-pressure (1 psi) is as follows:

$R > kW$ 1/3, whereas R is the distance in feet from an exploding charge of W pounds of trinitrotoluene (TNT). When R is in feet and W is in pounds, k = 45. When R is in meters and W is in kilograms, k = 18.

The concept of TNT equivalence (i.e, finding the mass of substance in question that will produce the same blast effect as a unit mass of TNT) has long been used in establishing safe separation distances for solid explosives.

Based on the previous analysis done for VEGP Units 1 and 2, the applicant identified six chemicals as potential hazards when transported by truck. The applicant used the U.S. Environmental Protection Agency (EPA) Tier II reports for Burke and Richmond Counties in Georgia, along with the EPA Landview database to confirm and/or update the list of chemicals for the analysis. The applicant also performed a traffic corridor evaluation, which showed that even fewer chemicals pass by the site now than was previously assumed in the analysis for Units 1 and 2. The applicant concluded that the only hazardous chemicals likely transported by truck in the vicinity of the site are gasoline and diesel/fuel oil.

Georgia State Highway 23 is the closest ground route to the VEGP site, by which the previously-identified chemicals are being transported by truck. The nearest point from State Highway 23 to the center of VEGP Units 1 and 2, is 4.7 miles and to the center of VEGP, Units 3 and 4, 4.2 miles. The applicant concluded that, due to the distance between Highway 23 and the proposed ESP site, any explosions induced by flammable clouds of these chemicals will not adversely affect the safe operation of the proposed units. The NRC staff independently confirmed these findings using the methodology described in RG 1.91. For an explosion from a flammable cloud, the maximum distance that would result in a peak incident blast pressure of 1 psi is conservatively determined to be 2479 feet from the road.

For an 8500-gallon gasoline truck carrying a TNT equivalent of 56,165 pounds, the critical distance would be 1723 feet from the explosion point. Since the above calculated critical distances of 2479 feet and 1723 feet for the two types of explosions discussed, are much less than 4.2 miles, the distance between Highway 23 (at its closest point) and proposed

Units 3 and 4, the NRC staff concludes that the potential explosion of a gasoline truck would not adversely impact the safe operation of the plant.

In addition to the above-discussed highway transit, gasoline is delivered to the site by tank wagon containing a maximum volume of 4000 gallons. For an explosion from a 4000 gallon truck, the NRC staff calculated the critical distance (beyond which the blast pressure would be less than 1 psi) to be 1340 feet. For an explosion from a flammable cloud in the equivalent circumstances, the critical distance is 1658 feet. The closest distance from the site delivery route to the power block circle is approximately 2000 feet. That distance is greater than the above calculated critical distances. Therefore, the NRC staff concludes that the potential explosion of a gasoline delivery tank truck would not have an adverse impact on the safety of the plant operation. Because of its higher quantity and TNT equivalent and because it is more volatile than diesel fuel, gasoline impacts are considered bounding for the truck-borne hazards evaluation.

<u>Pipelines and Mining Facilities</u>

No natural gas pipeline or mining facilities are located within 10 miles of the VEGP site. Based on RG 1.70, because there are no pipelines or mining activities within 5 miles of the VEGP site, the applicant did not evaluate potential hazards from this source.

<u>Waterway Traffic</u>

The potential impact of barge traffic was analyzed for VEGP, Units 1 and 2. However, the current use of the Savannah River and the lack of commercial facilities and barge slips/docks upstream of the plant indicate that there is no current or projected barge traffic on the Savannah River past the VEGP site. Because the Savannah River is not being used to transport chemicals by barge, a hazard evaluation was not required.

<u>Railroad Traffic</u>

The nearest railroad to the VEGP site is the CSX Railroad, which is approximately 4.5 miles northeast of the center point of VEGP, Units 1 and 2. Based on the information obtained from CSX, the top four U.S. Department of Transportation (DOT) qualified hazardous chemicals are cyclohexane (64 percent), anhydrous ammonia (9 percent), carbon monoxide (3 percent), and ETML (3 percent). Because cyclohexane is both flammable and toxic, it was analyzed in detail to evaluate the potential for an explosion hazard from a railcar and from a flammable vapor cloud.

For the explosion from a railcar, the equivalent TNT mass of 117.5 pounds, based on an Upper Flammability Limit (UFL) of 8.34 percent of cyclohexane at the point of release, would produce a peak overpressure of 1 psi at a distance of 220 feet from the railroad. For an explosion from a flammable vapor cloud, the TNT-equivalent maximum distance beyond which the blast pressure would be less than 1 psi is calculated to be 1026 feet from the railcar. The separation distance between the railroad and the proposed units is 4.5 miles, which is far greater than the above calculated critical distances. Even for a maximum railcar load of 132,000 pounds, the critical distance that could cause a peak overpressure of 1 psi to safety-related structures from an explosion or flammable vapor-cloud-induced explosion is calculated to be 2293 ft. Since the amounts of chemicals transported are much lower than the maximum railcar load, and that the actual distance (approximately 4.5 miles) between the railroad and the VEGP site is greater than the critical distance of 2293 ft, the NRC staff has determined that if such an explosion were to occur, it would not pose a hazard to safety-related structures at the plant.

2.2.3.3.2 Release of Hazardous Chemicals

Using the methodology found in RG 1.78, the applicant analyzed the potential impacts of hazardous chemical releases on control room habitability. RG 1.78 provides guidance on the detailed evaluation of such release events and describes assumptions and criteria for screening out release events that need not be considered in the evaluation of control room habitability. RG 1.78 provides that chemicals stored or situated at distances greater than 5 miles from the plant need not be considered because, if a release occurs at such a distance, atmospheric dispersion will dilute and disperse the incoming plume to such a degree that either toxic limits will never be reached or there would be sufficient time for the control room operators to take appropriate action. In addition, the probability of a plume remaining within a given sector for a long period of time is small. Likewise, if hazardous chemicals are known or projected to be shipped by rail, water, or road routes outside a 5-mile radius of nuclear power plant, the shipments need not be considered further for evaluation.

As another screening criteria, for stationary sources of hazardous chemicals within the 5-mile radius of a nuclear power plant, a detailed analysis need only be performed if the hazardous chemicals are in quantities greater than the limits provided in RG 1.78 for a toxicity limit and stable meteorological conditions. Mobile sources, within the 5-mile radius, need not be considered further if the total shipment frequency for all hazardous chemicals (i.e., all hazardous chemicals considered as a singular cargo category without further distinction of the nature of those chemicals) does not exceed the specified number by traffic type (10 shipments per year for truck traffic, 30 per year rail traffic, or 50 per year for barge traffic - these frequencies are based on transportation accident statistics, conditional spill probability given an accident, and a limiting criterion for the number of spills or releases). Frequent shipments (i.e., shipments exceeding the specified number by traffic type) do not need to be considered in detailed analysis if the quantity of hazardous chemicals is less than the quantity provided in RG 1.78 (as adjusted for the appropriate toxicity limit, meteorology, and control room air exchange rate).

Since there are no manufacturing plants, chemical plants, storage facilities, or oil or gas pipelines are located within 5 miles of the VEGP site, only the following potential scenarios were evaluated:

<u>**Release of Hazardous Chemicals from a Transportation Accident**</u>

The applicant concluded that the only hazardous chemicals likely to be transported by truck in the vicinity of the VEGP site are gasoline and diesel/fuel oil. Therefore, the control room habitability analysis conducted by the applicant only included those two chemicals. Because gasoline is more volatile than diesel/fuel oil, the applicant applied the flammable properties of gasoline for the purposes of the analysis. Per the analytical methodology in RG 1.78, the calculated toxic vapor concentration of gasoline at the control room resulting from a release of gasoline from a 8500 gallon truck on Georgia State Highway 23 (4.2 miles from VEGP, Units 3 and 4) is 34.9 parts per million, and from a 4000 gallon tank wagon during delivery (2000 feet from the center of the power block for Units 3 and 4) is 95.1 parts per million. The calculated vapor concentrations are much smaller than the toxicity limit of 300 parts per million (American Conference of Governmental Industrial Hygienists Threshold Limit Value) and, therefore, the applicant asserted that no adverse impact on control room habitability from the accidental release of gasoline or diesel/fuel oil is expected. The NRC staff has reviewed and verified the applicant's information through independent analysis. The NRC staff has found the applicant's methodology to be acceptable and the results and conclusions to be reasonable. Based on the above information, the NRC staff concludes that the accidental release of gasoline or diesel/fuel oil by truck transportation would not cause concentrations of these chemicals to affect control room habitability at or above the corresponding toxicity limits.

The information obtained by the applicant from CSX revealed that the railroad carried four major hazardous chemicals in 2005: cyclohexane, anhydrous ammonia, carbon monoxide, and ETMLs. Accidental spills of carbon monoxide or ETMLs are not expected to create a vapor hazard for the site, as they are molten nonhazardous materials. Therefore, evaluations were performed for cyclohexane and anhydrous ammonia. Assuming a railcar capacity of 67 tons of cyclohexane (based on RG 1.91 limit of 132,000 pounds for a railcar load) and 26 tons of anhydrous ammonia (analyzed previously for VEGP Units 1 and 2), the vapor concentrations at the control room, which is approximately 4.5 miles from railroad, were estimated based on stable atmospheric conditions using a windspeed of 1 meter per second (m/s). The calculated vapor concentration of 34.3 parts per million for cyclohexane is much less than the toxicity limit of 1300 parts per million, and the calculated concentration of 112 parts per million for anhydrous ammonia is also less than the toxicity limit of 300 parts per million. The NRC staff reviewed the applicant's calculations of the concentrations of these chemicals and conducted independent confirmatory analyses using the methodology provided in RG 1.78. In light of the above evaluation and analyses, the NRC staff finds that the applicant's approach and calculations are reasonable and its conclusions acceptable. Based on these estimated toxic vapor concentrations for these chemicals, the NRC staff has determined that the potential hazard from these chemicals is minimal and will not affect the safe operation of the proposed units.

<u>Potential Hazard from Major Depots or Storage Areas</u>

The applicant stated that the only chemical storage areas within 5 miles of the VEGP site are located at the SRS and the Plant Wilson combustion turbine plant. The original analysis performed for VEGP, Units 1 and 2 discussed the storage at SRS "D-Area" (which is 4.5 miles from the center of Units 1 and 2) and of the chemicals chlorine and ammonia. Since these chemicals (or any others) are no longer used at D-Area, the analysis for VEGP Units 3 and 4 considered only the chemicals stored at Plant Wilson.

The chemicals stored at Plant Wilson (approximately 5500 feet from the new power block of Units 3 and 4) consist of three 3-million gallon tanks of fuel oil, sulfuric acid, and several other chemicals in small quantities. Because the sulfuric acid and the other chemicals are present in small quantities and have low volatility and toxicity, the applicant stated that they do not pose a potential hazard to control room habitability. Therefore, the applicant only analyzed one of the 3-million gallon fuel oil tanks, as a bounding case, for the toxic vapor concentration from potential accidental release. The applicant estimated the vapor concentration of fuel oil to be less than 50 parts per million at 5500 feet from the storage tank. Since the calculated concentration is much less than the toxicity limit of 300 parts per million, the applicant concluded that the Plant Wilson fuel oil storage tanks do not present a hazard to VEGP Units 3 and 4. The NRC staff conducted a confirmatory analysis and found that the calculated concentration is much less than the toxicity limit of 300 parts per million.

<u>Potential Hazard from Onsite Storage Tanks</u>

SSAR, Table 2.2-5 lists the chemicals that are stored at VEGP. Of the many chemicals listed that are stored and used on the site, only three chemicals, hydrazine, phosphoric acid, and methoxypropylamine (MPA), were evaluated by the applicant for potential hazard effects that would be bounding. Phosphoric acid and MPA are new chemicals that are being used at VEGP, Units 1 and 2. The applicant stated that the other listed chemicals were not considered for evaluation based on low volatility, low toxicity, or the relatively small quantities stored. In evaluating the control room habitability conditions, the applicant used the guidelines of NUREG-0570, "Toxic Vapor Concentrations in the Control Room Following a Postulated Accidental Release," to determine the toxic concentrations of these chemicals at the control room intake.

Hydrazine is stored northeast of the VEGP Unit 1 reactor and is separated by a minimum distance of 1800 feet from Units 3 and 4. The applicant's analysis of the hydrazine for Units 1 and 2 showed that at least 2 minutes would be available between detection and the time the short-term toxicity limit (as defined in RG 1.78) would be reached. Since hydrazine storage is separated by 1800 feet for Units 3 and 4, the impact on the new units from an accidental release of hydrazine would be less than the impact on the existing VEGP Units 1 and 2. Due to the impact on control room habitability, these calculations will be evaluated at the time of the COL application. This is **COL Action Item 2.2-1**. When addressing this COL action item, Section 6.4 of the FSAR should also be taken into consideration.

Phosphoric acid is stored in a 5050-gallon tank at a distance of approximately 3200 feet from the air intake for the Unit 3 control room. The applicant calculated phosphoric acid concentration outside the control room intake under stable conditions (F stability) with 1 m/s windspeed to be 94 microgram/m^3, much lower than the 8-hour threshold limit value of 1 milligram/m^3 and the short-term exposure limit of 3 milligram/m^3.

The applicant had previously evaluated MPA for VEGP Units 1 and 2. The applicant calculated the MPA release concentration based on a 400-gallon release at 59 meters from the control room intake under atmospheric conditions of 2.5 m/s wind speed and G stability. Using these parameters, the applicant calculated the MPA concentration for VEGP Units 1 and 2 to be 1.5 parts per million, which is much lower than the short term exposure limit of 15 parts per million. Since VEGP Units 3 and 4 would be farther away from the MPA release point than VEGP Units 1 and 2, the MPA concentration at the new control room intake is expected to be lower than that calculated for VEGP Units 1 and 2.

SSAR Table 2.2-6 lists the chemicals that will be used at Units 3 and 4. However, the applicant did not provide the quantity of chemicals. Potential toxic concentrations of these chemicals based on their volatility, toxicity, and quantity, including their impact on control room habitability, will be evaluated at the time of the COL application. This is **COL Action Item 2.2-2**. When addressing this COL action item, Section 6.4 of the FSAR should also be taken into consideration.

The NRC staff used screening models (ALOHA, 2007; HPAC, 2005) to perform confirmatory analyses to independently determine the toxic concentrations of the above discussed chemicals. The NRC staff's estimated concentrations are comparable to those calculated by the applicant. Based on the NRC staff's confirmatory checks, the staff concludes that the applicant's assumptions, and its approach in determining the toxic concentrations of these chemicals at the control room intake, are reasonable and acceptable. Therefore, the NRC staff agrees with the applicant's conclusion that the control room will remain habitable for most release scenarios without any operator action. Furthermore, the applicant demonstrated that in the hydrazine release scenario, control room operators will have sufficient time to take emergency action (e.g., donning emergency breathing apparatus).

2.2.3.3.3 Fires

The preceding sections addressed the potential fire hazards associated with transportation accidents, industrial storage facilities, and onsite storage. The applicant considered the fire hazard from a forest fire resulting in release of potentially toxic chemicals CO, NO2, and CH4, and determined that such a scenario would produce only negligible concentrations outside the control room air intakes. In addition, because of the long distances separating the tree line from the control room, the NRC staff finds that there would be no adverse heat impact in the form of heat flux from the forest fire.

2.2.3.4 Radiological Hazards

Radiation monitoring of the main control room environment is provided by the radiation monitoring system. The habitability systems are capable of maintaining the main control room environment suitable for prolonged occupancy throughout the duration of postulated accidents that require protection from external fire, smoke, and airborne activity. In addition, safety related SSCs have been designed to withstand the efforts of radiological events and consequential releases. However, this site-specific information would be reviewed in Chapters 11 and 15 of a COL application.

2.2.3.5 Conclusion

The NRC staff has reviewed the applicant's potential accidents analysis using the procedures set forth in RS-002, Section 2.2.3. As discussed, the NRC staff has made confirmatory checks and calculations and has verified the applicant's evaluation of potential accidents by using screening models with conservative assumptions and comparing and verifying pertinent data available in the literature.

Based on these considerations, the NRC staff concludes that the potential accidents considered by the applicant would allow for a determination of whether a plant design is adequate to accommodate potential hazards in the site vicinity. Therefore, the NRC staff finds that, with respect to the hazards associated with evaluated potential accidents, the proposed site is acceptable for the planned units and the site meets the relevant requirements of 10 CFR 52.17, 10 CFR 100.20(b), and 10 CFR 100.21(e).

2.3 Meteorology

To ensure that a nuclear power plant or plants can be designed, constructed, and operated on an applicant's proposed ESP site in compliance with the Commission's regulations, the NRC staff evaluates regional and local climatological information, including climate extremes and severe weather occurrences that may affect the design and siting of a nuclear plant. The staff reviews information on the atmospheric dispersion characteristics of a nuclear power plant site to determine whether the radioactive effluents from postulated accidental releases, as well as routine operational releases, are within Commission guidelines. The staff has prepared Sections 2.3.1 through 2.3.5 of this SER in accordance with the review procedures described in RS-002, using information presented in Section 2.3 of the SSAR, responses to staff requests for additional information (RAIs), and generally available reference materials (as cited in applicable sections of RS-002).

2.3.1 Regional Climatology

2.3.1.1 Introduction

In Section 2.3.1 of the SSAR, the applicant presented information on the climatic conditions and regional meteorological phenomena (both the averages and extremes thereof) that could affect the design and operating bases of safety- and/or nonsafety-related SSCs for the proposed nuclear power plant. Specifically, the applicant provided the following information:

- data sources used to characterize the regional climatological conditions pertinent to the proposed site.

- a description of the general climate of the region with respect to types of air masses, synoptic features (high- and low-pressure systems), general airflow patterns (wind direction and speed), temperature and humidity, and precipitation (rain, snow, and sleet).

- frequencies and descriptions of severe weather phenomena that have affected the proposed site, including extreme wind, tornadoes, tropical cyclones, precipitation extremes, winter precipitation (hail, snowstorms, and ice storms), and thunderstorms (including lightning).

- a justification as to why the identification of meteorological conditions associated with the ultimate heat sink (UHS) maximum evaporation and drift loss of water and minimum water cooling is not necessary for a description of design-basis dry- and wet-bulb temperatures for the proposed site.

- a description of design-basis dry- and wet-bulb temperatures for the proposed site.

- the potentiality for restrictive air dispersion conditions and high air pollution at the proposed site.

Based on the above information, the applicant provided a table, SSAR Table 1-1, of proposed site characteristics. Site characteristics are the actual physical, environmental, and demographic features of a site and are used to verify the suitability of a proposed plant design for a site. The following are climatic site characteristics the applicant proposed to define the site:

- the maximum winter precipitation load (i.e., 100-year snowpack and 48-hour probable maximum winter precipitation (PMWP)) on the roofs of safety-related structures.

- tornado parameters, including maximum wind speed, maximum rotational and translational wind speed, the radius of maximum rotational wind speed, the maximum pressure drop, and the maximum rate of pressure drop.

- the 100-year return period straight-line (basic) wind speed.

- ambient air temperature and humidity extremes, including maximum dry-bulb (2-percent and 0.4-percent annual exceedance with concurrent mean wet-bulb temperatures; 100-year return period); minimum dry-bulb (99-percent and 99.6-percent annual exceedance; 100-year return period); and maximum wet-bulb (0.4-percent annual exceedance; 100-year return period).

- The site temperature basis for the AP1000, including the maximum safety dry-bulb temperature and coincident wet-bulb temperature; maximum safety noncoincident wet-bulb temperature; maximum normal dry-bulb temperature and coincident wet-bulb temperature; and maximum normal noncoincident wet-bulb temperature.

2.3.1.2 Regulatory Basis

The acceptance criteria for identifying regional climatological and meteorological information are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The staff considered the following regulatory requirements in reviewing the applicant's identification of regional climatological and meteorological information:

- 10 CFR 52.17(a), which requires that the application contain a description of the seismic, meteorological, hydrological, and geological characteristics of the proposed site.

- 10 CFR 100.20(c), which requires that the meteorological characteristics of the site, necessary for safety analysis or that may have an impact on plant design, be identified and characterized as part of the NRC's review of the acceptability of a site.

- 10 CFR 100.21(d), which requires that the physical characteristics of the site, including meteorology, geology, seismology, and hydrology be evaluated and site parameters established, such that the potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site.

The climatological and meteorological information assembled in compliance with the above regulatory requirements would be necessary to determine, at the COL stage, a proposed facility's compliance with the following requirements in Appendix A of 10 CFR Part 50:

- GDC 2, which requires that structures, systems and components important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

- GDC 4, "Environmental and Dynamic Effects Design Bases," which requires that SSCs important to safety be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, included loss-of-coolant accidents.

An ESP applicant, though, need not demonstrate compliance with the above GDC, with respect to regional climatology.

RS-002, Section 2.3.1 specifies that an application meets the above requirements, if the application satisfies the following criteria:

- The description of the general climate of the regions should be based on standard climatic summaries compiled by the National Oceanic and Atmospheric Administration (NOAA). Consideration of the relationships between regional synoptic-scale atmospheric processes and local (site) meteorological conditions should be based on appropriate meteorological data.

- Data on severe weather phenomena should be based on the standard meteorological records from nearby representative National Weather Service (NWS), military, or other stations recognized as standard installations which have long periods on record. The applicability of these data to represent site conditions during the expected period of reactor operation should be substantiated.

- Design basis straight-line wind velocity should be based on appropriate standards, with suitable corrections for local conditions.

- UHS meteorological data, as stated in RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," should be based on long-period regional records which represent site conditions.

- Freezing rain estimates should be based on representative NWS station data.

- High air pollution potential information should be based on U.S. EPA studies.

- All other meteorological and air quality data used for safety-related plant design and operating bases should be documented and substantiated.

To the extent applicable to the above-outlined acceptance criteria, the applicant applied the NRC-endorsed meteorological information selection methodologies and techniques found in the following:

- RG 1.23, "Onsite Meteorological Programs," which provides criteria for an acceptable onsite meteorological measurements program, which can be used to monitor regional meteorology site characteristics.

- RG 1.70, which describes the type of regional meteorological data that should be presented in SSAR Section 2.3.1.

- RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," which provides criteria for selecting the design-basis tornado parameters.

When independently assessing the veracity of the information presented by the applicant in SSAR Chapter 2.3.1, the NRC staff applied the same above-cited methodologies and techniques.

2.3.1.3 Technical Evaluation

The NRC staff reviewed the application, as supplemented by letters dated January 30, 2007 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML070330054);

March 26, 2007 (ADAMS Accession No. ML070880685); and March 30 2007 (ADAMS Accession No. ML070940221) to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding regional climatology. In reviewing and evaluating this information, the staff used (or relied on) none of the applicant's proposed design parameters and site interface values presented in SSAR Section 1.3.

2.3.1.3.1 Data Sources

The applicant characterized the regional climatology of the proposed VEGP site's area using data from the National Climatic Data Center (NCDC), including the NWS station in Augusta, Georgia, and from nine other nearby cooperative observer stations. Five of these cooperative observer stations are located in Georgia counties, including Burke, Jefferson, Jenkins, Richmond, and Screven. The other four stations are located in the South Carolina counties, including Aiken, Bamberg, Barnwell, and Orangeburg. The regional climatic observation stations used by the applicant are included in the list presented in SER Table 2.3.1-1.

The applicant also obtained information on mean and extreme regional climatological phenomena from a variety of sources, such as publications by the NCDC, the Air Force Combat Climatology Center (AFCCC), the American Society of Civil Engineers (ASCE), the National Oceanic and Atmospheric Administration–Coastal Services Center (NOAA-CSC), and the Southeast Regional Climate Center (SERCC).

In RAI 2.3.1-1, the NRC staff asked the applicant to explain how it selected the observation stations it used to characterize regional climatology in SSAR Section 2.3.1. The applicant responded by revising its SSAR to enumerate the following selection criteria:

- The applicant chose stations in "proximity" to the site (i.e., within the general site area, less than or equal to 50 kilometers).

- The applicant attempted to select stations surrounding the site equally in all directions, to the greatest extent possible.

- Where more than one station exists in the same general direction from the site, the applicant selected the station that recorded a more extreme value for one or more meteorological conditions or phenomena (e.g., rainfall, snowfall, temperatures).

In addition to the ten climatic stations identified by the applicant, the NRC staff reviewed data from an additional seven climatic stations. Generally, the staff used data from stations within 50 miles (80 kilometers) and with a period of record greater than 10 years. SER Table 2.3.1-1 lists the observation stations used by the staff, in addition to those used by the applicant, to evaluate the regional climatology characteristics of the site.

During a site audit conducted on December 6, 2006, the staff asked the applicant to include all applicable stations which recorded the most extreme value for a particular meteorological condition or phenomena. The applicant responded by revising its SSAR to include data from the Louisville and Bamberg observation stations.

The NRC staff also used information reported by the NWS, NCDC, NOAA-CSC, Storm Prediction Center, National Severe Storms Laboratory (NSSL), National Hurricane Center (NHC), SERCC, American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE), Structural Engineering Institute (SEI), AFCCC, and ASCE.

2.3.1.3.2 General Climate

The applicant described the proposed VEGP site's general climate as mild with short winters. The region often experiences long periods of mild weather in the autumn and spring, coupled with long hot summers. The predominant air mass over the region is maritime tropical. In the winter, continental polar air, associated with high-pressure systems that move southeastward out of Canada, periodically affects the region. However, in general, down sloping and land modification warm the cold air that reaches the proposed site.

The regional climate is primarily influenced by the Azores high-pressure system. During the summer, the Bermuda High and the Gulf High have the strongest influence on Georgia's precipitation and temperature patterns. These circulation patterns are less defined in the transitional seasons and winter months, because of the passage of synoptic and meso-scale weather systems.

The applicant stated that monthly precipitation exhibits a cyclical pattern, with one maximum during the winter into early spring and a second maximum during late spring into summer. These two precipitation maxima are related to eastward moving low-pressure systems and thunderstorm activity, respectively. During the summer and early autumn, heavy precipitation can also be associated with tropical cyclones.

The staff agrees with the applicant's description of the general climate of the region, which is consistent with the NCDC narrative, "Annual Summary with Comparative Data for Augusta, Georgia;" the NCDC climatic data summary for Augusta shows an annual mean wind speed of 6.1 miles per hour (mi/h) and an annual prevailing wind direction from the west-southwest.

2.3.1.3.3 Severe Weather

2.3.1.3.3.1 Extreme Wind

Estimating wind loading on plant structures involves identifying the site's "basic" wind speed, which is defined by ASCE/SEI 7-02, "Minimum Design Loads for Buildings and Other Structures," as the "3-second gust speed at 33 feet (10 meters) above the ground in Exposure Category C".[6] Using linear interpolation on a plot of basic wind speeds presented in ASCE/SEI 7-02 for the portion of the United States that includes the proposed VEGP site, the applicant defined the basic wind speed for the proposed site as 97 mi/h. This value is associated with a mean recurrence interval of 50 years. Using a conversion factor listed in ASCE/SEI 7-02, the applicant derived a 100-year return period 3-second gust wind speed site characteristic value of 104 mi/h, as presented in SER Table 2.3.1-4.

Based on Section C6.0 of ASCE/SEI 7-02, the ratio of the 100-year to 50-year mean recurrence interval values is typically 1.07, which means that the 50-year return period basic wind speed value of 97 mi/h corresponds to a 100-year return period basic wind speed value of 104 mi/h. Therefore, the staff concludes that a site characteristic 3-second gust basic wind speed value of 104 mi/h is acceptable.

2.3.1.3.3.2 Tornadoes

The applicant used an approximate 55-year period of tornado reports (January 1950 through April 2005) from the NCDC to calculate the probability of a tornado strike near the proposed VEGP site. The applicant stated that 348 tornadoes have been reported to have touched down in the vicinity (i.e., within a 2-degree latitude and longitude area) of the proposed ESP site. Following the methodology presented in WASH-1300, "Technical Basis for Interim Regional Tornado Criteria," issued May 1974, the applicant used the following formula to calculate the probability that a tornado will strike a particular location during any one year period:

$$Ps = n(a/A)$$

where:

Ps = mean tornado strike probability per year
n = average number of tornadoes per year in the area being considered
a = average individual tornado area
A = total area being considered

The applicant calculated the probability of a tornado strike in the vicinity of the proposed ESP site of 774×10^{-7} per year, or, put differently, a recurrence interval of once every 12,920 years. The staff verified the applicant's probabilistic calculation, using the same tornado database, "Storm Events for Georgia and South Carolina, Tornado Event Summaries," from NCDC.

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7 Exposure Category C is defined as open terrain with scattered obstructions, having heights generally less than 30 feet (9.1 meters). This category includes flat open country, grasslands, and all water surfaces in hurricane-prone regions.

The applicant chose the tornado site characteristics based on the proposed Revision 1 to RG 1.76 (Draft Regulatory Guide DG-1143). DG-1143 provides design basis tornado characteristics for three tornado intensity regions throughout the United States, each with a 10-7 probability of occurrence. The proposed VEGP site is adjacent to both tornado intensity regions I and II. The applicant chose to use the more conservative design-basis tornado region (region I) and, correspondingly, proposed the following tornado site characteristics:

Maximum Wind speed	300 mi/h
Maximum Translational Speed	60 mi/h
Rotational Speed	240 mi/h
Radius of Maximum Rotational Speed	150 feet
Pressure Drop	2.0 lbf/in.2
Rate of Pressure Drop	1.2 lbf/in.2/s

In March, 2007, revision 1 to RG 1.76 was issued. Revision 1 reconfirmed that the design-basis tornado wind speeds for new reactors should correspond to the exceedance frequency of 10-7 per year. The design-basis tornado wind speeds presented in Revision 1 to RG 1.76 are based on the Enhanced-Fujita (EF) scale, which relates the degree of damage from a tornado to the tornado's maximum wind speed. The original versions of RG 1.76 and DG-1143 were based on the original Fujita scale. The applicant's design-basis tornado site characteristics conservatively bound those presented in Revision 1 to RG 1.76. For example, Revision 1 to RG 1.76 suggests a design-basis tornado wind speed of 230 mi/h for the proposed VEGP site, whereas the applicant chose a site characteristic design-basis wind speed of 300 mi/h.

Because the applicant's design-basis tornado site characteristics conservatively bound those presented in Revision 1 to RG 1.76, the staff concludes that the applicant has chosen acceptable tornado site characteristics. SER Table 2.3.1-4 presents the tornado site characteristics for the proposed VEGP site in the list of regional climatic site characteristics.

2.3.1.3.3.3 Tropical Cyclones

According to information presented by the applicant, during the period of time between 1851 and 2004, 102 tropical cyclones centers passed within a 100-nautical mile (185-kilometer) radius of the proposed VEGP site. The applicant used the NOAA-CSC historical tropical database to derive these results. Using the same database, the staff was able to verify the statistics presented by the applicant. SER Table 2.3.1-3 presents the storm classifications and respective frequencies of tropical cyclones passing within 100 nautical miles of the site during the 154-year period tracked by the NOAA-CSC database.

Since 1850, only nine hurricanes of category 2 strength or greater, which had sustained (i.e., 1-minute average) winds greater than 96 mi/h, have impacted the 100-nautical mile area surrounding the proposed VEGP site. This translates to a recurrence interval of 0.06 years, or one hurricane of category 2 strength or greater every 17.1 years. Six of these category 2 and 3 storms that affected the 100-nautical mile area surrounding proposed site did so before 1900. No category 2 or 3 storms have affected the region since 1959.

The strongest recorded hurricane to pass within 100 nautical miles of the site was hurricane Gracie on September 29, 1959. Hurricane Gracie had sustained wind speeds of 120 mi/h as it crossed the Atlantic coastline approximately 100 nautical miles southeast of the proposed VEGP site. The forward speed of the storm, as it crossed the coastline, was about 12 mi/h, as reported by the NHC. Based on its forward speed, hurricane Gracie would have needed to travel approximately 7 hours overland to reach the proposed VEGP site, approximately 88 miles (142 kilometers) from the coast. The storm's sustained wind speeds had weakened to 70 mi/h within 6 hours after it crossed the coastline. Assuming the storm took a direct track over the proposed VEGP site, the maximum projected sustained winds at the site would have been 70 mi/h. The Hurricane Research Division, a specialized division of NOAA, recommends multiplying sustained winds by a factor of 1.3 to obtain 3-second gust estimates. This would have resulted in a 3-second gust wind speed of approximately 91 mi/h, well below the chosen 3-second gust basic wind speed site characteristic of 104 mi/h.

Although tropical systems generally weaken significantly before impacting the proposed VEGP site, they still can cause significant amounts of rainfall. The applicant reported that tropical cyclones produced at least 12 separate 24-hour and monthly rainfall records at eight NWS cooperative observer network stations in the vicinity of the proposed site's area. The staff has independently confirmed these statistics.

2.3.1.3.3.4 Precipitation Extremes

The applicant used historical climate data from 10 nearby observing stations, as listed in SER Table 2.3.1-1, to identify precipitation extremes (rainfall and snowfall) observed near the proposed VEGP site. Based on the similarity of precipitation extremes and a real distribution of the observing stations around the site, these data can be used to adequately represent precipitation extremes that might be expected to occur at the site.

In SSAR Table 2.3-3, the applicant provided a climatic summary for each of the utilized observation stations, including the ones with the maximum 24-hour rainfall and maximum monthly rainfall. The staff independently verified each of these rainfall records, using the NCDC "Cooperative Summary of the Day—Daily Surface Data (TD 3200/3210)" and confirmed that the statistics provided by the applicant are correct.

During a site audit conducted on December 6, 2006, the staff asked why the applicant did not use as input to SSAR Table 2.3-3 the monthly rainfall value of 22.16 inches at Louisville in October 1990, as reported in the NCDC "Climatology of the United States No. 20." The applicant responded in a letter dated January 30, 2007, that this value is suspect and most likely an error. The applicant used the NCDC "Cooperative Summary of the Day" and climate summaries from SERCC to show that the actual value should be 14.34 inches. The staff agrees with the applicant that the 22.16 inches is an error and accepts the overall highest monthly total of 17.32 inches, which occurred at Springfield.

Although most of the recorded precipitation extremes were associated with the occurrence of tropical cyclones, the overall highest 24-hour rainfall total and overall highest monthly rainfall total were not. On April 16, 1969, the 24-hour rainfall record in the area surrounding the proposed site was set at the Aiken 4NE Station, when 9.68 inches fell. The overall highest monthly total of 17.32 inches occurred during June 1973 in Springfield.

According to the applicant, the disruptive effects of any winter storm accompanied by frozen precipitation in the proposed VEGP site area can be significant. However, storms that produce significant amounts of snow are infrequent. With one exception, all of the 24-hour and monthly record snowfall totals around the proposed site were associated with a storm that occurred early in February 1973. The applicant originally reported that the highest daily and monthly snowfall totals were both 17.0 inches and occurred at the Blackville station in South Carolina (Most other surrounding stations recorded similar amounts, ranging from 14.0 to 16.0 inches). The staff found larger values of 19.0 inches and 22.0 inches for the daily and monthly snowfall records near the site--these occurred in

February 1973 at Bamberg, South Carolina. During a site audit conducted on December 6, 2006, the staff asked the applicant to justify not including Bamberg as one of the cooperative observation stations considered in the SSAR. The applicant responded by adding climatic data from Bamberg to the SSAR and using data recorded by the Bamberg station to help characterize the regional climatology of the proposed VEGP site.

The staff notes that large snowfalls are very rare in the vicinity of the proposed site. At Waynesboro, the climatic observation station closest to the proposed site, maximum monthly snowfall totals from 1940 through 2006 (except for 1973) annually have ranged between 2 and 4 inches; only 5 years in the 66-year period have had months with snowfall greater than 2 inches at the Waynesboro cooperative observation site.

The staff concludes that the applicant has adequately identified precipitation extremes that might be expected to occur at or around the site. SER Table 2.3.1-2 lists the highest precipitation extremes that have occurred in the vicinity of the site.

2.3.1.3.3.5 Winter Precipitation Loads

The methodology for assessing the potential winter precipitation load on the roofs of safety-related structures considers two climate-related components, the weight of the 100-year return period ground-level snowpack, and the weight of the 48-hour PMWP. Consistent with the staff's branch position on winter precipitation loads (NRC memorandum dated March 24, 1975, from Harold R. Denton to R.R. Maccary), the winter precipitation loads included in the combination of normal live loads considered in the design of a nuclear power plant that might be constructed on a proposed ESP site should be based on the weight of the 100-year snowpack or snowfall, whichever is greater, recorded at ground level. Likewise, the winter precipitation loads included in the combination of extreme live loads considered in the design of a nuclear power plant that might be constructed on a proposed ESP site should be based on the weight of the 100-year snowpack at ground level plus the weight of the 48-hour PMWP at ground level for the month corresponding to the selected snowpack. A COL or CP applicant may choose to justify an alternative method for defining the extreme winter precipitation load by demonstrating that the 48-hour PMWP could neither fall nor remain on top of the snowpack and/or building roofs.

The applicant identified a 100-year return period ground-level snowpack value of 10-pounds-force per square foot (lbf/ft^2) for the proposed VEGP site, which was determined in accordance with ASCE/SEI 7-02. The applicant estimated the 48-hour PMWP as 28.3 inches (water equivalent) of precipitation. The applicant derived this PMWP estimate by using the guidance provided in the NOAA Hydrometeorological Report No. 53 (HMR 53), "Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates—United States East of the 105th Meridian."

Between February 9 and 11, 1973, heavy snowfall impacted the proposed VEGP site. Snowfall totals recorded at most of the surrounding climatic data stations ranged from 14.0 to 17.0 inches, with the highest recorded snowfall of 22.0 inches occurring at Bamberg. The storm produced the most snowfall in the climatic period of record for the region. Precipitation records from SERCC, "Period of Record Daily Climate Summary for Bamberg, SC," indicate the amount of liquid equivalent (i.e., liquid depth if all the snow melted) was 7.79 inches for this event. An inch of liquid water is equivalent to 5.2 lbf/ft^2, and, correspondingly, 7.79 inches of liquid water yields a snowpack of 40.5 lbf/ft^2.

In RAI 2.3.1-2, the staff asked the applicant to justify the adequacy of the proposed snowpack site characteristic, 10 lbf/ft^2, in consideration of the effects of the previously-discussed February 1973 storm. The applicant responded that the liquid equivalent value from SERCC is most likely bad datum and should have been removed. The applicant also stated that Section C7, Table C7-1, of the ASCE standard specifically lists the Augusta NWS location as having a maximum observed ground snow load of 8 lbf/ft^2 over a period of 40 years. The NRC staff accepts the applicant's response, and the applicant's proposed snowpack site characteristic of 10 lbf/ft^2, because other liquid equivalent estimates from other stations for the February 9–11, 1973 event are much smaller (less than 2.40 inches for most stations). The following is a list of the total snowfall and liquid equivalent, as recorded by NCDC in its Summary of the Day publications, for several surrounding climatic stations for the February 1973 storm:

STATION	SNOWFALL	LIQUID EQUIVALENT
Augusta	14.0 inches	2.13 inches
Louisville	14.8 inches	1.55 inches
Midville	10.0 inches	1.97 inches
Millen	14.0 inches	2.30 inches
Waynesboro	14.0 inches	2.39 inches

The staff, thus, agrees with the applicant that the 7.79 inches liquid equivalent value from SERCC is most likely incorrect.

The applicant has identified the 48-hour PMWP site characteristic of 28.3 inches using data from HMR-53. The applicant determined its 48-hour PMWP site characteristic value by using linear interpolation between the 24- and 72-hour probable maximum precipitation (PMP) values for December (Figures 35 and 45 of HMR-53), which had the largest values among the winter months December–February. The value of 28.3 inches converts to an estimated weight of the 48-hour PMWP of 147 lbf/ft^2, assuming that 1 inch of liquid water is equivalent to 5.2 lbf/ft^2. Using the same data from HMR-53, the staff found that the applicant has adequately identified an appropriate estimate of the 48-hour PMWP.

SER Table 2.3.1-4 presents the staff-accepted winter precipitation site characteristics for the proposed VEGP site as part of the list of regional climatic site characteristics.

2.3.1.3.3.6 Hail, Freezing Rain, and Sleet

The following discussion on hail, freezing rain, and sleet is intended to provide a general climatic understanding of the severe weather phenomena in the site region but does not result in the generation of site characteristics for use as design or operating bases.

Hail can accompany severe thunderstorms and can be a major weather hazard, causing significant damage to crops and property. The applicant used the NOAA "Climate Atlas of the United States" to estimate that around the proposed VEGP site area, specifically to the northwest of the site, the annual mean number of days with hail of 0.75 inches or greater in diameter is approximately 1 to 2 per year. The applicant also stated that an extreme hailstorm event (i.e., hail with a diameter greater than 2.75 inches) was observed only once, on May 21, 1964, about 43 miles southeast of the proposed site.

The NCDC Storm Event Database, "Storm Events for Georgia, Query Results, Hail Event(s) Reported in Burke County, Georgia Between 01/01/1950 and 07/31/2006," reports that a total of 28 hail events with hail 0.75 inches or greater occurred in Burke County from January 1971 through May 2006. In four of those events, the hail had a diameter of 1.75 inches or greater.

The NRC staff notes that hailstorm events are point observations, which are often dependent on population density. Estimates of hail size can range widely based on the surrounding area population density and years considered. The applicant stated that Burke County can expect, on average, hail with a diameter of 0.75 inches or greater about 1 day per year and hail with a diameter of 1.0 inches or greater less than 1 day per year. The applicant also stated that the annual mean number of days reported with hail equal to or greater than 0.75 inches ranges from 1 to 2 days per year in the nearby, more populated counties of Richmond, Columbia, Aiken, and Edgefield. The annual mean number of days reported with hail equal to or greater than 1.0 inches ranges up to 1 day per year in those same counties. The staff verified the hail frequencies presented by the applicant from "The Climate Atlas of the United States." Based on the NSSL "Severe Thunderstorm Climatology, Total Threat," the staff finds that, considering data from 1980 through 1999, the total number of days per year with hail greater than 0.75 inches ranges from 2 to 4.

The applicant estimated that the highest average frequency of ice storms (i.e., sleet and freezing rain) occurs to the northeast, east, and southeast of the proposed VEGP site in South Carolina. These areas can expect an average of 3 to 5 days of freezing precipitation per year. Ice accumulations typically have a thickness of less than 1 inch.

The staff has independently confirmed and accepts the hail and ice storm frequencies provided by the applicant. The NCDC Storm Event Database, "Storm Events for Georgia, Query Results, Snow & Ice Event(s) Reported in Burke County, Georgia, Between 01/01/1950 and 07/31/2006," lists four ice events for Burke County in the period January 2002 through January 2005. "The Climate Atlas of the United States" estimates 3 to 5 days per year with freezing rain around the proposed VEGP site area. The staff notes that cold air damming events can bring cold air and an increased probability of ice storms during the winter months. In Jones, et al. (2002), the NCDC reports a 50-year return period uniform radial ice thickness of 0.75 inches because of freezing rain, with a concurrent 3-second gust wind speed of 30 mi/h for the proposed site area.

2.3.1.3.3.7 Thunderstorms

The following discussion on thunderstorms is intended to provide a general climatic understanding of the severe weather phenomena in the site region but does not result in the generation of site characteristics for use as design or operating bases.

The applicant estimated that, on average, approximately 52 days with thunderstorm occurrences happen per year in the site area. This frequency is taken from the NCDC local climatological data, annual summary with comparative data, for Augusta. The majority of thunderstorms recorded (60 percent) occurred between late spring and midsummer (i.e., from June through August). The applicant estimated that approximately 16 flashes to earth per square mile (6.2 flashes to earth per square kilometer) per year occur around the site. The staff finds this number appropriate based on similar values from "The Climate Atlas of the United States" (4.8-6 flashes to earth per square kilometer), a 5-year flash density map from Vaisala (4-8 flashes to earth per square kilometer), and a 1999 paper by G. Huffines and R.E. Orville, titled "Lightning Ground Flash Density and Thunderstorm Duration in the Continental United States: 1989-96" (3-7 flashes to earth per square kilometer). Assuming the size of the potential reactor area for the proposed Vogtle units is bounded by an area of 0.068 square miles (0.176 square kilometers), an approximate average of 1 lightning strike per year will occur in the reactor area.

2.3.1.3.4 Ultimate Heat Sink

The applicant has chosen a reactor design that does not use a cooling tower to release heat to the atmosphere following a loss-of-coolant accident. Instead, a passive containment cooling system (PCS) would provide the safety-related UHS. The applicant stated that the PCS is not significantly influenced by local weather conditions. If, at the COL or CP stage, the applicant chooses an alternative plant design that requires the use of a UHS cooling tower, the applicant will need to identify the appropriate meteorological site characteristics (i.e., maximum evaporation and drift loss and minimum water cooling conditions) used to evaluate the design of the chosen UHS cooling tower. At the time of the COL or CP, the staff will verify the design type and characteristics of the UHS. This is COL Action Item 2.3-1.

2.3.1.3.5 Temperatures

The applicant based its ambient air temperature and humidity site characteristics (e.g., the 0.4-percent, 2-percent, 99-percent, and 99.6-percent annual exceedance dry-bulb temperatures[8] and 0.4-percent annual exceedance wet-bulb temperature) on 1973–1996 Augusta data published by AFCCC in its 1999 long-term, engineering-related climatological data summaries. The values for the 0.4-percent, 2-percent, 99-percent, and 99.6-percent annual exceedance dry-bulb temperatures are 97 °F, 92 °F, 25 °F, and 21 °F, respectively. The staff performed an independent analysis for a longer period of record (1961–2006) using hourly data from Augusta, obtained from the NCDC "Integrated Surface Hourly Observations" data compilation. The staff calculated the same values as the applicant. Consequently, the staff finds the proposed site characteristics for ambient air temperature and humidity appropriate.

[8] The data presented by the applicant as minimum 1-percent and 0.4-percent annual exceedance values are referred to by the staff as 99-percent and 99.6-percent annual exceedance values throughout the SE.

The applicant based the mean coincident wet-bulb temperatures associated with the annual 2-percent and 0.4-percent exceedance dry-bulb temperatures on data in the AFCCC report "Engineering Weather Data." The staff has confirmed that the mean coincident wet-bulb temperatures of 75 °F and 76 °F associated with the 2-percent and 0.4-percent exceedance probabilities are appropriate based on values presented in the AFCCC report.

To determine the site characteristic 0.4-percent annual exceedance maximum wet-bulb temperature value, the applicant selected a value of 79 °F from the AFCCC report for Augusta based on data from 1973 through 1996. The staff evaluated Augusta wet-bulb data from 1961 through 2006 and produced the same exceedance value. Thus, the staff finds the applicant's value of 79 °F appropriate for the 0.4-percent annual exceedance maximum wet-bulb temperature site characteristic.

To calculate 100-year return maximum and minimum dry-bulb temperatures, the applicant performed linear regression using daily maximum and minimum dry-bulb temperatures from Augusta from the 30-year period between 1966 and 1995. The staff used a methodology presented in the 2001 ASHRAE Handbook ("Fundamentals") to check the applicant's 100-year return values. The ASHRAE methodology is based on the assumption that the annual maxima and minima are distributed according to the Gumbel (Type 1 Extreme Value) distribution. Based on techniques presented in Chapter 27 of the Handbook, the staff calculated 100-year return values of maximum dry-bulb temperature for Waynesboro, Augusta, and Louisville; and 100-year return values of minimum dry-bulb temperature for Waynesboro, Augusta, and Aiken. The staff included Aiken and Louisville in its analysis because those are the two observation stations where the all-time maximum (112 °F) and minimum (-4 °F) temperatures occurred in the vicinity of the proposed VEGP site. Louisville data are available for the past 77 years, and Aiken data are available for the past 94 years; thus, a reasonably extensive record exists on which to base climate records. Based on techniques in the ASHRAE handbook, the staff calculated 100-year return maximum and minimum dry-bulb temperature values which are bounded by the applicant's proposed 100-year return period maximum and minimum dry-bulb temperature site characteristic values of 115 °F and -8 °F, respectively. The applicant's proposed 100-year return period maximum and minimum dry-bulb temperature site characteristic values also bound the all-time maximum and minimum temperatures observed in the area surrounding the proposed VEGP site (i.e., 112 °F at Aiken, and -4 °F at Louisville). Therefore, the staff finds that the applicant's values of 115 °F and -8 °F are appropriate for the 100-year return period maximum and minimum dry-bulb temperature site characteristics.

The applicant used a linear regression technique on 1966–1995 data from Augusta to estimate the 100-year return period maximum wet-bulb temperature of 88 °F. The staff conducted a similar linear regression technique, and, in addition, used the technique presented in the ASHRAE handbook, as previously discussed above, to calculate a similar 100-year return value using 1961–2006 data from the Augusta NWS site. The maximum hourly wet-bulb temperature recorded at Augusta from 1961 through 2006 was 86 °F. Based on these results, the staff believes that the applicant's 100-year return maximum wet-bulb temperature site characteristic value of 88 °F is appropriate.

The applicant based many of the proposed site characteristics on data from Augusta. The staff accepts this approach because meteorological conditions at Augusta tend to be representative of the proposed VEGP site. In SER Section 2.3.3, the staff shows a comparison between onsite meteorological data and corresponding Augusta data. Temperature, dew point, wind speed, and wind direction measurements are very similar between the two observation stations.

At the time of any COL application, the applicant would have to compare site characteristics presented in the ESP against the corresponding site parameters listed in the design certification document (DCD).

The site characteristics discussed above are meant to encompass many potential designs and corresponding site parameters. Since the applicant has expressed an interest in using the AP1000 design in any future COL application, the applicant has identified additional site characteristics that directly correspond to temperature site parameters in the AP1000 DCD. The applicant provided the following definitions for the AP1000 DCD temperature site parameters:

- Maximum Safety Dry-Bulb Temperature and Coincident Wet-Bulb Temperature: These site parameter values represent a maximum dry-bulb temperature that exists for 2 hours or more, combined with the maximum wet-bulb temperature that exists in that population of dry-bulb temperatures.

- Maximum Safety Noncoincident Wet-Bulb Temperature: This site parameter value represents a maximum wet-bulb temperature that exists within a set of hourly data for a duration of 2 hours or more.

- Maximum Normal Dry-Bulb Temperature and Coincident Wet-Bulb Temperature: The dry-bulb temperature component of this site parameter pair is represented by a maximum dry-bulb temperature that exists for 2 hours or more, excluding the highest 1 percent of the values in an hourly data set. The wet-bulb temperature component is similarly represented by the highest wet-bulb temperature excluding the highest 1 percent of the data, although there is no minimum 2-hour persistence criterion associated with this wet-bulb temperature.

- Maximum Normal Noncoincident Wet-Bulb Temperature: This site parameter value represents a maximum wet-bulb temperature, excluding the highest 1 percent of the values in an hourly data set (i.e., a 1 percent exceedance), that exists for 2 hours or more.

The applicant identified the following AP1000 specific temperature site characteristics:

- a maximum safety dry-bulb temperature of 115 °F with a coincident wet-bulb temperature of 77.7 °F.

- a maximum safety noncoincident wet-bulb temperature of 83.9 °F.

- a maximum normal dry-bulb temperature of 94 °F with a coincident wet-bulb temperature of 78 °F.

- a maximum normal noncoincident wet-bulb temperature of 78 °F.

Initially, the applicant used a 30-year period of record, 1966 through 1995, from Augusta to define these site characteristics. In Open Item 2.3-1, the staff asked the applicant to base the AP1000 specific maximum safety dry-bulb and maximum safety wet-bulb temperatures on a more conservative 100-year return period. The applicant responded to Open Item 2.3-1 by providing a 100-year return period maximum safety dry-bulb temperature with a coincident wet-bulb temperature and maximum safety noncoincident wet-bulb temperature.

As previously discussed above, the staff has independently confirmed and accepts the applicant's 100-year dry-bulb temperature site characteristic of 115 °F. Since this value is based on a linear regression technique, there is no discrete measurement of the coincident wet-bulb temperature. The applicant estimated the safety coincident wet-bulb temperature based on the relationship between concurrent dry- and wet-bulb temperatures at Augusta from 1949 through 1995. The staff performed a similar analysis using hourly data from Augusta from 1961 through 2006 and believes the applicant's estimate is accurate.

The applicant calculated the 100-year return period maximum safety noncoincident wet-bulb temperature based on a linear regression technique. The staff used the technique presented in the ASHRAE handbook, as previously discussed above, to calculate a similar 100-year return value (i.e., ± 1 °F) using 1961–2006 hourly data from the Augusta NWS site. Thus, the staff believes the applicant's maximum safety noncoincident wet-bulb temperature estimate is appropriate for the site.

The maximum safety noncoincident wet-bulb temperature of 83.9 °F is lower than the previously discussed 100-year return period maximum wet-bulb temperature of 88 °F because, as defined above, it is based on a two hour persistence criteria; whereas, the 88 °F wet-bulb temperature is based on a one hour persistence criteria.

Since the applicant has determined a maximum safety dry-bulb temperature with a coincident wet-bulb temperature and a maximum safety noncoincident wet-bulb temperature based on a 100-year return period, the staff considers Open Item 2.3-1 closed.

As previously discussed above, the staff finds the applicant's estimates of 2-percent and 0.4-percent exceedance dry-bulb temperature and coincident wet-bulb temperature and 0.4-percent exceedance non-coincident wet-bulb temperature appropriate. The AP1000 specific maximum normal dry-bulb and wet-bulb temperatures are based on a 1-percent exceedance. The values are consistent with those previously discussed and thus acceptable to the staff.

2.3.1.3.6 Stagnation Potential

Large-scale episodes of atmospheric stagnation are not common in the region of the proposed site. Based on the 50-year period from 1948 through 1998, high-pressure stagnation conditions, usually accompanied by light and variable wind conditions, can be expected at the proposed VEGP site about 20 days per year, or about four cases per year with the mean duration of each case being about 5 days (Wang and Angell). Stagnation conditions usually occur during the months from May through October, with a peak in September. Winds are usually weakest in September due to influence from the Bermuda High pressure system.

The applicant also noted that, from a climatological standpoint, the lowest morning mixing heights occur in the autumn and are the highest during the winter. Conversely, afternoon mixing heights reach a seasonal minimum in the winter and a maximum during the summer, which is expected because of more intense summer heating. The applicant presented mixing height data from Athens, Georgia, which the applicant claims is reasonably representative of conditions at the proposed VEGP site.

The staff confirmed the information presented by the applicant regarding restrictive dispersion conditions as correct. Section 2.3.2 of this SER discusses the proposed VEGP site air quality conditions for design- and operating-basis considerations. Sections 2.3.4 and 2.3.5 of this SER discuss atmospheric dispersion site characteristics used to evaluate short-term post-accident airborne releases and long-term routine airborne releases, respectively.

2.3.1.3.7 Climate Change

As specified in RS-002, the applicability of data used to discuss severe weather phenomena that may impact the proposed ESP site during the expected period of reactor operation should be substantiated.

Long-term environmental changes and changes to the region resulting from human or natural causes may affect the applicability of the historical data for describing the site's climate characteristics. Although there is no scientific consensus regarding the issue of climate change, the staff believes current climate trends should be analyzed for the potential for ongoing environmental changes.

During a site audit conducted on December 6, 2006, the staff asked the applicant to evaluate trends in temperature and precipitation extremes in the proposed VEGP site vicinity and discuss whether such trends may be indicative of climatic change. In a letter dated January 30, 2007, the applicant stated that initial investigations showed no consistent long-term climate change in the proposed site area. The applicant also revised its SSAR to include a discussion of long-term climatic changes.

The applicant analyzed trends in temperature and rainfall normals / standard deviations over a 70-year period for successive 30-year intervals based on the NCDC "Climatography of the United States." The applicant stated that average temperature has increased only slightly (i.e., 0.2 to 0.3 °F) over the latest 30-year period and rainfall, on average, has increased by 1.5 inches over the same period.

The staff has confirmed and accepts the numbers provided by the applicant. The staff analyzed 1-year, 10-year, and 20-year trends in annual average daily maximum and minimum temperatures, annual extreme maximum and minimum temperatures, annual average precipitation, and annual extreme daily precipitation at Waynesboro and Augusta for potential indications of climate change using data from 1951 through 2004. The trends over 20 years show that annual extreme minimum temperatures have increased 2 °F and average annual precipitation has increased about 1.5 to 2.5 inches over the period of record. All other meteorological parameters showed no discernible signs of climate change.

The Intergovernmental Panel on Climate Change (IPCC) issued its Fourth Assessment Report on Climate Change in February 2007. The staff considered Chapter 11 in "Climate Change 2007: The Physical Science Basis, Contribution of Working Group I to the 4th Assessment Report of the Intergovernmental Panel on Climate Change," regarding the southeastern portion of the United States. The IPCC models projecting potential future climate change depend on human activity and land use. To account for this, the IPCC uses different global scenarios as input to the models. Chapter 11 of the IPCC report discusses the following three scenarios:

- (A2) "A more divided world with self-reliant, independently operating nations"
- (A1B) "A more integrated world with an emphasis on all energy sources"
- (B1) "A world more integrated and ecologically friendly" (i.e., less energy consumption and more cooperating nations)

During the 100-year period under the A1B scenario (i.e., 1980–1999 as compared to 2080–2099), the IPCC projection estimates that the proposed VEGP site may see an increase in average annual temperature of 3 °C and an increase in precipitation of 0 to 5 percent. Under the more and less extreme scenarios, increases in annual average temperature may range from 2 °C to 7.5 °C. The projection also shows a general decrease in snow depth as a result of delayed autumn snowfall and earlier spring snow melt.

The staff also analyzed climate-change-induced hurricane trends within 100 nautical miles of the site and found no discernible trends in hurricane frequency or intensity. The "Summary for Policymakers" based on the February 2007 IPCC report makes the following statement concerning tropical cyclones:

> Based on a range of models, it is likely that future tropical cyclones (typhoons and hurricanes) will become more intense, with larger peak wind speeds and more heavy precipitation associated with ongoing increases of tropical sea surface temperatures. (IPCC Sections 3.8, 9.5, and 10.3)

However, the question of whether hurricanes are becoming more destructive because of global warming is a contested issue in the scientific debate over climate change. A number of academic papers have been published either supporting or debunking the idea that warmer temperatures linked to human activity have created more intense storms, and the issue is currently unresolved (Dean; Eilperin; Kerr; Witze). Based on the current amount of scientific uncertainty regarding this subject, the staff believes the applicant has adequately addressed the issue of hurricanes and provided conservative site characteristics.

The applicant stated that the number of recorded tornado events has increased, in general, since detailed records were routinely kept beginning around 1950. However, some of this increase is attributable to a growing population, greater public awareness and interest, and technological advances in detection. These changes are superimposed on normal year-to-year variations. Consequently, the number of observations recorded within a 2-degree latitude and longitude square centered on the VEGP site reflects these effects. The staff has confirmed and accepts the applicant's statements regarding tornadoes. The "Summary for Policymakers" based on the February 2007 IPCC report states, "there is insufficient evidence to determine whether trends exist in small scale phenomena such as tornadoes, hail, lightning, and dust storms." (IPCC Sections 3.8 and 5.3).

In conclusion, the staff acknowledges that long-term climatic change resulting from human or natural causes may introduce changes into the most severe natural phenomena reported for the site. However, no conclusive evidence or consensus of opinion is available on the rapidity or nature of such changes. If in the future, the ESP site is no longer in compliance with the terms and conditions of the ESP (e.g., if new information shows that the climate has changed and that the climatic site characteristics no longer represent extreme weather conditions), the staff may seek to modify the ESP or impose requirements on the site in accordance with the provisions of 10 CFR 52.39, "Finality of Early Site Permit Determinations."

2.3.1.4 Conclusion

The NRC staff has evaluated the relevant sections of the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007, pursuant to the acceptance criteria described RS-002, Section 2.3.1 and applicable regulatory requirements of 10 CFR Part 52 and 10 CFR Part 100. The applicant has presented and substantiated information relative to the regional meteorological conditions. The staff has reviewed the information presented by the applicant and concludes that the identification and consideration of the regional and site meteorological characteristics meet the requirements of 10 CFR 52.17(a)(1), 10 CFR 100.20(c), and 10 CFR 100.21(d).

Table 2.3.1-1 - Regional Climatic Observation Stations

STATION NAME	COUNTY	STATE CLIMATIC DIVISION	DISTANCE FROM ESP SITE (km)	DIRECTION FROM ESP SITE	STATION ELEV. (m)	DIFF. FROM ESP SITE ELEV. (m)	YEARS OF DATA
Appling 2NW 1	Columbia	GA-6	69	NW	113	46	46
Augusta Bush Field 2	Richmond	GA-6	32	NW	40	-27	57
Augusta 1	Richmond	GA-6	41	NW	40	-27	13
Louisville 1 E 2	Jefferson	GA-6	59	SW	98	31	77
Midville Exp. Station 2	Burke	GA-6	51	SW	85	18	50
Millen 4 N 2	Jenkins	GA-6	36	SSW	59	-8	68
Newington 2	Screven	GA-6	65	SSE	64	-3	43
Sylvania 2 SSE 1	Screven	GA-6	47	SE	76	9	13
Waynesboro 2 S 2	Burke	GA-6	25	WSW	82	15	67
Allendale 2 NW 1	Allendale	SC-7	44	ESE	55	-12	26
Bamberg 2	Bamberg	SC-7	70	ENE	50	-17	57
Blackville 3W 2	Barnwell	SC-7	47	NE	99	32	93
Hampton 1 S 1	Hampton	SC-7	68	SSE	29	38	55
Aiken 5 SE 2	Aiken	SC-5	41	N	150	83	94
Clarks Hill 1 W 1	McCormick	SC-5	71	NW	116	49	56
Trenton 1 NNE 1	Edgefield	SC-5	68	NNE	189	122	47
Springfield 2	Orangeburg	SC-5	60	NNE	91	24	58

1 Climatic stations used by the staff only
2 Climatic stations used by both the staff and applicant

Data Reference: NCDC, "Local Weather Observation Station Record," October 2006.

Table 2.3.1-2 Climatic Precipitation Extremes within 50 Miles of the ESP Site

PARAMETER	SITE EXTREMES	STATION
Maximum 24-hr Rainfall	9.68 in.	Aiken 5SE
Maximum Monthly Rainfall	17.32 in.	Springfield
Minimum Monthly Rainfall	0 in.	Multiple
Maximum 24-hr Snowfall	19 in.	Bamberg
Maximum Monthly Snowfall	22 in.	Bamberg
Maximum Daily Snow Depth	19 in.	Bamberg

Table 2.3.1-3 - Tropical Cyclone Frequency within a 100-Nautical Mile Radius of the Proposed VEGP Site between 1851 and 2004

CLASSIFICATION	NUMBER OF OCCURRENCES	MAXIMUM SUSTAINED (1-MIN AVG) WIND SPEED RANGE
Saffir-Simpson Category 5 Hurricanes	0	>155 mi/h
Saffir-Simpson Category 4 Hurricanes	0	131–155 mi/h
Saffir-Simpson Category 3 Hurricanes	5	111–130 mi/h
Saffir-Simpson Category 2 Hurricanes	4	96–110 mi/h
Saffir-Simpson Category 1 Hurricanes	16	74–95 mi/h
Tropical Storms	46	39–73 mi/h
Tropical Depressions	23	<39 mi/h
Subtropical Storms	1	<74 mi/h
Subtropical Depressions	2	<39 mi/h
Extra-Tropical Storms	5	N/A

Table 2.3.1-4 - Regional Climatology Site Characteristics

SITE CHARACTERISTIC	VALUE		DESCRIPTION
Ambient Air Temperature and Humidity			
Maximum Dry-Bulb Temperature	2 percent annual exceedance	92 °F / 75 °F	The ambient dry-bulb temperature (and mean coincident wet-bulb temperature) that will be exceeded 2 percent of the time annually
	0.4 percent annual Exceedance	97 °F / 76 °F	The ambient dry-bulb temperature (and mean coincident wet-bulb temperature) that will be exceeded 0.4 percent of the time annually
	100-year return Period	115 °F	The ambient dry-bulb temperature that has a 1 percent annual probability of being exceeded (100-year mean recurrence interval)
Minimum Dry-Bulb Temperature	99 percent annual exceedance	25 °F	The ambient dry-bulb temperature below which dry-bulb temperatures will fall 1 percent of the time annually
	99.6 percent annual exceedance	21 °F	The ambient dry-bulb temperature below which dry-bulb temperatures will fall 0.4% of the time annually
	100-year return period	-8 °F	The ambient dry-bulb temperature for which a 1 percent annual probability of a lower dry-bulb temperature exists (100-year mean recurrence interval)
Maximum Wet-Bulb Temperature	0.4 percent annual exceedance	79 °F	The ambient wet-bulb temperature that will be exceeded 0.4 percent of the time annually
	100-year return period	88 °F	The ambient wet-bulb temperature that has a 1% annual probability of being exceeded (100-year mean recurrence interval)
Site Temperature Basis for AP1000			
Maximum Safety Dry-Bulb and Coincident Wet-Bulb	115 °F / 77.7 °F		These AP1000 specific site characteristics values represent a maximum dry-bulb temperature that exists for 2 hours or more, combined with the maximum wet-bulb temperature that exists in that population of dry-bulb temperatures.

SITE CHARACTERISTIC	VALUE	DESCRIPTION
Maximum Safety Wet-Bulb (Non-Coincident)	83.9 °F	This AP1000 specific site characteristic value represents a maximum wet-bulb temperature that exists within a set of hourly data for a duration of 2 hours or more.
Maximum Normal Dry-Bulb and Coincident Wet-Bulb	94 °F / 78 °F	The dry-bulb temperature component of this AP1000 specific site characteristics pair is represented by a maximum dry-bulb temperature that exists for 2 hours or more, excluding the highest 1 percent of the values in an hourly data set. The wet-bulb temperature component is similarly represented by the highest wet-bulb temperature excluding the highest 1 percent of the data, although there is no minimum 2-hour persistence criterion associated with this wet-bulb temperature.
Maximum Normal Wet-Bulb (Non-Coincident)	78 °F	This AP1000 specific site characteristic value represents a maximum wet-bulb temperature, excluding the highest 1 percent of the values in an hourly data set (i.e., a 1 percent exceedance), that exists for 2 hours or more.
Basic Wind Speed		
3-Second Gust	104 mi/h	The 3-second gust wind speed to be used in determining wind loads, defined as the 3-second gust wind speed at 33 feet above the ground that has a 1 percent annual probability of being exceeded (100-year mean recurrence interval)
Tornado		
Maximum Wind Speed	300 mi/h	Maximum wind speed resulting from passage of a tornado having a probability of occurrence of 10^{-7} per year
Maximum Translational Speed	60 mi/h	Translation component of the maximum tornado wind speed

SITE CHARACTERISTIC	VALUE	DESCRIPTION
Rotational Speed	240 mi/h	Rotation component of the maximum tornado wind speed
Radius of Maximum Rotational Speed	150 feet	Distance from the center of the tornado at which the maximum rotational wind speed occurs
Pressure Drop	2.0 $lbf/in.^2$	Decrease in ambient pressure from normal atmospheric pressure resulting from passage of the tornado
Rate of Pressure Drop	1.2 $lbf/in.^{2/s}$	Rate of pressure drop resulting from the passage of the tornado
Winter Precipitation		
100-Year Snowpack	10 lb/sq ft	Weight of the 100-year return period snowpack (to be used in determining normal precipitation loads for roofs)
48-Hour Probable Maximum Winter Precipitation	28.3 inches of water	PMP during the winter months (to be used in conjunction with the 100-year snowpack in determining extreme winter precipitation loads for roofs)

2.3.2 Local Meteorology

2.3.2.1 Introduction

In Section 2.3.2 of the SSAR, the applicant presented information on local (site) meteorological parameters. Specifically, the applicant provided the following information:

- a description of the local (site) meteorology in terms of airflow, atmospheric stability, temperature, water vapor, precipitation, fog, and air quality.

- an assessment of the influence on the local meteorology of construction and operation of the nuclear power plant that is planned to be constructed on the proposed site and its facilities, including the effects of plant structures, terrain modification, and heat and moisture sources resulting from plant operation.

- a topographical description of the site and its environs, as modified by the structures of the nuclear power plant that is planned to be built on the proposed site.

This section verifies that the applicant has identified and considered the meteorological and topographical characteristics of the site and the surrounding area, as well as changes that may result to those characteristics because of the construction and operation of the proposed facility.

2.3.2.2 Regulatory Basis

The acceptance criteria for identifying local meteorological parameters are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The staff considered the following regulatory requirements in reviewing the applicant's identification of local meteorological parameters:

- 10 CFR 52.17(a), which requires that the application contain a description of the seismic, meteorological, hydrological, and geological characteristics of the proposed site.

- 10 CFR 100.20(c), which requires that the meteorological characteristics of the site, necessary for safety analysis or that may have an impact on plant design, be identified and characterized as part of the NRC's review of the acceptability of a site.

- 10 CFR 100.21(c), which requires that site atmospheric dispersion characteristics be evaluated and dispersion parameters established such that (1) radiological effluent release limits associated with normal operation from the type of facility to be located at the site can be met for any individual located offsite; and (2) radiological dose consequences of postulated accidents shall meet the criteria set forth in 10 CFR 50.34(a)(1) for the type of facility proposed to be located at the site.

- 10 CFR 100.21(d), which requires that the physical characteristics of the site, including meteorology, geology, seismology, and hydrology be evaluated and site parameters established, such that the potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site.

The local meteorological information assembled in compliance with the above regulatory requirements would be necessary to determine, at the COL stage, a proposed facility's compliance with the following requirements in Appendix A, "General Design Criteria for Nuclear Power Plants," of 10 CFR Part 50:

- GDC 2, which requires that structures, systems and components important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions; and further requires that consideration be given to the most severe local weather phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

An ESP applicant, though, need not demonstrate compliance with the above GDC, with respect to local meteorology.

RS-002, Section 2.3.2 specifies that an application meets the above requirements, if the application satisfies the following criteria:

- Local meteorological data, based on onsite measurements and data from nearby NWS stations or other standard installations, should be presented in the format specified in RG 1.70.
- A complete topographical description of the site and environs set out to a distance of 50 miles from the site should be provided.
- A discussion and evaluation of the influence of a nuclear power plant of the type proposed to be constructed on the site on local meteorological and air quality conditions should be provided.

To the extent applicable to the above-outlined acceptance criteria, the applicant applied the NRC-endorsed meteorological information selection methodologies and techniques found in the following:

- RG 1.23, which provides criteria for an acceptable onsite meteorological measurements program to be used to monitor local (onsite) meteorology site characteristics.
- RG 1.70, which describes the type of local meteorological data that should be presented in SSAR Section 2.3.2.

When independently assessing the veracity of the information presented by the applicant in SSAR Chapter 2.3.2, the NRC staff applied the same above-cited methodologies and techniques.

2.3.2.3 Technical Evaluation

Using the approaches and methodologies described in RS-002 Section 2.3.2, the NRC staff reviewed the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007. In reviewing and evaluating the applicant's site meteorology, the staff used (or relied on) none of the applicant's proposed design parameters and site interface values presented in SSAR Section 1.3.

2.3.2.3.1 Local Meteorology Description

The applicant used data from the existing Vogtle meteorological monitoring program and 10 surrounding NWS observation stations (as listed in SSAR Section Table 2.3.1-2 and repeated in

SER Section 2.3.1) to describe local meteorology. The applicant used data from the onsite meteorological monitoring program to describe wind speed, wind direction, and atmospheric stability conditions; surrounding offsite observation stations were data sources for temperature, atmospheric moisture, precipitation, and fog conditions.

The applicant presented means and historical extremes of temperature, rainfall, and snowfall data from the 10 offsite observation stations listed in SSAR Section 2.3.1. SER Table 2.3.2-1 summarizes the overall extremes from those stations, as compiled by the applicant.

The staff evaluated the information regarding local meteorological conditions submitted by the applicant using data from the Vogtle onsite meteorological monitoring system, as well as climatic data reported in "Monthly Station Climate Summaries," "U.S. Monthly Climate Normals," and "Daily Surface Data" (all from NCDC) and "Period of Record Daily Climate Summaries for Georgia and South Carolina" from SERCC. The staff has confirmed the normal and extreme values presented by the applicant in SSAR Tables 2.3-3 and 2.3-5, respectively.

2.3.2.3.1.1 Airflow

The applicant presented hourly wind data from the Vogtle onsite meteorological monitoring program, as described in SSAR Section 2.3.3, from 1998 through 2002. The applicant also provided annual and seasonal wind roses based on 10-meter and 60-meter observation heights. The NRC staff confirmed that the wind directions from both levels are fairly similar. The prevailing annual wind direction for the site is generally from the southwest. Winds from the southwest predominate during the spring and summer, westerly winds predominate during the winter, and northeasterly winds predominate during the autumn months.

The applicant stated that annual average wind speeds at the 10- and 60-meter observation levels are 2.5 m/s and 4.6 m/s, respectively. This is consistent with the 6.1-meter measurement height annual average wind speed at Augusta, Georgia, of 2.7 m/s. The annual frequencies of calm wind conditions are 0.44 and 0.07 percent of the time for the 10-meter and 60-meter observation levels at the proposed VEGP site.

The staff reviewed the Vogtle onsite meteorological wind data from 1998 through 2002 for completeness and consistency. The wind measurements provided by the applicant had at least 95-percent data recovery. Initially, the staff did have concerns about the consistency of the data. The staff, having compared the 1998–2002 annual data used by the applicant to the 1972–1973, 1977-1978, 1978–1979, and 1980–1981 meteorological data presented in the original final safety analysis report (FSAR) for Vogtle Units 1 and 2, discovered that there were discrepancies between the two sets of data. During a site audit conducted on December 6, 2006, the staff asked the applicant to explain the differences in wind direction frequency at 60 meters and 10 meters during the spring, summer, and winter seasons, when comparing the submitted VEGP wind data to the original FSAR data for Vogtle Units 1 and 2. In its letter dated January 30, 2007, the applicant explained that while the winds are somewhat uniform (in that the overall peak sector for both the original FSAR data and the 1998–2002 data is the same (west)), there is some variability among the annual data due to the relatively low wind speeds at the site. The staff has confirmed that the wind speeds are typically light at the site and thus some degree of variability can be expected. When winds are light they are typically not produced by a large-scale pressure gradient (e.g., synoptic scale), rather by smaller, more random and turbulent motions (e.g., meso-scale).

During the December 2006 site audit, the staff also asked the applicant to explain the amount of variability in summer wind direction frequency between the two onsite observation heights of 10 and

60 meters. The applicant stated in its letter dated January 30, 2007 that it was revising the wind roses for the summer season to correct an error and would include the corrected wind roses in the next revision of the SSAR. In a letter dated March 26, 2007, the applicant also provided a revised onsite 1998–2002 database, in which periods of bad data were removed and coded as such. Based on an independent review of the revised onsite meteorological data, the staff accepts the changes and concludes that the onsite meteorological wind data from 1998 through 2002 are both complete and consistent.

The staff agrees with the applicant that the winds for the proposed VEGP site are predominately from the southwest through west sectors. The staff also agrees with the annual average wind speeds of 2.5 m/s and 4.6 m/s at 10 and 60 meters as presented by the applicant. The staff's conclusions are based on a comparison between the Vogtle onsite meteorological wind data and nearby Augusta climatological data, as presented in the NCDC 2004 "Local Climatological Data."

2.3.2.3.1.2 Atmospheric Stability

The applicant classified atmospheric stability in accordance with the guidance provided in the proposed Revision 1 to RG 1.23. Atmospheric stability is a critical parameter for estimating dispersion characteristics in SSAR Sections 2.3.4 and 2.3.5. Dispersion of effluents is greatest for extremely unstable atmospheric conditions (i.e., Pasquill stability class A) and decreases progressively through extremely stable conditions (i.e., Pasquill stability class G). The applicant primarily based its stability classification on temperature change with height (i.e., delta-temperature or $\Delta T/\Delta Z$) between the 60-meter and 10-meter height, as measured by the Vogtle onsite meteorological monitoring program between 1998 and 2002.

The applicant provided seasonal and annual frequencies of atmospheric stability classes for the 5-year period of record for the onsite data from 1998–2002. According to the applicant, there is a predominance of slightly stable (Pasquill stability class E) and neutral stability (Pasquill stability class D) conditions at the proposed VEGP site, ranging from 50 to 60 percent of the time, on a seasonal and annual basis. Extremely unstable conditions (Pasquill stability class A) occur most frequently during spring and summer, and extremely stable conditions (Pasquill stability class G) occur most frequently during the fall and winter months. Based on past experience with stability data at various sites, a predominance of slightly stable (Pasquill stability class E) and neutral (Pasquill stability class D) conditions at the proposed site is generally consistent with expected meteorological conditions.

During a site audit conducted on December 6, 2006, the staff asked the applicant to explain the decrease in frequency of extremely unstable conditions (Pasquill stability class A) from 1998-2000 to 2001–2002, and the increase in frequency of slightly stable conditions (Pasquill stability class E) from 2000 to 2001. The staff also asked the applicant to explain a decrease in the number of occurrences of unstable conditions (Pasquill stability classes A–C) in 2001 and 2002, as compared to 1998 through 2000. The applicant responded, in its letter dated January 30, 2007, that there has been a slight decreasing trend in stability class A over the past 5 years; however, when individual stability classes are combined into the following three basic stability categories, (1) unstable (A-C), (2) neutral (D-E), and (3) stable (F-G) the decreasing trend is not as significant. The applicant stated that the increase in stability class E frequency was due to a data error. This error was corrected in the revised meteorological database. The staff reviewed the revised meteorological database and has concluded that its concerns regarding stability class frequencies have been resolved.

As a qualitative check of the hourly stability data provided by the applicant, the staff created plots of stability class as a function of time of day for each individual year, and, additionally, the 5 years together. SER Figure 2.3.2-1 is a plot of the proposed VEGP site 1998–2002 hourly stability class data as a function of time of day. Unstable conditions (Pasquill stability classes A–C) generally occurred during the day, and stable conditions (Pasquill stability classes F–G) generally occurred during the night, as expected due to daytime heating and nighttime cooling.

During a site audit conducted on December 6, 2006, the staff asked the applicant to explain a daytime increase in the number of occurrences of stable conditions (Pasquill stability classes F and G) in 2001, which is not seen in the other years. The applicant responded, in its letter dated January 30, 2007, that this could be attributed to a data error. This error was corrected in the revised meteorological database. The staff has confirmed that this problem has been fixed.

Frequency of occurrence for each stability class is one of the inputs to the dispersion models used in SSAR Sections 2.3.4 and 2.3.5. The applicant included these data in the form of a joint frequency distribution (JFD) of wind speed and direction data as a function of stability class. A comparison of a JFD developed by the staff from the hourly data submitted by the applicant with the JFD developed by the applicant showed reasonable agreement.

The staff accepts the 5 years of stability data presented by the applicant in SSAR Section 2.3.2 as complete and adequate. The staff believes that these data are appropriate to use as input to the dispersion models discussed in SER Sections 2.3.4 and 2.3.5.

2.3.2.3.1.3 Temperature

The applicant characterized normal and extreme temperatures for the site based on the 10 surrounding observation stations listed in SSAR Section 2.3.1.1. The extreme maximum temperature recorded near the site is 112 °F, and the extreme minimum temperature recorded near the site is −4 °F. Annual average temperatures for the 10 surrounding observation stations in the site vicinity (which are based on the average of the daily mean maximum and minimum temperatures) range from 63.1 °F to 65.0 °F. The applicant stated that the annual average diurnal (day-to-night) temperature differences in the site vicinity range from 21.9 °F to 26.3 °F.

Using data from NCDC and SERCC, the staff reviewed the daily mean temperatures, the extreme temperatures, and the diurnal temperature ranges presented by the applicant. The staff confirmed the temperature characterizations, as presented in SSAR Section 2.3.2, and accepts them as correct.

2.3.2.3.1.4 Water Vapor

The applicant presented wet-bulb temperature, dew point temperature, and relative humidity data summaries from the Augusta NWS observation station to characterize the typical atmospheric moisture conditions near the proposed VEGP site.

Based on a 49-year period of record, the applicant indicated that the mean annual wet-bulb temperature is 56.7 °F. The highest monthly mean wet-bulb temperature is 72.7 °F during July, and the lowest monthly mean wet-bulb temperature is 40.3 °F during January. According to the applicant, the mean annual dew point temperature at Augusta is 51.9 °F, which also reaches its maximum during summer and minimum during winter. The applicant gives the highest monthly mean dew point temperature as 69.7 °F during July, and the lowest monthly mean dew point temperature as 34.4 °F during January.

Based on a 30-year period of record, the applicant indicates that relative humidity averages 72 percent on an annual basis. The average early morning relative humidity levels exceed 90 percent during August, September, and October. Typically, the relative humidity values reach their diurnal maximum in the early morning and diurnal minimum during the early afternoon.

The staff has verified and accepts as correct and appropriate the wet-bulb temperature, dew point temperature, and relative humidity data presented by the applicant. The staff reviewed the data listed in the NCDC "Augusta, Georgia, 2004 Local Climatological Data, Annual Summary with Comparative Data." Because of the proximity of Augusta to the proposed VEGP site and because of the similarity of topographic features at both locations (i.e., gently rolling terrain, adjacent to the Savannah River, and location within the broad river valley), the Augusta atmospheric moisture data should be typical of the atmospheric moisture conditions in the proposed site region. SER Section 2.3.1 discusses the wet-bulb site characteristics more quantitatively.

2.3.2.3.1.5 Precipitation

Based on data from the 10 surrounding observation stations, the applicant provided that the average annual precipitation (water equivalent) totals generally range from 43.85 to 48.57 inches. The highest average annual precipitation is 52.43 inches, which occurs at the Aiken 4NE Station.

According to the applicant, snowfall is infrequent, with normal annual totals ranging from 0.1 to 1.4 inches. SER Section 2.3.1 discusses in greater detail snowfall in the vicinity of the proposed VEGP site.

Using daily snowfall and rainfall data from NCDC and SERCC, the staff has independently verified the precipitation statistics presented in SSAR Section 2.3.2 and accepts them as accurate.

2.3.2.3.1.6 Fog

Augusta is the closest station to the proposed VEGP site that makes fog observations. The applicant stated that, based on a 54-year period of record, Augusta averages about 35.1 days per year of heavy fog conditions (e.g., visibility is reduced to one-quarter mile or less).

According to the applicant, the frequency of typical fog conditions at Augusta is expected to be similar to that at the proposed VEGP site because of the proximity and similarity of topographic features between the two locations. Both sites are located in gently rolling terrain, adjacent to the Savannah River, and are situated in a broad river valley.

The staff confirmed the applicant's assertion that the Augusta NWS station reports 35.1 days per year with heavy fog observations. The staff agrees that the frequency of fog conditions at Augusta is expected to be similar to that at the proposed VEGP site because of the proximity and similarity of topographic features at both locations.

2.3.2.3.1.7 Air Quality

The applicant provided that the proposed VEGP site is located in the Augusta—Aiken Interstate Air Quality Control Region. The counties within this region, including Burke County, have been designated as being in attainment or unclassified for all EPA criteria air pollutants (i.e., ozone, carbon monoxide, nitrogen dioxide, sulfur dioxide, particulate matter, and lead) (40 CFR 81.311, "Georgia," and 40 CFR 81.34, "Metropolitan Dayton Intrastate Air Quality Control Region").

According to the applicant, the proposed nuclear steam supply system (NSSS) and other radiological systems related to the proposed facility will not be sources of criteria pollutants or other hazardous air pollutants. Other proposed supporting equipment such as diesel generators, fire pump engines, auxiliary boilers, emergency station-blackout generators, and other nonradiological emission-generating sources are not expected to be, in the aggregate, a significant source of criteria pollutant emissions. The staff agrees with this assessment because these systems will be used on an infrequent basis.

Because the EPA has designated the proposed VEGP site area as being in attainment or unclassified for all criteria air pollutants and the new facility is not expected to be a significant source of air pollutants, the staff finds that the VEGP site air quality conditions should not be a significant factor in the design and operating bases for the facility.

2.3.2.3.2 Impacts on Local Meteorology

The applicant stated that the associated paved, concrete, or other improved surfaces resulting from the construction of the proposed nuclear facility are insufficient to generate discernible, long-term effects to local- or micro-scale meteorological conditions. Wind flow may be altered immediately adjacent to and downwind of larger site structures, but these effects will likely dissipate within 10 structure heights downwind. SER Section 2.3.3 discusses the effects of these larger structures on wind flow.

Although temperature may increase above altered surfaces, the effects will be too limited in their vertical profile and horizontal extent to alter local- or regional-scale ambient temperature changes. Any water vapor releases from the proposed 600-foot-high natural draft cooling towers will have insignificant effects on local meteorology because of the high release height of thermal/water vapor plumes.

Because of the limited and localized nature of the expected modifications associated with the proposed plant structures and the associated improved surfaces, the staff agrees with the applicant that the proposed facility will not have significant impact on local meteorological conditions to affect plant design and operation.

The use of natural draft cooling towers could create visible plumes under certain atmospheric conditions, which could cause shadowing of nearby lands and salt deposition. Ground-level icing would be insignificant, though, because of the low probabilities of ground-level plumes and freezing conditions. The staff finds that these projected atmospheric impacts will not have significant impact on local meteorological conditions to affect plant design and operation.

During a site audit conducted on December 6, 2006, the staff asked the applicant to clarify whether any terrain modifications are expected to result from construction of the proposed facility and how they may affect the local meteorological characteristics of the site. The applicant responded in its letter dated January 30, 2007, that although there will be excavation, landscaping, site leveling, and clearing associated with the construction of the new units, these alterations to the site terrain would be localized and would not represent a significant alteration to the flat-to-gently-rolling topographic character of the area and region around the site. Therefore, the overall meteorological characteristics of the site will not be affected. The staff agrees that these activities are too small-scale to impact the local meteorological characteristics of the site.

2.3.2.3.3 Topographic Description of the Site

The proposed VEGP site is located in Burke County, Georgia, west of the Savannah River on approximately 3169 acres of land. The applicant provided maps of topographic features within a 5-mile radius of the site. The applicant also provided terrain elevation profiles along each of the 16 standard 22.5-degree compass radials out to a distance of 50 miles. Based on these profiles, the applicant characterized the proposed site terrain as flat to gently rolling. The only significant nearby topographic feature mentioned by the applicant is the broad Savannah River valley. The staff agrees with this terrain characterization based on topography data from the USGS and a site visit. The staff concludes that the applicant provided all the necessary topographic information.

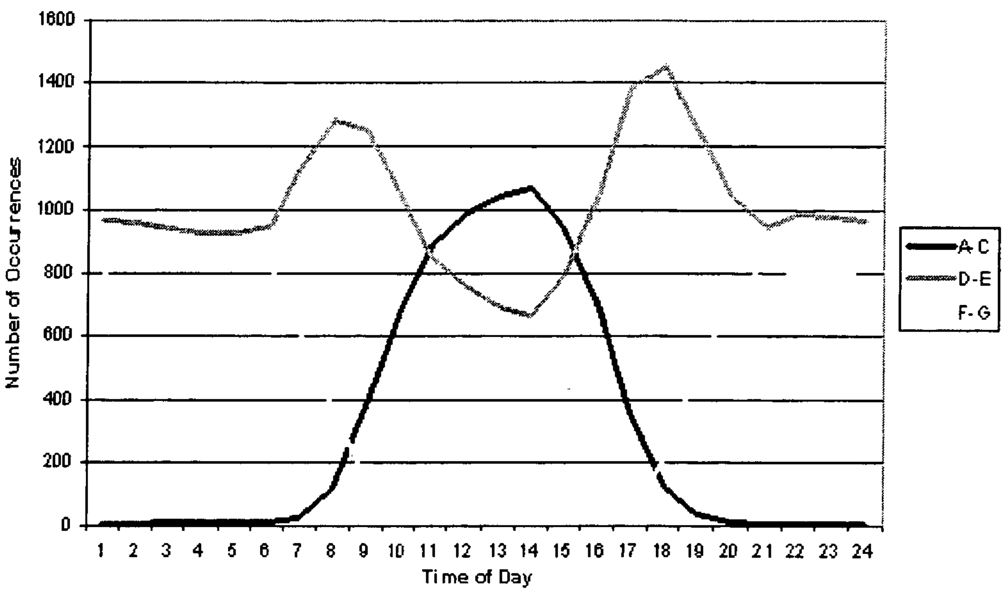
2.3.2.4 Conclusion

The NRC staff has evaluated the relevant sections of the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007, pursuant to the acceptance criteria of RS-002 Section 2.3.2 and applicable regulatory requirements of 10 CFR Part 52 and 10 CFR Part 100. As discussed above, the applicant has identified and provided acceptable consideration of the meteorological and topographical characteristics of the site and the surrounding area, including the potential impact on plant design and operation due to changes in local meteorology caused by plant construction and operation. Therefore, the staff finds that the applicant has provided the information required to address 10 CFR 52.17(a), 10 CFR 100.20(c), 10 CFR 100.21(c), and 10 CFR 100.21(d).

Table 2.3.2-1 - Offsite Temperature and Precipitation Extremes

PARAMETER	VALUE (DATE)	LOCATION
Maximum Temperature	112 °F (7/24/52)	Louisville 1E
Minimum Temperature	-4 °F (1/21/85)	Aiken 4NE
Maximum 24-hr Rainfall	9.68 in. (4/16/69)	Aiken 4NE
Maximum Monthly Rainfall	17.32 in. (6/73)	Springfield
Maximum 24-hr Snowfall	19.0 in. (2/10/73)	Bamberg
Maximum Monthly Snowfall	22.0 in. (2/73)	Bamberg

Figure 2.3.2-1 Vogtle 1998-2002 Hourly Stability Class Frequency



2.3.3 Onsite Meteorological Measurements Program

2.3.3.1 Introduction

In Section 2.3.3 of the SSAR, the applicant presented information concerning the onsite meteorological measurements program in support of its ESP application. Specifically, the applicant provided the following information:

- A description of meteorological instrumentation, including siting of sensors, sensor performance specifications, methods and equipment for recording sensor output, the QA program for sensors and recorders, and data acquisition and reduction procedures.

- Hourly meteorological data, including consideration of the period of record and amenability of the data for use in characterizing atmospheric dispersion conditions.

This section verifies that the applicant successfully implemented an appropriate onsite meteorological measurements program and that data from this program provide an acceptable basis for estimating atmospheric dispersion for DBA and routine releases from a nuclear power plant of the type specified by the applicant.

2.3.3.2 Regulatory Basis

The acceptance criteria for the development and implementation of an onsite meteorological program are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The staff considered the following regulatory requirements in reviewing the applicant's development and implementation of an onsite meteorological program:

- 10 CFR 52.17(a), which requires that the application contain a description of the seismic, meteorological, hydrological, and geological characteristics of the proposed site.

- 10 CFR 100.20(c), which requires that the meteorological characteristics of the site, necessary for safety analysis or that may have an impact on plant design, be identified and characterized as part of the NRC's review of the acceptability of a site.

- 10 CFR 100.21(c), which requires that site atmospheric dispersion characteristics be evaluated and dispersion parameters established such that (1) radiological effluent release limits associated with normal operation from the type of facility to be located at the site can be met for any individual located offsite; and (2) radiological dose consequences of postulated accidents shall meet the criteria set forth in
- 10 CFR 50.34(a)(1) for the type of facility proposed to be located at the site.

- 10 CFR 100.21(d), which requires that the physical characteristics of the site, including meteorology, geology, seismology, and hydrology be evaluated and site parameters established, such that the potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site.

The assessment and conclusions made in this section, regarding the site-specific adequacy of onsite meteorological instrumentation (including siting of sensors, sensor performance specifications, methods and equipment for recording sensor output, the QA program for sensors and recorders, and data acquisition and reduction procedures), are pertinent to the staff's evaluation, in SER Chapter 13, of the applicant's proposed emergency plan, in accordance with the following requirements of 10 CFR 50.47, "Emergency Plans," and 10 CFR Part 50, Appendix E, "Emergency Planning and Preparedness for Production and Utilization Facilities":

- 10 CFR 50.47(b), which requires that the onsite emergency response plan have adequate methods, systems, and equipment for assessing and monitoring actual or potential offsite consequences of a radiological emergency condition.

- 10 CFR Part 50, Appendix E, which requires emergency plans to have adequate provisions for equipment for determining the magnitude of and for continuously assessing impact of the release of radioactive materials to the environment.

The development and implementation of an onsite meteorological program is necessary for the collection of onsite meteorological information, so as to be able to demonstrate compliance, at the COL stage, with the numerical guides for doses contained in 10 CFR Part 50, Appendix I, "Numerical Guides for Design Objectives and limiting Conditions for Operation to Meet the Criterion 'As Low as Reasonable Achievable' for Radioactive material in Light-Water-Cooled Nuclear Power Reactor Effluents."

RS-002, Section 2.3.3 specifies that an application meets the above requirements, if the application satisfies the following criteria:

- The onsite meteorological measurements programs should produce data that describe the meteorological characteristics of the site and its vicinity for the purpose of making atmospheric dispersion estimate for both postulated accidental and expected routine airborne releases of effluents and for comparison with offsite sources to determine the appropriateness of climatological data used for design considerations. The criteria for an acceptable onsite meteorological measurements program are documented in the Regulatory Position, Section C, "Meteorological Monitoring Programs for Nuclear Power Plants," of RG 1.23.

To the extent applicable to the above-outlined acceptance criteria, the applicant applied the NRC-endorsed methodologies and parameters found in the following:

- RG 1.23, which provides criteria for an acceptable onsite meteorological measurements program, data from which are used as input to atmospheric dispersion models.

- RG 1.70, which provides guidance on information appropriate for presentation regarding an onsite meteorological measurements program.

- RG 4.2, "Preparation of Environmental Reports for Nuclear Power Stations," which states that the meteorological description of the site and its surrounding area should include data from the onsite meteorological program.

When independently assessing the sufficiency of the information presented by the applicant in SSAR Chapter 2.3.3, the NRC staff applied the same above-cited methodologies and parameters.

2.3.3.3 *Technical Evaluation*

Using the approaches and methodologies described in RS-002 Section 2.3.3, the NRC staff reviewed the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007. In reviewing and evaluating the applicant's onsite meteorological program, the staff used (or relied on) the following design parameters and site interface values proposed by the applicant in SSAR Section 1.3: building height, cooling tower height, cooling tower base diameter, and cooling tower diameter at the top.

The applicant used the existing onsite meteorological measurements program at the Vogtle facility (Units 1 & 2) to collect data for the proposed VEGP site and plans to continue to use this monitoring program to support operation of the proposed facility. If any changes are made to the monitoring program, the COL applicant should update the description of the proposed operational onsite meteorological measurements program at the time of the COL application in accordance with Section C.III.2.2.3.3 of RG 1.206, "Combined License Applications for Nuclear Power Plants."

2.3.3.3.1 Instrument Description

The Vogtle meteorological monitoring program began operation in 1979. Instruments for measuring pertinent meteorological parameters were mounted on a 45-meter tower located on a cleared area on the site. The facility updated the meteorological monitoring program in 1984 to meet the criteria of NUREG-0654, "Criteria for Preparation and Evaluation of Radiological Emergency Response Plans [RERP] and Preparedness in Support of Nuclear Power Plants." The updated monitoring equipment has observation heights at 10 and 60 meters above ground level. Measured data include wind speed and direction at 10 and 60 meters, temperature at 10 meters, differential temperature between 60 and 10 meters, dew point temperature at 10 meters, precipitation at the tower base, and sigma theta (wind direction standard deviation) at 10 and 60 meters. Currently, the original 45-meter tower is used as a backup meteorological monitoring system during periods of equipment failure on the 60-meter tower. The backup system can measure wind speed, wind direction, temperature, and sigma theta at the 10-meter level.

The meteorology tower is located about 4525 feet south of the proposed power block area. The applicant stated that the closest major structures to the meteorological measurement tower would be the proposed Unit 3 and 4 reactor buildings and proposed natural draft cooling towers. The cooling towers would be the largest structures in the vicinity of the meteorology tower and would have the greatest potential to influence the accuracy of future measurements because of the postulated downwind wake created by these structures.

The applicant stated that the region potentially affected by wake from the proposed cooling towers will extend about 1650 feet downwind. It based this value on the EPA 1981 version of the "Guideline for Determination of Good Engineering Practice Stack Height," which states that the distance downwind affected by the wake of a hyperbolically shaped natural draft cooling tower is about five times the width of the tower at the top of the structure. Since the closest cooling tower will be 3025 feet from the primary meteorological tower, the applicant determined that the primary meteorology tower will be outside of the potential wake zone.

RG 1.23 indicates that obstructions to flow (such as buildings) should be located at least 10 obstruction heights from the meteorological tower to prevent adverse building wake effects. Since the height of the proposed tallest power block structure is 234 feet above plant grade, the zone of turbulent flow created by the reactor buildings will be limited to about 2340 feet downwind. The staff concludes that building wake from the proposed reactor buildings will not cause any adverse affects on measurements because the meteorology tower is located 4525 feet south of the proposed power block area.

The 10-building-height distance of separation is typically applied to square or rectangular structures, whereas rounded and sloping structures such as hyperbolic natural draft cooling towers can be expected to produce a smaller wake zone. According to the applicant, the preliminary design for the natural draft cooling towers calls for them to be about 600 feet high, with a base diameter of 550 feet and a top diameter of 330 feet. In RAI 2.3.3-2, the staff asked the applicant to include the proposed natural draft cooling tower height and width as part of SSAR Table 1-1, which lists postulated design parameters, since this information is used to determine the potential wake effects from these towers. The applicant complied with this request.

Section 123 of the Clean Air Act as amended in 1990 defines good engineering practice stack height as the height necessary to ensure that emissions from a stack do not result in excessive concentrations of any air pollutant in the immediate vicinity of a source as a result of atmospheric downwash, eddies, and wakes which may be created by the source itself, by nearby structures, or by nearby terrain obstacles. The EPA defines "nearby structures" in its regulations (40 CFR 51.100(jj)(1)) as that distance up to five times the lesser of the height or the width dimension of a structure; that is, the downwind distance in which a structure is presumed to have a significant influence as a result of downwash, eddies, and wakes extends downwind approximately five times either the height or width (whichever is less) of the structure. The EPA regulatory guidance document for determining good engineering practice stack heights (EPA-450-4/80/023R, June 1985) also states that this area of influence becomes significantly smaller as the height to width ratio of a structure increases. Based on the EPA guidance for this type of structure, which will have a maximum width of 550 feet, the outermost boundary of influence exerted by the proposed cooling towers is estimated to be no more than 2750 feet. Since this distance is shorter than the 3025-foot separation between the proposed cooling towers and the primary meteorological tower, the staff concludes that the proposed natural draft cooling towers will not adversely affect measurements made at the primary meteorological tower. The staff calculated a larger area that may be affected by cooling tower wake because the updated 1985 EPA guidance used by the staff recommends using the maximum width of the structure, whereas the 1981 EPA guidance used by the applicant recommended using the width at the top of the structure for calculating potential wake influences.

The base of the primary tower is at an elevation similar to plant grade for the proposed facility, and the ground cover at the base of the tower is primarily native grass. The applicant stated that it evaluated minor structures in the vicinity of the primary meteorological tower as having no adverse effect on the measurements taken at the meteorological measurement tower. After conducting a site audit on December 6, 2006, the staff agrees with the applicant that the meteorology towers are sited in an appropriate area and these minor structures will have no adverse impact on the accuracy of measurements. The staff also noted during its site audit that the meteorology towers are located far enough from the surrounding tree line to prevent adverse effects on measurements.
SER Figure 2.3.3-1 shows the proposed layout of the VEGP site.

The primary meteorological equipment is mounted on a 200-foot Unarco-Rohn, Inc., Model 55G tower. All instrumentation (primary and backup) is mounted on a Tower Systems, Inc., Model TS-2500 instrument elevator system. The instruments are standard Climatronics products. The applicant uses Yokogawa digital equipment to receive the observations, which are displayed using the Meteorological Information and Dispersion Assessment System (MIDAS). The Climatronics Signal Conditioning Equipment is powered by dual (redundant) Hewlett Packard Model 6291A direct current power supplies.

During a site audit conducted on December 6, 2006, the staff reviewed the applicant's meteorology equipment calibration procedures in detail and found them to be adequate to ensure a reliable meteorological measurements program in accordance with RG 1.23. For example, the delta temperature calibration involves temperature baths using reference temperatures of 32 °F and 100 °F; the applicant checks to ensure on a regular basis that the delta-temperature instrumentation is taking accurate measurements. The applicant uses similar procedures for the other meteorological measurement equipment.

The applicant monitors the meteorology instruments at least once a week. Maintenance is performed in accordance with instrument manuals and is intended to maintain, at least, a 90-percent data recovery. From 1998–2002, the average data recovery rates are well above the RG 1.23 90-percent threshold.

Although all of the 5-year average recovery rates were still above 90 percent, the staff computed slightly different values for some of the annual data recovery rates. During a site audit conducted on December 6, 2006, the staff asked the applicant to verify the validity of the yearly data recovery statistics presented in the application. In a letter dated January 30, 2007, the applicant agreed with the values presented by the staff and stated that the hourly meteorological database was going to be updated. In RAI 2.3.3-1, the staff asked the applicant to provide the NRC with a copy of the updated hourly meteorological database. The applicant complied with this request. After receiving the updated and revised meteorological data, the staff was able to produce the same data recovery statistics as the applicant.

The applicant provided system performance specifications for the meteorological monitoring program, which are listed in SER Table 2.3.3-1. These values are consistent with RG 1.23 and thus accepted by the staff. Meteorological data samples are taken every 5 seconds and recorded as 15- and 60-minute averages. The 15-minute averages are used for emergency planning purposes, while the January 1998 through December 2002 hourly averages were used to compute the short-term and long-term diffusion estimates presented in SSAR Sections 2.3.4 and 2.3.5.

The description of meteorological instrumentation, including siting of sensors, sensor performance specifications, methods and equipment for recording sensor output, the QA program for sensors and recorders, and data acquisition and reduction procedures are in compliance with the guidelines of RG 1.23. Thus, the staff considers the meteorological instrumentation to be acceptable.

2.3.3.3.2 Meteorological Data

The applicant used the existing onsite meteorological measurements program from the Vogtle facility (Units 1 & 2) to collect hourly meteorological data. The applicant provided seasonal and annual summaries of onsite meteorological data in the SSAR, based on hourly measurements, from instrumentation mounted on the primary tower, taken over the 5-year period from 1998 through 2002. The applicant provided a copy of this 1998–2002 hourly database to the staff.

The staff performed a quality review of the 1998–2002 hourly meteorological database using the methodology described in NUREG-0917, "Nuclear Regulatory Commission Staff Computer Programs for Use with Meteorological Data," issued July 1982. The staff used computer spreadsheets to perform further review. During a site audit conducted on December 6, 2006, the staff notified the applicant that it had identified a few inconsistencies in the data (such as overly persistent wind directions or stability classes, temperature observations switching between degrees Celsius (°C) and Fahrenheit (°F), or delta-temperature measurements exceeding the auto-convective lapse rate) and asked the applicant for an explanation. The applicant responded in a letter dated January 30, 2007, that it would revise the onsite meteorological database to address these concerns. The staff reviewed a copy of this revised database and finds that the applicant has addressed all of the above concerns; a comparison between the JFD used by the applicant as input to the PAVAN and XOQDOQ atmospheric dispersion computer codes and a staff-generated JFD from the hourly database provided by the applicant shows that the two JFDs are similar.

To further check the validity and accuracy of the onsite meteorology data, the staff compared hourly data from the VEGP application to concurrent data obtained from the NCDC integrated hourly surface observations for Augusta. SER Table 2.3.3-2 compares 1998–2002 annual temperature, atmospheric moisture, wind speed, and wind direction statistics between the VEGP onsite data and the Augusta NWS data. The comparison of the 1998–2002 onsite temperature, atmospheric moisture, wind speed, and wind direction data with similar data recorded at Augusta for the same period of record shows that the Vogtle onsite data are reasonable.

Because of the reasonable correlation between the Augusta and Vogtle data, long-term temperature and atmospheric moisture data from Augusta are appropriate for determining the ambient air temperature and humidity site characteristics presented in SSAR Section 2.3.1. The Augusta annual maximum and minimum temperatures tend to be slightly more extreme than the Vogtle data. This implies that using Augusta data to characterize the extreme temperatures expected onsite is a conservative approach.

Based on an independent analysis of the onsite meteorological data and a comparison with hourly data from the Augusta NWS station, the staff accepts the 5 years of onsite data provided by the applicant as being representative of the site and an acceptable basis for estimating atmospheric dispersion for DBA and routine releases in SSAR Sections 2.3.4 and 2.3.5.

2.3.3.4 Conclusion

The NRC staff evaluated the relevant sections of the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007, pursuant to the acceptance criteria of RS-002 Section 2.3.3 and applicable regulatory requirements of 10 CFR Part 52 and 10 CFR Part 100. Based on the preceding discussion, the staff concludes that the applicant has successfully implemented an appropriate onsite meteorological measurements program and that data from this program provide an acceptable basis for estimating atmospheric dispersion for DBA and routine releases from a nuclear power plant of the type specified by the applicant. Therefore, the staff finds that the applicant has provided the information required to address 10 CFR 52.17(a)(1), 10 CFR 100.20(c), and 10 CFR 100.21(d). The staff also finds that analysis and conclusions regarding the site-specific adequacy of onsite meteorological instrumentation are sufficient to support the staff's evaluation of the applicant's proposed emergency plan, in SER Chapter 13, per 10 CFR 50.47 and 10 CFR Part 50, Appendix E.

Table 2.3.3-1 - Onsite Meteorological Monitoring Program Specifications

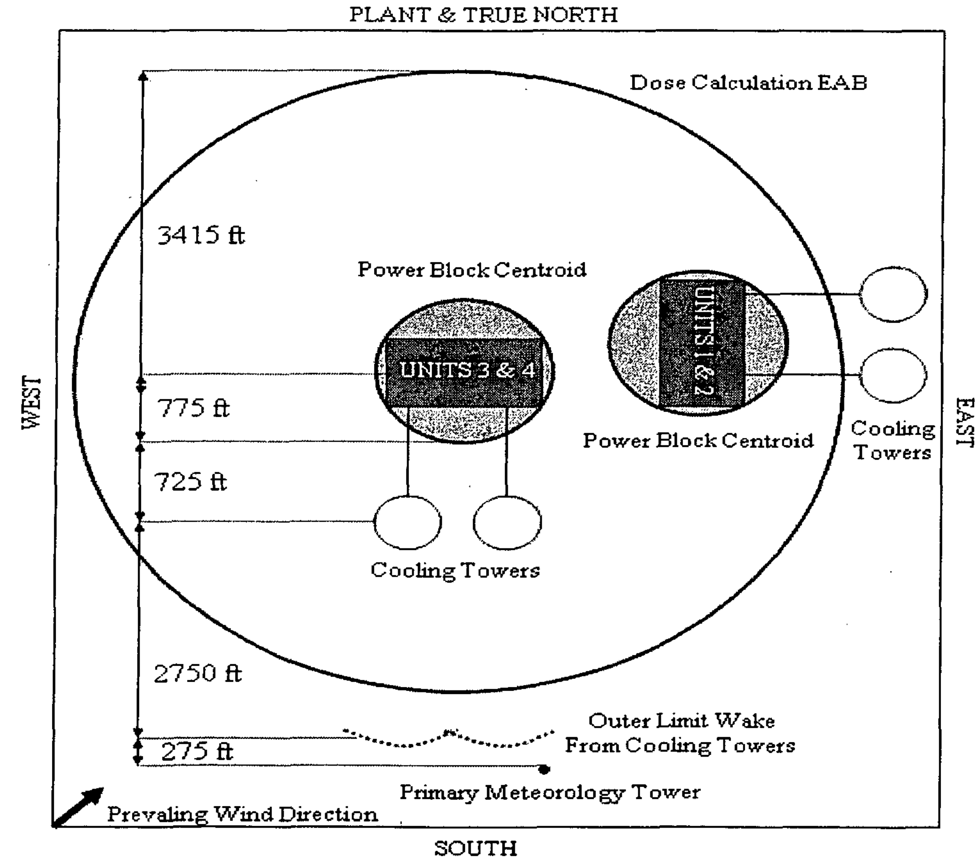
PARAMETER	RANGE	SYSTEM ACCURACY
Wind speed	0 - 100 mi/h	± 0.5 mi/h
Wind Direction	0 ° – 360 °	± 5 °
Ambient Temperature	-10 ° – 120 °F	± 0.9 °F
Differential Temperature	-5 ° – 10 °F	± 0.27 °F

Table 2.3.3-2 - Comparison of Augusta NWS and Vogtle Meteorology Observations

	ANNUAL AVERAGE TEMPERATURE		EXTREME MAXIMUM ANNUAL TEMPERATURE		EXTREME MINIMUM ANNUAL TEMPERATURE	
	AUGUSTA	VOGTLE	AUGUSTA	VOGTLE	AUGUSTA	VOGTLE
1998	65 °F	66 °F	103 °F	102 °F	19 °F	25 °F
1999	64 °F	65 °F	107 °F	104 °F	13 °F	17 °F
2000	63 °F	63 °F	101 °F	98 °F	13 °F	17 °F
2001	64 °F	64 °F	97 °F	94 °F	12 °F	20 °F
2002	64 °F	65 °F	101 °F	96 °F	16 °F	17 °F

	ANNUAL AVERAGE DEWPOINT		ANNUAL AVERAGE WIND SPEED		ANNUAL PREVAILING WIND DIRECTION	
	AUGUSTA	VOGTLE	AUGUSTA	VOGTLE	AUGUSTA	VOGTLE
1998	53 °F	53 °F	4.9 mi/h	5.1 mi/h	WSW	WSW
1999	51 °F	50 °F	5.3 mi/h	5.1 mi/h	WSW	SW
2000	52 °F	49 °F	5.1 mi/h	5.3 mi/h	WSW	SW
2001	52 °F	50 °F	5.1 mi/h	5.5 mi/h	WSW	W
2002	53 °F	51 °F	5.3 mi/h	5.2 mi/h	WSW	W

Figure 2.3.3-1 - Proposed Layout for VEGP Site



2.3.4 Short-Term Diffusion Estimates

2.3.4.1 Introduction

In Section 2.3.4 of the SSAR, the applicant presented information on atmospheric dispersion estimates for postulated accidental airborne releases of radioactive effluents to the EAB and the outer boundary of the LPZ. The applicant provided the following specific information:

- Atmospheric transport and diffusion models to calculate dispersion estimates (atmospheric dispersion factors, relative concentrations, or χ/Q values) for postulated accidental radioactive releases.

- Meteorological data summaries used as input to dispersion models.

- Diffusion parameters.

- Determination of χ/Q values used for assessment of consequences of postulated radioactive atmospheric releases from design-basis and other accidents.

This section verifies that the applicant has used appropriate atmospheric dispersion models and meteorological data to calculate relative concentrations at appropriate distances and directions from postulated release points for the evaluation of accidental airborne releases of radioactive material.

2.3.4.2 Regulatory Basis

The acceptance criteria for calculating atmospheric dispersion estimates for postulated accidental airborne releases of radioactive effluents are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The staff considered the following regulatory requirements in reviewing the applicant's calculation of atmospheric dispersion estimates for postulated accidental airborne releases of radioactive effluents

- 10 CFR 100.20(c), which requires that the meteorological characteristics of the site, necessary for safety analysis or that may have an impact on plant design, be identified and characterized as part of the NRC's review of the acceptability of a site.

- 10 CFR 100.21(c)(2), which requires that site atmospheric dispersion characteristics be evaluated and dispersion parameters established such that radiological dose consequences of postulated accidents shall meet the criteria set forth in 10 CFR 50.34(a)(1) for the type of facility proposed to be located at the site.

The applicant also originally identified Appendix E to 10 CFR Part 50 as applicable to SSAR Section 2.3.4. In RAI 2.3.4-2, the staff asked the applicant to explain how Appendix E applies to the development of the short-term (accidental release) atmospheric dispersion estimates presented in SSAR Section 2.3.4. The applicant responded by deleting the reference to Appendix E to 10 CFR Part 50 in SSAR Section 2.3.4.

RS-002, Section 2.3.4 specifies that an application meets the above requirements, if the application provides the following information:

- A description of the atmospheric dispersion models used to calculate relative concentrations (χ/Q values) in air resulting from accidental releases of radioactive material to the atmosphere. The models should be documented in detail and substantiated within the limits of the model so that the staff can evaluate their appropriateness to site characteristics, plant characteristics (to the extent known), and release characteristics.

- Meteorological data used for the evaluation (as input to the dispersion models) which represent annual cycles of hourly values of wind direction, wind speed, and atmospheric stability for each mode of accidental release.

- The variation of atmospheric diffusion parameters used to characterize lateral and vertical plume spread as a function of distance, topography, and atmospheric conditions, as related to measured meteorological parameters. The methodology for establishing these relationships should be appropriate for estimating the consequences of accidents within the range of distances which are of interest with respect to site characteristics and established regulatory criteria.

- Cumulative probability distributions of relative concentrations (χ/Q values) describing the probabilities of these χ/Q values being exceeded. These cumulative probability distributions should be presented for appropriate distances and time periods as specified in Section 2.3.4.2 of RG 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)." The methods of generating these distributions should be adequately described.

- Relative concentrations used for assessment of consequences of atmospheric radioactive releases from design-basis and other accidents.

To the extent applicable to the above-outlined acceptance criteria, the applicant applied the NRC-endorsed analytical methodologies, models and parameters found in the following:

- RG 1.23, which provides criteria for an acceptable onsite meteorological measurements program, data from which are used as input to atmospheric dispersion models.

- RG 1.70, which states that the SSAR should provide atmospheric estimates at the EAB and outer boundary of the LPZ for appropriate time periods up to 30 days after an accident based on the most representative meteorological data and potential impacts of topography on atmospheric dispersion site characteristics.

- RG 1.111, "Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors," which provides acceptable methods for characterizing annual average atmospheric transport and diffusion conditions for evaluating the consequences of radiological releases at the EAB and outer boundary of the LPZ.

- RG 1.145, "Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants," which provides acceptable methods for characterizing atmospheric dispersion conditions for appropriate time periods up to 30 days for evaluating the consequences of DBA radiological releases to the EAB and outer boundary of the LPZ.

- RG 1.183, "Alternative Radiological Source Terms for Evaluating Design Basis Accidents at Nuclear Power Reactors," which provides criteria on the use of alternative radiological source terms for evaluating the consequences of DBAs.

- RG 4.7, which provides criteria on the amount of meteorological data necessary to ensure the generation of representative atmospheric dispersion site characteristics.

The applicant originally identified RG 1.78 as applicable to SSAR Section 2.3.4. In RAI 2.3.4-3, the staff asked the applicant to explain how RG 1.78 applies to the development of the short-term (accidental release) atmospheric dispersion site characteristics presented in SSAR Section 2.3.4. The applicant responded by deleting the reference to RG 1.78 for SSAR Section 2.3.4.

When independently assessing the veracity of the information presented by the applicant in SSAR Chapter 2.3.4, the NRC staff applied the same above-cited methodologies, models and parameters.

2.3.4.3 Technical Evaluation

Using the approaches and analytic methodologies described in RS-002 Section 2.3.4, the NRC staff reviewed the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007. In reviewing and evaluating the applicant's short-term atmospheric dispersion estimates, the staff used (or relied on) only the elevation of the post-accident release point from the design parameters and site interface values presented by the applicant in SSAR Section 1.3.

2.3.4.3.1 Atmospheric Dispersion Mode

The applicant used the computer code PAVAN (NUREG/CR-2858, "PAVAN: An Atmospheric Dispersion Program for Evaluating Design-Basis Accidental Releases of Radioactive Materials from Nuclear Power Stations,") to estimate χ/Q values at the EAB and at the outer boundary of the LPZ for potential accidental releases of radioactive material. The PAVAN model implements the methodology outlined in RG 1.145.

The PAVAN code estimates χ/Q values for various time-average periods ranging from 2 hours to 30 days. The meteorological input to PAVAN consists of a joint frequency distribution (JFD) of hourly values of wind speed and wind direction by atmospheric stability class. In response to RAI 2.3.4-5, the applicant provided a copy of the input file used to compute the χ/Q values listed in SSAR Section 2.3.4. The staff used this input file, as well as the hourly meteorological data, to verify the χ/Q values presented by the applicant, as discussed in SER Section 2.3.4.3.4.

The χ/Q values calculated through PAVAN are based on the theoretical assumption that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the point of release and all distances for which χ/Q values are calculated.

For each of the 16 downwind direction sectors (e.g., N, NNE, NE, ENE), PAVAN calculates χ/Q values for each combination of wind speed and atmospheric stability at the appropriate downwind distance (i.e., the EAB and the outer boundary of the LPZ). The χ/Q values calculated for each sector are then ordered from greatest to smallest and an associated cumulative frequency distribution is derived based on the frequency distribution of wind speed and stabilities for each sector. The smallest χ/Q value in a distribution will have a corresponding cumulative frequency equal to the wind direction frequency for that particular sector. PAVAN determines for each sector an upper envelope curve based on the derived data (plotted as χ/Q versus probability of being exceeded), such that no plotted point is above the curve. From this upper envelope, the χ/Q value, which is equaled or exceeded 0.5 percent of the total time, is obtained. The maximum 0.5 percent χ/Q value from the 16 sectors becomes the 0–2 hour "maximum sector χ/Q value."

Using the same approach, PAVAN also combines all χ/Q values independent of wind direction into a cumulative frequency distribution for the entire site. An upper envelope curve is determined, and the program selects the χ/Q value which is equaled or exceeded 5.0 percent of the total time. This is known as the 0–2 hour "5-percent overall site χ/Q value."

The larger of the two χ/Q values, either the 0.5-percent maximum sector value or the 5-percent overall site value, is selected to represent the χ/Q value for the 0–2 hour time interval (note that this resulting χ/Q value is based on 1-hour averaged data but is conservatively assumed to apply for 2 hours).

To determine χ/Q values for longer time periods (i.e., 0–8 hour, 8–24 hour, 1–4 days, and 4-30 days), PAVAN performs a logarithmic interpolation between the 0–2 hour χ/Q values and the annual average (8760-hour) χ/Q values for each of the 16 sectors and overall site. For each time period, the highest among the 16 sector and overall site χ/Q values is identified and becomes the short-term site characteristic χ/Q value for that time period.

2.3.4.3.2 Meteorological Data Input

The meteorological input to PAVAN used by the applicant consisted of a JFD of wind speed, wind direction, and atmospheric stability based on hourly onsite data from January 1998 through December 2002. The wind data were obtained from the 10-meter level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-temperature) measurements taken between the 60-meter and 10-meter levels on the onsite meteorological tower.

As discussed in SER Section 2.3.3, the staff considers the 1998–2002 onsite meteorological database suitable for input to the PAVAN model.

2.3.4.3.3 Diffusion Parameters

The applicant chose to implement the diffusion parameter assumptions outlined in RG 1.145, as a function of atmospheric stability, for its PAVAN model runs. The staff evaluated the applicability of the PAVAN diffusion parameters and concluded that no unique topographic features (such as rough terrain, restricted flow conditions, or coastal or desert areas) preclude the use of the PAVAN model for the VEGP site. Therefore, the staff finds that the applicant's use of diffusion parameter assumptions, as outlined in RG 1.145, was acceptable.

2.3.4.3.4 Relative Concentration for Accident Consequences Analysis

The applicant modeled one ground-level release point and did not take credit for building wake effects. Ignoring building wake effects for a ground-level release decreases the amount of atmospheric turbulence assumed to be in the vicinity of the release point, resulting in higher (more conservative) χ/Q values. A ground-level release assumption is therefore acceptable to the staff.

The applicant defined a "dose calculation" EAB as a circle that extends 0.5 mile beyond the power block area.[9] Consequently, the applicant executed PAVAN using a distance from release point to the

[9] Because the power block area is defined as being within a 775-foot-radius circle centered on a point between the two proposed AP1000 units, the dose calculation EAB can also be defined as a circle with a radius of 3,415 feet from the proposed power block centroid.

dose calculation EAB of 0.5 mile (800 meters) for all downwind sectors. The applicant stated that because the dose calculation EAB is circumscribed the "true" (actual) EAB for the site, any χ/Q values produced by PAVAN will be conservative estimates. The staff verified that the dose calculation EAB is within the true EAB for the site and is therefore acceptable to the staff.

The outer boundary of the LPZ for the proposed facility is a 2-mile-radius circle centered on the existing power block. The applicant chose to use a downwind distance of 1.4 miles (2304 meters) for all direction sectors for calculating LPZ χ/Q values because this is the shortest distance in any direction from the proposed power block area boundary to the predefined LPZ. The use of the shortest distance results in higher (more conservative) χ/Q values and is therefore acceptable to the staff.

SER Table 2.3.4-1 lists the short-term atmospheric dispersion estimates for the dose calculation EAB and the outer boundary of the LPZ that the applicant derived from its PAVAN modeling run results. The applicant identified these χ/Q values as site characteristics in SSAR Table 1-1 because these are the atmospheric dispersion site characteristics used by the applicant to demonstrate compliance with the terms of 10 CFR 100.21(c)(2) for the radiological dose consequences of postulated accidents.

The applicant originally identified the 0.5-percent maximum sector EAB χ/Q value as being larger than the 5-percent overall site EAB χ/Q value. In contrast, by way of confirmatory analysis, the staff found the 5-percent overall site χ/Q value to be the larger of the two values. In RAI 2.3.4-4, the staff asked the applicant to confirm which of the two χ/Q values is more limiting for the site. The applicant responded that a new PAVAN run, using the revised meteorological database discussed in SER Section 2.3.3, verified the staff's results: the 5-percentile overall site EAB χ/Q value did indeed bound the 0.5-percentile maximum sector EAB χ/Q value.

The staff confirmed the applicant's atmospheric dispersion estimates by running the PAVAN computer model and obtaining similar results (i.e., plus or minus 4 percent).

In light of the foregoing, the staff accepts the short-term χ/Q values presented by the applicant. The staff will include the short-term χ/Qs listed in SER Table 2.3.4-1 as site characteristics in any ESP that the NRC may issue for the VEGP site.

2.3.4.4 Conclusion

The NRC staff has evaluated the relevant sections of the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007, pursuant to the acceptance criteria described in RS-002 Section 2.3.4 and the applicable regulatory requirements of 10 CFR Part 52 and 10 CFR Part 100. As discussed above, the applicant provided meteorological data and an atmospheric dispersion model that are appropriate for the characteristics of the site. Therefore, the staff concludes that representative atmospheric transport and diffusion conditions have been calculated at the EAB and the outer boundary of the LPZ, and, thus, that the applicant has provided the information required to comply with the applicable provisions of 10 CFR Part 52 and 10 CFR 100.21(c)(2).

Table 2.3.4-1 - Short-Term (Accidental Release) Atmospheric Dispersion Site Characteristics

SITE CHARACTERISTIC	VALUE	DEFINITION
0–2 hr χ/Q value @ EAB	3.49×10^{-4} s/m^3	The atmospheric dispersion coefficients used in the design safety analysis to estimate dose consequences of accidental airborne releases.
0–8 hr χ/Q value @ LPZ outer boundary	7.04×10^{-5} s/m^3	
8–24 hr χ/Q value @ LPZ outer boundary	5.25×10^{-5} s/m^3	
1–4 day χ/Q value @ LPZ outer boundary	2.77×10^{-5} s/m^3	
4–30 day χ/Q value @ LPZ outer boundary	1.11×10^{-5} s/m^3	

2.3.5 Long-Term Diffusion Estimates

2.3.5.1 Introduction

In Section 2.3.5 of the SSAR, the applicant presented its atmospheric dispersion estimates for routine releases of radiological effluents to the atmosphere. Specifically, the applicant provided the following information:

- atmospheric dispersion models used to calculate concentrations in air and the amount of material deposited as a result of routine releases of radioactive material to the atmosphere.
- points of routine release of radioactive material to the atmosphere, the characteristics of each release mode, and the location of potential receptors for dose computations.
- meteorological data used as input to dispersion models.
- diffusion parameters.
- relative concentration factors (χ/Q values) and relative deposition factors (D/Q values) used to assess the consequences of routine airborne radioactive releases.

This section verifies that the applicant has used appropriate atmospheric dispersion models and meteorological data to calculate relative concentration and relative deposition at appropriate distances and directions from postulated release points for the evaluation of routine airborne releases of radioactive material.

2.3.5.2 Regulatory Basis

The acceptance criteria for calculating atmospheric dispersion estimates for routine releases of radiological effluents are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The staff considered the following regulatory requirements in reviewing the applicant's calculation of atmospheric dispersion estimates for routine releases of radiological effluents:

- 10 CFR 100.20(c), which requires that the meteorological characteristics of the site, necessary for safety analysis or that may have an impact on plant design, be identified and characterized as part of the NRC's review of the acceptability of a site.
- 10 CFR 100.21(c)(1), which requires that site atmospheric dispersion characteristics be evaluated and dispersion parameters established such that radiological effluent release limits associated with normal operation from the type of facility to be located at the site can be met for any individual located offsite.

Characterization of atmospheric transport and diffusion conditions is necessary for estimating the radiological consequences of routine releases of radioactive materials to the atmosphere, so as to demonstrate compliance, at the COL stage, with the numerical guides for doses contained in 10 CFR Part 50, Appendix I, "Numerical Guides for Design Objectives and limiting Conditions for Operation to Meet the Criterion 'As Low as Reasonable Achievable' for Radioactive Material in Light-Water-Cooled Nuclear Power Reactor Effluents."

The applicant originally identified in its application Appendix E to 10 CFR Part 50 as applicable to SSAR Section 2.3.5. In RAI 2.3.5-3, the staff asked the applicant to explain how Appendix E applies to the development of the long-term (routine release) atmospheric dispersion estimates presented in SSAR Section 2.3.5. The applicant responded by deleting the reference to Appendix E to 10 CFR Part 50 in SSAR Section 2.3.5.

RS-002, Section 2.3.5 specifies that an application meets the above requirements, if the application provides the following information:

- A description of the atmospheric dispersion models used to calculate concentrations in air and the amount of material deposited as a result of routine releases of radioactive material to the atmosphere. The models should be sufficiently documented and substantiated to allow a review of their appropriateness for site characteristics, plant characteristics (to the extent known), and release characteristics.

- A discussion of the relationship between atmospheric diffusion parameters, such as vertical plume spread, and measured meteorological parameters. Use of these parameters should be substantiated as to their appropriateness for use in estimating the consequences of routine releases from the site boundary to a radius of 50 miles from the plant site.

- Meteorological data used as input to the dispersion models. Data used for this evaluation should represent hourly average values of wind speed, wind direction, and atmospheric stability which are appropriate for each mode of release. The data should reflect atmospheric transport and diffusion conditions in the vicinity of the site throughout the course of a year.

- Relative concentration (χ/Q) and relative deposition (D/Q) values used for assessment of consequences of routine radioactive gas releases.

- Points of routine release of radioactive material to the atmosphere, the characteristics of each release mode, and the location of potential receptors for dose computations.

To the extent applicable to the above-outlined acceptance criteria, the applicant applied the NRC-endorsed analytical methodologies, models and parameters found in the following:

- RG 1.23, which provides criteria for an acceptable onsite meteorological measurements program, data from which are used as input to atmospheric dispersion models.

- RG 1.70, which states that the SSAR should provide realistic estimates of annual average atmospheric transport and diffusion characteristics out to a distance of 50 miles from the plant, including a detailed description of the model used and a calculation of the maximum annual average χ/Q value at or beyond the site boundary for each venting location.

- RG 1.109, "Calculation of Annual Doses to Man from Routine Releases of Reactor Effluents for the Purpose of Evaluating Compliance with 10 CFR Part 50, Appendix I," which presents identification criteria to be used for specific receptors of interest.

- RG 1.111, which provides acceptable methods for characterizing atmospheric transport and diffusion conditions for evaluating the consequences of routine effluent releases.

- RG 1.112, "Calculation of Releases of Radioactive Materials in Gaseous and Liquid Effluents from Light-Water-Cooled Power Reactors," which provides criteria for identifying release points and release characteristics.

When independently assessing the veracity of the information presented by the applicant in SSAR Chapter 2.3.5, the NRC staff applied the same above-cited methodologies, models and parameters.

2.3.5.3 Technical Evaluation

Using the approaches and analytic methodologies described in RS-001 Section 2.3.5, the NRC staff reviewed the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007. In reviewing and evaluating the applicant's long-term atmospheric dispersion estimates, the staff used (or relied on) none of the applicant's proposed design parameters and site interface values presented in SSAR Section 1.3, but did rely on the routine release point elevation, containment building minimum cross-sectional area, and the equivalent structural height values presented by the applicant in SSAR Section 2.3.5.

2.3.5.3.1 Atmospheric Dispersion Model

The applicant used the NRC-sponsored computer code XOQDOQ (described in NUREG/CR-2919, "XOQDOQ Computer Program for the Meteorological Evaluation of Routine Effluent Releases at Nuclear Power Stations,") to estimate χ/Q and D/Q values resulting from routine releases. The XOQDOQ model implements the methodology outlined in RG 1.111.

The XOQDOQ model is a straight-line Gaussian plume model based on the theoretical assumption that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. In predictions of χ/Q and D/Q values for long time periods (i.e., annual averages), the plume's horizontal distribution is assumed to be evenly distributed within the downwind direction sector (e.g., "sector averaging").

Because geographic features such as hills, valleys, and large bodies of water can potentially influence dispersion and airflow patterns, terrain recirculation factors can be used to adjust the results of a straight-line trajectory model such as XOQDOQ to account for terrain-induced flows, recirculation, or stagnation. In RAI 2.3.5-5, the staff asked the applicant to explain why it did not use terrain recirculation factors, which were used in Chapter 8 of Revision 21 of the VEGP Offsite Dose Calculation Manual (ODCM, dated October 1, 2003), in developing the long-term χ/Qs presented in the VEGP SSAR. The applicant responded that the topographic features in the site vicinity do not require the use of terrain recirculation factors and that the analyses reported in the Unit 1/Unit 2 FSAR did not use these factors. The applicant also stated that most terrain recirculation factors used in the ODCM for ground-level releases are about 1. Based on SSAR Figure 2.3-15, topographical descriptions in SSAR Section 2.3.1, and a site audit conducted on December 6, 2006, the staff agrees with the applicant that the site can be characterized as having open terrain with gently rolling hills. Thus, the staff concludes that XOQDOQ modeling results are applicable to the site and no unique topographic features (such as valley, desert, or overall water trajectories) preclude the use of the model for the proposed VEGP site.

2.3.5.3.2 Release Characteristics and Receptors

The applicant modeled one ground-level release point, assuming a minimum building cross-sectional area of 2,926 square meters and a containment "equivalent" structure height of 65.6 meters. The staff asked the applicant in RAI 2.3.5-1 to provide the basis for the calculation of the containment building minimum cross-sectional area and equivalent structural height. In its response, the applicant stated that the equivalent structure height was determined by dividing the building cross-sectional area by the width of the proposed reactor containment at the bottom.

A ground-level release is a conservative assumption resulting in higher χ/Q and D/Q values when compared to a mixed-mode (e.g., part-time ground, part-time elevated) release or a 100-percent elevated release, as discussed in RG 1.111. A ground-level release assumption is therefore acceptable to the staff.

The applicant executed XOQDOQ using a distance from the release point to the dose calculation EAB of 0.5 mile (800 meters) for all downwind sectors as discussed in SSAR Section 2.3.4.3. The applicant also placed receptors of interest (i.e., resident, meat animal, and vegetable garden) in all compass directions at a downwind distance of 1,071 meters. This distance is based on the closest of these receptors (the nearest resident in the west-southwest sector), as identified in the VEGP "Annual Radiological Environmental Operating Report (AREOP) for 2004," produced by Southern Company (ADAMS Accession No. ML051380059). This is a conservative assumption and is therefore acceptable to the staff. SER Table 2.3.5-1 compares the AREOP distances and the distances used as input to the XOQDOQ model.

2.3.5.3.3 Meteorological Data Input

The meteorological input to XOQDOQ consists of a JFD of wind speed, wind direction, and atmospheric stability based on hourly onsite data from January 1998 through December 2002. The wind data were obtained from the 10-meter level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-temperature) measurements taken between the 60-meter and 10-meter levels on the onsite meteorological tower.

As discussed in SER Section 2.3.3, the staff considers the 1998–2002 onsite meteorological database suitable for input to the XOQDOQ model.

2.3.5.3.4 Diffusion Parameters

The applicant chose to implement the diffusion parameter assumptions outlined in RG 1.111, as a function of atmospheric stability, for its XOQDOQ model runs. The staff evaluated the applicability of the XOQDOQ diffusion parameters and concluded that no unique topographic features (such as valley, desert, or over water trajectories) preclude the use of the XOQDOQ model for the VEGP site. Therefore, the staff finds that the applicant's use of diffusion parameter assumptions, as outlined in RG 1.111, was acceptable.

2.3.5.3.5 Resulting Relative Concentration and Relative Deposition Factors

SER Table 2.3.5-2 lists the long-term atmospheric dispersion and deposition estimates for the dose calculation EAB and special receptors of interest that the applicant derived from its XOQDOQ modeling results. The applicant identified these χ/Q and D/Q values as site characteristics in SSAR Table 1-1 because these are the atmospheric dispersion site characteristics used by the applicant to demonstrate compliance with the terms of 10 CFR 100.21(c)(1) for the radiological dose consequences related to routine operation.

In response to RAI 2.3.5-6, the applicant provided long-term atmospheric dispersion and deposition estimates for all 16 radial sectors from the site boundary, to a distance of 50 miles from the proposed facility, in SSAR Table 2.3-18. The COL applicant will need to use this information to show that the proposed plant's gaseous radiological waste systems include all items of reasonably demonstrated technology that, when added to the system sequentially and in order of diminishing cost-benefit return, can, for a favorable cost-benefit ratio, effect reductions in dose to the population reasonably expected to be within 50 miles of the reactor, in accordance with the requirements of Section II.D of Appendix I to 10 CFR Part 50.

The χ/Q values presented in SER Table 2.3.5-2 reflect several plume radioactive decay and deposition scenarios. Section C.3 of RG 1.111 states that radioactive decay and dry deposition should be considered in radiological impact evaluations of potential annual radiation doses to the public, resulting from routine releases of radioactive materials in gaseous effluents. Section C.3.a of RG 1.111 states that an overall half-life of 2.26 days is acceptable for evaluating the radioactive decay of short-lived noble gases and an overall half-life of 8 days is acceptable for evaluating the radioactive decay for all iodines released to the atmosphere.

Definitions for the χ/Q categories listed in the headings of SER Table 2.3.5-2 are as follows:

- Undepleted/No Decay χ/Q values are χ/Qs used to evaluate ground-level concentrations of long-lived noble gases, tritium, and carbon-14. The plume is assumed to travel downwind, without undergoing dry deposition or radioactive decay.

- Undepleted/2.26-Day Decay χ/Q values are χ/Qs used to evaluate ground-level concentrations of short-lived noble gases. The plume is assumed to travel downwind, without undergoing dry deposition, but is decayed, assuming a half-life of 2.26 days, based on the half-life of xenon-133m.

- Depleted/8.00-Day Decay χ/Q values are χ/Qs used to evaluate ground-level concentrations of radioiodine and particulates. The plume is assumed to travel downwind, with dry deposition, and is decayed, assuming a half-life of 8.00 days, based on the half-life of iodine-131.

The applicant provided a copy of its XOQDOQ input file in response to RAI 2.3.5-4. Using this information as well as the updated meteorological data provided by the applicant in its March 30, 2007 letter, the staff confirmed the applicant's χ/Q and D/Q values by running the XOQDOQ computer code and obtaining the same results.

In light of the foregoing, the staff accepts the long-term χ/Q and D/Q values presented by the applicant. The staff will include the long-term atmospheric dispersion and deposition factors listed in SER Table 2.3.5-2 as site characteristics in any ESP that the NRC might issue for the VEGP site.

2.3.5.4 Conclusion

The NRC staff evaluated the relevant sections of the application, as supplemented by letters dated January 30, 2007, March 26, 2007, and March 30, 2007, pursuant to the acceptance criteria of RS-002 Section 2.3.5 and applicable regulatory requirements of 10 CFR Part 52 and 10 CFR Part 100. As discussed above, the applicant has provided meteorological data and an atmospheric dispersion model that are appropriate for the characteristics of the site and release points. Therefore, the staff concludes that the applicant has calculated representative atmospheric transport and diffusion conditions for 16 radial sectors from the site boundary to a distance of 50 miles and for the specific receptor locations.

Therefore, the applicant has provided the information required to address 10 CFR 52.17(a), 10 CFR 100.20, and 10 CFR 100.21(c)(1). The staff also concludes that the applicant's characterization of long-term atmospheric transport and diffusion conditions would be appropriate, at the COL stage, for use in demonstrating compliance with the numerical guides for doses contained in Appendix I to 10 CFR Part 50.

Table 2.3.5-1 - Distances between the Proposed Units 3 and 4 Power Block and Receptors of Interest[10]

RECEPTOR	DOWNWIND DIRECTION SECTOR	DISTANCE COMPILED FROM THE AREOP	DISTANCE USED
Nearest Resident	N	2032 m	1071 m
	NNE	>8045 m	1071 m
	NE	>8045 m	1071 m
	ENE	>8045 m	1071 m
	E	>8045 m	1071 m
	ESE	7118 m	1071 m
	SE	7327 m	1071 m
	SSE	7410 m	1071 m
	S	6835 m	1071 m
	SSW	7068 m	1071 m
	SW	3633 m	1071 m
	WSW	1071 m	1071 m
	W	5024 m	1071 m
	WNW	2069 m	1071 m
	NW	>8045 m	1071 m
	NNW	1946 m	1071 m
Meat Animal	N	>8045 m	1071 m
	NNE	>8045 m	1071 m
	NE	>8045 m	1071 m
	ENE	>8045 m	1071 m
	E	>8045 m	1071 m
	ESE	>8045 m	1071 m
	SE	>8045 m	1071 m
	SSE	7414 m	1071 m
	S	>8045 m	1071 m
	SSW	6736 m	1071 m
	SW	7155 m	1071 m
	WSW	6366 m	1071 m
	W	6170 m	1071 m
	WNW	>8045 m	1071 m
	NW	2400 m	1071 m
	NNW	>8045 m	1071 m
Vegetable Garden	N	>8045 m	1071 m
	NNE	>8045 m	1071 m
	NE	>8045 m	1071 m
	ENE	>8045 m	1071 m
	E	>8045 m	1071 m
	ESE	>8045 m	1071 m
	SE	>8045 m	1071 m
	SSE	>8045 m	1071 m
	S	>8045 m	1071 m
	SSW	>8045 m	1071 m
	SW	>8045 m	1071 m
	WSW	4273 m	1071 m
	W	>8045 m	1071 m
	WNW	4458 m	1071 m
	NW	5899 m	1071 m
	NNW	>8045 m	1071 m

10 Note that 2004 AREOP did not report any milk-giving animals (either cows or milk) within a 5-mile radius of the proposed VEGP site.

Table 2.3.5-2 - Long-Term (Routine Release) Atmospheric Dispersion Site Characteristics

SITE CHARACTERISTIC	VALUE	DEFINITION
Annual Average Undepleted/No Decay χ/Q Value @ EAB, northeast, 0.5 mile	5.5×10^{-6} s/m^3	The maximum annual average EAB undepleted/no decay atmospheric dispersion factor (χ/Q value) for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/2.26-Day Decay χ/Q Value @ EAB, northeast, 0.5 mile	5.5×10^{-6} s/m^3	The maximum annual average EAB undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Depleted/8.00-Day Decay χ/Q Value @ EAB, northeast, 0.5 mile	5.0×10^{-6} s/m^3	The maximum annual average EAB depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average D/Q Value @ EAB, northeast and east-northeast, 0.5 mile	1.7×10^{-8} 1/m^2	The maximum annual average EAB relative deposition factor (D/Q value) for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Resident, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average resident undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/2.26-Day Decay χ/Q Value @ Nearest Resident, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average resident undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Depleted/8.00-Day Decay χ/Q Value @ Nearest Resident, northeast, 0.67 mile	3.0×10^{-6} s/m^3	The maximum annual average resident depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average D/Q Value @ Nearest Resident, northeast, east-northeast, and east, 0.67 mile	1.0×10^{-8} 1/m^2	The maximum annual average resident D/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Meat Animal, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average meat animal undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/2.26-Day Decay χ/Q Value @ Nearest Meat Animal, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average meat animal undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Depleted/8.00-Day Decay χ/Q Value @ Nearest Meat Animal, northeast, 0.67 mile	3.0×10^{-6} s/m^3	The maximum annual average meat animal depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average D/Q Value @ Nearest Meat Animal, northeast, east-northeast, and east, 0.67 mile	1.0×10^{-8} 1/m^2	The maximum annual average meat animal D/Q value for use in determining gaseous pathway doses to the maximally exposed individual.

SITE CHARACTERISTIC	VALUE	DEFINITION
Annual Average Undepleted/No Decay χ/Q Value @ Nearest Vegetable Garden, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average vegetable garden undepleted/no decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Undepleted/2.26-Day Decay χ/Q Value @ Nearest Vegetable Garden, northeast, 0.67 mile	3.4×10^{-6} s/m^3	The maximum annual average vegetable garden undepleted/2.26-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average Depleted/8.00-Day Decay χ/Q Value @ Nearest Vegetable Garden, northeast, 0.67 mile	3.0×10^{-6} s/m^3	The maximum annual average vegetable garden depleted/8.00-day decay χ/Q value for use in determining gaseous pathway doses to the maximally exposed individual.
Annual Average D/Q Value @ Nearest Vegetable Garden, northeast, east-northeast, and east, 0.67 mile	1.0×10^{-8} 1/m^2	The maximum annual average vegetable garden D/Q value for use in determining gaseous pathway doses to the maximally exposed individual.

2.4 Hydrologic Engineering

2.4.1 Hydrologic Description

2.4.1.1 Introduction

Attachment 2 of RS-002 [Review Standard] discusses the site characteristics that could affect the safe design and siting of proposed plant or plants. Section 2.4 of the applicant's SSAR describes the hydrological setting and the data used in the applicant's safety conclusions regarding hydrology. The NRC staff's review of the SSAR covers: (1) interface of the plant with the hydrosphere; (2) hydrological causal mechanisms; (3) surface and ground water use; (4) data that forms the basis of the applicant's analysis and conclusions; (5) alternate conceptual models; (6) consideration of other site-related evaluation criteria; and (7) additional information for applications under 10 CFR Part 52.

The VEGP site is located on the southwest side of the Savannah River (SNC 2007). The VEGP site currently hosts two nuclear power plants, VEGP Units 1 and 2. The VEGP application proposed the addition of two new nuclear power reactors at the VEGP site (SNC 2007).

2.4.1.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), addresses the hydrologic characteristics of a proposed site that may affect the consequences of an escape of radioactive material from the facility. Applicants should determine factors important to hydrologic radionuclide transport, described in 10 CFR 100.20(c)(3), by using onsite measurements. 10 CFR 100.20(c) also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

Section 2.4.1 of RS-002 provides the following criteria that was used by the NRC staff to evaluate this SSAR section.

- To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the applicant's SSAR should describe the surface and subsurface hydrologic characteristics of the site and region. This description should be sufficient to assess the acceptability of the site and the potential for those characteristics to influence the design of the SSCs of a nuclear unit(s) that might be constructed on the proposed site.

- Meeting Section 2.4.1 of RS-002 provides reasonable assurance that the hydrologic characteristics of the site and potential hydrologic phenomena will pose no undue risk to the type of facility proposed for the site. Further, it provides reasonable assurance that such a facility will pose no undue risk of radioactive contamination to surface or subsurface water from either normal operations or as the result of a reactor accident.

- To meet the requirements of the hydrologic aspects of 10 CFR Part 52 and 10 CFR Part 100, the applicant's SSAR should form the basis for the hydrologic engineering analysis with respect to subsequent sections of the application for an ESP. Therefore, completeness and clarity are of paramount importance. Maps should be legible and adequate in their coverage to substantiate applicable data. Site topographic maps should be of good quality and of sufficient scale to allow independent analysis of preconstruction drainage patterns. Data on surface water users, location with respect to the site, type of use, and quantity of surface water used are necessary. Inventories of surface water users should be consistent with regional hydrologic inventories reported by applicable Federal and State agencies. The description of the hydrologic characteristics of streams, lakes, and shore regions should correspond to those of the USGS, NOAA, Soil Conservation Service (SCS), USACE, or appropriate State and river basin agencies. Applicants should describe all existing or proposed reservoirs and dams (both upstream and downstream) that could influence conditions at the site. Descriptions may be obtained from reports of USGS, U.S. Bureau of Reclamation (USBR), USACE, and others. Generally, reservoir descriptions of a quality similar to those contained in pertinent datasheets of a standard USACE hydrology design memorandum are adequate. Tabulations of drainage areas, types of structures, appurtenances, ownership, seismic and spillway design criteria, elevation-storage relationships, and short- and long-term storage allocations should be provided.

2.4.1.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the applicant's technical information presented in the SSAR; and (2) NRC staff's technical evaluation of the hydrology near the site, including appropriateness of the data used by the applicant in its SSAR.

2.4.1.3.1 Technical Information Presented by the Applicant

In Section 2.4 of the SSAR, the applicant described the site area and the facilities that currently exist on the proposed site, including the hydrological and geological setting. In addition, the description included the hydrologic characteristics of the Savannah River Basin along with the major dams and multipurpose projects that manage water supply and provide flood control within the basin. The applicant described that the VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia, 26 miles southeast of Augusta, Georgia, and 100 miles north-northwest of Savannah, Georgia (SNC, 2006). The VEGP site is located approximately 150 river miles upstream of the mouth of the Savannah River. Elevations in the Savannah River basin range from sea level at the mouth to 5030 ft mean sea level (MSL) at Little Bald Peak in North Carolina. The Savannah River system drains a total of 10,577 square miles. The contributing watershed area of the Savannah River near the VEGP site is approximately 8304 square miles. There are 14 dams in the Savannah River Basin upstream of the VEGP site (SNC, 2006) owned and operated by the U.S. Army Corps of Engineers (USACE) or one of several power generation companies in Georgia and South Carolina. The entire 312-mile reach of the Savannah River is regulated by three major USACE multipurpose projects. The three reservoirs created by these projects are

Hartwell Lake and Dam, Richard B. Russell Lake and Dam, and J. Strom Thurmond Lake and Dam (also known as Clarks Hill Lake and Dam).

The applicant mentioned that the average daily discharge at the USGS gauge 02197320, Savannah River near Jackson, SC, which is located approximately six river miles upstream of the VEGP site, based on 31 years of data is 8913 cubic feet per second (cfps) (SNC, 2006). Based on the same record, the average discharge at this location varies from 7216 cfps in September to 11,347 cfps in March.

The applicant described that the VEGP site is located on a high bluff on the west bank of the Savannah River and has an area of approximately 3169 acres (SNC, 2006). The grade elevations of proposed Units 3 and 4 will be 220 feet MSL or higher. Approximately 4 miles from the VEGP site, Georgia State Highway 23 runs along a topographic ridgeline. The ridgeline separates drainages that generally flow northeast towards the Savannah River from drainages that generally flow to the southwest.

The applicant also detailed the local site drainage at the VEGP site, the current water uses within the Savannah River Basin, and the proposed water consumption for the two new units. A storm water drainage system exists on the VEGP site. This system was developed during construction of existing Units 1 and 2 and provides drainage away from the site. Surface runoff from the high ground where Units 1 and 2 are located is collected in four major drainage channels that are aligned with access roads and railroad facilities (SNC, 2006). The outfall of the drainage channels is to the north, the south, the east, and the west of the site.

The applicant described that annual peak discharges in the Savannah River at Augusta, Georgia, reported by the USGS based on observed streamflow at gauge 02197000, located approximately 48.7 miles upstream of the VEGP site, are presented in the SSAR (SNC, 2006). The annual peak discharges were estimated by USGS for water years (October 1 of the previous calendar year through September 30 of current year) 1796, 1840, 1852, 1864, 1865, and 1876. The maximum annual peak discharge in the period of record is 350,000 cfps, observed on October 2, 1929. The oldest annual peak discharge, on January 17, 1796, was estimated from reported river stages using slope-conveyance methods. The estimated values of the peak discharge on this date vary from 280,000 cfps for a reported stage of 38 feet to 360,000 cfps for a reported maximum flood stage of 40 feet. Based on the elevation of the USGS gauge 02197000 being 96.58 feet MSL, the maximum historic flood elevation of the Savannah River at Augusta, Georgia is estimated between 134.6 and 136.6 feet MSL (SNC, 2006).

Average daily and annual peak discharge data for nine streamflow gauges maintained by the USGS on the Savannah River were used in preparation of SSAR Sections 2.4.11 and 2.4.2, respectively.

Unregulated annual peak discharge values for the period after 1952 were estimated by modeling using the 1990 reservoir operation rules and the stage-storage-discharge characteristics of the three major USACE projects. Estimates of regulated peak discharge values for the period prior to 1952 were also generated using the same approach. Four USGS topographic quadrangles were used to create a map of the topography at the VEGP site. Cross-section profiles of the Savannah River at several locations were used in the SSAR. Air temperature records from eight NWS meteorological stations were used to analyze historical air temperature variations in the SSAR.

2.4.1.3.2 NRC Staff's Technical Evaluation

The NRC staff reviewed the description of the site region, general location and hydrologic interfaces of the VEGP site, and the description of the local site drainage provided by the applicant. The NRC staff independently obtained descriptions and maps of the general region surrounding the VEGP site. The NRC staff created Figure 2.4.1-1 that shows a map of the region where the VEGP site is located. The estimated distances from the VEGP site to the Georgia cities of Augusta, Waynesboro, and Savannah, are 25.7, 14.8, and 83.2 miles, respectively.

The Savannah River Basin straddles the State boundary between Georgia and South Carolina (Figure 2.4.1-2). The NRC staff created the map shown in Figure 2.4.1-2 by using USGS hydrologic unit codes geographical information system (GIS) coverages from the Natural Resources Conservation Service Geospatial Data Gateway. The Savannah River Basin consists of 9 level 4 and 312 level 6 hydrologic unit codes (Seaber et al., 1987), with a total area of 10,218 square miles. The area of the Savannah River Basin estimated from the GIS coverages is 3.4 percent less (10,218 square miles versus 10,577 square miles) than that reported by SNC (2006). The NRC staff's research indicated that the Nature Conservancy (2007) reports the area of the Savannah River Basin as 10,577 square miles. The contributing drainage area at the streamflow gauge at Hardeeville, South Carolina, about 10 miles above the mouth of the Savannah River, is approximately 10,250 square miles (Cooney et al., 2005). The differences in the reported drainage areas for the Savannah River Basin are minor and are not expected to result in any significant differences in estimation of the probable maximum participation (PMP) or the probable maximum flood (PMF) for the Savannah River Basin. The estimation of the drainage area is an intermediate step in the determination of the probable maximum flood in streams and rivers.

Based on its independent assessment, the NRC staff concluded that the applicant presented sufficient information related to hydrologic description in SSAR Section 2.4.1. Later sections of this SER describe the NRC staff's review of hydrological causal mechanisms, water uses, data, and conceptual models.

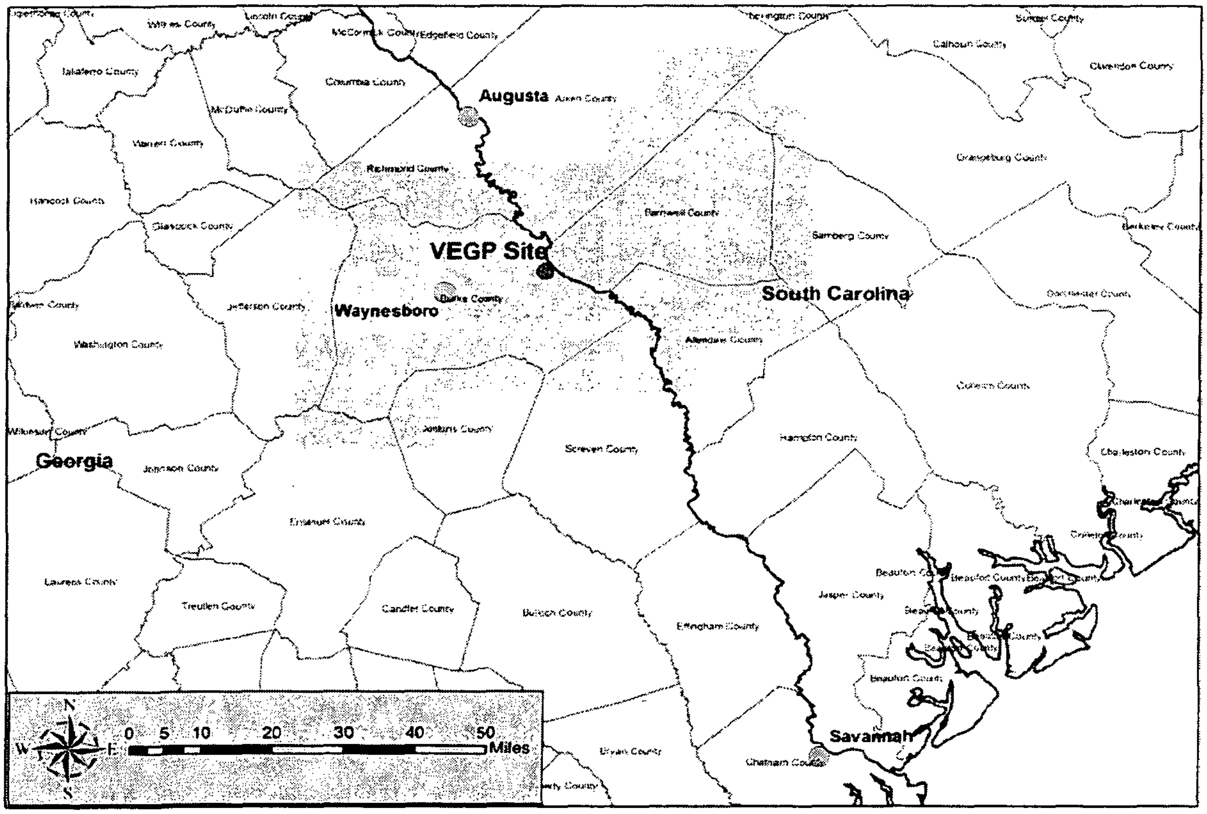


Figure 2.4.1-1 - Location map of the VEGP site

The cities of Augusta, Waynesboro, and Savannah are 25.7, 14.8, and 83.2 miles from the site, respectively. The Savannah River marks the state boundary between South Carolina and Georgia near the VEGP site.

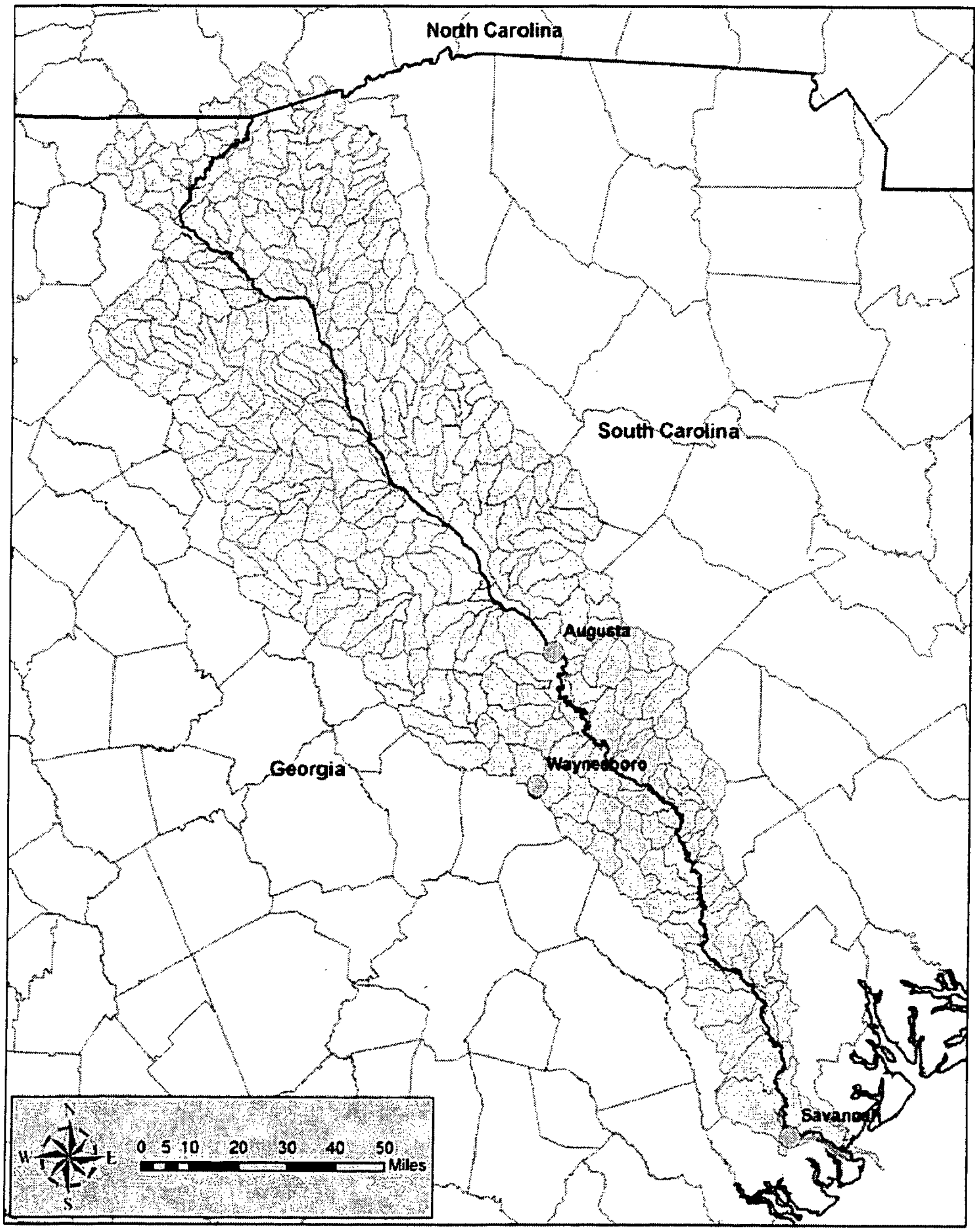


Figure 2.4.1-2 - The Savannah River Basin that straddles the state boundary between Georgia and South Carolina. Portions of the headwaters lie in North Carolina.

2.4.1.4 Conclusion

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the hydrologic description at the proposed site. Section 2.4.1 of RS-002 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the hydrology in the vicinity of the site and site regions, including interface of the plant with the hydrosphere, hydrological causing mechanisms, surface and ground water uses, spatial and temporal data sets, and alternate conceptual models of site hydrology.

Therefore, the NRC staff concludes that the identification and consideration of the hydrological setting of the site set forth above are acceptable and meet the applicable requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). In view of the above, the NRC staff finds the applicant's proposed site characterization related to the hydrological setting for the ESP application to be acceptable.

2.4.2 Floods

Section 2.4.2 of the SSAR identified historical flooding (defined as occurrences of abnormally high water stage or overflow from a stream, floodway, lake, or coastal area) at the proposed site or in the region of the site. The applicant, in Section 2.4.2 of the SSAR, summarized and identified the individual types of flood-producing phenomena, and combinations of flood-producing phenomena, considered in establishing the flood design bases for safety-related plant features. In addition, the SSAR covered the potential effects of local intense precipitation. Although topical information may appear in SSAR Sections 2.4.3 through 2.4.7 and Section 2.4.9, the types of events considered and the controlling event are reviewed in this section of the SER.

The NRC staff reviews the flood history and the potential for flooding for the sources and events listed below. Factors affecting potential runoff (such as urbanization, forest fire, or change in agricultural use), erosion, and sediment deposition are considered in the NRC staff's review. In addition to describing flood history, the applicant also determined the local intense precipitation on the site in order to estimate local flooding. Local intense precipitation is reported as a site characteristic used in site grading design. The NRC staff's review of the SSAR covered" (1) local flooding on the site and drainage design; (2) stream flooding; (3) surges; (4) seiches; (5) tsunami; (6) seismically induced dam failures (or breaches); (7) flooding caused by landslides; (8) effects of ice formation in water bodies; (9) combined events criteria; (10) consideration of other site-related evaluation criteria; and (11) additional information for 10 CFR Part 52 applications.

2.4.2.1 Introduction

The VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia, 26 miles southeast of Augusta, Georgia, and 100 miles north-northwest of Savannah, Georgia (SNC, 2006). The VEGP site is located approximately

150 river miles upstream of the mouth of the Savannah River. Elevations in the Savannah River basin range from sea level at the mouth to 5030 feet MSL at Little Bald Peak in North Carolina. The Savannah River system drains a total of 10,577 square miles. The contributing watershed area of the Savannah River near the VEGP site is approximately 8304 square miles.

There are 14 dams in the Savannah River Basin upstream of the VEGP site (SNC, 2006), which are owned and operated by the USACE or one of several power generation companies in Georgia and South Carolina. The three major USACE multipurpose projects regulate the entire 312-mile reach of the Savannah River. The three reservoirs created by these projects are Hartwell Lake and Dam, Richard B. Russell Lake and Dam, and J. Strom Thurmond Lake and Dam (also known as Clarks Hill Lake and Dam).

The VEGP site is located on a high bluff on the west bank of the Savannah River and has an area of approximately 3169 acres (SNC, 2006). The grade elevations of the proposed Units 3 and 4 will be 220 feet MSL or higher. Approximately 4 miles from the VEGP site, Georgia State Highway 23 runs along a topographic ridgeline. The ridgeline separates drainages that generally flow northeast toward the Savannah River from drainages that generally flow to the southwest.

Potential causes of floods at the VEGP site are local runoff from intense point-rainfall near the site and flooding in the Savannah River caused by precipitation in the river basin or floods from cascading failure of upstream dams on the river. The VEGP site is located approximately 150 river miles inland from the ocean; therefore, flooding caused by surges, seiches, and oceanic tsunamis is unlikely to occur. Section 2.4.7 of the SERs addresses Ice-related events that may result in flooding.

2.4.2.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

Section 2.4.2 of RS-002 provides the review guidance that the NRC staff used to evaluate this SSAR section.

- To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the SSAR should contain a description of the surface and subsurface hydrologic characteristics of the site and region and an analysis of the PMF. This description should be sufficient to assess the acceptability of the site and the potential for those characteristics to influence the design of plant SSCs important to safety. Meeting this guidance provides reasonable assurance that the hydrologic characteristics of the site and potential hydrologic phenomena will pose no undue risk to the type of facility proposed for the site.

As stated in Section 2.4.2 of RS-002, to judge whether the applicant has met the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the NRC uses the following criteria:

- For SSAR Section 2.4.2.1 (Flood History), the NRC staff compares the potential flood sources and flood response characteristics of the region and site identified in its review (as described in the review procedures) to those identified by the applicant. If similar, the NRC staff accepts the applicant's conclusions. If, in the NRC staff's opinion, significant discrepancies exist, the applicant must provide additional data, reestimate the effects on a nuclear unit(s) of a specified type that might be constructed on the proposed site, or revise the applicable flood design bases, as appropriate.

- For SSAR Section 2.4.2.2 (Flood Design Considerations), the applicant's estimate of controlling flood levels is acceptable if it is no more than 5 percent less conservative than the NRC staff's independently determined (or verified) estimate. If the applicant's SSAR estimate is more than 5 percent less conservative, the applicant should fully document and justify its estimate of the controlling level. Alternatively, the applicant may accept the NRC staff's estimate.

- For SSAR Section 2.4.2.3 (Effects of Local Intense Precipitation), the applicant's estimates of the local PMP and the capacity of site drainage facilities (including drainage from the roofs of buildings and site ponding) are acceptable if the estimates are no more than 5 percent less conservative than the corresponding NRC staff assessment. Similarly, conclusions relating to the potential for any adverse effects of blockage of site drainage facilities by debris, ice, or snow should be based upon conservative assumptions of the storm and vegetation conditions likely to exist during storm periods. If a potential hazard does exist (e.g., the elevation of ponding exceeds the elevation of plant access openings), the applicant should document and justify the local PMP basis.

- The NRC staff used the appropriate sections of several documents to determine the acceptability of the applicant's data and analyses in meeting the requirements of 10 CFR Part 52 and 10 CFR Part 100. RG 1.59, Revision 2, "Design Basis Floods for Nuclear Power Plants," issued August 1977, provides guidance for estimating the design-basis flooding considering the worst single phenomenon, as well as combinations of less severe phenomena. The NRC staff used the publications of USGS, NOAA, SCS, USACE, applicable State and river basin authorities, and other similar agencies to verify the applicant's data relating to the hydrologic characteristics and extreme events in the region.

2.4.2.3 *Technical Evaluation*

The technical evaluation consists of: (1) a review of the applicant's technical information presented in the SSAR; and (2) the NRC staff's technical evaluation to determine the potential for site flooding due to various flooding mechanisms.

2.4.2.3.1 Technical Information Presented by the Applicant

Flood History

In Section 2.4.2 of the SSAR, the applicant characterized the historical flooding in streams near the VEGP site using the discharge record at the USGS gauge 02197000, located on the Savannah River at Augusta, Georgia, approximately 48.7 river miles upstream of the site (SNC, 2006). The maximum annual peak flood discharge of 350,000 cfps was reported on October 2, 1929. The discharge on January 17, 1796 was estimated to be between 280,000 cfps for a reported stage of 38 feet (USGS, 2006; gauge datum at 96.58 feet MSL) and 360,000 cfps for a reported stage of 40 feet (USGS, 1990). Based on an elevation of 96.58 feet MSL for the Augusta, Georgia stream gauge datum, the applicant concluded that the historical maximum stage of the Savannah River near the VEGP site is, therefore, between 134.6 and 136.6 feet MSL.

The applicant noted that the average annual peak discharges have declined since the three dams were constructed on the Savannah River (SNC, 2006).

Design-Basis Flood

The applicant selected the design-basis flood from several flooding scenarios including an approximate estimate of the PMF, flooding caused by local intense precipitation on local drainages, and potential dam-failure-generated floods with coincident wind setup and wave runup (SNC, 2006). Flooding from storm surges, seiches, and tsunamis was not considered since the VEGP site is located approximately 150 river miles inland from the Atlantic Coast (SNC, 2006).

The applicant determined that the design-basis flood for the VEGP site is a flood generated by an upstream breach of dams with coincident wind setup and wave runup. SSAR Section 2.4.4 provides a detailed estimation of this flooding event, which was reviewed by the NRC staff in Section 2.4.4 below.

Local Intense Precipitation

The local intense precipitation was estimated from the recommendations of Hydrometeorological Report Nos. 51 and 52 (SNC, 2006). The 6-hour, 10-square miles PMP depth was estimated from Hydrometeorological Report No. 51 for the location of the VEGP site. A multiplier for the VEGP site was estimated from Hydrometeorological Report No. 52 that, when applied to the 6-hour, 10-square miles PMP depth, yielded the 1-hour, 1-square mile PMP depth. Another set of multipliers for the VEGP site was also obtained from Hydrometeorological

Report No. 52. This set of multipliers was applied to the 1-hour, 1-square mile PMP depth to obtain PMP depths at 30, 15, and 5 minutes. The applicant's local intense precipitation is presented in Table 2.4.2-1.

Table 2.4.2-1 - Local Intense Precipitation Depths for Various Durations at the VEGP Site

Duration	Area (square miles)	Multiplier	Applied to	Local Intense Precipitation (inches)
6 hours	10	NA	NA	31.0
1 hour	1	0.620	6-hour, 10-square miles value	19.2
30 minutes	1	0.736	1-hour, 1-square mile value	14.1
15 minutes	1	0.509	1-hour, 1-square mile value	9.8
5 minutes	1	0.323	1-hour, 1-square mile value	6.2

2.4.2.3.2 NRC Staff's Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the data and methods presented in the applicant's SSAR. Sections 2.4.2 through 2.4.7, and 2.4.9 of the SER describe the NRC staff's review of various flooding mechanisms. Based on these reviews, the NRC staff verified that the design-basis flooding scenario at the VEGP site consisted of a domino-type dam-failure scenario-generated flood, and coincident wind setup and wave runup scenario.

The NRC staff independently estimated the local intense precipitation for the VEGP site in order to verify applicant's submission in SSAR Section 2.4.2. Hydrometeorological Report No. 52 recommends that local intense precipitation or point precipitation be estimated as a 1-hour, 1-square mile PMP event. Hydrometeorological Report No. 52 presents a set of maps of estimated PMP depths for several durations ranging from 6 to 72 hours and several areas ranging from 10 to 20,000 square miles. The PMP approach only addressed areas 10 square miles and larger and durations of 6 hours and greater. In order to estimate PMP depths at a point (essentially a 1 square mile area) and for durations of 1 hour and less, Hydrometeorological Report No. 52 recommends the use of a set of multipliers to first estimate the 1-hour, 1-square mile PMP depth from the 6-hour, 10-square miles PMP depth followed by the application of the multipliers to the 1-hour, 1-square mile PMP depth to obtain shorter-duration PMP depths for a 1-square mile area.

The 6-hour, 10-square miles PMP for the VEGP site location was estimated from the PMP depth map corresponding to 6-hour duration and 10-square miles drainage area. Hydrometeorological Report No. 52 maps of multipliers were used to obtain the set of multipliers for the VEGP site. Table 2.4.2-2 shows the NRC staff's estimate of the local intense precipitation.

Table 2.4.2-2 - The NRC Staff-estimated Local Intense Precipitation Depths for Various Durations at the VEGP Site

Duration	Area (square miles)	Multiplier	Applied to	Local Intense Precipitation (inches)
6 hours	10	NA	NA	31.0
1 hour	1	0.621	6-hour, 10-square miles value	19.3
30 minutes	1	0.738	1-hour, 1-square mile value	14.2
15 minutes	1	0.509	1-hour, 1-square mile value	9.8
5 minutes	1	0.323	1-hour, 1-square mile value	6.2

The NRC staff concluded that the local intense precipitation values reported by the applicant in the SSAR are essentially identical (less than 5% different) to those independently estimated by the NRC staff and, thus, are acceptable. The local intense precipitation values reported by the applicant in Table 2.4.2-3 of the SSAR will be used as a site characteristic for the VEGP site.

2.4.2.4 Conclusion

The NRC staff independently confirmed the local intense precipitation values estimated and presented by the applicant in SSAR Section 2.4.2. The local intense precipitation values reported by the applicant in Table 2.4.2-3 of the SSAR will be used as a site characteristic for the VEGP site. As discussed in Section 2.4.4 of this SER, the NRC staff also verified that the controlling flood for the VEGP site consists of a domino-type dam failure scenario-generated flood and coincident wind setup and wave runup scenario.

The applicant has presented and substantiated sufficient information pertaining to the local intense precipitation, flooding causal mechanisms, and the controlling flooding mechanism at the proposed site. RS-002, Section 2.4.2 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the local intense precipitation, flooding causal mechanisms, and the controlling flooding mechanism in the vicinity of the site and site regions. The applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area, and reasonable combinations of these phenomena in establishing the design-basis information pertaining to the local intense precipitation, flooding causal mechanisms, and the controlling flooding mechanism. The applicant's analysis contained sufficient margin for the limited accuracy, quantity, and period of time in which the historical data has been accumulated. As documented in SERs for previous licensing actions, the NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in these site characteristics. Accordingly, the NRC staff concludes that the use of these methodologies results in site characteristics containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. The site characteristics previously identified are acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the local intense precipitation, flooding causal mechanisms, and the controlling flooding mechanism set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d).

In view of the above, the NRC staff finds the applicant's proposed site characteristics related to the local intense precipitation for inclusion for the ESP application to be acceptable.

2.4.3 Probable Maximum Flood (PMF) On Streams And Rivers

In this section of the SSAR, the applicant developed the hydrometeorological design basis to determine the extent of any flood protection required for those SSC necessary to ensure the capability to shut down the reactor and maintain it in a safe shutdown condition. The NRC staff's review of the SSAR covers: (1) design bases for flooding in streams and rivers; (2) design bases for site drainage; (3) consideration of other site-related evaluation criteria; and (4) additional information for 10 CFR Part 52 applications.

2.4.3.1 Introduction

The VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia; 26 miles southeast of Augusta, Georgia; and 100 miles north-northwest of Savannah, Georgia (SNC, 2006). The VEGP site is located approximately 150 river miles upstream of the mouth of the Savannah River. The Elevations in the Savannah River basin range from sea level at the mouth to 5030 feet MSL at Little Bald Peak in North Carolina. The Savannah River system drains a total of 10,577 square miles. The contributing watershed area of the Savannah River near the VEGP site is approximately 8304 square miles.

A PMP in the watershed of the Savannah River can cause a flood near the site. The NRC staff's evaluation in this section consisted of verifying the applicant's approach for estimating the PMF in the Savannah River near the VEGP site and independently estimating the PMF.

2.4.3.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

To evaluate the information provided in SSAR 2.4 per the above acceptance criteria, applicant applied the NRC-endorsed analytical methodologies found in RG 1.59, Revision 2, issued August 1977.

Section 2.4.3 of RS-002 provides the review guidance used by the NRC staff to evaluate this SSAR section.

- To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the SSAR should contain a description of the hydrologic characteristics of the site and region and an analysis of the PMF. This description should be sufficient to assess the acceptability of the site and the potential for those characteristics to influence the design of SSCs important to safety for a nuclear unit(s) of a specified type that might be constructed on the proposed site. Meeting this guidance provides reasonable assurance that any hydrologic phenomena of severity up to and including the PMF will pose no undue risk to the type of facility proposed for the site.

- To judge whether the applicant has met the requirements of the hydrologic aspects of 10 CFR Part 52 and 10 CFR Part 100, the NRC uses specific criteria.

- The PMF, as defined in RG 1.59, has been adopted as one of the conditions to be evaluated in establishing the applicable stream and river flooding design basis referenced in GDC 2. PMF estimates are needed for all adjacent streams or rivers and site drainage (including the consideration of PMP on the roofs of safety-related structures). The criteria for accepting the applicant's PMF-related design basis depend on one of the following three conditions:

    1. The elevation attained by the PMF (with coincident wind waves) establishes a necessary protection level to be used in the design of the facility.

    2. The elevation attained by the PMF (with coincident wind waves) is not controlling; the design-basis flood protection level is established by another flood phenomenon (e.g., the probable maximum hurricane (PMH)).

    3. The site is "dry"; that is, the site is well above the elevation attained by a PMF (with coincident wind waves).

- When condition (1) is applicable, the NRC staff will assess the flood level. The NRC staff may perform this assessment independently from basic data, by detailed review and checking of the applicant's analyses, or by comparison with estimates made by others that have been reviewed in detail. The applicant's estimates of the PMF level and the coincident wave action are acceptable if the estimates are no more than 5 percent less conservative than the NRC staff estimates. If the applicant's estimates of discharge are more than 5 percent less conservative than the NRC staff's, the applicant should fully document and justify its estimates or accept the NRC staff estimates.

- When condition (2) or (3) applies, the NRC staff analyses may be less rigorous. For condition (2), acceptance is based on the protection level estimated for another flood-producing phenomenon exceeding the NRC staff estimate of PMF water levels. For condition (3), the site grade should be well above the NRC staff assessment of PMF water levels. The evaluation of the adequacy of the margin (difference in flood and site elevations) is generally a matter of engineering judgment. Such judgment is based on the confidence in the flood-level estimate and the degree of conservatism in each parameter used in the estimate.

- The NRC staff used the appropriate sections of several documents to determine the acceptability of the applicant's data and analyses. RG 1.59 provides guidance for estimating the PMF design basis. Publications by NOAA and USACE may be used to estimate PMF discharge and water level conditions at the site, as well as coincident wind-generated wave activity.

2.4.3.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the applicant's technical information presented in the SSAR; and (2) NRC staff's technical evaluation to determine the potential for site flooding due to PMF.

2.4.3.3.1 Technical Information Presented by the Applicant

The proposed site grade for the new units is 220 feet MSL. The applicant reviewed studies and analysis that were performed for the existing VEGP units to verify that its conclusions are valid for proposed units. The applicant also performed an approximate PMF estimation as described in RG 1.59 to alternatively estimate the maximum flood stage in the Savannah River near the VEGP site.

Previous Studies

For the original VEGP Units 1 and 2, the applicant used two approaches in determining the PMF in the Savannah River near the VEGP site.

- The first approach used PMP values estimated from Hydrometeorological Report Nos. 51 and 52 and routed the PMP using the U.S. Army Corps of Engineers (USACE) HEC-1 Flood Hydrograph Computer Program. The watershed that was upstream of the Thurmond Dam was characterized by NWS-estimated unit hydrographs of 10 subbasins. The applicant used the USACE DAMBRK computer program to model separately the valley storage below the Thurmond Dam. The peak PMF discharge at the VEGP site was reported as 895,000 cfps when ignoring valley storage and as 540,000 cfps when accounting for valley storage. The associated flood water surface elevations were 136 feet MSL and 126 feet MSL, respectively. The flood water surface elevation with coincident wind wave action was reported as 163 feet MSL and 153 feet MSL.

- In the second approach, the USACE DAMBRK computer program was used to route the USACE-derived PMF outflow hydrograph from the Thurmond Dam to the VEGP site and combining the PMF outflow hydrograph with the PMF discharge of the drainage area downstream of this dam. The PMF discharge in the Savannah River near the VEGP site was estimated as 710,000 cfps with a corresponding water surface elevation of 138 feet MSL. The PMF water surface elevation with coincident wind wave action was estimated as 165 feet MSL.

Approximate PMF Estimation

The applicant used the alternative method for estimation of the PMF described in RG 1.59. The PMF values corresponding to 100, 500, 1000, 5000, 10,000, and 20,000 square miles of contributing areas were obtained from PMF isoline maps given in RG 1.59. The applicant estimated a best-fit power curve to this data and used the estimated power curve to predict the PMF in the Savannah River near the VEGP site. The applicant estimated that the PMF at the VEGP site corresponding to a contributing area of 8,304 square miles is 920,000 cfps.

In SSAR Section 2.4.4, the applicant simulated floods caused by dam failure to determine the flood water surface elevation that corresponded to the PMF discharge from a stage-discharge relationship obtained from a steady-state backwater analysis for the Savannah River. The flood water surface elevation corresponding to the peak PMF discharge was 138.8 feet MSL.

As described in SSAR Section 2.4.4, the applicant used a 50 miles per hour windspeed over a fetch of 11 miles to estimate the wind setup and wave runup. The estimated wind setup and wave runup was 11.3 feet. The PMF water surface elevation with coincident wind wave action was estimated as 150.1 feet MSL, 69.9 feet below the proposed site grade. As such, the applicant concluded that the VEGP site is a dry site.

2.4.3.3.2 NRC Staff's Technical Evaluation

NRC staff's technical evaluation consisted of reviewing the data and methods presented in the applicant's SSAR. The NRC staff independently estimated the PMF and performed an assessment of impacts for flooding on the VEGP site.

In order to verify the applicant's submittal related to PMF in the Savannah River near the VEGP site, the NRC staff carried out an independent and conservative estimate of the PMF. The NRC staff first estimated the PMP in the Savannah River Basin, as described in Hydrometeorological Report Nos. 51 and 52. The cumulative PMP depths for 6, 12, 24, 48, and 72 hours were obtained from the PMP maps in Hydrometeorological Report No. 51 for drainage areas of 10, 200, 1000, 5000, 10,000, and 20,000 square miles (Table 2.4.3-1). The NRC staff plotted a set of depth-area-duration curves for the PMP values (Figure 2.4.3-1).

Table 2.4.3-1 - PMP Depths for Various Drainage Areas and Durations near the VEGP Site

Area (square miles)	Duration (hours)				
	6	12	24	48	72
10	31.0	37.0	43.8	48.2	51.0
200	23.0	27.9	35.0	38.0	42.0
1000	16.9	22.5	28.5	33.5	35.2
5000	9.7	14.0	19.3	23.8	27.5
10000	7.4	11.1	15.8	20.0	23.3
20000	5.4	8.8	12.5	16.2	19.2

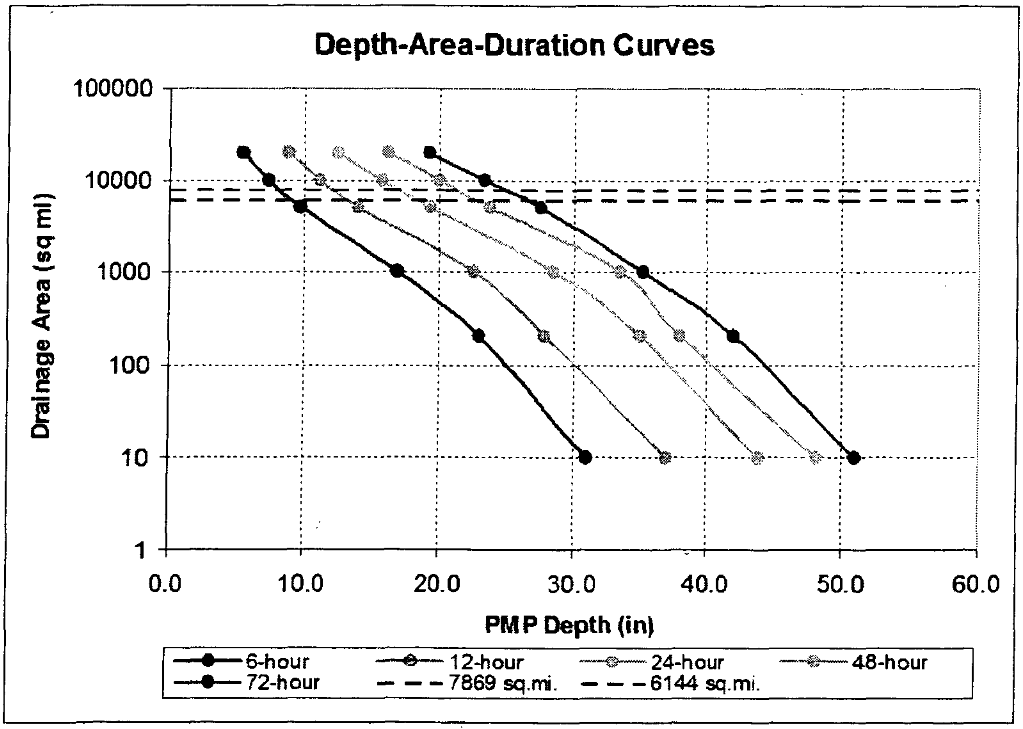


Figure 2.4.3-1 - PMP Depth-Area-Duration Curves Near the VEGP site

The drainage area at the VEGP site was estimated from the hydrologic unit codes that drain areas upstream of the site. The NRC staff estimated the drainage area at the VEGP site to be 7869 square miles. The cumulative PMP values for durations of 6, 12, 24, 48, and 72 hours were then estimated for the corresponding drainage area of the Savannah River near the VEGP site from the depth-area-duration plot (Table 2.4.3-2).

Table 2.4.3-2 - Cumulative PMP for the Savannah River Drainage Area Upstream of the VEGP Site

Area (square miles)	Duration (hours)				
	6	12	24	48	72
7869	8.2	12.1	17.1	21.3	24.9

The incremental PMP depths were calculated from the estimated cumulative PMP depths and the recommended procedure of the American National Standards Institute/American Nuclear Society (ANSI/ANS) Standard 2.8-1992 to estimate the time distribution of the 72-hour PMP storm at 6-hour increments (Table 2.4.3-3).

Table 2.4.3-3 - Incremental 6-hourly PMP Values of the 72-hour PMP Storm for the Savannah River Drainage Near the VEGP Site

6-hr period	Depth (inches)	Group	ANSI/ANS-2.8-1992 Rearrange	PMP Depth (inches)	Time (hour)
1	8.20	1	2.50	1.05	6
2	3.90		3.90	1.05	12
3	2.50		8.20	1.05	18
4	2.50		2.50	1.05	24
5	1.05	2	1.05	2.50	30
6	1.05		1.05	3.90	36
7	1.05		1.05	8.20	42
8	1.05		1.05	2.50	48
9	0.90	3	0.90	0.90	54
10	0.90		0.90	0.90	60
11	0.90		0.90	0.90	66
12	0.90		0.90	0.90	72

In order to estimate the flooding hazard at the VEGP site from a PMF in the Savannah River, the NRC staff adopted a bounding approach. The NRC staff started with a very conservative scenario under which the PMF is obtained by assuming that no losses occur during the PMP event and all of the runoff generated within the drainage area of the Savannah River upstream of the VEGP site is instantaneously delivered to the river near the VEGP site. Under this extremely conservative scenario of PMF generation, the NRC staff estimated the peak PMF discharge in the Savannah River near the VEGP site as 6.94 million cfps by multiplying the drainage area with the precipitation depth during the 6–hour period with maximum estimated PMP precipitation. Then the volume of water thus obtained was converted to an average discharge during that 6-hour period. The stage-discharge relationship estimated during the review of dam failure-generated floods, described in Section 2.4.4 of this report, indicated that the water surface elevation corresponding to a discharge of 6.94 million cfps would exceed the site grade. The NRC staff determined that this first PMF estimation approach was unnecessarily conservative. Therefore the NRC staff refined its approach for estimating the PMF in the Savannah River near the VEGP site.

In this new approach, the NRC staff estimated the PMF inflow into the Thurmond Lake and then the routed outflow from the Thurmond Dam to the VEGP site. The NRC staff estimated the PMP storm over the 6144 square miles of contributing area for Thurmond Lake, following the same procedure described above for estimation of the PMP storm for the 7689 square miles contributing area at the VEGP site. The NRC staff estimated the maximum depth of PMP for any 6-hour duration in the PMP storm for the contributing area of the Thurmond Lake to be 8.9 inches. In addition, the NRC staff estimated the corresponding maximum PMF inflow into Thurmond Lake assuming no losses and instantaneous translation as 5.9 million cfps. The NRC staff postulated that this inflow will then be released from the Thurmond Dam and flow downstream to the VEGP site. In Section 2.4.4, the NRC staff computed the flood from the cascading failure of the Russell Dam located upstream of the Thurmond Dam followed by the failure of the Thurmond Dam itself. The inflow into the Thurmond Lake due to the upstream failure of the Russell Dam was 6.5 million cfps. The NRC staff estimated the corresponding peak discharge as 2.5 million cfps and the corresponding water surface elevation as 170.1 feet MSL in the Savannah River near the VEGP site after being attenuated along the 70-mile river reach between the Thurmond Dam and the site. The PMF generated by a PMP in the drainage area of the Thurmond Lake would produce an inflow (5.9 million cubic feet per second) less severe than that generated by the postulated failure of the Russell Dam upstream of the Thurmond Lake (6.5 million cfps). Therefore, the NRC staff concluded that the PMF inflow into the Thurmond Lake is bounded by inflow into the Thurmond Lake caused by the postulated breach of the Russell Dam.

The NRC staff postulated that the outflow from the Thurmond Dam would combine with the flood response from the contributing area downstream of the dam and upstream of the VEGP site during the PMP event. This contributing area is 1545 square miles in size (7689 square miles contributing area at the VEGP site – 6144 square miles contributing area for the Thurmond Lake). The NRC staff estimated the peak PMF runoff from this contributing area by conservatively assuming that no losses occur during the PMP event, that the runoff generated anywhere in this area is instantaneously translated to the VEGP site, and that the timing of the peak flow from this area coincides with that of the peak flow of the discharge from the Thurmond Lake routed to the VEGP site. The NRC staff estimated the peak discharge from the 1545 square miles contributing area downstream of the Thurmond dam as approximately 1.4 million cfps (8.2 inches of excess rainfall over 1545 square miles of drainage area converted to average discharge over a duration of six hours).

The NRC staff conservatively estimated the combined peak discharge in the Savannah River near the VEGP site by adding the bounding peak discharge of 2.5 million cfps near the VEGP site to the peak PMF discharge of 1.4 million cfps from the 1545 square miles of contributing area downstream of the Thurmond Dam and upstream of the VEGP site. The bounding peak PMF discharge in the Savannah River near the VEGP site is thus estimated as 3.9 million cfps. This peak discharge is less than the 5.9 million cfps needed to raise the stillwater elevation in the Savannah River to inundate the proposed site grade of 220 feet MSL.

The NRC staff estimated the maximum wind wave runup at the VEGP site corresponding to an ANSI/ANS-2.8-1992-recommended windspeed of 50 miles per hour and a maximum fetch of 11 miles, as approximately 19 feet (see Section 2.4.4 of this SER). The NRC staff also estimated the stillwater elevation corresponding to a discharge of 3.9 million cfps in the

Savannah River near the VEGP site using the stage-discharge function estimated in Section 2.4.4 of this SER. The NRC staff-estimated stillwater elevation corresponding to a discharge of 3.9 million cfps was 194.8 feet MSL. The bounding maximum water surface elevation accounting for wind wave action was, therefore, 213.8 feet MSL (194.8 feet MSL + 19 feet). The staff emphasizes that this NRC-estimated bounding value is very conservative (beyond any scenario that would be plausibly expected), and the staff does consider the applicant's model and calculated PMF value to be acceptable. The NRC staff concluded, therefore, that the VEGP site will remain dry during a bounding PMF event in the Savannah River watershed. This conclusion meets the criterion (3) described above in Section 2.4.3.2.

2.4.3.4 Conclusion

The VEGP site is a dry site with respect to floods in rivers and streams. All safety-related SSC will be placed above the highest flood water surface elevation.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the PMF on streams and rivers at the proposed site. RS-002, Section 2.4.3 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the PMF on streams and rivers. Furthermore, the applicant considered local flooding of the site drainage under local intense precipitation in establishing design-basis information pertaining to flooding, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the probable maximum floods on streams and rivers set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d).

In view of the above, the NRC staff finds the applicant's analysis related to the PMF on streams and rivers for the ESP application to be acceptable.

2.4.4 Potential Dam Failures

In this section of the site SSAR (SSAR), the hydrological design basis is developed to ensure that any potential hazard to the safety-related facilities resulting from the failure of onsite, upstream, and downstream water control structures are considered in plant design. The NRC staff's review of the SSAR covers: flood waves from severe breaching of an upstream dam; domino-type or cascading dam failures; dynamic effects of dam-failure induced flood waves on structures; loss of water supply at the plant due to failure of a downstream dam; effects of sediment deposition and erosion; failure of onsite water control or storage structures; potential effects of seismic and non-seismic information on the postulated design bases and how they relate to dam failures in the vicinity of the site and the site region; and additional information for 10 CFR Part 52 applications.

2.4.4.1 Introduction

The VEGP Site is located at Savannah River mile 150.9, and three large dams lie upstream of the site. Hartwell Dam, located 138 miles upstream of the VEGP site; Richard B. Russell Dam, located 108 miles upstream of the site; and J. Strom Thurmond Dam, located 71 miles upstream of the VEGP site, respectively (USACE 1996). Floods initiated by a domino-type failure of these upstream dams were found to produce a peak discharge and peak stage at the site that was larger than flood waves discussed in Section 2.4.3 of this SER (i.e., waves induced by rainfall events alone).

2.4.4.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).
- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).
- 10 CFR 100.23, "Geologic and Seismic Siting Criteria," as it relates to establishing the design-basis flood resulting from seismic dam failure.

To evaluate the information provided in SSAR 2.4 per the above acceptance criteria, the applicant applied the NRC-endorsed analytical methodologies found in the following:

- RG 1.70, Revision 3, issued November 1978
- RG 1.29, "Seismic Design Classification"
- RG 1.59, Revision 2, issued August 1977
- RG 1.102, Revision 1, "Flood Protection for Nuclear Power Plants," issued September 1976.

Section 2.4.4 of RS-002 provides the review guidance that the NRC staff used to evaluate this SSAR section.

- The regulations at 10 CFR Part 52 and 10 CFR Part 100 apply to SSAR Section 2.4.4 because it addresses the site's physical characteristics, including hydrology, considered by the Commission when determining its acceptability to host a nuclear unit(s). To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the SSAR should contain a description of the hydrologic characteristics of the region and an analysis of potential dam failures. The description should be sufficient to assess the acceptability of the site and the potential for those characteristics to influence the design of SSCs important to safety. Meeting this criterion provides reasonable assurance that the effects of high water levels resulting from the failure of upstream dams, as well as those of low water levels resulting from the failure of a downstream dam, will pose no undue risk to the type of facility proposed for the site.

- The regulation at 10 CFR 100.23 requires consideration of geologic and seismic factors in determining site suitability. Specifically, 10 CFR 100.23(c) requires an investigation of the geologic and seismic site characteristics to permit evaluation of seismic effects on the site. Such an evaluation must consider seismically induced floods, including failure of an upstream dam during an earthquake.

- The regulation at 10 CFR 100.23 applies to SSAR Section 2.4.4 because it requires investigation of seismic effects on the site. Such effects include seismically induced floods or low water levels, which constitute one element in the Commission's consideration of the suitability of proposed sites for nuclear power plants. RG 1.70 provides more detailed guidance on the investigation of seismically induced floods, including results for seismically induced dam failures and antecedent flood flows coincident with the flood peak. Meeting this guidance provides reasonable assurance that, given the geologic and seismic characteristics of the proposed site, a nuclear unit(s) of a specified type could be constructed and operated on the proposed site without undue risk to the health and safety of the public, with respect to those characteristics.

- To judge whether the applicant has met the requirements of 10 CFR Part 52, 10 CFR Part 100, and 10 CFR 100.23 as they relate to dam failures, the NRC uses the following criteria:

    - The NRC staff will review the applicant's analyses and independently assess the coincident river flows at the site and at the dams being analyzed. ANSI/ANS-2.8-1992 provides guidance on acceptable river flow conditions to be assumed coincident with the dam failure event. To be acceptable, the applicant's estimates of the flood discharge resulting from the coincident events (which may include landslide-induced failures) should be no more than 5 percent less conservative than the NRC staff estimates. If the applicant's estimates differ by more than 5 percent, the applicant should fully document and justify its estimates or accept the NRC staff estimates.

    - The applicant should identify the location of dams and potentially likely or severe modes of failure, as well as dams or embankments built to impound water for a nuclear unit(s) that might be constructed on the proposed site. The applicant should discuss the potential for multiple, seismically induced dam failures and the domino failure of a series of dams. Approved USACE and Tennessee Valley Authority models should be used to predict the downstream water levels resulting from a dam breach. First-time use of other models will necessitate complete model description and documentation. The NRC staff will review the model theory, available verification, and application to determine the acceptability of the model and subsequent analyses. For cases that assume something other than instantaneous failure, the conservatism of the rate of failure and shape of the breach should be well documented. The applicant should present a determination of the peak flow rate and water level at the site for the worst possible combination of dam failures, a summary analysis that substantiates the condition as the critical permutation, and a description of and the bases for all coefficients and methods used. In addition, the effects of other concurrent events on plant safety, such as blockage of the river and waterborne missiles, should be considered.

- The effects of coincident and antecedent flood flows (or low flows for downstream structures) on initial pool levels should be considered. Depending upon estimated failure modes and the elevation difference between plant grade and normal river levels, it may be acceptable to use conservative, simplified procedures to estimate flood levels at the site. For cases in which calculated flood levels employing simplified methods are at or above plant grade and use assumptions which cannot be demonstrated as conservative, it will be necessary to use unsteady flow methods to develop flood levels at the site. The methods described in RS-002 (ADAMS Accession No. ML040700094), are acceptable to the NRC staff; however, other criteria could be acceptable with proper documentation and justification. Applications should summarize the computations, coefficients, and methods used to establish the water level at the site for the most critical dam failures. Coincident wind-generated wave activity should be considered in a manner similar to that discussed in Section 2.4.3 of RS-002.

2.4.4.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the information provided by the applicant; and (2) the NRC staff's technical evaluation to determine the potential for site flooding resulting from dam failure.

2.4.4.3.1 Technical Information Presented by the Applicant

In the SSAR, the applicant presented the potential for a domino-type failure of Russell and Thurmond dams to induce flooding at the VEGP site. The applicant performed the calculation using the USACE developed Hydrologic Engineering Center River Analysis System (HEC-RAS) numerical model (2005a). The NRC staff obtained the related input files though a RAI 2.4.1-1 (Enclosure Attachment 2). The applicant's simulation conservatively estimated the volume of the dams upstream of Russell Reservoir, and placed the entire flood volume of these dams in Russell Reservoir at the start of the simulation.

The applicant stated in the SSAR that Russell Dam was breached by overtopping in the HEC-RAS model. After investigating the applicant's model input files, the NRC staff determined that the dam was actually breached by a piping-type failure placed midway up the dam (elevation 420 feet MSL). The dam was assumed to breach 2 hours after the start of the simulation.

The SSAR describes how the applicant chose its breach parameters, and how the selection process applied references from the relevant technical literature. The applicant selected methods that were described in the US Bureau of Reclamation (USBR), Department of Interior (1998) Predication of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment, Dam Safety Office, Water Resources Research Laboratory. These USBR methods are accepted current engineering practices. Breaches of both dams extend the full height of the each dam, and the HEC-RAS model defined them using three parameters: bottom width of the breach, left and right side slope, and breach formation time. For the Russell Dam, the bottom width was 750 feet, the side slopes were 2, and the breach time was 1.0 hour. For the Thurmond Dam, the bottom width was 755 feet, the side slopes were 2, and the breach time was 1.0 hour.

The SSAR states that the applicant assigned the initial water surface elevation in Thurmond Reservoir to be 344.7 feet MSL. After reviewing the applicant's HEC-RAS input files, the NRC staff determined that the actual initial elevation assumed in the model analysis was 342.1 feet MSL. The applicant correctly described elevation 342.1 feet MSL to be the Standard Project Flood (SPF) elevation for Thurmond Reservoir (USACE 1996).

The applicant's computed results for the unsteady dam beach and routing analysis was a peak water surface elevation of 166.8 feet MSL at the VEGP site. The computed peak flow at the VEGP site was approximately 2.3 million cfps. The applicant also computed the wave runup due to the maximum wave height. Based on ANS/ANSI 2.8 (1992), a 50 miles per hour wind was applied to the longest fetch (11.1 miles) during passage of the flood wave. The resulting maximum wave height was 7.5 feet, with a corresponding maximum runup height of 11.3 feet. After combining the runup height and the peak flood stage, the applicant computed the maximum flood level at the VEGP site as 178.1 feet MSL. This elevation is 41.9 feet below site grade.

2.4.4.3.2 NRC Staff's Technical Evaluation

NRC staff independently reviewed the applicant's estimate of the flood water height at the VEGP site resulting from a domino-type failure of upstream dams. This evaluation consisted of a steady flow analysis, used to compute the Savannah River discharge necessary for the water surface elevation at the site to reach the site grade, and (b) an unsteady flow analysis, used to compute the maximum stage and discharge in the Savannah River should an upstream domino-type dam failure occur.

Steady Flow Analysis

The NRC staff performed a steady flow analysis to compute the stage versus discharge rating curve at the VEGP site. The analysis used the current public release of HEC-RAS, version 4.0, which is a numerical model developed by the USACE HEC (HEC-RAS, 2006).

In response to RAI 2.4.1-1, the applicant provided electronically the initial geometric description of Russell and Thurmond dams and the Savannah River cross-sections between river miles 259.2 and 99.4. The applicant stated in SSAR Section 2.4.4.2 that these data were supplied in HEC-RAS format directly from the USACE, Savannah River District. The NRC staff's analysis utilized the latest public release of HEC-RAS, a numerical model developed by the HEC, USACE (HEC-RAS 2006). The NRC staff independently confirmed the geometric description of the dams and cross-sections using USACE (1996) and a 30-meter digital elevation model (DEM) data from the USGS.

The applicant-developed HEC-RAS model was modified by the NRC staff to remove cross-sections and reservoirs upstream of Thurmond Dam tailrace for the steady-state flow analysis. The NRC staff then applied a series of constant flow upstream boundary conditions ranging between 3,800 and 6,400,000 cfps to compute the rating curve for the Savannah River adjacent to the site. Based on this rating curve, the river discharge at the site necessary for the static water surface elevation to reach elevation 220 feet MSL is approximately 5.9 million cfps. This discharge is greater than 2.5 times the peak unsteady-flow discharge computed by the applicant as passing at the VEGP site during the dam break analysis. However, as discussed below, the discharge conservatively estimated by the NRC staff, using the unsteady flow analysis, did not exceed 5.9 million cfps.

Unsteady Flow Analysis

The NRC staff performed an unsteady flow analysis to examine the sensitivity of the applicant's model parameters. Using the model input files provided by the applicant, this analysis used a bounding assumption to simplify the distribution of impounded water in the Savannah River basin upstream of Thurmond Dam. This assumption assigned, as an initial condition of the model, the volume of water impounded in Russell Reservoir to be equal to the maximum volume of water impounded by all dam upstream, including Russell Dam. In other words, the initial Russell Reservoir volume assigned by the applicant, and used by the NRC staff in the unsteady-flow analysis, was 8,022,500 acre-ft. As shown in Table 2.4.4.1, this initial impounded volume was greater than the cumulative impounded volume of all reservoirs in the Savannah River watershed upstream of Russell Dam.

The NRC staff's analysis was similar to the applicant's in that Russell Dam was assumed to breach early in the simulation, followed by an overtopping breach of Thurmond Dam downstream. Both the applicant's and the NRC staff's analyses excluded all bridges and dams downstream of Thurmond Dam, which could constrict the flow of the flood wave and hence attenuate the flood at the VEGP site. The NRC staff assumed that the initial water surface elevation in Thurmond Reservoir was at the SPF level (elevation of 342.1 feet). The initial Savannah River discharge passing through Thurmond Dam before the breach and downstream, including at the VEGP Site, was 560,000 cfps. This discharge represents the SPF maximum estimated outflow at Thurmond Dam (USACE 1996).

Table 2.4.4.1 - Storage Volumes of Reservoirs Upstream of Russell Dam

Dam	River System	River Mile above Savannah River Mouth (1)	Maximum Storage (acre-feet) (2)
Bad Creek	Keowee	368.6	33,892
Jocassee	Keowee	366.5	1,287,788
Keowee	Keowee	351.5	955,586
Burton	Tallulah	381.4	108,000
Nacoochee	Tallulah	377.1	8,100
Mathis-Terrora	Tallulah	362.8	31,000
Tallulah Falls	Tallulah	359.9	2,400
Tugaloo	Tugaloo	358.1	42,200
Yonah	Tugaloo	354.9	11,700
Hartwell	Savannah	288.9	3,438,700
Russell	Savannah	259.1	1,488,166
Total			7,407,532

(1) From USACE (1996)
(2) From NID (2007)

The Russell Dam breach simulated by the applicant extended from the thalweg (elevation 345 feet) and to the top of the dam. The final bottom width of the breach was 750 feet, and the breach side slope was 2, resulting in a top width of 1350 feet. These breach parameters are reasonable, and fall within the range suggested by USBR (1998). However, to test the sensitivity of the model to these selected values, the NRC staff increased the total breach area by 50 percent (a more conservative assumption). Specifically, the breach bottom width was increased to 975 feet, the side slope was increased to 4, and the top width was increased to 2175 feet. The impact of this 50 percent increase in total breach area was to increase the peak discharge from Russell Dam, from 4.5 million cfps to 6.5 million cfps (approximately 45 percent increase in peak discharge).

The Thurmond Dam breach occurred approximately 2.5 hours after the Russell Dam breach, when the water surface elevation exceeded the top of the dam by 0.1 feet (i.e., elevation 351.1 feet). The applicant's Russell Dam breach parameters were that the final dam breach extended from the top to the bottom (elevation 200 feet) of the dam, with a bottom width of 755 feet, top width of 1359 feet, and side slopes of 2. These breach parameters are reasonable, and fall within the range suggested by USBR (1998). However, to test the sensitivity of the model to these selected values, the NRC staff increased the breach area by 50 percent (a more conservative assumption). NRC staff assigned the breach bottom width to be 981.5 feet, top width of 2189.5 feet, and side slopes of 4. The impact of this 50 percent increase in breach area was to increase the peak discharge issuing from Thurmond Dam. Under this scenario, with both Russell and Thurmond dam breach areas increased by 50 percent, the increase in peak Thurmond Dam discharge was from 5.5 million cfps to

7.8 million cfps (approximately 41 percent increase). The peak water surface elevation at Thurmond Dam also increased from 352.4 feet to 353.0 feet.

After the peak flood wave passed Thurmond Dam, the peak was attenuated because of the large overbank areas between Thurmond Dam and the VEGP site. Much of the overbank lengths in this region are very broad, with some overbank areas extending laterally from the river for more than 5 miles.

The NRC staff's evaluation mentioned above assumes that the time for the full breach to develop was 1.0 hour. As described in USBR (1998), the breach formation time could take anywhere from 0.1 to 1.0 hour for engineered, compacted earth dams, using the 1987 Engineering Guidelines for the Evaluation of Hydropower Projects, FERC 0119-1, Office of Hydropower Licensing, Federal Energy Regulatory Commission (FERC) method. The sensitivity of the HEC-RAS model to this parameter was tested by decreasing the parameter to 0.1 hour. The simulation results show that the Russell Dam discharge increased to 6.7 million cfps. However, the overtopping breach at Thurmond Dam did not increase with the decrease in breach formation time. Maximum breach discharge is a function of maximum water surface elevation at the dam, and, due to the rapidity of the breach, the maximum stage at the dam was lowered by 2.4 feet (350.6 feet versus 353.0 feet). As expected, the maximum stage adjacent to the VEGP site was also lower with the 0.1 hour (169.9 feet) versus the 1.0 hour breach formation time. Therefore, the 1.0 hour breach formation time parameter was used for the NRC staff's final analysis.

The NRC staff computed the peak discharge at the VEGP site, after it was attenuated along the 70 miles between Thurmond Dam and the site, with approximately 2.5 million cfps. The hydrograph of water surface elevation in the Savannah River near the VEGP site is shown in Figure 2.4.4-1 of the SER. The applicant computed the peak static water surface elevation at the VEGP site as 166.8 feet (Southern 2007). The NRC staff's analysis, with a 50 percent increase in breach area, produced a peak water surface elevation of 170.1 feet at the site, an increase in peak flood stage of 3.3 feet.

In order to satisfy the combined effects guidance in ANS/ANSI 2.8 (1992), the maximum wave height and associated maximum wave runup were computed and added to the peak flood wave elevation. The windspeed for the site was assumed to be 50 miles per hour following the guidelines in ANS/ANSI 2.8 (1992). Based on an estimated fetch of 11.2 miles, the maximum wave height was computed to be 9.8 feet using procedures discussed in USACE (2006). In Section 2.4.4 of the SSAR, the applicant stated that the embankment slope near the site will be 2H:1V. Given this slope value and the maximum wave height, the maximum wave runup at the VEGP site was determined to be 19 feet. Combining this value with the peak static water surface elevation determined with the NRC staff's more conservative breach parameters results in a maximum flood elevation at the VEGP site of 189.1 feet MSL. Even with a more conservative estimate of breaching parameters, the peak flood wave is 30.9 feet below the plant grade (elevation 220 feet MSL). Therefore, the NRC staff concludes that the VEGP site will not be affected by the potential failure of dams upstream of the site. The NRC staff did not apply the "no more than 5% less conservative" criterion to determine the agreement between the NRC staff's estimate of the maximum flood discharge and the corresponding water surface elevation and that of the applicant's from dam-break flooding in the Savannah River. The NRC staff only applies this criterion to compare agreement between the results obtained by the applicant and the results from the NRC staff's independent analysis when the complexity and the conservativeness of the two analyses are the same. Since the NRC staff's independent analysis of the dam-break flooding in the Savannah River is a bounding analysis that is more conservative than the analysis performed by the applicant, the NRC staff did not apply the above-mentioned criterion. The NRC staff, based on its independent analysis of dam-break flooding in the Savannah River, determined that the VEGP site would not flood during the postulated dam-break scenario. Thus the NRC staff agrees with the applicant that the VEGP site is "dry."

2.4.4.4 Conclusion

It is possible that dams upstream of the VEGP site could fail and potentially cause a domino-type cascading failure of multiple dams. However, this failure of upstream dams would not affect the VEGP site. The analysis performed by the applicant follows methods accepted in current engineering practice. The NRC staff reviewed these results by first computing the rating curve at the site, and determining that the peak flood wave discharge that was necessary to reach plant grade was more than 2.5 times the peak flood computed by the applicant. The NRC staff then adjusted the breach parameters in the applicant's HEC-RAS model to examine the sensitivity of model results. Although the peak wave could be increased using more conservative values than standard engineering practice, the resulting peak flood wave passing the VEGP site was still below the site grade by more than 30 feet. Therefore, NRC staff concludes the site is dry, and that safe operation and/or shutdown of the plant will not be affected by failure of dams upstream of the site.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the effects of dam failures at the proposed site. RS-002, Section 2.4.4 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the effects of dam failures. Furthermore, the applicant considered dam failures in establishing design-basis information pertaining to flooding and safety-related water supply, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in these site characteristics, as documented in SERs from previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in site characteristics containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the site characteristics identified in this section are acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the dam failures set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). The NRC staff finds the applicant's proposed site characteristics related to the maximum flood elevation, wind run-up, and combined effects maximum flood elevation associated with dam failures for the ESP application to be acceptable.

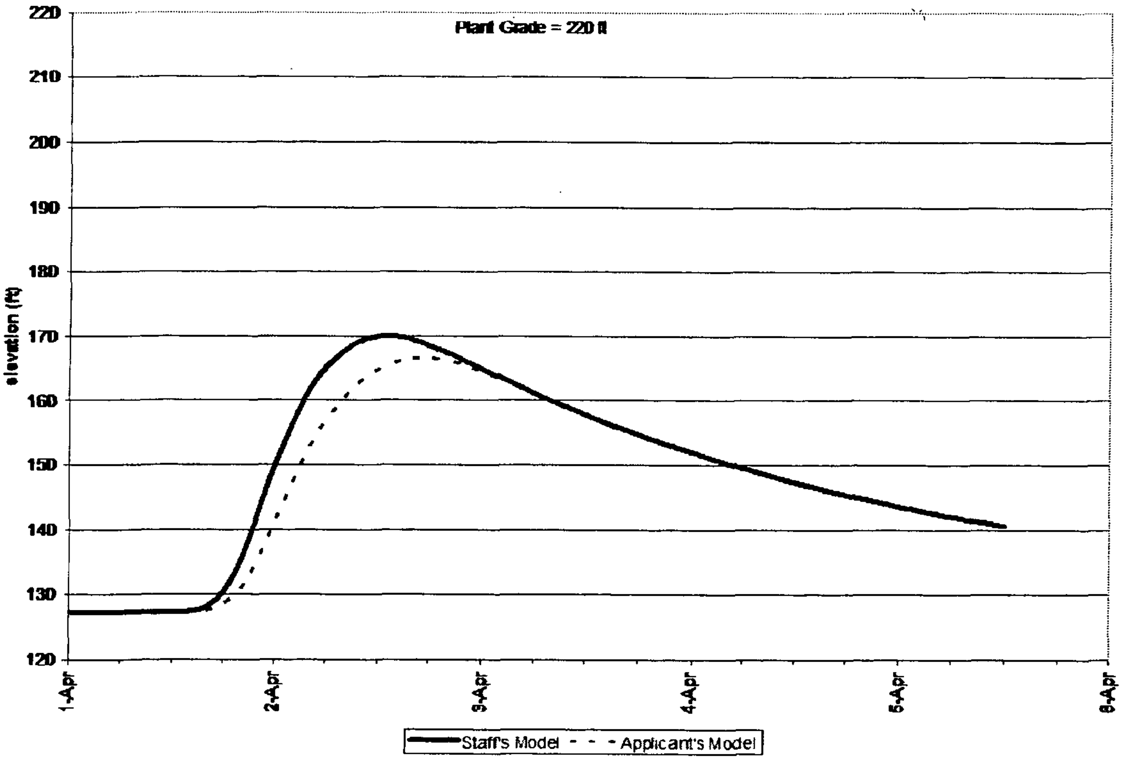


Figure 2.4.4-1 - Stage hydrograph at the VEGP Site

2.4.5 Probable Maximum Surge And Seiche Flooding

In this section of the SSAR, the hydrometeorological design basis is developed to ensure that any potential hazard to the safety-related facilities due to the effects of probable maximum surge and seiche is considered in plant design. The NRC staff's review of the SSAR covers: (1) probable maximum hurricane; (2) probable maximum wind storm; (3) seiche and resonance; (4) wave runup; (5) effects of sediment erosion and deposition; (6) consideration of other site-related evaluation criteria; and (7) additional information for 10 CFR Part 52 applications.

2.4.5.1 Introduction

The VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia, 26 miles southeast of Augusta, Georgia, and 100 miles north-northwest of Savannah, Georgia (SNC, 2006). The VEGP site is located approximately 150 river miles upstream of the mouth of the Savannah River. The grade elevation of the existing VEGP units and the new proposed units is 220 feet MSL.

The Savannah River is the only large body of water that could potentially flood the VEGP site due to surge and seiche effects. Section 2.4.4 discuss the increase in water surface elevation along one bank from the wind blowing across the river's surface.

2.4.5.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).
- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

To evaluate the information provided in SSAR 2.4 per the above acceptance criteria, applicant applied the NRC-endorsed analytical methodologies found in the following:

- RG 1.70, Revision 3, issued November 1978
- RG 1.29
- RG 1.59, Revision 2, issued August 1977

- RG 1.102, Revision 1, issued September 1976

- RG 1.125, Revision 1, "Physical Models for Design and Operation of Hydraulic Structures and Systems for Nuclear Power Plants," issued October 1978

Section 2.4.5 of RS-002 provides the review guidance used by the NRC staff to evaluate this SSAR section.

- To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the applicant's safety assessment should contain a description of the surface and subsurface hydrologic characteristics of the region and an analysis of the potential for flooding caused by surges or seiches. This description should be sufficient to assess the acceptability of the site and the potential for a surge or seiche to influence the design of SSCs important to safety for a nuclear unit(s) of a specified type that might be constructed on the proposed site. Meeting this requirement provides reasonable assurance that the most severe flooding likely to occur as a result of storm surges or seiches will not pose an undue risk to the type of facility proposed for the site.

- If it has been determined that surge and seiche flooding estimates are necessary to identify flood design bases, the NRC will consider the applicant's analysis to be complete and acceptable if it addresses the following areas and if the NRC staff can independently and comparably evaluate them based on the applicant's submission.

- All reasonable combinations of PMH, moving squall line, or other cyclonic windstorm parameters are investigated, and the most critical combination is selected for use in estimating a water level.

- Models used in the evaluation are verified or have been previously approved by the NRC staff.

- Detailed descriptions of bottom profiles are provided (or are readily obtainable) to enable an independent NRC staff estimate of surge levels.

- Detailed descriptions of shoreline protection and safety-related facilities are provided to enable an independent NRC staff estimate of wind-generated waves, runup, and potential erosion and sedimentation.

- Ambient water levels, including tides and sea level anomalies, are estimated using NOAA and USACE publications, as described below.

- Combinations of surge levels and waves that may be critical to the design of a nuclear unit(s) of a specified type that might be constructed on the proposed site are considered, and adequate information is supplied to allow a determination that no adverse combinations have been omitted.

- This section of the SSAR may also state with justification that surge and seiche flooding estimates are not necessary to identify the flood design basis (e.g., the site is not near a large body of water).

- Hydrometeorological estimates and criteria for the development of PMHs for East and Gulf Coast sites, squall lines for the Great Lakes, and severe cyclonic windstorms for all lake sites by USACE, NOAA, and the NRC staff are used for evaluating the conservatism of the applicant's estimates of severe windstorm conditions, as discussed in RG 1.59. USACE and NOAA criteria call for variation of the basic meteorological parameters within given limits to determine the most severe combination that could result. The applicant's hydrometeorological analysis should be based on the most critical combination of these parameters.

- Data from publications by NOAA, USACE, and other sources (such as tide tables, tide records, and historical lake level records) are used to substantiate antecedent water levels. These antecedent water levels should be as high as the 10-percent exceedance monthly spring high tide, plus a sea-level anomaly based on: (1) the maximum difference between recorded and predicted average water levels for durations of 2 weeks or longer for coastal locations; or (2) the 100-year recurrence interval high water for the Great Lakes. In a similar manner, the NRC staff independently analyzes the storm track, wind fields, effective fetch lengths, direction of approach, timing, and frictional surface and bottom effects to ensure that the applicant selected the most critical values. Models used to estimate surge hydrographs that the NRC staff has not previously reviewed and approved are verified by reproducing historical events, with any discrepancies in the model being on the conservative (i.e., high) side.

- The NRC staff uses USACE criteria and methods, as generally summarized in RS-002, as a standard to evaluate the applicant's estimate of coincident wind-generated wave action and runup.

- The NRC staff uses USACE criteria and methods, as generally summarized in RS-002, and other standard techniques to evaluate the potential for oscillation of waves at natural periodicity.

2.4.5.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the technical information provided by the applicant; and (2) NRC staff's technical evaluation to determine the potential for site flooding due to surge and seiche.

2.4.5.3.1 Technical Information Presented by the Applicant

The proposed site grade for the new units is 220 feet MSL. The applicant reported three major hurricanes, defined as those of Category 3 or larger (Saffir/Simpson Hurricane Scale) that have affected the Atlantic coast of Georgia between 1841 and 2004 (SNC, 2006). The most severe observed hurricane with a landfall location within 100 miles of the Savannah River estuary was

Hurricane Hugo, which made landfall near Charleston, South Carolina (SNC, 2006). The applicant reported that Hurricane Hugo produced a 20-ft storm surge in the Cape Romain-Bulls Bay area in South Carolina.

The applicant estimated the probable maximum surge height at the mouth of the Savannah River using the RG 1.59 values of 28.2 feet mean low water (MLW) at Folly Island, South Carolina, and 33.9 feet MLW at Jekyll Island, Georgia, which are located northeast and southwest of the Savannah River estuary, respectively (SNC, 2006). The applicant obtained from ANSI/ANS-2.8 (1992) the 10 percent exceedance high tide at the Savannah River estuary as 9.0 feet MLW with MLW at the entrance to Savannah River being at 1.2 feet below MSL. The applicant estimated the probable maximum surge water surface elevation with a coincident 10 percent exceedance high tide at the mouth of the Savannah River as 32.3 feet MLW or 31.1 feet MSL (SNC, 2006).

The applicant noted that probable maximum surge data from RG 1.59 do not include hurricanes after 1975. Inclusion of the more recent hurricane data in RG 1.59 could have slightly altered the probable maximum surge estimate (SNC, 2006).

The applicant postulated that a probable maximum surge at the mouth of the Savannah River would only have an insignificant effect near the VEGP site because the surge height would dissipate before reaching the VEGP site, which is located approximately 151 river miles inland from the mouth, and the proposed site grade is 220 feet MSL (SNC, 2006).

2.4.5.3.2 NRC Staff's Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the data, the references, and the methods presented in the applicant's SSAR.

The NRC staff reviewed the references provided by the applicant in the SSAR and agreed that three hurricanes exceeding Category 3 have been reported by Blake et al. (2007) on the Georgia coastline within 100 miles of Savannah, Georgia. The NRC staff downloaded historical hurricane track data for the Atlantic basin from the NOAA Coastal Services Center (2007) and created a map of these hurricane tracks in the vicinity of the VEGP site (Figure 2.4.5-1). The NRC staff determined from this map that three Category 4 hurricanes and five Category 3 hurricanes have come within 150 miles and 100 miles of the VEGP site, respectively. One Category 1 and one Category 2 hurricane came within 50 miles of the VEGP site. Within a 25 mile-radius of the Savannah River Estuary (Figure 2.4.5-2), four Category 3 hurricanes have been observed. Within a 50 mile-radius of the Savannah River Estuary, six Category 3 and one Category 4 hurricane have occurred (Figure 2.4.5-2). Based on these historical data, the NRC staff concluded that storm surges caused by severe hurricanes that exceed Category 4 can occur in the vicinity of the Savannah River Estuary.

The NRC staff reviewed the probable maximum surge estimation performed by the applicant. The NRC staff concluded that the applicant appropriately applied the method described in Appendix C of RG 1.59 to the Savannah River estuary location. In addition, the NRC staff finds that the applicant's estimate of total probable maximum surge height of 32.3 feet MLW or 31.1 feet MSL is acceptable.

The NRC staff reviewed the location of the VEGP site in relation to the Savannah River Estuary, and concluded that effects of storm surge and seiche at the site would likely be small. To quantitatively bound these effects, the NRC staff used the HEC-RAS model described in Section 2.4.4 of this SER. The downstream boundary condition, applied at river mile 99.4, of the NRC staff's unsteady flow analysis was modified to a constant stage height. The selected height for this analysis was elevation 119.7 feet MSL. This elevation is the sum of the peak flood stage at the model's boundary during the dam break simulation (elevation 88.6 feet MSL) and the computed maximum storm surge occurring at the mouth of the Savannah River using RG 1.59 (31.1 feet). This estimate of storm surge at river mile 99.4 does not take into account attenuation of the surge that would occur between the mouth and the model boundary. The peak stage at the site computed during the domino-type failure of the upstream dams using this revised downstream boundary condition was elevation 172.1 feet MSL, which is 47.9 feet below the site grade. Wind blowing along the water surface could increase the water surface elevation along one bank. These effects were computed in Section 2.4.4 to be approximately 19 feet. Combining these effects results in a water surface elevation of 191.1 feet MSL, which is 28.9 feet below the site grade. Therefore, the NRC staff concluded that the probable maximum surge and seiche will not affect the VEGP site.

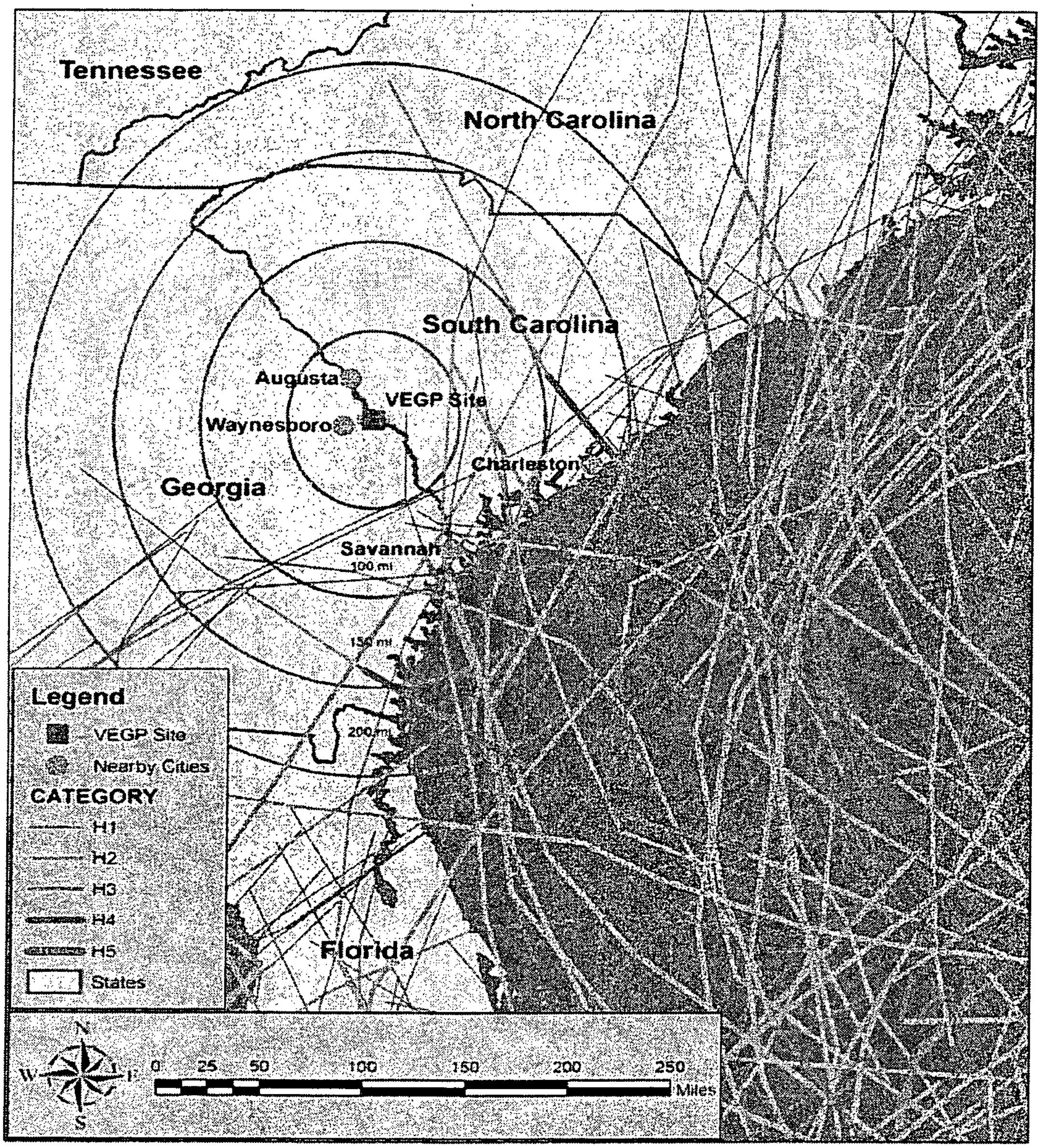


Figure 2.4.5-1 - Hurricane tracks near the VEGP site. The hurricane track data was downloaded from the NOAA Coastal Services Center and all hurricanes (Category H1 through H5) from the dataset were selected to show on the map.

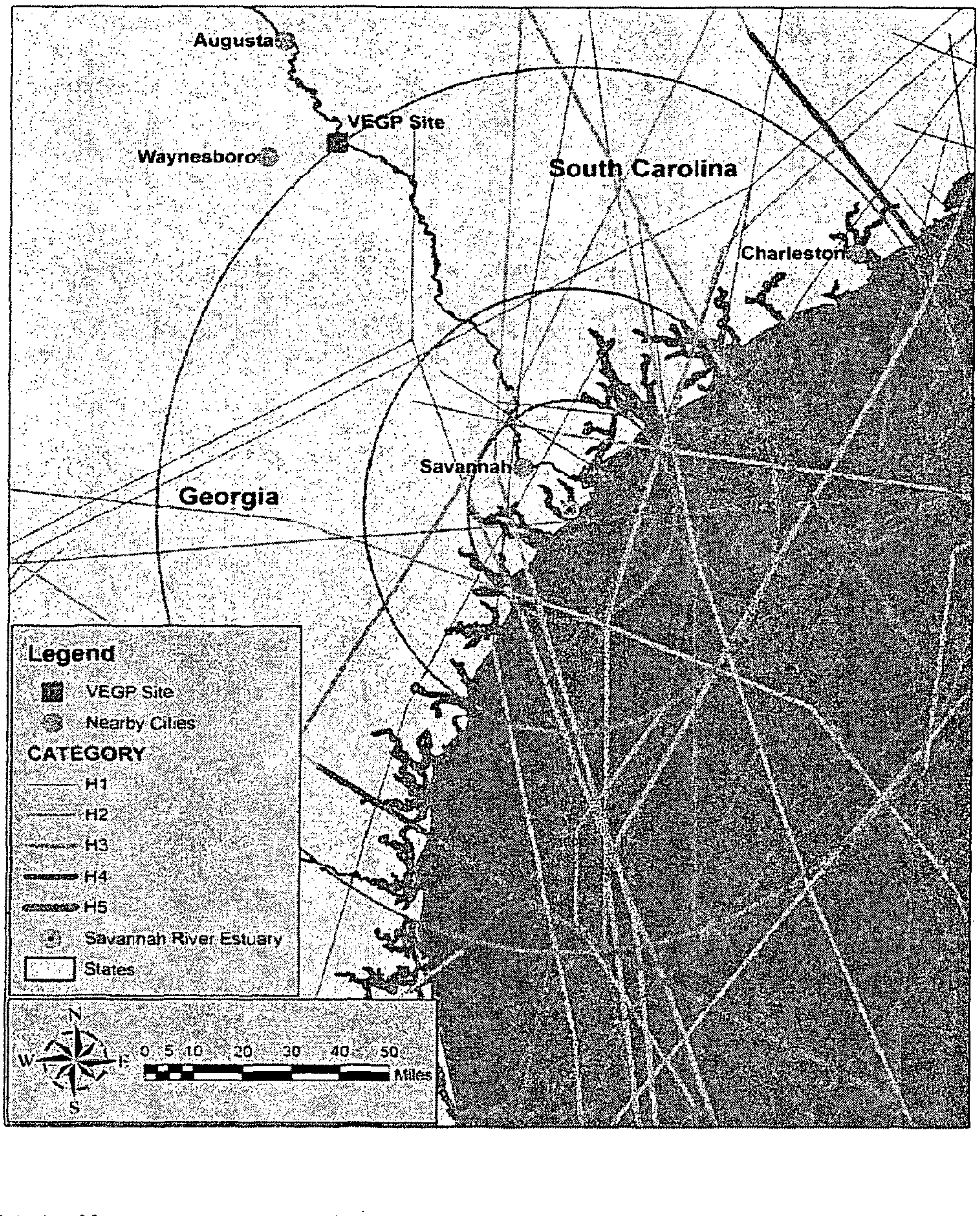


Figure 2.4.5-2 - Hurricane tracks near the Savannah River Estuary. The hurricane track data was downloaded from the NOAA Coastal Services Center and all hurricanes (Category H1 through H5) from the data set were selected to show on the map.

2.4.5.4 Conclusion

A probable maximum surge in the Savannah River Estuary can occur. However, this probable maximum surge does not affect the VEGP site. The VEGP site is also not affected by seiche because the site is located approximately 150 river miles inland from the ocean and there are no large bodies of water in the vicinity. All safety-related SSC will be placed above the highest flood water surface elevation that is controlled by flooding in the Savannah River resulting from cascading upstream dam failures.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the effects of storm surge and seiche at the proposed site. Section 2.4.5 of RS-002 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the effects of storm surge and seiche. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the effects of surge and seiche near the site, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of surge and seiche phenomena set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). The NRC staff finds the applicant's analysis related to surge and seiche for the ESP application to be acceptable.

2.4.6 Probable Maximum Tsunami Hazards

In this section of the SSAR, the geohydrological design basis is developed to ensure that any plant design considers potential hazards to the safety-related facilities due to the effects of probable maximum tsunami. The NRC staff's review of the SSAR covers: (1) historical tsunami data; (2) probable maximum tsunami; (3) tsunami propagation models; (4) wave runup, inundation, and drawdown; (5) hydrostatic and hydrodynamic forces; (6) debris and water-borne projectiles; (7) effects of sediment erosion and deposition; (8) consideration of other site-related evaluation criteria; and (9) additional information for 10 CFR Part 52 applications.

2.4.6.1 Introduction

The VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia; 26 miles southeast of Augusta, Georgia; and 100 miles north-northwest of Savannah, Georgia (SNC, 2006). The VEGP site is located approximately 150 river miles upstream of the mouth of the Savannah River. The grade elevation of the existing VEGP units and the proposed new units is 220 feet MSL.

A probable maximum tsunami can be caused near the mouth of the Savannah River by a tsunamigenic source in the Atlantic Ocean. There are no large inland bodies of water near the VEGP site in which a tsunami may be generated.

2.4.6.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.23, as it relates to investigating the tsunami potential at the site.

To evaluate the information provided in SSAR 2.4 per the above acceptance criteria, applicant applied the NRC-endorsed analytical methodologies found in the following:

- RG 1.70, Revision 3, issued November 1978
- RG 1.29
- RG 1.59, Revision 2, issued August 1977
- RG 1.102, Revision 1, issued September 1976
- RG 1.125, Revision 1, issued October 1978

Section 2.4.6 of RS-002 provides the following review guidance used by the NRC staff to evaluate this SSAR section. The acceptance criteria for this section are based on meeting the requirements of the following regulations:

- The regulations at 10 CFR 52.17(a) and 10 CFR 100.20(c) require that the NRC take into account the site's physical characteristics (including seismology, meteorology, geology, and hydrology) when determining its acceptability to host a nuclear unit(s). The regulations at

10 CFR Part 52 and 10 CFR Part 100 apply to RS-002, Section 2.4.6, because they address the physical characteristics, including hydrology, considered by the Commission when determining the acceptability of the proposed site. To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the SSAR should contain a description of the hydrologic characteristics of the coastal region in which the proposed site is located and an analysis of severe seismically induced waves. The applicant's description should be sufficient to assess the site's acceptability and the potential for a tsunami to influence the design of SSCs important to safety for a nuclear unit(s) of specified type that might be constructed on the proposed site. Meeting this requirement provides reasonable assurance that the most severe flooding likely to occur as a result of a tsunami will pose no undue risk to the type of facility proposed for the site.

- The regulation at 10 CFR 100.23(c) requires that the NRC consider the geologic and seismic factors when determining suitability of the site. Pursuant to 10 CFR 100.23(c), an investigation must be completed to obtain geologic and seismic data necessary for evaluating seismically induced floods and water waves. This regulation also applies to RS-002, Section 2.4.6, because it requires the investigation of distantly and locally generated waves or tsunamis that have affected or could affect a proposed site, including available evidence regarding the runup or drawdown associated with an historic tsunami in the same coastal region and local features of coastal topography that might modify runup or drawdown. RG 1.70 provides more detailed guidance on the investigation of seismically induced flooding.

- Though not required at the ESP stage, the applicant for a COL must demonstrate compliance with general design criteria [GDC] 2 as it relates to designing SSCs important to safety to withstand the effects of a tsunami.

- To judge whether the applicant has met the requirements of 10 CFR Part 52, 10 CFR Part 100, and 10 CFR 100.23 with respect to tsunamis and the analysis thereof, the NRC uses the following criteria:

- If it has been determined that tsunami estimates are necessary to identify flood or low-water design bases, the NRC will consider the applicant's analysis to be complete if it addresses the following areas and if the NRC staff can independently and comparably evaluate them based on the applicant's submission:

  - All potential distant and local tsunami generators, including volcanoes and areas of potential landslides, are investigated, and the most critical ones are selected.

  - Conservative values of seismic characteristics (source dimensions, fault orientation, and vertical displacement) for the tsunami generators selected are used in the analysis.

  - The NRC staff previously approved or verified all models used in the analysis. RG 1.125 provides guidance in the use of physical models of wave protection structures.

  - Bathymetric data are provided (or are readily obtainable).

- Detailed descriptions of shoreline protection and safety-related facilities are provided for wave runup and drawdown estimates. RG 1.102 provides guidance on flood protection for nuclear power plants.

- Ambient water levels, including tides, sea level anomalies, and wind waves, are estimated using NOAA and USACE publications, as described below.

- If the applicant adopts RG 1.59, Position 2, the design basis for tsunami protection of all safety-related facilities identified in RG 1.29 should be shown at the COL stage to be adequate in terms of the time necessary for implementation of any emergency procedures.

- The applicant's estimates of tsunami runup and drawdown levels are acceptable if the estimates are no more than 5 percent less conservative than the NRC staff's estimates. If the applicant's estimates are more than 5 percent less conservative (based on the difference between normal water levels and the maximum runup or drawdown levels) than the NRC staff's, the applicant should fully document and justify its estimates or accept the NRC staff's estimates.

- This section of the SSAR will also be acceptable if it states that the criteria used to determine that tsunami flooding estimates are not necessary to identify the flood design basis (e.g., the site is not near a large body of water).

2.4.6.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the applicant's technical information presented in the SSAR; and (2) NRC staff's technical evaluation to determine the potential for tsunami hazards at the site.

2.4.6.3.1 Technical Information Presented by the Applicant

The applicant stated in SSAR Section 2.4.6 that since the VEGP site is not located on an open ocean coast of a large body of water, a tsunami would not produce maximum water level at the site (SNC, 2006).

The Atlantic Ocean is subject to infrequent seismic and volcanic activities that have resulted in few recorded tsunamis. The most notable Atlantic tsunami was generated by the Great Lisbon Earthquake of 1755. The earthquake generated a tsunami that traveled across the Atlantic and produced waves 10 to 15 feet in height on the Caribbean coasts and computer models suggested a wave height of 10 feet along the east coast of the U.S.

The applicant estimated that effects of any tsunami with similar height approaching the Savannah River estuary would be dissipated before reaching the VEGP site, which is located approximately 151 river miles inland and has a grade elevation of 220 feet MSL (SNC, 2006).

2.4.6.3.2 NRC Staff's Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the data and the references presented in the applicant's SSAR. The NRC staff also carried out a hierarchical review of tsunamis near the VEGP site.

The NRC staff carried out a search of the National Geophysical Data Center (NGDC) Tsunami Runup Database to locate all reported tsunami runups on the U.S. east coast. This search returned reported tsunami runup events in the general region of the Savannah River estuary that are shown on the map below (Figure 2.4.6-1).

The NGDC database did not contain the actual runup heights for several of the runup locations shown on the map (Figure 2.4.6-1). The NGDC database reported an observed runup height less than 1 foot at Charleston, South Carolina, near the Savannah River Estuary resulting from the 1929 Grand Banks submarine landslide-generated tsunami. The NGDC database lists the 1886 earthquake in Charleston, South Carolina as having generated three runup events in Copper River, South Carolina and Jacksonville and Mayport in Florida. Runup heights at the three locations are not available. The event description in the NGDC database lists extensive damage to Charleston, South Carolina by a "mighty tidal," presumably the tsunami wave (NGDC, 2007a).

The NGDC tsunami runup database lists the tsunami caused by the 1755 Great Lisbon Earthquake as resulting in runups on the east coast of the U.S. However, the NGDC database does not include runup heights on the east coast of the U.S. (NGDC, 2007b). A computer modeling of the tsunami wave generated by the 1755 Great Lisbon Earthquake suggested runups of approximately 10 feet on the U.S. east coast (Mader, 2001).

Based on the historical tsunami data near the Savannah River estuary, the NRC staff concluded that the region is subject to tsunamis but there is not enough historical data to ascertain the severity of runups near the Savannah River estuary. In order to determine whether tsunamis pose a hazard to the VEGP site, the NRC staff adopted a bounding approach.

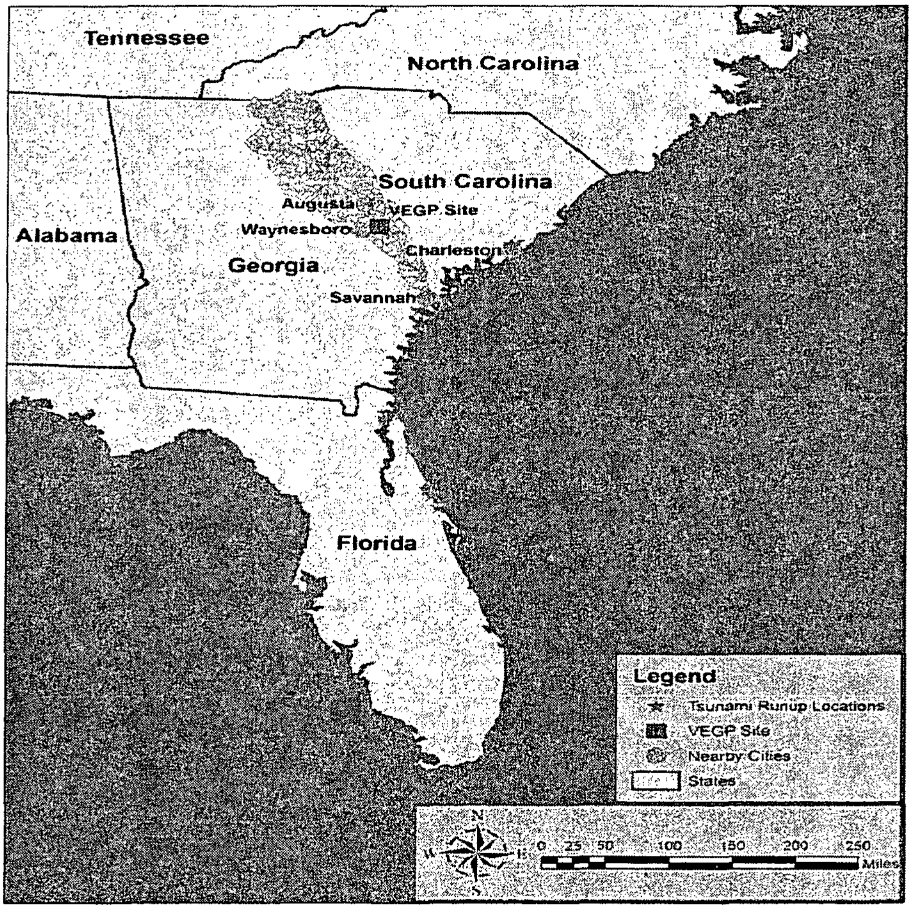


Figure 2.4.6-1 - Locations of Tsunami Runups Reported in the NGDC Tsunami Runup Database near the Savannah River Estuary

The NRC staff evaluated three metrics related to the geographical and topographical location of the site in relation to tsunami wave inundation: (1) distance of the site from the shoreline; (2) upriver distance of the site from the shoreline; and (3) elevation of the site relative to the shoreline. These three metrics specifically address: (1) if the site is located within the horizontal extent of the tsunami wave inundation zone; (2) if the tsunami wave can produce a bore in the Savannah River that may travel upstream to the site; and (3) if the tsunami wave can run up to site grade.

The NRC staff's search of the NGDC tsunami database revealed that the maximum observed horizontal distance of inundation during a tsunami is approximately 3.4 miles. The accounts from the 2004 Sumatra tsunami indicated the maximum extent of horizontal distance could be 5.0 miles from the shoreline on the island of Sumatra, Indonesia. The VEGP site is located more than 100 miles inland from the east coast of the U.S. Since the distance of the site from the shoreline is an order of magnitude more than the maximum observed horizontal inundation distance from a tsunami, the NRC staff concluded that a tsunami arriving at the Savannah River Estuary from the Atlantic Ocean will not inundate the VEGP site.

The NRC staff's search of the NGDC tsunami database revealed that the maximum observed tsunami runup, defined as the highest ground elevation the waters from a tsunami reached, is 1720 feet caused by the giant Lituya Bay subaerial landslide on July 10, 1958. There have been other tsunamis caused by landslides in Lituya Bay on October 27, 1936, on an unspecified day in 1853, and on September 10, 1899, which had reported runups of 490 feet, 394 feet, and 200 feet, respectively. The NGDC tsunami database also reports runups of 820 feet and 738 feet on May 18, 1980 in Spirit Lake located in the Washington State, which was caused by the catastrophic collapse of the north flank of the Mount St. Helens dome and the subsequent pyroclastic flow into the lake. The NGDC tsunami database also contains a few observed runups exceeding 150 feet (Table 2.4.6-1).

The tsunami events that caused runups exceeding 150 feet have properties that are not similar to those at the Savannah River Estuary. The Lituya Bay tsunami events are characterized by subaerial landslides in a very narrow inlet bay flanked by steep and high slopes. The Spirit Lake events were cause by the catastrophic failure of the north flank of the Mount St. Helens volcano. The 1674 tsunami runups on Ambon Island, Indonesia were caused by a near-field tsunamigenic earthquake in the Banda Sea. The events in Japan and Russia and those in Alaska were generated by tsunamigenic sources in the Pacific Ocean. The NRC staff concluded that none of these runup events can be considered representative of tsunamigenic conditions that may affect the Savannah River Estuary. Therefore, the NRC staff carried out a search for tsunami runups with tsunamigenic sources located in the Atlantic Ocean and in the Caribbean Sea, the most likely locations of tsunamigenic sources relevant to the Savannah River Estuary. Table 2.4.6-2 shows the results of this search.

Table 2.4.6-1 - Tsunami Runups Exceeding 150 Feet in the NGDC Tsunami Database

Date			Cause*	Country	Location	Runup (feet)
Year	Month	Day				
1958	7	10	3	USA	Lituya Bay, Alaska	1720
1980	5	18	6	USA	Spirit Lake West, Washington	820
1980	5	18	6	USA	Spirit Lake East, Washington	738
1936	10	27	8	USA	Lituya Bay, Alaska	490
1853	--	--	8	USA	Lituya Bay, Alaska	394
1674	2	17	1	Indonesia	Ceyt, Ambon Island	328
1674	2	17	1	Indonesia	Hila, Ambon Island	328
1674	2	17	1	Indonesia	Hitu Peninsula, Ambon Island	328
1674	2	17	1	Indonesia	Lima, Ambon Island	328
1741	8	29	5	Japan	Sado Island	295
1788	7	21	1	USA	Unga Island, Alaska	289
1788	8	6	1	USA	Unga Island, Alaska	289
1771	4	24	1	Japan	Ishigaki Island	280
1899	9	10	3	USA	Lituya Bay, Alaska	200
1737	10	17	0	Russia	Bering and Commander Islands	197
1771	4	24	1	Japan	Shiraho	197
1771	4	24	1	Japan	Ara	185
1792	5	21	5	Japan	Shimbara	180
1964	3	28	3	USA	Valdez Inlet, Alaska	170
2004	12	26	1	Indonesia	Labuhan, NW Coast of Sumatra	167
1650	9	29	6	Greece	West Coast Patmos	164
2004	12	26	1	Indonesia	Rhiting, Aceh, Sumatra	160
1771	4	24	1	Japan	Nobaruzaki	153

* Cause Codes:

0: Unknown
1: Earthquake
2: Questionable Earthquake
3: Earthquake and Landslide
4: Volcano and Earthquake
5: Volcano, Earthquake, and Landslide
6: Volcano
7: Volcano and Landslide
8: Landslide
9: Meteorological
10: Explosion
11: Astronomical Tide

Table 2.4.6-2 - Runups Exceeding 30 Feet Caused by Tsunamigenic Sources in the Atlantic Ocean and the Caribbean Sea

Date			Cause*	Country	Location	Runup (feet)
Year	Month	Day				
1755	11	1	1	Portugal	Lagos	98
1954	10	--	0	Greenland	Aputiteq Point	60
1755	11	1	1	Portugal	Lisbon	40
1894	11	21	6	Ireland	West Coast	40
1867	11	18	1	Guadeloupe	Deshaies	33
1867	11	18	1	Guadeloupe	Sainte-Rose	33
1900	10	29	1	Venezuela	Puerto Tuy	33

The 1755 Great Lisbon Earthquake, the only known great teletsunami in the Atlantic basin, produced runups of nearly 100 feet in Lagos, Portugal and approximately 40 feet in Lisbon, Portugal. According to the NGDC tsunami database, reported runups at Saint Martin harbor and Samana Bay in the Dominican Republic, both in the Caribbean Sea, were approximately 15 feet and 12 feet, respectively. Computer modeling of the tsunami waves generated by the 1755 Great Lisbon Earthquake, Mader (2001) estimated the runup heights on the east coast of the U.S. to be approximately 10 feet.

Based on the above data, the NRC staff concluded that all known tsunami runups on the Atlantic coast of the U.S. have been at least an order of magnitude less than the elevation of the site grade of the proposed new units at the VEGP site.

A tidal bore is a solitary, non-linear, shallow-water undular wave (Chen, 2003) that is caused by a large tide and typically propagates upstream in a slowly flowing estuary. The tidal bore is hydraulically similar to a traveling hydraulic jump characterized by supercritical flow upstream of the estuary. The formation of supercritical flow in the estuary is a necessary condition for the formation of a tidal bore (Chen, 2003). Supercritical flow is described by the Froude number, the ratio of inertial to gravity forces in open channel flow (Chow, 1959), exceeding 1.0. The Froude number is expressed by

$$Fr = V/(gL)^{1/2} \qquad (1)$$

where V is the velocity of flow, g is the acceleration due to gravity, and L is a characteristic length taken as the hydraulic depth for open channels. The hydraulic depth is defined as the ratio of the cross sectional area of discharge normal to the direction of flow to the top width of the free surface (Chow, 1959). For wide rectangular channels, therefore

$$Fr = V / (gh)^{1/2} \qquad (2)$$

where h is the depth of flow. Therefore, the criteria for supercritical flow in wide, rectangular channels, Fr >= 1.0, can also be stated as

$$V \geq (gh)^{1/2} \tag{3}$$

The right hand side of equation (3) is the celerity, or speed, of a shallow-water wave. Therefore, when the Froude number exceeds 1.0, the velocity of flow exceeds shallow-water wave celerity.

Tidal bores are rare occurrences. Bartsch-Winkler and Lynch (1988) presented a catalog of worldwide occurrences and characteristics of tidal bores. This catalog listed 67 known locations where tidal bores occur. The only documented occurrences of tidal bores in the U.S. are those in the Knik and Turnagain Arms of Cook Inlet in Alaska (Bartsch-Winkler and Lynch, 1988). The NRC staff's additional search did not find any reference to the formation of a tidal bore in the Savannah River Estuary. The NRC staff concluded that a tsunami-induced bore traveling upstream from the mouth of the Savannah River would not occur.

A tsunami that causes a runup near the mouth of the Savannah River would have to reach an elevation of 220 feet MSL more than 100 miles inland in order to inundate the VEGP site. Both these metrics are an order of magnitude greater than the maximum estimated tsunami runup on the Atlantic coast near the site and the maximum reported horizontal extent of tsunami inundation anywhere, respectively. Based on the data pertaining to the geographical and topographical location of the VEGP site as it relates to tsunamis, the NRC staff concluded that a tsunami at the mouth of the Savannah River would not affect the VEGP site, which is located more than 100 miles from the mouth and at a grade elevation of 220 feet MSL.

2.4.6.4 Conclusion

The VEGP site is not affected by probable maximum tsunami. All safety-related SSC will be placed above the highest flood water surface elevation that is controlled by flooding in the Savannah River resulting from to cascading upstream dam failures.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the effects of probable maximum tsunami hazards at the proposed site. RS-002, Section 2.4.6 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating the effects of probable maximum tsunami hazards. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the probable maximum tsunami hazards, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the probable maximum tsunami hazards set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). The

NRC staff finds the applicant's proposed analysis related to probable maximum tsunami hazards for the ESP application to be acceptable.

2.4.7 Ice Effects

This section of the applicant's SSAR develops the hydrometeorological design basis to ensure that ice-induced hazards do not affect safety-related facilities and water supply. The applicant is responsible for providing site characteristics and other hydrometeorological parameters related to ice formation at or near the site to the organization responsible for review of the SSCs to ascertain whether the mechanical or structural design basis for the plant properly considers ice effects on potentially affected SSC. The review covers: (1) historical ice accumulation; (2) high and low water levels; (3) ice sheet formation; (4) ice-induced forces and blockages; (5) consideration of other site-related evaluation criteria; and (6) additional information for 10 CFR Part 52 applications.

2.4.7.1 Introduction

The VEGP site is located on the southeast side of the Savannah River, approximately 15 miles east-northeast of Waynesboro, Georgia, 26 miles southeast of Augusta, Georgia, and 100 miles north-northwest of Savannah, Georgia (SNC 2007). The VEGP site is located approximately 150 river miles upstream of the mouth of the Savannah River. The grade elevation of the existing VEGP units and the new proposed units is 220 feet MSL.

The site may be affected by icing in the Savannah River near the site. There are no large inland bodies of water near the VEGP site and no water reservoirs are proposed for safety-related use.

2.4.7.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

To evaluate the information provided in SSAR 2.4 per the above acceptance criteria, applicant applied the NRC-endorsed analytical methodologies found in the following:

- RG 1.59, Revision 2, issued August 1977.

- The regulations at 10 CFR 52.17(a) and 10 CFR 100.20(c) require that the NRC take into account the site's physical characteristics (including seismology, meteorology, geology, and hydrology) when determining its acceptability for hosting a nuclear power reactor(s). To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the SSAR should contain a description of any icing phenomena with the potential to result in adverse effects to the intake structure or other safety-related facilities for a nuclear unit(s) of a specified type that might be constructed on the proposed site. Applicants should describe ice-related characteristics historically associated with the site and region, and they should perform an analysis to determine the potential for flooding, low water, or ice damage to safety-related SSCs. The analysis should be sufficient to evaluate the site's acceptability and to assess the potential for those characteristics to influence the design of SSCs important to safety for a nuclear unit(s) of a specified type that might be constructed on the proposed site. Meeting this guidance provides reasonable assurance that the effects of potentially severe icing conditions will pose no undue risk to the type of facility proposed for the site.

- Publications by NOAA, USGS, USACE, and other sources are used to identify the history and potential for ice formation in the region. The historical maximum depths of icing should be noted, as well as mass and velocity of any large, floating ice bodies. The phrase, "historical low water ice affected," or similar phrases in streamflow records (USGS and State publications) will alert the reviewer to the potential for ice effects. The following items should be considered and evaluated, if necessary:

  - The regional ice and ice jam formation history should be described to enable an independent determination of the need for including ice effects in the design basis.

  - If the potential for icing is severe, based on regional icing history, it should be shown that water supplies capable of meeting safety-related needs are available from under the ice formations postulated and that safety-related equipment could be protected from icing. If this cannot be shown, it should be demonstrated that alternate sources of water are available that could be protected from freezing and that the alternate source would be capable of meeting safety-related requirements in such situations.

  - If floating ice is prevalent, based on regional icing history, potential impact forces on safety-related intakes should be considered. The structural design basis should include dynamic loading caused by floating ice. (This item will be addressed at the COL or CP stage.)

  -If ice blockage of the river or estuary is possible, it should be demonstrated that the resulting water level in the vicinity of the site has been considered. If this water level would adversely affect the intake structure or other safety-related facilities of a nuclear unit(s) of a specified type that might be constructed on the proposed site, it should be demonstrated that it would not also adversely affect an alternate safety-related water supply.

- The applicant's estimates of potential ice flooding or low flows are acceptable if the estimates are no more than 5 percent less conservative than the NRC staff estimates. If the applicant's estimates are more than 5 percent less conservative than the NRC staff's, the applicant should fully document and justify its estimates or accept the NRC staff estimates.

2.4.7.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the applicant's technical information presented in the SSAR; and (2) NRC staff's technical evaluation to determine the potential for ice-related hazards at the site.

2.4.7.3.1 Technical Information Presented by the Applicant

The applicant used air temperature records from eight locations, including seven cooperative stations, around the VEGP site to analyze historical extreme air temperature variations (SNC 2007). The applicant also used air temperature data from onsite measurements.

The climate at the VEGP site consists of short, mild winters and long, humid summers (SNC 2007). At the Augusta, Georgia station, based on 129 years of records, January is the coldest month with a mean temperature of 46.8 °F. Among the eight stations, the lowest air temperature was -4.0 °F at Aiken, South Carolina in January 1985. During the same period, the air temperature at the VEGP site was -0.1 °F, with air temperatures remaining below freezing (32 °F) for approximately 50 hours (SNC 2007). Onsite measurements from 1984 to 2002 showed that mean daily air temperature remained below freezing for a maximum of three consecutive days (SNC 2007).

Historical water temperature data from five USGS gauging stations located on the Savannah River covering an area that includes the VEGP site showed that the minimum water temperature is observed in the month of February and varies from 39.2 °F and 42.8 °F (SNC 2007).

Based on historical air and water temperature records, the applicant concluded that it is very unlikely that surface or frazil ice formation would occur in the Savannah River in the vicinity of the proposed intake location of the new VEGP units (SNC 2007).

The applicant reported in SSAR Section 2.4.7 that the USACE Ice Jam Database includes no recorded ice jam events in the lower reaches of the Savannah River. The existence of dams and reservoirs on the Savannah River upstream of the VEGP site reduce the possibility of any surface ice or ice floes moving downstream (SNC 2007). Since the water temperature in the lower reach of the Savannah River consistently remains above freezing, the applicant concluded that formation of frazil ice or ice jams is very unlikely at the proposed intake location for the new VEGP units.

The proposed VEGP units would use a closed-cycle cooling system with cooling towers for the circulating water system cooling (SNC 2007). Makeup water for the circulating water system cooling towers will be supplied from the Savannah River using a new intake system comprising of an intake canal and a pump intake structure located upstream of the existing river intake system for VEGP Units 1 and 2 (SNC 2007).

The reactors for the proposed VEGP units will use passive UHS systems that do not require any safety-related water supply (SNC 2007). The proposed reactors would have a non-safety related auxiliary heat sink service water system that will be used for shutdown, normal operations, and anticipated operational events (SNC 2007). The makeup water to the service water system will be supplied from groundwater wells or an onsite water storage tank (SNC 2007). No water will be necessary from the Savannah River or any other open surface water source for the proposed reactors' UHS (SNC 2007). The applicant concluded, therefore, that any ice event in the Savannah River will not have an impact on the safe operation of the proposed units (SNC 2007).

2.4.7.3.2 NRC Staff's Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the data and the references presented in the applicant's SSAR.

The NRC staff carried out a review of historical air temperature data near the VEGP site. The stations used by the NRC staff and their periods of record are shown in Table 2.4.7-1.

Table 2.4.7-1 - Meteorological stations near the VEGP site used by the NRC staff

Name (State)	COOP ID	Start Date	End Date
Augusta Bush Field Airport (Georgia)	090495	03/01/1949	04/30/2007
Louisville 1E (Georgia)	095314	01/01/1893	03/31/2007
Midville Experiment Station (Georgia)	095863	06/01/1957	03/31/2007
Millen 4N (Georgia)	095882	11/01/1891	12/31/1998
Newington (Georgia)	096323	09/01/1956	02/28/2003
Waynesboro 2S (Georgia)	099194	11/01/1893	02/28/2007
Aiken 5SE (South Carolina)	380074	01/01/1893	03/31/2007
Bamberg (South Carolina)	380448	08/01/1951	01/31/2007
Blackville 3W (South Carolina)	380764	06/01/1894	07/31/2002

In reviewing the daily minimum air temperature record at these stations, the NRC staff determined that the lowest daily minimum air temperature, -4 °F, was observed at the Aiken 5SE station on January 21, 1985. The range of the lowest daily minimum air temperatures at all stations was 0 °F to -4 °F. The NRC staff estimated the mean daily minimum air temperature during the winter months, December through March, for all stations (see Table 2.4.7-2). None of these temperatures was below freezing.

Table 2.4.7-2 - Mean Daily Minimum Air Temperatures During the Months of December Through March for All Stations Used in the NRC Staff's Review

Name (State)	Mean Daily Minimum Air Temperature (°F)			
	December	January	February	March
Augusta Bush Field Airport (Georgia)	34.7	33.5	35.8	42.3
Louisville 1E (Georgia)	49.2	49.9	55.7	62.4
Midville Experiment Station (Georgia)	37.1	35.5	38.3	45.2
Millen 4N (Georgia)	38.1	37.6	39.8	45.9
Newington (Georgia)	38.8	36.4	39.4	45.5
Waynesboro 2S (Georgia)	42.3	41.5	45.5	52.5
Aiken 5SE (South Carolina)	39.0	37.8	40.7	47.3
Bamberg (South Carolina)	37.4	35.5	37.9	43.8
Blackville 3W (South Carolina)	52.1	54.4	59.4	67.8

The NRC staff also identified the longest consecutive period during which the mean daily air temperature (estimated as the average of the daily minimum and maximum temperatures) was below freezing at each of the stations (see Table 2.4.7-3). The longest duration, that of nine days, of mean daily air temperature below freezing was observed at the Aiken station from January 13 to January 21, 1893.

According to USACE (2002), frazil ice forms in turbulent, supercooled water that is not covered by an ice layer. The NRC staff identified the maximum number of consecutive days that mean daily air temperature falls below 18 °F for each of the stations (Table 2.4.7-3a). Two consecutive days of mean daily air temperatures below 18 °F were observed twice at Waynesboro 2S and once at Blackville 3W. At all other stations experienced only 1 consecutive day with the mean air temperature below 18 °F.

In response to NRC staff's RAI 2.4.1-1, the applicant provided water temperature data at the Shell Bluff Landing site, which is located approximately 11 river miles upstream of the VEGP site. The NRC staff reviewed water temperature data supplied by the applicant. The period of record for these monthly water temperatures was from January 30, 1973 to August 13, 1996. From these data, the NRC staff computed the following water temperature statistics: the minimum water temperature was 41.0 °F, the average water temperature was 63.4 °F, the median water temperature was 64.4 °F, and the maximum water temperature was 81.0 °F.

Based on its independent review of air temperature data near the VEGP site, the NRC staff concluded that the occurrenceS of air temperatures below freezing at and near the VEGP site are brief and infrequent. Although air temperature could fall below 18 °F in the vicinity of the VEGP site, the duration of such a freezing spell would be unlikely to exceed two days. Since the water temperatures in the Savannah River near the site have never approached freezing (minimum water temperature estimated from 13 years of monthly data was 41.0 °F), the NRC staff concluded that the VEGP site would not support the formation of frazil ice.

Table 2.4.7-3 - Longest Consecutive Period of Mean Daily Air Temperature below Freezing for All Stations Used in the NRC Staff's Review

Name (State)	Longest Consecutive Period of Mean Daily Air Temperature Below Freezing	
	Duration (days)	Dates
Augusta Bush Field Airport (Georgia)	6	01/10/1982 – 01/15/1982, 12/30/2000 – 04/01/2001
Louisville 1E (Georgia)	8	01/14/1893 – 01/21/1893
Midville Experiment Station (Georgia)	4	02/16/1958 – 02/19/1958, 01/08/1970 – 01/11/1970, 12/23/1989 – 12/26/1989
Millen 4N (Georgia)	5	01/13/1912 – 01/17/1912, 01/25/1940 – 01/29/1940
Newington (Georgia)	5	01/16/1977 – 01/20/1977
Waynesboro 2S (Georgia)	6	12/30/1917 – 01/04/1918, 01/11/1982 – 01/16/1982
Aiken 5SE (South Carolina)	9	01/13/1893 – 01/21/1893
Bamberg (South Carolina)	5	02/01/1980 – 02/05/1980, 12/31/2000 – 01/04/2001
Blackville 3W (South Carolina)	5	12/30/1899 – 01/03/1900

Table 2.4.7-3a - Number of Days with Minimum Daily Temperature at or below 18 °F

Name (State)	Longest Consecutive Period of Mean Daily Air Temperature Below 18 °F
Augusta Bush Field Airport (Georgia)	1
Louisville 1E (Georgia)	1
Midville Experiment Station (Georgia)	1
Millen 4N (Georgia)	1
Newington (Georgia)	1
Waynesboro 2S (Georgia)	2
Aiken 5SE (South Carolina)	1
Bamberg (South Carolina)	1
Blackville 3W (South Carolina)	2

The proposed units at the VEGP site have no safety-related water requirement and would not use any safety-related intakes. Consequently, formation of ice sheets, forces induced by ice, and blockages caused by ice are not areas of concern for this review.

The NRC staff searched the USACE Ice Jam Database for ice jam events reported in the states of Georgia, North Carolina, and South Carolina (CRREL, 2007a; 2007b; 2007c). The Ice Jam Database contains no ice jams reported in Georgia and South Carolina (CRREL, 2007d; 2007f). There are two ice jams reported in North Carolina (CRREL 2007e), one on the Neuse River and the other on the Missouri River. Based on these search results, the NRC staff concluded that ice jams in the Savannah River near the VEGP site are not likely.

The NRC staff proposed a site characteristic related to frazil ice that states that hydrometeorologic conditions at the VEGP site do not support formation of frazil ice.

2.4.7.4 Conclusion

Based on its review and independent analysis of data available publicly and those provided by the applicant, the NRC staff concluded that icing in the vicinity of the VEGP site is unlikely. Since the proposed units have no requirement other than initial filling and occasional makeup purposes, for continuous safety-related water supply, no safety-related water reservoirs or canals, intakes, and structures will be used. Therefore, the NRC staff concluded that ice effects will not affect safety of the proposed units.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the identification and evaluation of ice effects at the proposed site. Section 2.4.7 of RS-002 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating ice effects at the site. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the hydrologic interface of the plant with the site, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this site characteristic, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in a site characteristic containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the site characteristic previously identified is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the site characteristic related to ice effects set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). The NRC staff finds the applicant's proposed site characteristic related to ice effects for the ESP application to be acceptable.

2.4.8 Cooling Water Canals and Reservoirs

This section of the applicant's SSAR develops the hydraulic design basis for canal and reservoirs used to transport and impound water supplied to the safety-related structures, systems, and components (SSCs). The NRC staff's review of the SSAR covers (1) hydraulic design bases for protection of structures, (2) hydraulic design bases of canals, (3) hydraulic design bases of reservoirs, (4) consideration of other site-related evaluation criteria, and (5) 10 CFR Part 50, Appendix A, GDC 44, for CP and OL applications, as it relates to providing a UHS for normal operating and accident conditions.

2.4.8.1 *Introduction*

The VEGP site is located on the southwest side of the Savannah River (SNC 2008a). The two proposed plant units will use a closed-cycle cooling system with cooling towers. The Savannah River will provide makeup water for the cooling towers' evaporative and other losses using a new intake system consisting of a 200-foot-long intake canal and an intake structure.

The proposed units at the VEGP site will not rely on external sources of safety-related UHS cooling water. The applicant has not proposed any safety-related cooling water supply canals and reservoirs.

2.4.8.2 *Regulatory Basis*

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in the site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit

- 10 CFR 100.20(c), with respect to the requirement that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit

To satisfy the hydrologic requirements of 10 CFR Part 52 and 10 CFR Part 100, the applicant's SSAR should describe the cooling water canals and reservoirs for a nuclear power plant of the specified type that might be constructed on the proposed site. The analysis related to cooling water canals and reservoirs should be sufficient to evaluate the site's acceptability and to assess the potential for those characteristics to influence the design of SSCs important to safety for a nuclear power plant of the specified type that might be constructed on the proposed site. Meeting this requirement provides reasonable assurance that the capacities of cooling water canals and reservoirs are adequate.

2.4.8.3 *Technical Evaluation*

The technical evaluation consists of (1) a review of the technical information presented in the application, and (2) the NRC staff's technical evaluation to determine the acceptability of the design bases for canals and reservoirs.

2.4.8.3.1 Technical Information Presented by the Applicant

The proposed VEGP units will use a closed-cycle cooling system with cooling towers for condenser heat removal during normal operation (SNC 2008a). To replenish the water losses from evaporation, drift, and blowdown, the Savannah River will supply makeup water at a maximum rate of approximately 57,784 gallons per minute (SNC 2008a). The makeup water intake system for the proposed units will be located upstream of the intake for the existing units (SNC 2008a).

The proposed plants for the new VEGP units use a passive UHS with in-plant storage of safety-related cooling water (SNC 2008a). The proposed plant design does not require an external water-cooled UHS (SNC 2008a). The makeup water intake that will supply water to the condenser heat removal system will not be safety related (SNC 2008a). Because the proposed VEGP units will not rely on the Savannah River for safety-related water supply, low-water conditions in the river will not affect safety-related SSCs (SNC 2008a).

2.4.8.3.2 Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the data and the references presented in the applicant's SSAR in its various revisions. The ESP SER with Open Items was based on SSAR, Revision 2 (SNC 2007), and this final ESP SER is based on SSAR, Revision 4 (SNC 2008a) and Revision 4S-2 (SNC 2008b).

On the basis of its initial review of the information presented in the SSAR, the NRC staff concluded that, as proposed in the application, the new VEGP Units 3 and 4 would not rely on any external water source for safety-related cooling water. The applicant did not propose any safety-related canals or reservoirs as a source for cooling water. However, safety-related water would be needed for initial filling and occasional makeup purposes. In this regard, the applicant did not provide design parameters for these values. This omission was designated Open Item 2.4-1.

The NRC staff identified in Section 2.4.8 of the ESP SER with Open Items a permit condition stating that VEGP Units 3 and 4 will not rely on any external water source for safety-related cooling water other than initial filling and occasional makeup water. This permit condition precluded the use of onsite surface and ground water for safety-related water supply except for initial filling and occasional makeup water.

The NRC staff discussed these issues with the applicant and reviewed the water components of the passive containment cooling system of a nuclear power reactor design that fits within the bounding parameters provided in the proposed permit application. The applicant stated that storage volume for each of the two water tanks would be approximately 800,000 gallons (SNC 2007g). The applicant also stated that the VEGP Units 3 and 4 water storage tanks will require initial filling and occasional makeup water to these tanks. For the VEGP site, the applicant proposes to use ground water as the source of water for the tanks, as described in SSAR Section 2.4.12.2 and Table 2.4.12-12 (SNC 2008b). The NRC staff determined that the capacity of the three existing and two proposed deep ground-water wells at the VEGP site under the current groundwater use permit issued by the State of Georgia Environmental Protection Division to SNC for 5.5 million gallons a day (MGD) annual average flow will be sufficient for initial filling and occasional makeup water supply, due to evaporative losses, to the two tanks providing water to the passive containment cooling system. The staff determined that neither the initial filling of the two tanks and occasional makeup involves reliance on external sources of safety-related UHS cooling water. Apart from the water stored in these two tanks to supply water to the passive containment cooling system, no other water is required by any safety-related system. Therefore, Open Item 2.4-1 is now closed, and the permit condition stated above is not required.

2.4.8.4 Conclusion

As proposed, VEGP Units 3 and 4 will not rely on any external water source for safety-related cooling water except for initial filling and makeup water. The units will not use any safety-related canals or reservoirs. The SSAR should address the requirements of 10 CFR Part 52 and 10 CFR Part 100 as they relate to identifying and evaluating design bases of canals and reservoirs at the site. As set forth above, the applicant presented and substantiated sufficient information pertaining to the design bases of canals and reservoirs at the proposed site.

Therefore, the NRC staff concludes that the identification and consideration of the safety-related canals and reservoirs set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). The NRC staff finds the applicant's site characterization related to canals and reservoirs acceptable for the ESP application.

2.4.9 Channel Diversions

In this section of the applicant's SSAR, the geohydrologic design basis is developed to ensure that the plant and essential water supplies will not be adversely affected. This review includes stream channel diversions away from the site (which may lead to loss of safety related water) and stream channel diversions towards the site (which may lead to flooding). Additionally, in such an event, the applicant needs to show that alternate water supplies are available to safety-related equipment. The NRC staff's review of the SSAR covers: (1) historical channel diversions; (2) regional topographic evidence; (3) ice causes; (4) flooding of site due to channel diversion; (5) human-induced causes of channel diversion; (6) alternate water sources; (7) consideration of other site-related evaluation criteria; and (8) additional information for 10 CFR Part 52 applications.

2.4.9.1 Introduction

The VEGP site is located on the southwest side of the Savannah River (SNC 2007). The site is located on a plateau with natural drainages that drain water away from the site in all directions. The proposed site grade for the new units is 220 feet MSL. The two proposed units will use a closed-cycle cooling system with cooling towers. Make-up water for the cooling towers' evaporative and other losses will be supplied from the Savannah River using a new intake system consisting of a canal and an intake structure.

The proposed units at the VEGP site will not rely on safety-related cooling water from the Savannah River. The highest water surface elevation caused by flooding in the Savannah River is 178.1 feet MSL, more than 30 feet below the proposed site grade.

2.4.9.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c) and 10 CFR 100.20(d), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

Section 2.4.9 of RS-002 provides the following criteria that were used by the NRC staff to evaluate this SSAR section.

- Channel diversion or realignment poses the potential for flooding or for an adverse effect on the supply of cooling water for a nuclear unit(s) of a specified type that might be constructed on the proposed site. Therefore, it is one physical characteristic that must be evaluated pursuant to 10 CFR 100.21(d). The consideration of the 10 CFR 100.21(d) criteria in this evaluation provides reasonable assurance that the effects of flooding caused by channel diversion resulting from severe natural phenomena will pose no undue risk to the type of facility proposed for the site.

- To judge whether the applicant has met the requirements of 10 CFR Part 52 and 10 CFR Part 100 as they relate to channel diversion, the NRC uses the following criteria:

    - A description of the applicability (potential adverse effects) of stream channel diversions is necessary.

    - Historical diversions and realignments should be discussed.

    - The topography and geology of the basin and its applicability to natural stream channel diversions should be addressed.

    - If applicable, the safety consequences of diversion and the potential for high or low water levels caused by upstream or downstream diversion to adversely affect safety-related facilities, water supply, or the UHS should be addressed. RG 1.27 provides guidance on acceptable UHS criteria.

2.4.9.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the technical information presented in the application; and (2) NRC staff's technical evaluation to determine the effects of potential channel diversions near the site.

2.4.9.3.1 Technical Information Presented by the Applicant

The applicant provided information related to physiographic, topographic, hydrologic, and geologic characteristics of the region within which the VEGP site is located (SNC, 207). Based on these data, the applicant concluded that it could not completely discount diversion of the river channel in this region (SNC 2007).

The applicant stated that although meandering of the river channel upstream and downstream of the VEGP site can be observed on topographic maps, the Savannah River near the VEGP site has a relatively straight and stable reach from River Mile 143 to River Mile 152 and the river plan-form did not change between 1965 and 1989 as inferred from USGS topographic maps (SNC 2007). The applicant also stated that the flow in the Savannah River is controlled by upstream multipurpose projects in the Savannah River system (SNC 2007). The effect of the control on the Savannah River results in lowering of peak flows and augmentation of low flows with an associated reduction in the morphological activity of the river (SNC 2007). The applicant concluded that it is unlikely the river will be diverted away from the VEGP site due to natural causes.

2.4.9.3.2 NRC Staff's Technical Evaluation

The NRC staff's technical evaluation consisted of a review of the approach presented in the applicant's SSAR.

As proposed in the application, the new VEGP Units 3 and 4 will not rely on any external water source for safety-related cooling water. The applicant did not propose any safety-related intakes for cooling water from the Savannah River. The NRC staff concluded that diversion of the Savannah River away from the VEGP site for any cause would not adversely affect the safety of the proposed VEGP Units 3 and 4.

The topographic elevations within the floodplain adjacent to the Savannah River northeast of the VEGP site are approximately 90 feet MSL and lower. The proposed grade elevation of the VEGP Units 3 and 4 is 220 feet MSL. In order to cause flooding at the VEGP site, the Savannah River would have to erode through more than 100 feet of terrain. Upstream dams regulate peak flood discharges in the Savannah River near the VEGP site and the river plan-form near the VEGP site is relatively straight. Based on these topographic, morphologic, and hydrologic characteristics, the NRC staff concluded that it is unlikely that flooding at the VEGP site can occur due to the Savannah River diverting towards the VEGP site.

2.4.9.4 Conclusion

As proposed, VEGP Units 3 and 4 will not rely on any external water source for safety-related cooling water. The NRC staff concluded that diversion of the Savannah River away from the VEGP site for any reason would not result in an adverse effect on safety of proposed VEGP Units 3 and 4. Based on topographic, morphologic, and hydrologic characteristics of the Savannah River, the NRC staff concluded that flooding of the VEGP site due to the river diverting towards the site is unlikely.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the identification and evaluation of channel diversions at the proposed site. Section 2.4.9 of RS-002 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating channel diversions affecting the site. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the hydrologic interface of the plant with the site, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the channel diversion characterization set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d).

In view of the above, the NRC staff finds the applicant's site characterization related to channel diversions to be acceptable for the ESP application.

2.4.10 Flooding Protection Requirements

In this section of the applicant's SSAR, the locations and elevations of safety-related facilities and those of structures and components required for protection of safety-related facilities are compared with design-basis flood conditions to determine if flood effects need to be considered in plant design or in emergency procedures. The NRC staff's review of the SSAR covers: (1) safety-related facilities exposed to flooding; (2) type of flooding protection; (3) emergency procedures; (4) consideration of other site-related evaluation criteria; and (5) additional information for 10 CFR Part 52 applications.

2.4.10.1 Introduction

The VEGP site is located on the southwest side of the Savannah River (SNC 2007). The proposed site grade for the new units is 220 feet MSL. The proposed units at the VEGP site will not rely on safety-related cooling water from the Savannah River.

2.4.10.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in site vicinity:

- 10 CFR 52.17(a), with respect to the requirement that the application contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.20(c), also requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

The regulation at 10 CFR 100.20(c) requires estimation of the PMF using historical data. Meeting this requirement provides reasonable assurance that the effects of flooding or a loss of flooding protection resulting from severe natural phenomena will pose no undue risk to the type of facility proposed for the site.

To judge whether the applicant has met the requirements of 10 CFR Part 52 and 10 CFR Part 100 as they relate to flooding protection, the NRC uses the following criteria:

- The applicability (potential adverse effects) of a loss of flooding protection should be described.

- Historical incidents of shore erosion and flooding damage should be discussed.

- The topography and geology of the basin and its applicability to damage as a result of flooding should be addressed.

If applicable, the safety consequences of a loss of flooding protection and the potential to adversely affect safety-related facilities, water supply, or the UHS should be addressed. RG 1.27 provides guidance on acceptable UHS criteria.

2.4.10.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the technical information presented in the application; and (2) NRC staff's technical evaluation to determine flooding protection requirements.

2.4.10.3.1 Technical Information Presented by the Applicant

The applicant stated that entrances and openings of all safety-related SSCs will be placed at or above the proposed site grade of 220 feet MSL (SNC 2007). The design-basis flood elevation in the Savannah River is 178.1 feet MSL (SNC 2007). The applicant concluded that safety-related SSC of the proposed VEGP Units 3 and 4 will not be exposed to flooding from the Savannah River.

The applicant stated that the effects of local intense precipitation will be considered in the design of site drainage system (SNC 2007). The applicant committed to designing the site drainage system such that all safety-related SSC would be safe from flooding from local intense precipitation (SNC 2007). All drainage structures such as culverts, storm drains, and bridges would be assumed to be blocked during the local intense precipitation event (SNC 2007).

2.4.10.3.2 NRC Staff's Technical Evaluation

In the preceding sections of this report, the NRC staff estimated the highest water surface elevation due to flooding in the Savannah River and concluded that it is well below the proposed site grade. The NRC staff concluded that protection from flooding in the Savannah River is not needed for a safety-related SSC if its entrances and openings are located above the proposed site grade of 220 feet MSL.

2.4.10.4 Conclusion

The proposed site grade of 220 feet MSL is safe from flooding in the Savannah River. The entrances and openings of all safety-related SSC that are located above the proposed site grade would be safe from flooding.

As set forth above, the applicant has presented and substantiated sufficient information pertaining to the flood protection measures at the proposed site. RS-002, Section 2.4.10 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating flood protection measures at the site. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the flooding protection requirements at the site, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the applicant's analysis previously identified are acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the flooding protection requirement analysis set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). In view of the above, the NRC staff finds the applicant's analysis related to flooding protection requirements to be acceptable for the ESP application.

2.4.11 Low Water Considerations

In this section of the applicant's SSAR, natural events that may reduce or limit the available safety-related cooling water supply, are identified and the applicant ensures that an adequate water supply will exist to shut down the plant under conditions requiring safety-related cooling. The NRC staff's review of the SSAR covers: (1) low water from drought; (2) low water from other phenomena; (3) effect of low water on safety-related water supply; (4) water use limits; (5) consideration of other site-related evaluation criteria; and (6) additional information for 10 CFR Part 52 applications.

2.4.11.1 Introduction

The VEGP site is located on the southwest side of the Savannah River (SNC 2007). The proposed units at the VEGP site will not rely on safety-related cooling water from any external source, including the Savannah River and groundwater.

2.4.11.2 Regulatory Basis

The acceptance criteria for this section relate to the following regulations:

- 10 CFR Part 52 and 10 CFR Part 100 require that hydrologic characteristics be considered in the site evaluation.

- 10 CFR 100.23 requires that siting factors to be evaluated must include the cooling water supply.

Section 2.4.11 of RS-002 provides the following criteria that were used by the NRC staff to evaluate this SSAR section.

- The regulations at 10 CFR Part 52 and 10 CFR Part 100 require that the evaluation of a nuclear power plant site consider the hydrologic characteristics. To satisfy the requirements of 10 CFR Part 52 and 10 CFR Part 100, the applicant's SSAR should describe the surface and subsurface hydrologic characteristics of the site and region. In particular, the UHS for the cooling water system may consist of water sources that could be affected by the site's hydrologic characteristics that may reduce or limit the available supply of cooling water for safety-related SSCs, such as those resulting from river blockage or diversion, tsunami runup and drawdown, and dam failure.

- Meeting the requirements of 10 CFR Part 52 and 10 CFR Part 100 provides reasonable assurance that severe hydrologic phenomena, including low-water conditions, will pose no undue risk to the type of facility proposed for the site.

- As required by 10 CFR 100.23, siting factors, including cooling water supply, must be evaluated for a nuclear unit. The evaluation of the emergency cooling water supply for a nuclear power plant(s) of a specified type that might be constructed on the proposed site should consider river blockages, diversions, or other failures that may inhibit the flow of cooling water, tsunami runup and drawdown, and dam failures.

- The regulation at 10 CFR 100.23 applies to this section because the UHS for the cooling water system consists of water sources that are subject to natural events that may reduce or limit the available supply of cooling water (i.e., the heat sink). Natural events such as river blockages, diversions, or other failures that may inhibit the flow of cooling water, tsunami runup and drawdown, and dam failures should be conservatively estimated to assess the potential for these characteristics to influence the design of those SSCs important to safety for a nuclear unit(s) of a type specified by the applicant that might be constructed on the proposed site. The available water supply should be sufficient to meet the needs of the unit(s) to be located at the site. Specifically, those needs include the maximum design essential cooling water flow, as well as the maximum design flow for normal plant needs at power and at shutdown.

- The specific criteria discussed in the paragraphs below assess the applicant's ability to meet the requirements of the hydrologic aspects of the above regulations. Acceptance is based primarily on the adequacy of the UHS to supply cooling water for normal operation, anticipated operational occurrences, safe shutdown, cooldown (first 30 days), and long-term cooling (periods in excess of 30 days) during adverse natural conditions.

Low Flow in Rivers and Streams

- For essential water supplies, the low-flow/low-level design for the primary water supply source is based on the probable minimum low flow and low level resulting from the most severe drought that can reasonably be considered for the region. The low-flow/low-level site parameters for operation should not allow shutdowns caused by inadequate water supply to trigger the frequent use of emergency systems.

- Low Water Resulting from Surges, Seiches, or Tsunami

- For coastal sites, the applicant should postulate the appropriate PMH wind fields at the ESP stage to estimate the maximum winds blowing offshore, thus creating a probable minimum surge level. Low-water levels on inland ponds, lakes, and rivers caused by surges should be estimated based on the probable maximum winds oriented away from the plant site. The same general analysis methods discussed in Sections 2.4.3, 2.4.5, and 2.4.6 of RS-002 apply to low-water estimates resulting from the various phenomena discussed. If the site is susceptible to such phenomena, minimum water levels resulting from setdown (sometimes called runout or rundown) from hurricane surges, seiches, and tsunamis should be verified at the COL or CP stage to be higher than the intake design basis for essential water supplies.

Historical Low Water

- If historical flows and levels are used to estimate design values by inference from frequency distribution plots, the data used should be presented to allow for an independent determination. The data and methods of NOAA, USGS, SCS, USBR, and USACE are acceptable.

Future Controls

- This section is acceptable if water use and discharge limitations (both physical and legal), which are already in effect or under discussion by the responsible Federal, State, regional, or local authorities and which may affect the water supply for a nuclear unit(s) of a type specified by the applicant that might be constructed on the proposed site, have been considered and are substantiated by reference to reports of the appropriate agencies. The design basis should identify and take into account the most adverse possible effects of these controls to ensure that essential water supplies are not likely to be negatively affected in the future.

2.4.11.3 Technical Evaluation

The technical evaluation consists of: (1) a review of the technical information presented in the application; and (2) NRC staff's technical evaluation to determine effects of low water conditions.

2.4.11.3.1 Technical Information Presented by the Applicant

The applicant stated that proposed VEGP Units 3 and 4 will not use any external water sources for safety-related cooling water supply (SNC 2007).

2.4.11.3.2 NRC Staff's Technical Evaluation

The applicant stated that proposed VEGP Units 3 and 4 will not need any external water sources for safety-related cooling water supply for continuous use. While, the NRC staff determined that initial filling and occasional makeup water requirements for two water storage tanks exist, as described in Section 2.4.8.3.2 of this report, the NRC staff determined that low water conditions will not affect any safety-related SSCs.

2.4.11.4 Conclusion

The proposed VEGP Units 3 and 4 will not rely on any external source of water supply for safety-related cooling on a continuous basis; therefore, low water conditions will not affect any safety-related SSCs. RS-002, Section 2.4.11 provides that the SSAR should address the requirements of 10 CFR Parts 52 and 100 as they relate to identifying and evaluating low water conditions affecting the site. As set forth above, the applicant has presented and substantiated sufficient information pertaining to the identification and evaluation of low water conditions at the proposed site.

Therefore, the NRC staff concludes that the identification and consideration of the low water conditions set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d). In view of the above, the NRC staff finds the applicant's site characterization related to low water considerations for inclusion in an ESP for the applicant's site to be acceptable.

2.4.12 Ground Water

2.4.12.1 Introduction

This section of the applicant's SSAR evaluates the hydrogeological characteristics of the site and describes the effects of ground water on the plant foundations and the reliability of safety-related water supply and dewatering systems. The NRC staff's review of the SSAR covers: (1) local and regional ground-water characteristics and use; (2) effects on plant foundations and other safety-related SSCs; (3) reliability of ground-water resources and systems used for safety-related purposes; (4) reliability of dewatering systems; and (5) consideration of other site-related evaluation criteria.

The proposed VEGP Units 3 and 4 are to be located on a topographic ridge perpendicular to the Savannah River that forms a boundary between two watersheds. The watershed to the northwest is dominated by Mallard Pond and an unnamed drainage creek from it that discharges to the Savannah River. The watershed to the southeast is dominated by Daniels Branch, Telfair Pond, and Beaverdam Creek. Beaverdam Creek discharges to the Savannah River. Construction of the proposed facilities may alter the topography of the site and alter recharge to the unconfined aquifer in the immediate vicinity of the proposed units. Ground water has no safety-related role in the operation of the proposed VEGP units; however, the three existing and two proposed deep groundwater wells at the VEGP site will be sufficient for initial filling and occasional makeup water supply to the two tanks providing water to the passive containment cooling system.

Section 2.4.13 of this SER provides a complete discussion and evaluation of accidental radioactive releases (i.e., the release, migration, and the resulting hazard).

2.4.12.2 Regulatory Basis

The acceptance criteria for this section relate to the following regulations:

- 10 CFR Part 52 and 10 CFR Part 100 requires the site evaluation to consider hydrologic characteristics.
- 10 CFR 100.23 sets forth the criteria to determine the suitability of design bases for a nuclear unit of specified type that might be constructed on the proposed site with respect to its seismic characteristics. This section also requires applicants to ensure the adequacy of the cooling water supply for emergency and long-term shutdown decay heat removal, taking into account information concerning the physical, including hydrological, properties of the materials underlying the site.

As specified in 10 CFR 100.20(c), the NRC must consider the site's physical characteristics (including seismology, meteorology, geology, and hydrology) when determining its acceptability to host a nuclear unit.

The regulation at 10 CFR 100.20(c)(3) requires that the NRC address factors important to hydrologic radionuclide transport using onsite characteristics. To satisfy the hydrologic requirements of 10 CFR Part 100, the staff's review of the applicant's SSAR should verify the description of ground-water conditions at the proposed site and the effect of the construction and operation of a nuclear unit of specified type that might be constructed on the site on those conditions. Meeting this requirement provides reasonable assurance that the release of radioactive effluents from a unit of specified type that might be constructed on the proposed site will not significantly affect the ground water at or near the site.

The regulation at 10 CFR 100.23 requires that the evaluation consider geologic and seismic factors when determining the suitability of the site and the acceptability of the design for each nuclear power plant. In particular, 10 CFR 100.23(d)(4) requires consideration of the physical properties of materials underlying the site when designing a system to supply cooling water for emergency and long-term shutdown decay heat removal.

Though not required at the ESP stage, the applicant for a COL must demonstrate compliance with GDC 2 as it relates to designing SSCs important to safety to withstand the effects of natural phenomena.

To judge whether the applicant has met the requirements of the hydrologic aspects of 10 CFR Part 52 and 10 CFR Part 100, the NRC used the following criteria:

- Section 2.4.12.1 of the SSAR must fully describe regional and local ground-water aquifers, sources, and sinks. In addition, it must describe the type of ground-water use, wells, pump, storage facilities, and the flow needed for the proposed plants of specified type that might be constructed on the site. If ground water is to be used as an essential source of water for safety-related equipment, the design basis for protection from natural and accident hazard phenomena must be compared to RG 1.27 guidelines. This section must adequately describe and reference the bases and data sources.

- Section 2.4.12.2 of the SSAR must describe present and projected local and regional ground-water use. This section must discuss and tabulate existing uses, including amounts, water levels, location, drawdown, and source aquifers. It must also indicate flow directions, gradients, velocities, water levels, and the effects of potential future use on these parameters, including any possibility for reversing the direction of ground-water flow. In addition, SSAR Section 2.4.12.2 must identify any potential ground-water recharge area within the influence of the proposed plants of specified type that might be constructed on the site, as well as the effects of construction, including dewatering. This section must also discuss the influence of existing and potential future wells with respect to ground water beneath the site and describe and reference the bases and data sources. RS-002 discusses certain studies concerning ground-water flow problems.

- Section 2.4.12.3 of the SSAR must discuss the need for and extent of procedures and measures, including monitoring programs, to protect present and projected ground-water users. These items are site specific and will vary with each application.

To evaluate whether the applicant has met the requirements of 10 CFR 50.55, "Conditions of Construction Permits," the NRC uses the following criteria:

- SSAR Section 2.4.12.4 should describe the design bases (and development thereof) for ground-water-induced loadings on subsurface portions of safety-related SSCs at the COL stage. If a permanent dewatering system is employed to lower design-basis ground-water levels, the applicant must provide the bases for the design of the system and determination of the design basis for ground-water levels. The application must provide information regarding the following:

  — all structures, components, and features of the system

  — the reliability of the system as related to available performance data for similar systems used at other locations

  — the various soil parameters (such as permeability, porosity, and specific yield) used in the design of the system

  — the bases for determination of ground-water flow rates and areas of influence to be expected

  — the bases for determination of time available to mitigate the consequences of system failure where system failure could cause design bases to be exceeded

  — the effects of malfunctions or failures (such as a single failure of a critical active component or failure of circulating water system piping) on system capacity and subsequent ground-water levels

  — a description of the proposed ground-water level monitoring program and outlet flow monitoring program

- If wells are proposed for safety-related purposes, the applicant must describe the hydrodynamic design bases (and development thereof) for protection against seismically induced pressure waves, which should be consistent with site characteristics.

2.4.12.3 Technical Evaluation

This section reviews the applicant's information and evaluates the effects of ground water.

2.4.12.3.1 Technical Information Presented by the Applicant

In Section 2.4.12 of the SSAR, both Revision 4 (SNC 2008a) and Revision 4-S2 (SNC 2008b), Southern Nuclear Operating Company (Southern) presented information and data describing the local and regional ground-water systems and use, monitoring or safeguard requirements, and design basis for subsurface hydrologic loading. Much of the information and data was available in Revision 2 of the SSAR (SNC 2007) and was described in the ESP SER with Open Items; however, a substantial body of work on groundwater models and modeling of the VEGP site was included in Revision 4-S2 (SNC 2008b) and the responses to the additional RAIs (SNC 2008c).

The VEGP site is located on a ridge perpendicular to the Savannah River which lies to the northeast. This ridge separates two drainages. Mallard Pond and an unnamed drainage stream lie to the northwest, and Red Branch, Daniels Branch, Telfair Pond, and Beaverdam Creek lie to the southeast (SNC 2008a, Part 2, Section 2.4.1.2.2).

The applicant described the hydrogeology in Section 2.4.12.1.1 of the SSAR (SNC 2008b). The thickness of Coastal Plain sediments varies from less than 200 feet at the fall line to 4000 feet at the coastline, and is approximately 1000 feet thick at the site (SNC 2008b, Section 2.4.12.1.1). A surface topography of gently rolling hills ranges in elevation from 80 feet above MSL to nearly 300 feet above MSL in the immediate vicinity of the VEGP site (SNC 2008a, Part 3, Sections 2.4.1 and 2.6.1). Developed portions of the site have ground surface elevations of approximately 220 feet MSL (SNC 2008b, Section 2.4.12, pg. 2.4.12-1, and Figure 2.4.12-1). The Savannah River has incised the Coastal Plain sediments and formed steep bluffs exhibiting topographic relief of nearly 150 feet from the river to the developed portions of the existing VEGP site (SNC 2008a, Part 3, Section 2.6.1).

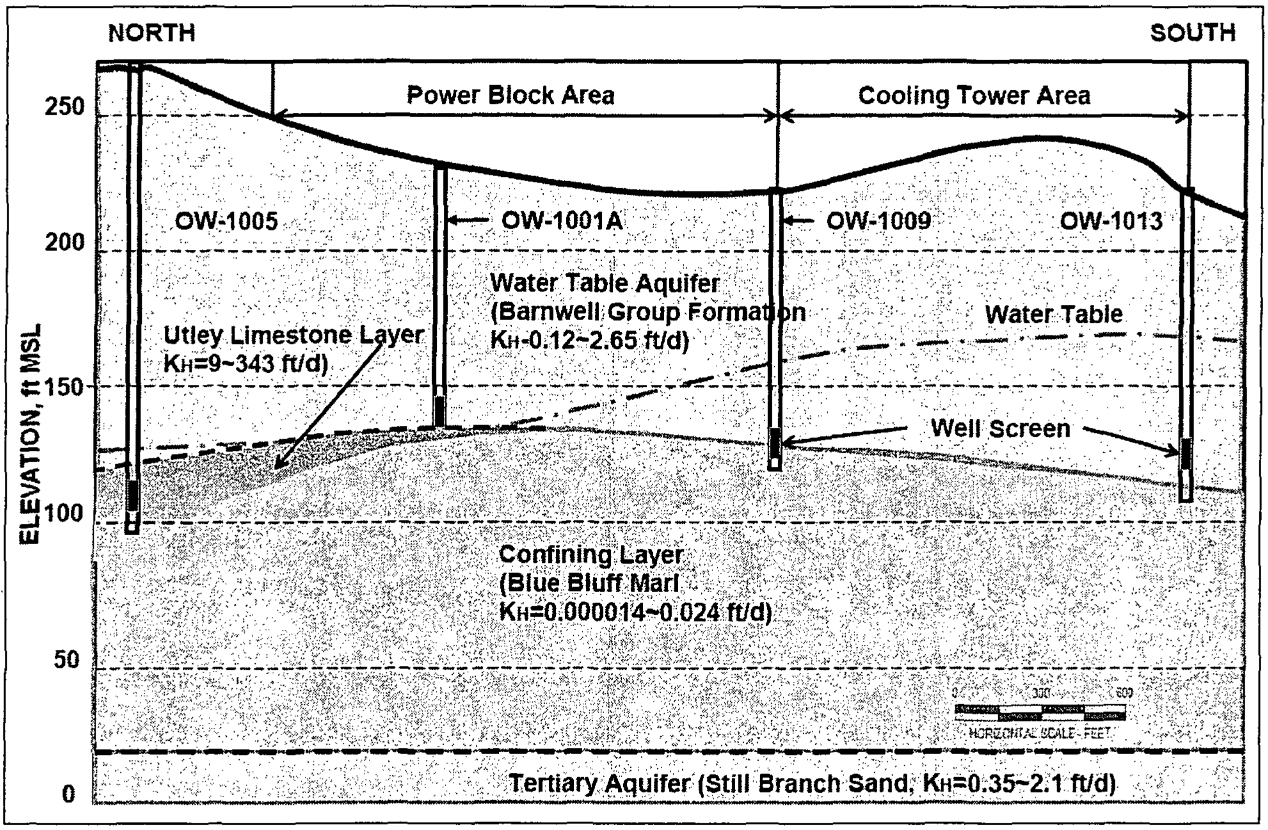


Figure 2.4.12-1 Hydrogeologic cross-section of the Water Table aquifer at the Vogtle site (KH is the horizontal hydraulic conductivity)

Precipitation onto outcrops of aquifer sediments creates a ground-water source. Locally, net infiltration from precipitation recharges the Water Table aquifer (SNC 2008b, Section 2.4.12.1.1). Net infiltration from precipitation recharges the locally confined Tertiary and Cretaceous aquifers at outcrops of these formations nearer the fall line (SNC 2008b, Section 2.4.12.1.1).

The applicant stated that the Water Table aquifer discharges to ground-water wells and local drainages, including springs and seeps that ultimately drain to the Savannah River (SNC 2008b, Section 2.4.12.1.2). Figure 2.4.12-7 of the SSAR (SNC 2008b) depicts the piezometric surface of the Water Table aquifer and implies that ground-water flow throughout the proposed powerblock area is moving to the north-northwest and Mallard Pond. Depictions of the piezometric surface from 1971 (see SNC 2003 drawing AX6DD329) and 1984 (see SNC 2003 drawing AX6DD330) reveal the evolution of decline in the piezometric surface of the Water Table aquifer.

The applicant stated that the Tertiary aquifer drains to the Savannah River (see Figure 2.4.12-14 in SNC 2008b) and discharges to wells, natural springs, and subaqueous outcrops presumed to exist offshore (SNC 2008b, Section 2.4.12.1.2). Discharge to the Savannah River occurs where the river has completely eroded the Blue Bluff Marl confining layer (SNC 2008b, Section 2.4.12.1.2). Depictions of the piezometric surface from 1971 (see SNC 2003 drawing AX6DD327) and 1984 (see SNC 2003 drawing AX6DD328) reveal the evolution of the piezometric surface of the Tertiary aquifer.

The applicant concluded that piezometric head data for observation wells OW-1001 and OW-1001A were invalid and removed the data from the ESP application (SNC 2008b, Section 2.4.12.1.3, pg. 2.4.12-12). The well screen for OW-1001A ranges in elevation from 146.13 to 136.13 feet MSL (SNC 2008b, Section 2.4.12.1.3). In the vicinity of the proposed VEGP Unit 4, which is close to these wells, the top of the Blue Bluff Marl is located between 121.9 feet and 138.2 feet MSL (SNC 2008a, Part 2, Section 2.5.1.2.3.2 and Figure 2.5.1-47), with the lower value in the vicinity of OW-1001A. Omission of these data and information led the applicant to interpolate other nearby measurements and assign a piezometric head value to this location of approximately 147 feet (SNC 2008b, Figure 2.4.12-7) when the information suggests a head value less than the screened interval.

The applicant reported hydraulic properties of the Barnwell Formation sediments and included the range of hydraulic conductivity measurements for the Utley Limestone from 3,250 to 125,400 feet/year (9 to 343 feet/day). The applicant derived a value for effective porosity of 0.34 (SNC 2008b, Section 2.4.12.1.4) from the median specific gravity and moisture content measurements for Barnwell sediments. Using ground-water data from June 2005 through July 2007, the applicant estimated a hydraulic gradient of 0.014 feet/feet to apply to the Water Table aquifer across the site (SNC 2008b, Section 2.4.12.1.3).

The applicant reported a range of 480 to 1220 feet/year (1.3 to 3.3 feet/day) for hydraulic conductivity values in the engineered backfill (SNC 2008b, Section 2.4.12.1.4). The applicant obtained this value from the prior postconstruction testing of backfill regions underlying VEGP Units 1 and 2, as reported in the updated final safety analysis report (UFSAR), Table 2.4.12-14 (SNC 2003). The applicant used a value of 0.34 for the porosity of the engineered backfill, as applied in the FSAR for VEGP Units 1 and 2 (SNC 2003, Sections 2.4.13.1.1 and 2.4.12.2.4.3, and Table 2.4.12-14).

The applicant reported hydraulic properties of the Tertiary aquifer sediments (SNC 2008b, Section 2.4.12.1.4, Table 2.4.12-3). These include a range of hydraulic conductivities from 0.35 to 2.1 feet/day with a geometric mean of 0.83 feet/day, an effective porosity of 0.31, and a storage coefficient of $1.0x10^{-4}$. The applicant estimated a hydraulic gradient of 0.005 feet/feet to apply to a distance of 5600 feet between the center of the proposed powerblock area and the Savannah River.

In Section 2.4.12.1.4 of SNC 2008b and Appendix 2.4B of SNC 2008c, the applicant presents the development and application of a two-dimensional, single-layer, steady-state ground-water model of the Water Table aquifer underlying the VEGP site. The model domain includes the watersheds on either side of the ridge on which VEGP Units 3 and 4 are proposed to be sited and is bounded above by the land surface and below by the top of the Blue Bluff Marl. The model varied spatially the hydraulic conductivity assignments to represent the presence or absence of the possibly more conductive Utley Limestone unit. In addition, the model assigned engineered fill areas associated with existing and proposed VEGP units the maximum hydraulic conductivity of engineered backfill measured at VEGP Units 1 and 2. The aquifer recharge rate assignments accounted for variations in surface slopes, vegetative cover, and land use, including structures and paved areas.

The applicant executed a series of simulations for seven alternative models. The seven models involved different combinations of hydraulic conductivity and recharge to calibrate the model (SNC 2008b, Section 2.4.12.1.4, pg. 2.4.12-18). The applicant also considered the seven model simulations to represent alternative conceptual models of the site and aquifer. The seven models include the following:

1. uniform hydraulic conductivity and recharge (single values of each for the entire model domain)

2. uniform hydraulic conductivity, variable recharge (open and forested areas, buildings and pavement)

3. accounting for thickness of the Utley Limestone (variable hydraulic conductivity, model 2 recharge pattern and values)

4. simplified Utley Limestone (simplified version of model 3)

5. high conductivity zone upstream of Mallard Pond (acknowledges Utley cave and spring)

6. low conductivity zone in southwestern part of model domain (attempt to reduce bias in model results; in models 1 through 5 the predicted hydraulic head in Daniels Branch, Telfair Pond watershed, is lower than observed while predicted head in Mallard Pond watershed is higher than observed)

7. simplified version of model 6

The applicant stated that, while the solutions obtained with models 6 and 7 were very similar and close to the measured water levels, model 7 provided the best match with the observed data and was selected for analysis of the postconstruction setting (SNC 2008b). The applicant analyzed travel time by using model 7 to simulate the travel path from the VEGP Unit 4 auxiliary building to the upper reaches of Mallard Pond. Essentially, the ground water moved through three regions of the model—the saturated engineered backfill, the aquifer from the excavation (backfill) to the high conductivity zone above Mallard Pond, and the high conductivity zone to Mallard Pond. The applicant predicted travel times through the three zones to be 2.4 years, 3.2 years, and 1.1 years for a total ground-water travel time of 6.7 years (see Figure 78 in Appendix 2.4B, SNC 2008b).

The applicant provided data about regional and local ground-water use (SNC 2008b, Section 2.4.12.2, pg. 2.4.12-23). The application lists permits issued by the State of Georgia Environmental Protection Division for ground-water withdrawals that exceed 100,000 gallons per day during any single month for municipal, industrial, and agricultural users. In addition, users are listed as shown in the Safe Drinking Water Information System maintained by EPA. The applicant provided the locations of the nearest examples of each of these ground-water users. The application summarizes current well location and usage by VEGP Units 1 and 2.

The applicant also provided a forecast of water resource usage in Burke County and summarized the projected ground-water use for the proposed units. Part 3 of the application (i.e., the environmental report) includes additional information and data (SNC 2008a, Part 3, Section 2.3.2).

Regarding the reliability of ground-water resources and systems used for safety-related purposes, the applicant stated that a future plant that fits within the bounding parameters provided in the proposed permit application has a passive safety-related UHS. Consequently, no safety-related ground-water supplies are necessary except for initial fill up and occasional makeup water (SNC 2008b, Section 2.4.12, pg. 2.4.12-1).

The applicant stated that the plant grade for the proposed units is elevation 220 feet MSL, and the foundation embedment depth is 39.5 feet from plant grade (SNC 2008b, Section 2.4.12, pg. 2.4.12-1). The elevation of containment and auxiliary building foundations is approximately 180.5 feet MSL. The applicant stated that the maximum ground-water elevation of the Water Table aquifer underlying the proposed VEGP units is 165 feet MSL (SNC 2008a, Part 2, Table 1-1). Regarding the reliability of dewatering systems, the applicant stated that a future plant that fits within the bounding parameters provided in the proposed permit application will not require a permanent dewatering system to lower the design-basis ground-water level because all safety-related SSCs are well above the highest recorded water table elevation in the powerblock area (SNC 2008b, Section 2.4.12.4, pg. 2.4.12-25).

The applicant stated that the excavated natural materials will be replaced with compacted structural fill with properties that provide an adequate factor of safety against liquefaction (SNC 2008a, Part 2, Section 2.5.4.8.3.1). The applicant reported confirmatory liquefaction analyses in Section 2.5.4.8 (SNC 2008a, Part 2, Section 2.5.4.8). The applicant concluded that the liquefaction potential of the compacted structural fill was not a concern and materials comprising the Blue Bluff Marl had an adequate factor of safety against liquefaction (SNC 2008a, Part 2, Section 2.5.4.8.4).

The applicant committed to review and evaluate existing SNC ground-water monitoring programs and observation well locations for adequacy and to describe that evaluation and the resulting long-term ground-water monitoring program for the proposed units in the COL application (SNC 2008b, Section 2.4.12.3, pg. 2.4.12-24).

2.4.12.3.2 Technical Evaluation

The technical evaluation by NRC staff is presented below for each of the specific RS-002 acceptance criteria. As a result of a series of requests, beginning at the initial site audit conducted in January 2007, the applicant has revised Section 2.4.12 of the SSAR with each revision of the application. The applicant provided the latest version of this FSAR section to the NRC as a supplement to Revision 4 of the application (SNC 2008b).

In an initial request for additional information (RAI) the NRC staff asked the applicant for (1) an interpretation of field observations and the potential for an alternative conceptual model allowing communication between the Water Table aquifer and the Tertiary aquifer, (2) a description of the process to develop the conceptual model (i.e., alternatives considered and the methodology used by the model to account for transient behavior), and (3) all available location information on the sediments related to the Water Table aquifer (e.g., thickness and continuity of the Barnwell sands, silts and clays, the Utley Limestone, and the Lisbon Formation). Southern responded to these requests (SNC 2007c) and incorporated new material in Revision 2 of the SSAR.

The NRC staff issued the SER with Open Items and included Open Item 2.4-2, which requested that the applicant provide an improved and complete description of the local hydrological conditions, including alternative conceptual models, to demonstrate that the design basis related to ground-water-induced loadings would not be exceeded. Future projections were needed of the impact on the Water Table aquifer arising from potential changes in land use and aquifer recharge as a result of construction of the proposed facilities. The applicant developed a ground-water model of the Water Table aquifer and incorporated its description and results into Revision 3 of the SSAR.

The NRC staff's review of the ground-water model described in SSAR, Revision 3, as well as model input and output, revealed issues with model convergence, mass balance, and calibration bias. The NRC staff also realized that alternative conceptual models were not presented. Rather, the applicant presented a sequence of models used to achieve calibration of a single conceptual model. The staff raised these concerns with the applicant at a public meeting at the NRC in Rockville, Maryland, on April 8, 2008, at a site audit at the applicant's consultant's offices in Frederick, Maryland, on April 9, 2008, and through additional RAIs dated July 22, 2008. The applicant addressed these issues in the supplement to Revision 4 of the application (SNC 2008b) and in responses to the RAIs (SNC 2008c).

The applicant's analysis, which was initially based entirely on field data and the assumption that postconstruction ground-water levels would not exceed prior measured levels, evolved into an analysis based on field data, a model of the Water Table aquifer, and postconstruction projections of the water table. This final analysis provided reasonable assurance that the design basis related to ground-water-induced loadings would not be exceeded.

Local and Regional Ground-Water Characteristics and Use

Based on a review of USGS documents (Clarke and West 1997, 1998; Cherry 2006; Cherry and Clarke 2007), State of Georgia documents, Huddlestun and Summerour (1996), and Summerour et al. (1994, 1998), the NRC staff determined that the applicant's description of the regional and local hydrogeologic conditions is accurate with one potential exception-ground-water flow within the Water Table aquifer may not always be from the powerblock area to the north-northwest and Mallard Pond. The NRC staff's investigations of the site and review of topographic maps confirm that the proposed location is on a ridge perpendicular to the Savannah River and separating drainages to the north-northwest (e.g., Mallard Pond) and to the south-southeast (e.g., Daniels Branch, Telfair Pond, and Beaverdam Creek).

The NRC staff confirmed that the recorded piezometric surface contour plots, including seasonal and climatic fluctuations of the Water Table aquifer, indicate ground-water movement toward the north-northwest and Mallard Pond from release points within the powerblock area. However, a number of lines of reasoning, described below, led the NRC staff to question whether this would be the only ground-water flow and contaminant migration direction for future accidental effluent release events.

First, the applicant stated that the piezometric head level in the Water Table aquifer is a function of the topography and recharge, which both change in the vicinity of the proposed VEGP Units 3 and 4. Substantial areas of the proposed site will be leveled and made impervious by construction of buildings and paved surfaces. Other substantial areas of the proposed site will be leveled and might be made more transmissive (i.e., able to accept more recharge) by converting them to gravel surfaces that would be maintained essentially vegetation free. Stormwater management facilities that will be constructed to route runoff from significant storm events away from the site could reduce potential infiltration rates. Each of these actions implies a potentially substantial change in the net infiltration to the Water Table aquifer in the immediate vicinity of the proposed VEGP Units 3 and 4. The applicant's model of the Water Table aquifer (SNC 2008b, 2008c) includes an evaluation of current, spatially varying recharge patterns and postconstruction changes to recharge resulting from changes in land use and vegetation. In addition, the NRC staff has used the applicant's model and conservatively analyzed a higher postconstruction recharge with a lower hydraulic conductivity assigned to the engineered backfill in the excavated region.

Second, the NRC staff's review of the historical piezometric head contours in the Water Table aquifer for the years 1971 (see SNC 2003, drawing AX6DD329), 1984 (see SNC 2003, drawing AX6DD330), and 2005 (see SNC 2008b, SSAR Figure 2.4.12-7) revealed evidence of change that has occurred since 1971 in the piezometric head as a result of the construction and operation of VEGP Units 1 and 2. This suggests that the assumption that the current piezometric surface will exist after construction and during operation of the proposed units is not realistic. However, the NRC staff notes that the broad and essentially flat area created for construction of the proposed VEGP Units 3 and 4 does represent a current local topographic high, and it is likely that the highest postconstruction recharge rates within the region disturbed by construction would be in the vicinity of the cooling tower area and not near the powerblock area. Thus, while the same ground-water surface will not exist, the location of the ground-water high divide will remain in the vicinity of the proposed cooling towers.

Finally, the NRC staff used the applicant's model of the Water Table aquifer to evaluate the sensitivity of the model solution to drain boundary condition elevations, to the use of minimum light detection and ranging (LiDAR) data rather than average LiDAR data in drain cells, to the use of drain cells instead of constant head boundary conditions for the perennial reach of Daniels Branch, and to postconstruction conditions more extreme than those evaluated by the applicant. In the latter cases, the staff evaluated the origin of releases to the watershed that lies to the southeast of the proposed facilities. To do this, the staff first assigned drain boundary condition cells elevations consistent with the land surface and conductance consistent with neighboring cells. This did not result in a substantial change in the model solution. The NRC staff next used minimum rather than average LiDAR to set drain elevations in the Daniels Branch drainage to evaluate ground-water movement to that drainage. This modification in the model boundary condition did not substantially change the essential feature of the applicant's model in this regard (i.e., that ground water moved beneath and was not intercepted in the upper reach of the Daniels Branch). The staff then used a drain boundary condition in the perennial reach of the Daniels Branch which did cause the cell ground-water level prediction to increase (i.e., the predicted ground-water elevation in the drainage was higher than in the constant head boundary condition model). However, ground water continued to discharge to the perennial reach of the streambed, but at a lower rate. Next, the staff used a series of recharge rate cases to evaluate the sensitivity of the applicant's results. These post construction cases included the hydraulic conductivity of the engineered fill (3.3 feet/day) in the excavation and a suite of high expected value and low recharge rates applied to the powerblock area and the cooling tower area. None of the cases revealed discharge to the Daniels Branch drainage; however, one case exhibited ground-water flow under the streambed. In addition, the case in which a high recharge was applied to both the proposed powerblock and cooling tower areas resulted in movement of some pathways directly toward the Savannah River from the southeast corner of the powerblock. However, such a result is not plausible because the powerblock grounds are actually engineered (e.g., sloped, paved) to promote runoff rather than infiltration and recharge. If comparable recharge rates were applied to VEGP Units 1 and 2 then flow toward the river from the proposed VEGP Units 3 and 4 would not occur. Thus, the staff attempted to test the hypothesis that ground water from the powerblock could discharge to the other watersheds but did not do so. However, because a pathway from the powerblock into the Daniels Branch drainage was demonstrated, by the staff, the uncertainty in the aquifer structure and hydraulic properties compels the staff to view this pathway as plausible and to continue to examine the alternative conceptual model of ground-water flow from the powerblock being intercepted by the upper reaches of the Daniels Branch. SER Section 2.4.13 further discusses alternative conceptual models of the future ground-water pathway.

The NRC staff confirmed the applicant's hydraulic conductivity values for the Water Table aquifer. The NRC staff independently determined that the USGS-derived minimum and maximum range of transmissivity values based on field data (i.e., 500 feet2/day to 9500 $feet^2$/day or 3700 gallons/day/feet to 71,000 gallons/day/feet) (Clarke and West 1998, Table 3), when combined with the local thickness of the Water Table aquifer (i.e., approximately 30 feet), are indicative of the higher values of the Utley Limestone of the Barnwell Formation cited by the applicant.

The NRC staff's review of the SSAR (SNC 2008b, Section 2.4.12) and USGS documents (Clarke and West 1997, 1998; Cherry 2006; Cherry and Clarke 2007) supports the applicant's interpretation that the Tertiary aquifer drains toward the Savannah River. The sequence of piezometric head maps from 1971 (see SNC 2003 drawing AX6DD327), 1984 (see SNC 2003 drawing AX6DD328), and the seasonal fluctuations in the 2005 to 2006 time period (see SNC 2008b, SSAR Figures 2.4.12-14 through 2.4.12-18) indicate the direction that ground-water flow has been maintained. These piezometric head data reveal a pattern of decline in head values over time, but the change will not affect both the existing and future groundwater uses.

Regarding the applicant's reported values of hydraulic conductivity in the Tertiary aquifer, the NRC staff independently reviewed USGS minimum and maximum ranges of transmissivity estimates based on field data (1,346 to 91,200 gallons/day/foot) and on regional simulation (100 to 185,000 gallons/day/foot) (Clarke and West 1998, Table 12). When combined with the local thickness of the Tertiary aquifer (approximately 182 feet), the USGS data bracket the central value of hydraulic conductivity provided by the applicant (i.e., 0.83 feet/day), but are generally higher.

One purpose of using an alternative conceptual model is to acknowledge the uncertainty in the interpretation of field observations and data sets that are by their nature incomplete. An example lies in the interpretation of data available from observation wells OW-1001 and OW-1001A. A poorly constructed and slowly responding well (i.e., OW-1001) may still provide valid data, until the validity of the data are disproved by completion of a competent observation well at the location. Observations of hydraulic head below the screened interval elevation of a well (i.e., OW-1001A) are obviously not valid as head observations; however, they suggest that the hydraulic head at that location is below the bottom of the screen (i.e., 136.13 feet). Again, until they are replaced with a competent observation well and an unambiguous data set, OW-1001 and OW-1001A provide information that suggests an alternate interpretation of local communication between the Water Table and Tertiary aquifers. Data from Borehole B-1004 in the vicinity of these observation wells suggest that the Blue Bluff Marl is approximately 95 feet thick at this location (SNC 2008a, Part 2, Figure 2.5.1-51). The data and information from the two observation wells are consistent with ground-water movement from the Water Table aquifer into the Tertiary aquifer at this location; however, the thickness of the marl unit suggests the integrity of this confining unit. Section 2.4.13 of this SER further discusses this alternate conceptual model.

The NRC staff reviewed aspects of the ground-water system that led to the applicant's statement that ground-water in South Carolina neither affects nor is affected by VEGP site operation. The NRC staff reviewed the USGS ground-water model of the region that included the VEGP site in Georgia as well as the SRS in South Carolina (Clarke and West 1998; Cherry 2006). This recent USGS work presents a current interpretation of ground-water data and provides insight into where the Savannah River has incised confining zones, allowing releases to occur from confined aquifers into the Savannah River alluvium and hence to the Savannah River. The deep confined aquifers of the Cretaceous aquifer system (i.e., described as the Dublin and Midville aquifer systems in USGS reports) are not incised by the river opposite the VEGP site, but are incised several miles upstream (Clarke and West 1998, Figure 5). Therefore, the confining zones are intact beneath the Savannah River opposite the VEGP site. This allows complete communication of ground water in the Cretaceous aquifer between the States of Georgia and South Carolina. Accordingly, at the request of NRC staff, the USGS analyzed alternate water use rates at the VEGP site using its regional model to predict impacts and ground-water origins (Cherry and Clarke 2007). For those scenarios that examined the anticipated pumping rate for the proposed VEGP Units 3 and 4, the ground water appeared to originate in the upland areas of Georgia, with none of the recharge originating in South Carolina.

Water use data for a period of 20 years ending in the year 2000 suggest that withdrawal rates for surface water and ground water remained nearly unchanged (Fanning 2003) in the vicinity of the VEGP site. Projected water demand in Burke County, Georgia, indicates an increase of 50 percent by 2035 (Rutherford 2000). In South Carolina, analysts project an increase of 50 percent by 2045 (SC DNR 2004). However, despite these projections, a recent USGS report assigned lower ground-water pumping rates for the region in the future (i.e., through 2020) than have occurred during the recent drought (Cherry 2006, Figure 34). This suggests that stress on the ground-water resource was highest during the recent drought and could now diminish. Future demand includes production from the Water Table aquifer; however, wells in the Water Table aquifer are relatively low-production wells providing ground water for domestic use. Such wells exhibit a relatively local drawdown and, when located on the VEGP property boundary, are so distant from the proposed powerblock area that they would not substantially influence the elevation of the water table or the pathway of accidental releases.

The aquifers of interest in the evaluation of safety-related issues are the unconfined or Water Table aquifer and the uppermost confined or Tertiary aquifer. The two aquifers are separated by the Blue Bluff Marl formation, which has a thickness of approximately 63 feet (SNC 2008b). An accidental release to ground water would contaminate the Water Table aquifer. It is possible, but perhaps unlikely, that hydraulic communication exists between the Water Table and Tertiary aquifers. However, such communication, if it exists, could lead to an accidental release reaching the Tertiary aquifer. The staff conducted a confirmatory analysis of this scenario and documented the results in Section 2.4.13 of this SER. Based on its review of available data on the piezometric levels of these aquifers, the NRC staff concludes that they are influenced by local changes in aquifer characteristics and water use and discharge locally to surface drainage systems that ultimately discharge to the Savannah River. Changes in ground-water use with a potential to affect regional ground-water characteristics (i.e., the deep confined or Cretaceous aquifer system) over the long term will not influence the safety-related analysis of the ground-water system, which focuses on the unconfined or Water Table aquifer.

Effects on Plant Foundations and Other Safety-Related Structures, Systems, and Components

The proposed VEGP Units 3 and 4 will have foundations for the containment and auxiliary buildings at elevation 180.5 feet MSL. The applicant's parameter for maximum water table elevation or design ground-water level is 165 feet MSL (SNC 2008a, Part 2, Table 1-1). The applicant based this ground-water level on monitoring of the unconfined aquifer over the past decade. The plant grade elevation is 220 feet MSL. Foundations of all safety-related structures will be on structural backfill that will be placed above the Blue Bluff Marl on an engineered fill. The excavated natural materials will be replaced with compacted structural fill with properties that provide an adequate factor of safety against liquefaction (SNC 2008a, Part 2, Section 2.5.4.8.3.1). The maximum ground-water level from the site parameter list for the plant fitting within the bounding parameters in the proposed permit application is 2 feet below the design grade elevation. Therefore, the safety-related structural requirement for a plant that fits within the bounding parameters in the proposed permit application located at the proposed VEGP site is a ground-water elevation less than 218 feet MSL.

Based on the maximum observed ground-water level of 165 feet MSL, the water table elevation of the unconfined aquifer will not contribute a buoyant force on the nuclear island structure, which will have a foundation elevation at or higher than 180.5 feet MSL. However, after construction activity and modification of surface condition of the area surrounding the safety-related plant structures, changes in land use and ground-water recharge will likely alter the elevation of the ground-water table.

As part of the SER with Open Items, the NRC staff wrote, "The applicant should provide an improved and complete description of the current and future local hydrological conditions, including alternate conceptual models, to demonstrate that the design bases related to groundwater-induced loadings on subsurface portions of safety-related SSCs would not be exceeded. Alternatively, the applicant can provide design parameters for buoyancy evaluation of the plant structures." This was Open Item 2.4-2.

In response, the applicant has provided additional data from COL borings, revised its interpretations of data sets, and developed a ground-water model of the Water Table aquifer. The applicant's model of the Water Table aquifer (SNC 2008b, 2008c) includes an evaluation of current, spatially varying recharge patterns and of post-construction changes to recharge resulting from changes in land use and vegetation. These additional data and analyses have allowed the NRC staff to evaluate alternative conceptual models, alternative directions of ground-water movement, and the effects of ground-water-induced loadings on subsurface portions of safety-related SSCs.

The NRC staff used the applicant's model and analyzed a higher post construction recharge assignment to the powerblock and cooling tower areas, along with a lower hydraulic conductivity assignment to the engineered backfill in the excavated region of the powerblock area. Using a hypothetical high recharge rate of half of the precipitation (i.e., 24 inches/year) and a low hydraulic conductivity in the engineered backfill (i.e., the minimum of observed values in engineered backfill for VEGP Units 1 and 2 or 1.3 feet/day), the predicted hydraulic head was still below the foundations of all proposed structures and well below the design requirement of a plant that fits within the bounding parameters in the proposed permit application (i.e., a maximum water table elevation of 218 feet MSL). Therefore, based on its independent analysis, the NRC staff finds the applicant's site characteristic value for the maximum ground-water elevation at the VEGP site to be acceptable. This elevation will be far enough below the site grade so as to not represent a safety concern for the plant fitting within the bounding parameters proposed in the application. This analysis by NRC staff enables closure of Open Item 2.4-2. Therefore, Open Item 2.4-2 is closed.

Reliability of Ground-Water Resources and Systems Used for Safety-Related Purposes

Any plant that fits within the bounding parameters provided in the proposed permit application will not need ground water for safety-related use. Therefore, the NRC staff did not evaluate the reliability of the ground-water source for safety-related use. The NRC staff determined that the proposed VEGP Units 3 and 4 will have no SSCs that rely on ground water for a safety-related use other than initial filling and occasional makeup to water storage tanks associated with the passive containment cooling system.

Reliability of Dewatering Systems

The applicant proposed no permanent dewatering systems as part of the operation of the proposed VEGP Units 3 and 4. On the basis of the field data and the applicant's ground-water model results, as well as its own modeling efforts, the NRC staff concludes that a permanent dewatering system will not be required for a future plant fitting within the bounding parameters provided in the proposed permit application.

2.4.12.4 Conclusion

As set forth above, the applicant has substantiated sufficient information pertaining to the identification and evaluation of the effects of ground water in the vicinity of the proposed site. Section 2.4.12, "Groundwater," of RS-002 directs the applicant to address in the SSAR the requirements of 10 CFR Part 52 and 10 CFR Part 100 as they relate to identifying and evaluating the effects of ground water in the vicinity of the site and site regions. Furthermore, the applicant considered the most severe natural phenomena historically reported for the site and surrounding area while describing the hydrologic interface of the plant with the site with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this site characteristic, as documented in the SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in a site characteristic containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the NRC staff considers the identifed site characteristic for the highest ground water elevation to be acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application.

Therefore, the NRC staff concludes that the identification and consideration of the ground-water elevation characteristic set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.23(d)(4). In view of the above, the NRC staff finds the proposed hydrology-related site characteristic to be acceptable for inclusion in an ESP for the applicant's site.

2.4.13 Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters

2.4.13.1 Introduction

This section of the applicant's SSAR evaluates the hydrogeological characteristics of the site in terms of the effects of accidental releases of radioactive liquid effluents in ground and surface waters on existing uses and known and likely future uses of ground and surface water resources. The NRC staff's review of the applicant's SSAR, described in this section, addresses only accidental releases of radioactive liquid effluent with regard to surface and subsurface site characteristics. The NRC staff's review of the SSAR covers (1) alternate conceptual models, (2) characteristics that affect transport, (3) pathways, and (4) consideration of other site-related evaluation criteria.

This section of the SER reviews the applicant's process to identify and quantify the accidental radioactive liquid effluent release, its pathway to the accessible environment, and its migration and attenuation in surface waters and ground waters.

2.4.13.2 Regulatory Basis

The acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100. The NRC staff considered the following regulatory requirements in reviewing the identification of potential hazards in the site vicinity:

- 10 CFR 52.17(a) requires the application to contain information regarding the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit.

- 10 CFR 100.20(c) requires that the review take into account the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit.

- 10 CFR 100.21(d) requires that the physical characteristics of the site (including seismology, meteorology, geology, and hydrology) must be evaluated and site parameters established such that potential threats from such physical characteristics will pose no undue risk to the type of facility to be located at the site.

Section 2.4.13 of RS-002 provides the following criteria that the NRC staff used to evaluate this SSAR section:

- Compliance with 10 CFR Part 52 and 10 CFR Part 100 requires that the NRC consider the local geologic and hydrologic characteristics when determining the acceptability of a site to host a nuclear unit. The geologic and hydrologic characteristics of the site may have a bearing on the potential consequences of radioactive materials escaping from a nuclear unit of specified type that might be constructed on the proposed site. An applicant should plan special precautions if a reactor will be located at a site where a significant quantity of radioactive effluent could accidentally flow into nearby streams or rivers or find ready access to underground water tables.

- These criteria apply to RS-002, Section 2.4.13, because the reviewer evaluates a site's hydrologic characteristics with respect to the potential consequences of radioactive materials escaping from a nuclear unit of specified type that might be constructed on the proposed site. The review considers the radionuclide transport characteristics of ground water and surface water environments with respect to accidental releases to ensure that current and future users of ground water and surface water are not adversely affected by an accidental release from a nuclear unit of specified type that might be constructed on the proposed site. RG 1.113, Revision 1, "Estimating Aquatic Dispersions of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I," issued April 1977, and RG 4.4, "Reporting Procedure for Mathematical Models Selected to Predict Heated Effluent Dispersion in Natural Water Bodies," issued May 1974, provide guidance in the selection and use of surface water models for analyzing the flow field and dispersion of contaminants in surface waters.

- Meeting the requirements of 10 CFR Part 52 and 10 CFR Part 100 provides reasonable assurance that accidental releases of liquid effluents to ground water and surface water, and their adverse impact on public health and safety, will be minimized.

- To judge whether the applicant has met the requirements of 10 CFR Part 52 and 10 CFR Part 100 with respect to accidental releases of liquid effluents, the NRC uses the following criteria:

  — The applicant should describe radionuclide transport characteristics of the ground-water environment with respect to existing and future users. In addition, the applicant should describe estimates and bases for coefficients of dispersion, adsorption, ground-water velocities, travel times, gradients, permeabilities, porosities, and ground-water or piezometric levels between the site and existing or known future surface water and ground-water users. These estimates and bases should be consistent with site characteristics. The application should identify potential pathways of contamination to ground-water users and describe and reference data sources.

  — The applicant should describe transport characteristics of the surface water environment with respect to existing and known future users for conditions which reflect worst-case release mechanisms and source terms to postulate the most pessimistic contamination from accidentally released liquid effluents. The applicant should also describe estimates of physical parameters necessary to calculate the transport of liquid effluent from the points of release to the site of existing or known future users. The application should identify potential pathways of contamination to surface water users and describe and reference sources of information and data. The NRC staff will base its acceptance on its evaluation of the applicant's computational methods and the apparent completeness of the set of parameters necessary to perform the analysis.

  — Mathematical models are acceptable to analyze the flow field and dispersion of contaminants in ground water and surface water, providing that the models have been verified by field data and use conservative site-specific hydrologic parameters. Furthermore, conservatism should guide the selection of the proper model to represent a specific physical situation. Radioactive decay and sediment adsorption may be considered, if applicable, providing that the adsorption factors are conservative and site specific. RG 1.113 guides in the selection and use of surface water models. RS-002 discusses the transport of fluids through porous media.

2.4.13.3 Technical Evaluation

This section consists of (1) a review of the information provided by the applicant and (2) the NRC staff's evaluation of the applicant's submittal.

2.4.13.3.1 Technical Information Presented by the Applicant

In Section 2.4.13 of the SSAR, Revision 4-S2 (SNC 2008b), Southern presented information and data describing a postulated accidental release of radioactive liquid effluents in ground water and surface water. Southern also described (1) the conceptual models of the site, (2) characteristics that affect radionuclide transport, (3) contamination pathways, and (4) other site-related evaluation criteria.

In SSAR Section 2.4.13.1.1, the applicant selected the accident scenario from the information provided by the reactor vendor for the future plant fitting within the bounding parameters provided in the SSAR. The accident scenario is an instantaneous release from an effluent holdup tank located at the lowest level of the auxiliary building within the powerblock area (SNC 2008b). The applicant stated that the effluent holdup tank has a volume of 28,000 gallons, and a postulated rupture leads to a loss of 80 percent of that volume or 22,400 gallons in accordance with Branch Technical Position (BTP) 11-6. In its analysis, the applicant assumed that the release instantaneously enters the backfilled region of the Water Table aquifer, which underlies the auxiliary building, and displaces all pore water in a space 21 feet wide, 21 feet long, and 20 feet deep.

The applicant presented field observations of the current Water Table aquifer and a model of the aquifer in a variety of post construction settings to conclude that ground water will flow north in the future from the proposed powerblock area toward Mallard Pond (SNC 2008b, Figure 2.4.13-1). Southern concluded that the most critical release pathway in the ground-water environment will be from the proposed VEGP Unit 4 auxiliary building northward to the south side of Mallard Pond. The travel distance scaled from the curvilinear pathway shown in Figure 78 of Appendix 2.4B (SNC 2008b) revealed an approximate distance of 2550 feet; 150 feet through backfill, 1200 feet through undisturbed aquifer to a point south of observation well OW-1005, and an additional 1200 feet to the south side of Mallard Pond through an undisturbed but higher conductivity segment of aquifer. Using a ground-water model of the Water Table aquifer to trace the pathway of contaminants, the applicant reported travel times through the three curvilinear aquifer segments of 2.4, 3.2, and 1.1 years, respectively, for a total travel time of 6.7 years from the release point below the auxiliary building to Mallard Pond.

In SSAR Section 2.4.12.1.4 (SNC 2008b), the applicant reported hydraulic properties of the Barnwell Formation sediments used in the safety analyses and included the range of hydraulic conductivity measurements for the Utley Limestone from 3,250 to 125,400 feet/year (9 to 343 feet/day). The applicant also derived a value for effective porosity of 0.34 (SNC 2008b, Section 2.4.12.1.4) from the median specific gravity and moisture content measurements. The applicant estimated a maximum hydraulic gradient of 0.014 feet/feet to apply to the Water Table aquifer in the vicinity of the proposed Units 3 and 4 (SNC 2008b, Section 2.4.12.1.3). A maximum gradient of 0.023 feet/feet can be derived from the hydraulic head data for the aquifer between OW-1005 and Mallard Pond. The applicant used the ground-water model and estimated the travel times for the last two segments in the aquifer as 3.2 and 1.1 years respectively, for a total of 4.3 years.

In SSAR Section 2.4.12.1.4 (SNC 2008b), the applicant reported the range of measured hydraulic conductivity values in the engineered backfill as 480 to 1220 feet/year (1.3 to 3.3 feet/day). As reported in UFSAR Table 2.4.12-14, the applicant obtained these values from the prior postconstruction testing of backfill regions underlying existing VEGP Units 1 and 2 (SNC 2003). The applicant also estimated the backfill porosity to be 0.34 based on information from the UFSAR (SNC 2003). An estimate of the hydraulic gradient in the engineered backfill is the same as in the surrounding Water Table aquifer, a maximum estimated value of 0.014 ft/ft. The applicant used the ground-water model and estimated the travel time to be 2.4 years.

The applicant also postulated an alternative release pathway from the powerblock area through the Tertiary aquifer to the Savannah River (SNC 2008b, Figure 2.4.13-2). In SSAR Section 2.4.12.1.4, Table 2.4.12-3, the applicant reported hydraulic properties of the Tertiary aquifer sediments (SNC 2008b) used in the safety analyses and included a range of hydraulic conductivities from 0.35 to 2.1 feet/day, with a geometric mean of 0.83 feet/day and an effective porosity of 0.31. The applicant estimated a maximum hydraulic gradient of 0.005 feet/feet to apply to a distance of 5600 feet between the center of the powerblock and the Savannah River (SNC 2008b, Section 2.4.12.1.4). Based on the geometric mean of the hydraulic conductivity, the maximum gradient, and the effective porosity, the applicant estimates the travel time to be 1142 years.

As the applicant described, Mallard Pond is controlled by a combination of standpipe and spillway with discharge to a stream that ultimately discharges to the Savannah River (SNC 2007c, 2008b). The applicant identified two companies as the nearest downstream industrial surface water users; both withdraw water from the Savannah River and are located near River Mile 45, about 106 miles downstream of VEGP (SNC 2008b, Section 2.4.13.1.2.1).

For the Mallard Pond drainage pathway, the applicant's analysis considered (1) radionuclide decay associated with travel times in the ground-water pathway, (2) adsorption and decay during a retarded travel time for sorbed radionuclides in the groundwater pathway and the dilution of the ground water released to Mallard Pond (i.e., 0.094 gallons/ per minute) in the stream below the pond (i.e., 1125 gallons/minute). The applicant performed analytical tests to estimate distribution coefficients for cobalt, strontium, and cesium. The minimum values of the distribution coefficient from 16 soil samples, identified by the applicant as being representative of backfill material, were 1.4 milliliters per gram (mL/g) for cobalt, 6.0 mL/g for strontium, and 3.5 mL/g for cesium. Minimum values from three samples of aquifer materials, identified by the applicant as being representative of Barnwell Group sediments, were 3.9 mL/g for cobalt, 14.4 mL/g for strontium, and 22.7 mL/g for cesium. Ground-water wells withdrawing aquifer water did not intercept either of the pathways analyzed by the applicant.

In RAI 2.4.13-2 (SNC 2007c), the NRC staff requested that the applicant evaluate the potential for chelation and complexation agents (e.g., organic acids) to mix with radiological liquid effluents and adversely impact sorption phenomena. The NRC staff requested that the applicant clearly state whether or not mixing with chelation agents was possible. In its RAI response (SNC 2007c), the applicant stated that the site does not prohibit the use of chelating agents, but does require a comprehensive evaluation before their use. The applicant stated that it will tightly control any future use of chelating agents at VEGP and that it does not anticipate using chelating agents if they could come in contact with radioactive materials. In summary, the applicant stated that it would be extremely unlikely for radioactive liquids to come into contact with chelating agents.

In RAI 2.4.13-3 (SNC 2007c), the NRC staff requested that the applicant more fully describe the basis for the estimated ground-water flow into Mallard Pond and provide all data supporting the dilution of the release in surface water flow within the Mallard Pond drainage. In SSAR Section 2.4.13.1.3.1, the applicant fully described the ground-water release (SNC 2008b) and provided a calculation package detailing the measurements made for Mallard Pond and its downstream drainage (SNC 2007c). This calculation package, dated September 27, 1985, documents field observations made during June and July of 1985. These measurements represent single moment-in-time measurements. The applicant's calculation package states that the discharge downstream of the confluence of the Mallard Pond drainage and West Branch drainage ranges from 800 to 1200 gallons/minute (SNC 2007c). The applicant used a discharge rate of 1125 gallons/minute in calculations of the release dilution (SNC 2008b, Section 2.4.13.1.3.1). After the NRC issued the SER with Open Items, the applicant developed a ground-water model of the Water Table aquifer and provided simulations of postconstruction events that better describe future ground-water flow in the vicinity of the proposed VEGP Units 3 and 4 (SNC 2008b, Appendix 2.4B)

Of the 56 radionuclides in the effluent holdup tank inventory (SNC 2008b, Table 2.4.13-1), the applicant only identified 10 that will require more than decay in the ground-water pathway to be reduced to less than 1 percent of their maximum effluent concentration limits (ECLs) (SNC 2008b). The 10 radionuclides were H-3, Mn-54, Fe-55, Co-60, Sr-90, Ag-110m, I-129, Cs-134, Cs-137, and Ce-144.

In SSAR Section 2.4.13.1.3.1, the applicant identified eight radionuclides that will require more than decay and adsorption in the ground-water pathway to be reduced to less than 1 percent of their ECLs (SNC 2008a). Distribution coefficients were only available for cobalt, strontium, and cesium. Following inclusion of adsorption and decay associated with retarded travel time, the applicant identified the remaining eight radionuclides requiring further analysis as H-3, Mn-54, Fe-55, Sr-90, Ag-110m, I-129, Cs-137, and Ce-144.

The applicant applied dilution downstream of Mallard Pond to the decayed radioisotope concentrations entering Mallard Pond from the Water Table aquifer. The applicant's estimated concentration of each radioisotope downstream of the dilution location is below its respective ECLs. The highest contributor to dose is H-3, which, according to the applicant, represents nearly 6 percent of its ECL (SNC 2008b, Section 2.4.13.1.3.1, Table 2.4.13-5). The applicant calculated the cumulative measure, (i.e., the sum of all ratios), and reported 0.058, which is less than one and meets the requirement in Note 4 in Appendix B, "Annual Limits on Intake (ALIs) and Derived Air Concentrations (DACs) of Radionuclides for Occupational Exposure; Effluent Concentrations; Concentrations for Release Sewerage," to 10 CFR Part 20, "Standards for Protection against Radiation" (SNC 2008b, Section 2.4.13.1.4).

The applicant noted that it demonstrated compliance for a point along the stream within the restricted area which does not represent a potable water source. The applicant stated that the stream is a gaining stream (i.e., it does not discharge to ground water) which discharges to the Savannah River. The applicant identified the Savannah River as being the nearest potable water supply in an unrestricted area. The applicant indicated that a conservative representation of Savannah River flow is the 100-year drought flow of 3298 cubic feet/second (1,480,000 gallons/minute) while the tributary flow rate is 1125 gallons/minute, thus the additional dilution would further reduce radionuclide concentration by a factor of about a 1,000 (SNC 2008b, Section 2.4.13.1.4).

For the alternative Tertiary aquifer pathway mentioned above, the applicant stated that, using only the radioactive decay in the Tertiary aquifer pathway, the cumulative measure applied to ground-water quality before discharge to the Savannah River (i.e., the sum of all ratios) is 0.036. Therefore, this value is in compliance with 10 CFR Part 20 limits (SNC 2008b, Section 2.4.13.1.4).

In SSAR Section 2.4.13.2, the applicant stated that no outdoor tanks contain liquid radioactive waste in the reactor design under consideration; therefore, no accident scenario is projected to result in a liquid effluent release directly to the surface water environment (SNC 2008b).

2.4.13.3.2 Technical Evaluation

The NRC staff has divided its technical evaluation into four topics—alternate conceptual models, characteristics that affect radionuclide transport, contamination pathways, and contaminant transport analyses.

The applicant provided this section of the application to the NRC as a supplement to Revision 4 of the application (SNC 2008b). As a result of a series of requests, beginning at the initial site audit conducted in January 2007, the applicant has revised Section 2.4.13 of the SSAR with each revision of the application.

The staff issued an initial RAI on March 15, 2007, which asked the applicant to describe and discuss (1) the process followed to establish the conceptual model for the plausible transport pathways and travel times, (2) the process used to evaluate the potential of chelating agents (e.g., organic acids) that may combine with radionuclides and influence the movement of radionuclides in the environment, and (3) the process used to estimate the ground-water flux carrying an accidental release from the powerblock to Mallard Pond. Southern responded to these requests (SNC 2007c) and incorporated revisions into Revision 2 of the SSAR.

The NRC staff issued the SER with Open Items and included Open Item 2.4-3 asking that the applicant include an analysis providing assurance that it had considered an adequate number of combinations of release location and plausible alternative pathways. The NRC staff cited the inevitable change in site hydrology (e.g., changes in surface material and vegetation, slope, infiltration or recharge, runoff) as potentially significant in forecasts of aquifer response to construction of the proposed facility and potential future ground-water pathways. The NRC staff's analysis, which did not apply adsorption because of the potential impact of chelating agents, concluded that dilution in the Savannah River was required to meet the requirements of Table 2, Column 2, of Appendix B to 10 CFR Part 20. Accordingly, public access to ground water or surface water before its discharge to the river was an issue, and the staff included Open Item 2.4-4 requesting that the applicant specify the nearest point of public access along each potential pathway.

In response to these open items, the applicant developed a ground-water model of the Water Table aquifer and incorporated its description and results into Revision 3 of the SSAR. The applicant exercised the model using alternative combinations of the magnitude and distribution of recharge rates, the magnitude and distribution of hydraulic conductivity, and external and internal boundary conditions. In addition to revising the section to reflect the application of a ground-water model, the applicant better described the point of public exposure for each of the pathways analyzed.

The NRC staff's review of the ground-water model results in Revision 3 of the SSAR, as well as model input and output, revealed issues with model convergence, mass balance, and calibration bias. The NRC staff also noted that the applicant did not present alternative conceptual models. Instead, the applicant presented a sequence of models used to achieve calibration of a single conceptual model. The staff raised these concerns with the applicant at a public meeting at NRC in Rockville, Maryland, on April 8, 2008, at a site audit at the applicant's consultant's offices in Frederick, Maryland, on April 9, 2008, and in RAIs sent on July 22, 2008. The applicant addressed these concerns in its supplement to Revision 4 of the application (SNC 2008b) and in responses to the RAIs (SNC 2008c).

The applicant's analysis of radioactive liquid effluent pathways, which was originally based entirely on field data and the assumption that prior pathways would not be altered in the future, evolved into an analysis based on field data, as well as a model of the Water Table aquifer, enabling a more thorough analysis of plausible postconstruction conditions.

Alternate Conceptual Models

Transport of an accidental release of radioactive liquid effluent is viewed as a combinatorial problem with multiple possible environmental pathways. Among all plausible alternative conceptual models and pathways, the critical one results in the plausible yet conservative release consequence that is ultimately of interest for the site safety evaluation.

In general, the process of determining plausible pathways is uncertain because of spatially and temporally varying characteristics and because the release may occur in the future after substantial change has or may have occurred to the local landscape and near-field hydrology of the proposed site. This is even more important in the case of the VEGP site because it sits atop a ground-water divide and thus is very sensitive to changes in hydraulic conductivity and recharge. The existing hydrology of the site does not necessarily represent the future hydrology of the site. Construction of a large industrial facility such as the proposed nuclear power plants can lead to substantial change to the postconstruction landscape and hydrologic features of this site. These changes lead to alterations in the distribution of recharge in the vicinity of the proposed plants and in the water table of the aquifer underlying the proposed site.

The applicant developed a two-dimensional, single-layer, steady-state ground-water model of the Water Table aquifer underlying the VEGP site (SNC 2008b). Section 2.4.12 of this SER describes this model. Based on field data and the results of the simulation of seven alternative ground-water models, the applicant concluded that all contaminants released from the Nuclear Island area at the proposed VEGP Units 3 and 4 would move to the north and discharge to

Mallard Pond. Upon evaluation of the modeling results, the NRC staff concluded that this alternative pathway is perhaps the most plausible of alternative pathways. The applicant used model 7 to define, using tracer particles, plausible ground-water pathways and simulate the travel path from the proposed VEGP Unit 4 auxiliary building to the upper reach of Mallard Pond. Essentially, the ground water moved through three regions of the model—the saturated engineered backfill, the aquifer from the excavation (backfill) to the high conductivity zone above Mallard Pond, and through the aquifer's high conductivity zone to Mallard Pond. As described in Section 2.4.12, the applicant predicted travel times through the three zones to be 2.4 years, 3.2 years, and 1.1 years, respectively, for a total ground-water travel time of 6.7 years. Section 2.4.13.1.3.1 of the SSAR (SNC 2008b) further describes this pathway through the Water Table aquifer, which the NRC staff evaluates below.

The applicant presented an alternative ground-water pathway involving the Tertiary aquifer in Section 2.4.13.1.3.2 of the SSAR (SNC 2008b). The Blue Bluff Marl appears to be of substantial thickness and low hydraulic conductivity in the vicinity of the proposed construction; however, based on an alternative interpretation of field data (i.e., the possibility that ground water could move from the Water Table aquifer into the Tertiary aquifer) that cannot be completely excluded, the applicant evaluated a Tertiary aquifer pathway. The NRC staff considers this pathway to be plausible but unlikely. This pathway requires that a release to the Water Table aquifer be transported through the underlying mud unit, ultimately releasing to and moving through the confined Tertiary aquifer and discharging into the Savannah River opposite the site.

As described in Section 2.4.12 of this SER, the NRC staff used the applicant's model of the Water Table aquifer to evaluate the sensitivity of the model's solution to drain boundary condition elevations, to the use of minimum LiDAR data to define drainages, and to a variety of postconstruction conditions more extreme than those evaluated by the applicant. The staff used a matrix of recharge rates applied to the powerblock area and cooling tower area to explore the potential for change in the water table to yield alternative pathways for releases from the powerblock area. In addition, the staff evaluated the sensitivity of the simulation to the hydraulic conductivity of the backfill by assuming a less permeable or less conductive material. Using the matrix of recharge rates, the staff analyzed combinations of the following—powerblock area recharge high (i.e., half precipitation, 24 inches/year), expected (i.e., one-eighth precipitation, 6 inches/year), or zero, and cooling tower area recharge high ((i.e., half precipitation, 24 inches/year), expected (i.e., quarter precipitation, 12 inches/year), or zero.

A review of surface treatments and slopes within the powerblock and cooling tower areas reveals that it is unlikely that recharge rates inside a powerblock area would ever be greater than those inside a cooling tower area. Slopes, surface materials, and surface water control structures within the powerblock area are designed to conduct water away, especially during high precipitation events. Lesser slopes, gravel-covered surfaces, and surfaces maintained free of vegetation are typical of cooling tower areas, and all substantially increase the potential for recharge, especially during normal precipitation events. Accordingly, cases involving high and expected, high and low, and expected and low recharge for the powerblock and cooling tower areas, respectively, are implausible.

Given the historical measurements of the Water Table aquifer, as well as the natural flow and discharge of the Water Table aquifer to surrounding ravines or drainages, at least four potential ground-water pathway directions could be evaluated relative to the plausible combinations of recharge and hydraulic conductivity that contribute to a calibrated model. These potential ground-water pathways include ground-water flow from the powerblock toward (1) the Mallard Pond drainage, (2) the Daniels Branch drainage, (3) the Savannah River, and (4) an unnamed drainage located south of the VEGP Units 1 and 2 cooling towers. The applicant-produced ground-water model (SNC 2008b, 2008c, Appendix 2.4B) served as the starting point for the analysis. This model reproduces the general magnitude and location of the present-day ground-water high and surrounding contours. The staff then made perturbations to recharge rates and hydraulic conductivity to evaluate alternative pathways.

For all plausible recharge rate cases, as well as in the case of a lower conductivity backfill material, no ground-water pathway beginning inside the proposed powerblock area resulted in a simulated discharge to the Daniels Branch drainage or to the drainage located south of the VEGP Units 1 and 2 cooling towers. The high recharge cases with both maximum- and minimum-field-measured backfill hydraulic conductivity values did yield pathways that flow under the upper reaches of the Daniels Branch; however, the ground water was simulated to be below the streambed and it did not discharge into the Daniels Branch. In these same two cases, Water Table aquifer pathways were simulated that discharged into the Savannah River; however, this is an artifact of the case and not necessarily realistic. The model assigned higher recharge rates to the VEGP Units 3 and 4 powerblock and cooling tower areas than to the comparable VEGP Units 1 and 2 areas. If the model treated all powerblock and cooling tower areas similarly, the resulting higher water table that would underlie VEGP Units 1 and 2 would preclude ground-water movement directly towards the Savannah River from the VEGP Units 3 and 4 powerblock. For all plausible recharge rate cases, the majority of pathway traces showed ground-water movement to the north and traces beginning inside the powerblock area released to Mallard Pond.

However, the NRC staff postulated plausible pathways by conservatively extending the release points outside the proposed power block area. Based on measured hydraulic heads, site topography, and model simulations, the NRC staff concludes that, of the four possible ground-water pathways in the Water Table aquifer leading to the receptor, the Mallard Pond drainage pathway is the most plausible, the Daniels Branch drainage pathway is plausible but perhaps unlikely, the Savannah River drainage pathway is implausible, and the drainage to the south of VEGP Units 1 and 2 cooling towers is implausible. The decision to categorize the Daniels Branch drainage as plausible but unlikely results from (1) the ability to configure a relatively simple model and create pathways from the proposed powerblock area to ground water underlying the upper reaches of the Daniels Branch drainage, (2) uncertainty in future recharge rates and their spatial distribution, and (3) uncertainty in the magnitude and spatial distribution of the hydraulic conductivity of the Barnwell Group sediments, including the Utley Limestone, in the vicinity of the proposed facility. Thus, the uncertainties that exist with regard to the existing hydrogeological setting and future conditions require the NRC staff to conclude that the Daniels Branch pathway is plausible but perhaps unlikely. The possible Water Table aquifer pathways toward the Savannah River and toward the drainage located south of the VEGP Units 1 and 2 cooling towers did not conform to known aspects of the field setting; therefore, the staff determined that they were implausible. The following sections on the characteristics that affect transport and pathways evaluate the pathways found to be plausible in terms of their compliance with 10 CFR Part 20, Appendix B, Table 2.

The applicant provided parameters for an accidental release, including the tank, its relative location in the facility, its volume, and its contents. The applicant specified a single possible location for the accidental release of radioactive liquid effluents. The NRC staff postulated that a release could occur anywhere within the powerblock area. This assumption allows the identification of all potential alternative pathways and the selection of the most critical ones to conservatively estimate accidental release consequences.

The NRC staff found that the applicant's analysis in the SSAR was sufficient with respect to data (e.g., both past and present) and with respect to the model developed, thus enabling the staff to perform its evaluation. However, the NRC staff concluded that the additional ground-water pathway it identified previously (i.e., the pathway from the proposed powerblock area to the Daniels Branch drainage) is plausible. In the SER with Open Items, the NRC wrote that the applicant's SSAR, Revision 2, was incomplete because it did not consider the inevitable change in hydrology, and, hence, the potential change in flow direction within the Water Table aquifer for some release locations within the powerblock area. The analysis provided no assurance that the applicant had considered an adequate number of combinations of release locations and feasible pathways. This was Open Item 2.4-3. The applicant did develop and apply a model of the Water Table aquifer and has included ground-water pathways in both the Water Table and Tertiary aquifer. Therefore, Open Item 2.4-3 is closed.

Characteristics that Affect Transport

The NRC staff independently determined that the USGS-derived minimum and maximum range of transmissivity values based on field data (i.e., 500 to 9500 feet^2/day) (Clarke and West 1998, Table 3), when combined with the local thickness of the Water Table aquifer (i.e., approximately 30 feet), provide hydraulic conductivities ranging from 16.5 to 316 feet/day that are indicative of the values for the Utley Limestone of the Barnwell Formation cited by the applicant (i.e., 3,250 to 125,400 feet/year or 9 to 343 feet/day based on aquifer tests (SNC 2008a, Section 2.4.12). In model 7, the applicant identified hydraulic conductivity values of 32, 100, and 8 feet/day applied to three zones of the Water Table aquifer. The applicant assigned the majority of the model domain a value of 32 feet/day; it assigned a zone immediately upgradient of Mallard Pond a value of 100 feet/day, and it assigned the southwestern quadrant of the model domain the low value of 8 feet/day. A sensitivity case based on model 7 used hydraulic conductivity values of 25, 65, and 5 feet/day and divided the center of the model into a low and high zone; the remainder of the model was assigned the middle value. In this case, the applicant assigned the majority of the model domain associated with Utley Limestone the highest value, 65 feet/day, and assigned a small zone between the proposed location of the VEGP Units 3 and 4 the lowest value, 5 feet/day. Overall, the NRC staff found the model values to be comparable to the applicant data and USGS values of hydraulic conductivity.

The NRC staff reviewed the applicant's prior estimates of the magnitude of the hydraulic gradient (i.e., 0.014 and 0.023 for the backfill to OW-1005 segment and the OW-1005 to Mallard Pond segments, respectively), effective porosity (i.e., 0.34 and 0.31), and ground-water flux (i.e., 0.094 gallons/minute into Mallard Pond) and found them appropriate for simple, conservative effluent transport analyses. Ultimately, the applicant used the model-based values of hydraulic conductivity and hydraulic gradient to derive travel time along a pathway. The beginning of this section and the entirety of Section 2.4.12 summarize the NRC staff's review of the applicant's ground-water model. On the basis of its review, the staff concludes that the ground-water model exhibits mass balance and convergence.

The NRC staff reviewed the hydraulic properties assigned by the applicant to the engineered backfill. The applicant's analysis of transport characteristics in the engineered backfill relies on the observed maximum hydraulic conductivity of the existing units' engineered backfill (1220 feet/year, 3.3 feet/day) and the estimated values of effective porosity (0.34) and hydraulic gradient taken from the Water Table model. The NRC staff also used the minimum measured hydraulic conductivity (480 feet/year or 1.3 feet/day) in sensitivity analyses. The staff notes that the entire range of hydraulic conductivity for the backfill is below the range applied in the model to the natural sediments of the Water Table aquifer. This is not unexpected given the relatively high compaction and well-graded sediments of the backfill material, especially compared to portions of the Barnwell Group sediments, including the Utley Limestone, which are known to be more conductive.

Regarding the applicant's reported values of hydraulic conductivity in the Tertiary aquifer, the NRC staff independently reviewed the USGS minimum and maximum ranges of transmissivity estimates based on field data (180 to 12,200 feet2/day) and regional simulation (13 to 24,700 feet2/day) (Clarke and West 1998, Table 12). When combined with the local thickness of the Tertiary aquifer (approximately 182 feet), the USGS data, while being generally higher, do bracket the central value of hydraulic conductivity provided by the applicant (i.e., 0.83 feet/day). The NRC staff reviewed the applicant's estimates of the magnitude of the hydraulic gradient (i.e., 0.005) and effective porosity (i.e., 0.309). Ultimately, the NRC staff's use of the highest observed transmissivity value attributed to the Tertiary aquifer (i.e., 2.1 feet/day) ensures a conservative estimate of pore-water velocity and travel time (i.e., 450 years). The NRC staff notes that the applicant employed the geometric mean of the hydraulic conductivity values (i.e., 0.83 feet/day) and an effective porosity of 0.309 and calculated a travel time of 1142 years. Such a value represents the central tendency of the travel time and should not be viewed as overly conservative.

The applicant has not stated that it will avoid the use of complexants or chelating agents at the proposed VEGP Units 3 and 4. In response to RAI 2.4.13-2 (SNC 2007c), Southern indicated that it does not prohibit the use of chelating agents; rather it requires a comprehensive evaluation prior to use. Southern's statements suggest that, while it stopped routine use of chelating agents a number of years ago, circumstances could result in a mixture of chelating agents and radioactive liquid effluent. Accordingly, the NRC staff's analysis assumed that complexants or chelating agents may be present.

The NRC staff reviewed the applicant's estimate of streamflow necessary to dilute the radiological effluent released through the Water Table aquifer into Mallard Pond after an accident. For the streamflow dilution, the applicant used a measured streamflow of 1125 gallons/minute at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch, which is a single moment-in-time measurement made in June and July 1985. The NRC staff determined that a lower streamflow than that measured by the applicant is feasible. Because the data were not gathered during the most severe drought of record (USACE 2006), the NRC staff concludes that it is reasonable to assume that the discharge from Mallard Pond could cease entirely for a period of time. It should also be noted that the stream downstream of Mallard Pond crosses the VEGP property boundary and then reenters the VEGP property before discharging to the Savannah River (SNC 2008b, Section 2.4.13.1.4). Thus, the discharge from Mallard Pond enters the public domain before its discharge to the Savannah River.

The applicant stated that the magnitude of the 100-year drought flow of the Savannah River was 3298 cubic feet per second (cfps) (1.48x106 gallons/minute). The minimum release from Thurmond Dam is currently set at 3600 cfps (1.616x106 gallons/minute) by the U.S. Army Corps of Engineers. A USGS streamflow gauge near the VEGP site shows higher flows, suggesting that at low flows the Savannah River actually picks up some additional flow between Thurmond Dam and the VEGP site. These additional flows are contributed by and consistent with tributary and ground-water discharges flowing into the Savannah River. The staff determined that, based on the above, 3600 cfps is a conservative estimate of monthly and annual flows.

The applicant believes that the drainage below Mallard Pond, when it enters the Hancock Landing property, does not represent a potable water supply and that 10 CFR Part 20 requirements do not apply. The applicant identified the Savannah River as the potential water supply to which 10 CFR Part 20 compliance applies and identified the closest surface water withdrawal downstream of the release as two industrial surface water users, both located about 106 miles downstream of the VEGP site. However, the NRC staff does not concur with this selection and instead determined based on the information provided by the applicant that the intersection between the Creek below Mallard Pond and the Hancock Landing property is the point of compliance. The staff evaluated both points of compliance and determined that for both points, 10 CFR Part 20 limits can be met. In addition, although the staff disagrees with the applicant's point of compliance for 10 CFR Part 20 limits, the staff concurs that the applicant adequately demonstrated that 10 CFR Part 20 limits can be met downstream of Mallard Pond, inside the exclusion area boundary (i.e. before reaching an unrestrictred area).

Contamination Pathways

To bound the most severe radiological consequences of radioactive liquid effluent release, the NRC staff postulated plausible alternative pathways to the accessible environment. The NRC staff concludes that the Mallard Pond drainage would likely intercept most accidental release pathways originating inside the powerblock area of the proposed VEGP Units 3 and 4. However, the future direction of ground-water flow within the Water Table aquifer may change, and it is not unreasonable to expect that some accidental release locations within the powerblock area could result in releases moving to the west and south. Such releases could flow into the upper reaches of the Daniels Branch drainage and ultimately to the Savannah River. Another feasible accidental release pathway would involve transport from the Water Table aquifer into the Tertiary aquifer, with subsequent migration toward and discharge into the Savannah River from the Tertiary aquifer. The NRC staff concludes that these three pathways represent plausible alternate pathways for the transport of an accidental release of radioactive liquid effluents and analyzed all three.

The NRC staff reviewed the Mallard Pond drainage accidental release pathway postulated by the applicant, and, assuming no credit for adsorption because of the potential presence of chelating agents, concludes that such a release and pathway analysis would require inclusion of release and dilution into the Savannah River to ensure that radionuclide concentrations meet site suitability requirements (10 CFR Part 20, Appendix B, Table 2).
The postulated release posed by the applicant is conservative because it ignores the leak containment and detection systems associated with the effluent holdup tank; the integrity of the engineered system, including the foundation of the auxiliary building; the time required to move through the vadose zone; the dispersal of contaminants in the vadose zone and aquifer; and the opportunity to remediate contaminant plumes in the ground-water environment.

Contaminant Transport Analysis

The NRC staff reviewed the applicant's calculations regarding the inventory, its accidental release, and its decay, adsorption, and dilution during transport through the environment. The NRC staff concludes that the applicant's use of adsorption to allow additional decay of cobalt, strontium, and cesium isotopes during retarded travel times was not warranted given the potential for chelating agents to be present. The NRC staff also concludes that neither the analysis nor the data adequately support the flow measurements and dilution calculations performed by the applicant for the Mallard Pond drainage north of the proposed VEGP Units 3 and 4. Consequently, it is reasonable to assume that flow from Mallard Pond ceased in the past and could cease in the future during times of extreme drought because of the standpipe discharge control structure. Neglecting adsorption and onsite dilution, the NRC staff determined that release from the drainage to the Savannah River will require mixing with approximately 10 percent of the Savannah River low flow (i.e., 160,000 gallons/minute) to achieve concentrations meeting the site suitability requirements (i.e., a sum of fractions less than one).

The NRC staff considered alternate subsurface conceptual models and release locations, with the release moving in another direction (e.g., towards the southwest), and determined that a pathway leading to the upper reaches of the Daniels Branch drainage was plausible but unlikely. As in the case of the Mallard Pond drainage analysis, the potential presence of chelating agents precludes the application of adsorption phenomena, and the release could not meet the 10 CFR Part 20 requirements before reaching the site boundary. Such a pathway (i.e., the Daniels Branch drainage) could pose a greater threat than the Mallard Pond drainage pathway quantified by the applicant in SSAR Section 2.4.13 (SNC 2008b).

The NRC staff concludes that, in addition to alternate conceptual models involving the direction of ground-water flow in the Water Table aquifer, an alternate conceptual model exists that suggests possible local communication between the unconfined Water Table aquifer and the confined Tertiary aquifer. The NRC staff determined that limited evidence indicates the possibility of a local hydraulic flaw in the aquitard separating these two aquifers. If an accidental release from the proposed VEGP Units 3 and 4 were to be intercepted by such a local communication region of the Water Table aquifer, then the staff concludes that the release could move into the Tertiary aquifer and move toward and discharge into the Savannah River. Using the maximum hydraulic conductivity cited by the applicant for the Tertiary aquifer, the shortest travel time to the river would be approximately 450 years. After accounting for decay during this travel time, of all radionuclides listed (SNC 2008b, Table 2.4.13-1), only I-129 and

Cs-137 would require future concentration reduction by mixing or dilution in the Savannah River. The NRC staff determined that dilution in only 76 gallons/minute of flow in the Savannah River (i.e., less than 0.005 percent of the 3600 cfps low flow) would be required to achieve the level of less than 1 percent of their ECLs. In this instance, the hierarchical process followed by the NRC staff to evaluate alternate conceptual models yields a release that is of less consequence than either a release through Mallard Pond or to the Daniels Branch drainage.

When the SER with Open Items was released, the NRC staff's review of the release location, migration, attenuation, and dilution of the radioactive liquid effluent release was incomplete. As stated in Open Item 2.4-3, the applicant had not considered a sufficient number of alternate conceptual models to identify potential release points and pathways. In addition, the analysis of the Mallard Pond drainage pathway raised an issue concerning the point of compliance, and the staff required the applicant to specify the nearest point along each potential pathway that was accessible to the public. This was Open Item 2.4-4. Later, the applicant provided the analysis of pathways and radionuclide transport through Revision 4 (2008b) and the response to RAIs (2008c). Also, the applicant provided a map of the site boundary and noted that the stream draining the Mallard Pond drainage does leave the site and reenters it before discharging to the Savannah River. It is also clear from the applicant's map that the stream draining to the upper reaches of the Daniels Branch leaves the site just before entering Lower Debris Basin 2. Therefore, Open Item 2.4-4 is closed.

The NRC Staff conducted a further analysis of the Mallard Pond and upper Daniels Branch drainages. The staff determined the catchment areas for both watersheds and applied monitored runoff rates from unregulated watersheds in the region to estimate the minimum monthly runoff rate for the Mallard Pond and upper Daniels Branch drainages. The catchment areas were based on standard 10-meter resolution USGS digital elevation models (DEMs) acquired from the U.S. Department of Agriculture Geospatial Gateway. The DEM for each catchment was checked for anomalous sinks or peaks and processed to produce flow direction and flow accumulation data. The staff identified a drainage outlet location at the intersection of the drainage channel and site boundary. Using these inputs, the staff used the ArcGIS "watershed" function to trace the catchment boundary and determine the catchment area. The area of the Mallard Pond catchment was 3.266 square kilometers, and the upper Daniels Branch catchment was 3.122 square kilometers. The staff used stream gauge data from six unregulated watersheds in Georgia and South Carolina to quantify the runoff from the VEGP watersheds. One gauge had a duration of record from 1929 to present, another from 1949 to present, and all others were of relatively short duration. The staff determined streamflow or runoff as a function of watershed area for these watersheds and defined the minimum watershed flow as the average of the lowest 12-month period. In other words, the staff used a 12-month floating window to search the data and define the 12-month period with the lowest annual flow of record. The average flow for that year was considered to be the minimum watershed flow. The minimum watershed flow for the Mallard Pond drainage was 279 gallons/minute, and for the upper reaches of the Daniels Branch drainage it was 267 gallons/minute.

The migration and fate of an accidental release of a radioactive liquid effluent can be estimated by assuming that (1) migration from the engineered backfill is the same or nearly the same for both pathways, (2) chelating agents are not present, and therefore, the minimum measured distribution coefficients are assumed to conservatively represent cobalt, strontium, and cesium movement, and (3) the runoff measured at other nearby unregulated watersheds is an appropriate surrogate for minimum annual runoff at watersheds on and adjacent to the VEGP site. For the analysis of the Mallard Pond drainage, key data include the travel times through the backfill and aquifer (i.e., 2.4 and 4.3 years (SNC 2008b)), the ground-water flux from the engineered backfill carrying the radioactive contamination, (i.e., 0.094 gallons/minute (SNC 2008b)), and the minimum distribution coefficients for backfill and aquifer materials (see FSAR Table 2.4.13-3 (SNC 2008b)). The resulting sum of fractions, where the fraction is the ratio of radionuclide concentration to its effluent concentration limit, is 0.235, which is below the requirement of one (10 CFR Part 20, Appendix B, Table 2).

For the analysis of the upper reach of the Daniels Branch drainage, key data include the travel times through the backfill and aquifer, the ground-water flux from the backfill, and the minimum distribution coefficients. The staff assumed the travel time through the backfill to be the same in both cases (i.e., 2.4 years). The staff also assumed that travel through the aquifer occurs from the engineered backfill to the nearest reach of Daniels Branch drainage, approximately 1500 feet away, and occurs at a ground-water velocity comparable to that currently observed. This results in a travel time estimate of 2.6 years. The resulting sum of fractions for this pathway is 0.336, which is also below the requirement of one (10 CFR Part 20, Appendix B, Table 2).

The NRC staff's analysis demonstrates that a release to the ground-water environment of a radioactive liquid effluent will meet the requirements of 10 CFR Part 20, Appendix B, Table 2. However, use of the minimum distribution coefficients in the analysis implies that no chelating agents can be comingled with the radioactive liquid effluents. Therefore, COL Action Item 2.4-1 requires that the COL or CP applicant confirm that no chelating agents will be comingled with radioactive waste liquids and that such agents will not be used to mitigate an accidental release. Alternatively, the COL or CP applicant may repeat experiments that include chelating agents to produce the distribution coefficients, and incorporate these newly determined distribution coefficients into the analysis to demonstrate that the requirements of 10 CFR Part 20, Appendix B, Table 2, are satisfied.

2.4.13.4 Conclusion

As set forth above, the applicant has substantiated sufficient information pertaining to the identification and evaluation of the effects of accidental releases of radioactive liquid effluents in ground and surface waters on existing users and known and likely future users of ground and surface water resources in the vicinity of the proposed site. Section 2.4.13 of RS-002 indicates that the SSAR should address the requirements of 10 CFR Part 100 as they relate to identifying and evaluating the effects of accidental releases of radioactive liquid effluents in ground and surface waters on existing users and known and likely future users in the vicinity of the site. Furthermore, the applicant considered the most severe natural phenomena historically reported for the site and surrounding area while describing the hydrologic interface of the plant with the site with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated. The NRC staff has generally accepted the methodologies used to determine the severity of the phenomena reflected in this analysis, as documented in the SERs for previous licensing actions. Accordingly, the NRC staff concludes that the use of these methodologies results in an analysis containing sufficient margin for the limited accuracy, quantity, and period of time in which the data have been accumulated. In view of the above, the staff considers the applicant's analysis to be acceptable for use in establishing the design bases for those SSCs important to safety as may be proposed in a COL or CP application.

The NRC staff concludes that the identification and consideration of accidental releases of radioactive liquid effluents in ground and surface waters set forth above are acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.21(d).

2.4.14 Site Characteristics

This section of the SER lists site characteristics and bounding parameters recommended by the NRC staff for inclusion in the ESP that may be granted for the VEGP site as given in table below.

Table 2.4.14-1 - Proposed Site Characteristics Related to Hydrology

SITE CHARACTERISTIC	VALUE	DEFINITION
Proposed Facility Boundaries	Figure 2.4.14-1	The site boundary within which all safety-related SSC will be located.
Highest Ground Water Elevation	165 feet MSL at the Water Table Aquifer	The highest elevation of the water table within the site boundaries.
Maximum Flood Elevation (maximum hydrostatic water surface elevation due to a postulated upstream dam breach scenario)	166.79 feet MSL	The stillwater elevation, without accounting for wind-induced waves that the water surface reaches during a flood event.
Wind run-up (to add to the maximum flood elevation)	11.31 feet	The water surface elevation reached by wind-induced waves running up on the shore.
Combined Effects Maximum Flood Elevation	178.10 feet MSL;	The water surface elevation obtained by adding wind run-up to the highest flood level.
Local Intense Precipitation	19.2 inches during 1 hour 6.2 inches during 5 minutes	The depth of PMP for duration of one hour on a one square-mile drainage area. The surface water drainage system should be designed for a flood produced by the local intense precipitation. The local intense precipitation is specified by SSAR Table 2.4.2-3 (see Table 2.4.2-1 of this SER).
Frazil Ice	The ESP site does not have the potential for the formation of frazil and anchor ice	Ice crystals that form in turbulent, open waters in presence of supercooling. Frazil ice is very sticky and may lead to blockages of intake screens and trash racks.

Table 2.4.14-2 Bounding Parameters

Bounding Parameters	Value	Definition
Plant Grade Elevation	220 feet MSL	The elevation of the finished ground surface that prevents the flood produced by the local intense precipitation from affecting the safety-related SSCs.

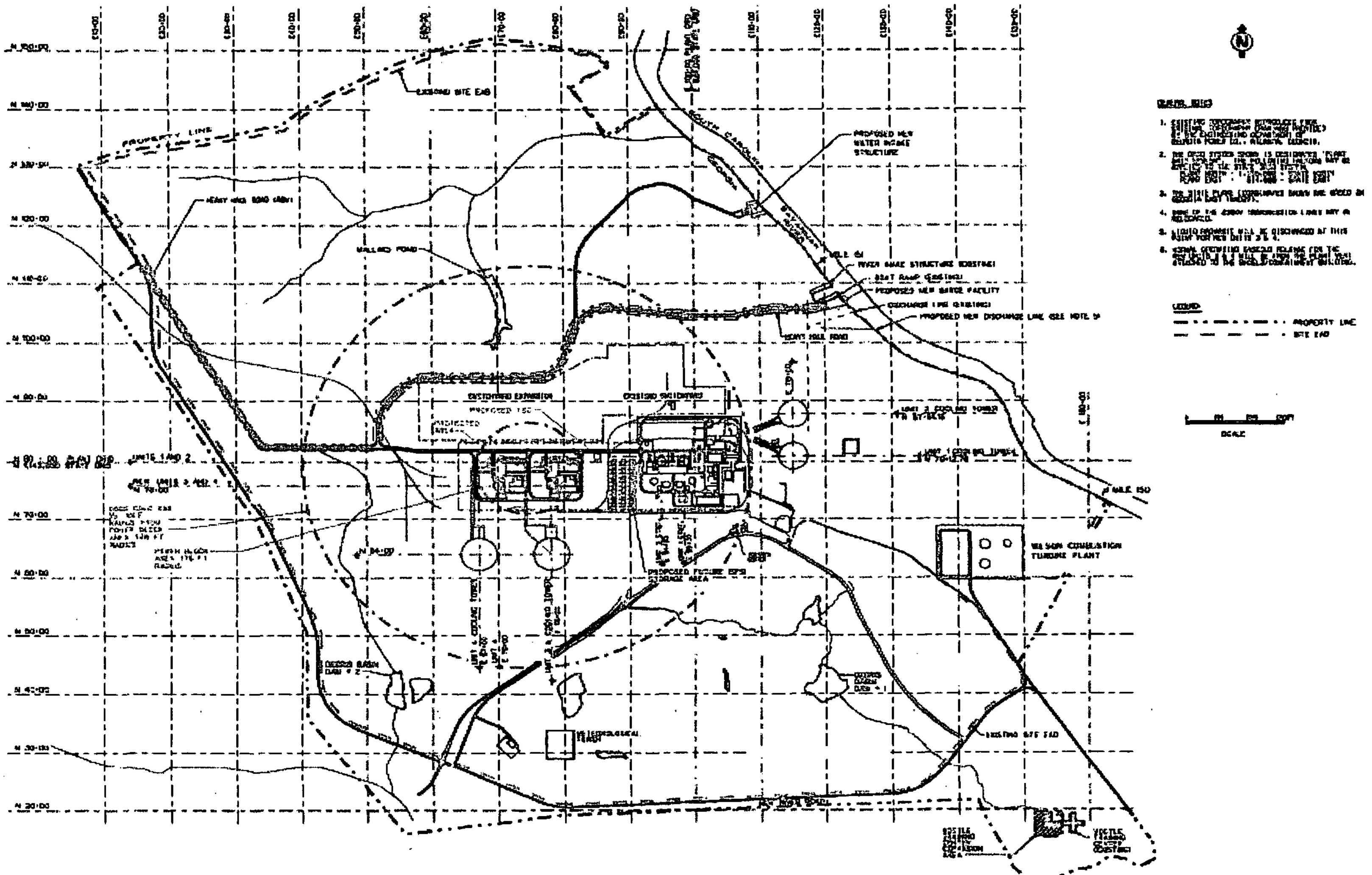


Figure 2.4.14-1 - The Proposed facility boundary for the VEGP site (Taken from SSAR Figure 1-4).

2.5 Geology, Seismology, and Geotechnical Engineering

In Section 2.5, "Geology, Seismology, and Geotechnical Engineering," of the VEGP SSAR, the applicant described geologic, seismic, and geotechnical engineering properties of the VEGP ESP site. SSAR Section 2.5.1, "Basic Geologic and Seismic Information," presents information on geologic and seismic characteristics of the VEGP site and region surrounding the site. SSAR Section 2.5.2, "Vibratory Ground Motion," describes the vibratory ground motion assessment for the ESP site through a PSHA and develops the SSE ground motion. SSAR Section 2.5.3, "Surface Faulting," evaluates the potential for surface tectonic and non-tectonic deformation at the ESP site. SSAR Sections 2.5.4, "Stability of Subsurface Materials and Foundations," 2.5.5, "Stability of Slopes," and 2.5.6, "Embankments and Dams," describe foundation and subsurface material stability at the ESP site.

The applicant reviewed reports from previous investigations for the existing VEGP Units 1 and 2 as a starting point for the characterization of the geologic, seismic, and geotechnical engineering properties of the site. The applicant also referred to published geologic literature and seismicity data, new borehole data for the proposed VEGP Units 3 and 4, seismic reflection and refraction surveys, and detailed investigations of the nearby SRS. Results of the investigations and analyses performed by the applicant for each of the SSAR Sections (2.5.1 to 2.5.6) provide information used to determine the SSE, as described in NRC RG 1.165 titled, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion."

The applicant defined the following four terms for areas in which investigations for the VEGP ESP site occurred, as designated by RG 1.165.

Site region: an area within 320 km (200 mi) of the site location.
Site vicinity: an area within 40 km (25 mi) of the site location.
Site area: an area within 8 km (5 mi) of the site location.
Site: an area within 1 km (0.6 mi) of the proposed VEGP Units 3 and 4 locations.

This RG also provides guidance on recommended levels of investigation for each of these areas.

The applicant also used the seismic source and ground motion models published in the EPRI's (1986) "Seismic Hazard Methodology for the Central and Eastern United States [CEUS as the starting point for its seismic hazard evaluation. The applicant used the procedures recommended in RG 1.165 for performing the probabilistic seismic hazard analysis (PSHA) for the ESP site, and employed the performance-based approach described in RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion" for determining the SSE.

The applicant conducted field investigations, examined relevant geologic literature, and concluded that no geologic or seismic hazards have the potential to affect the VEGP ESP site, except for the Charleston seismic zone and a small magnitude local earthquake occurring in the site region. The applicant also concluded that there is only limited potential for non-tectonic surface deformation within the 8 km (5 mi) site area radius, and that this potential could be mitigated by excavation of shallow deposits overlying the foundation bearing unit.

This SER, compiled by the NRC staff, is divided into six main sections, 2.5.1 to 2.5.6, which parallel the six main sections included in the applicant's SSAR. Each of the six SER sections is then divided into four sub-sections: (1) "Technical Information in the Application" that describes the contents of the SSAR, the investigations performed by the applicant, and the results; (2) "Regulatory Basis" that provides a summary of the regulations and NRC regulatory guides used by the applicant to formulate the SSAR; (3) "Technical Evaluation" that describes the staff's evaluation of what the applicant did, including any requests for additional information (RAI's), open items, and any confirmatory analyses performed by the NRC staff; and (4) the final "Conclusions" sub-section for each main section that documents whether or not the applicant provided a thorough characterization for the site and if its results provide an adequate basis for the conclusions made by the applicant.

2.5.1 Basic Geologic and Seismic Information

Section 2.5.1.1 of this SER provides a summary of relevant geologic and seismic information contained in SSAR Section 2.5.1 of the VEGP application. SER Section 2.5.1.2 provides a summary of the regulations and guidance used by the applicant to perform its investigation. SER Section 2.5.1.3 provides a review of the staff's evaluation of SSAR 2.5.1, including any requests for additional information, any open items, and any confirmatory analyses performed by the staff. Finally, SER Section 2.5.1.4 provides an overall summary of the applicant's conclusions, as well as the staff's conclusions, restates any bases covered in the application, and confirms that regulations were met or fulfilled by the applicant.

In SSAR Section 2.5.1, the applicant described geologic and seismic characteristics of the VEGP site region and site area. SSAR Section 2.5.1.1, "Regional Geology," describes the geologic and tectonic setting of the site region (within a 320 km (200 mi) radius), and SSAR Section 2.5.1.2, "Site Geology," describes the structural geology of the site area (within an 8 km (5 mi) radius). In SSAR Section 2.5.1, the applicant also provided an update of geologic, seismic and geophysical data for the VEGP site and then reviewed the updated information, pursuant to RG 1.165, to determine whether any of the data published since the mid-1980's requires an update to the 1986 EPRI seismic source model.

The applicant developed SSAR Section 2.5.1 based on information derived from the review of previously prepared reports for existing VEGP Units 1 and 2, and published geologic literature, new boreholes drilled for potential VEGP Units 3 and 4, and seismic reflection and refraction surveys conducted for the ESP application. The applicant also used recently published literature to supplement and update existing geologic and seismic information.

2.5.1.1 Technical Information in the Application

2.5.1.1.1 Regional Geologic Description

SSAR Section 2.5.1.1, "Regional Geology," discusses the physiography, geomorphology, geologic history, stratigraphy, and geologic setting within a 320 km (200 mi) radius of the VEGP site. The applicant reviewed previous reports prepared for VEGP Units 1 and 2, as well as geophysical data and published geologic literature, in order to compile the regional geologic description. The applicant collected new data in order to assess whether or not the Pen Branch fault is a capable tectonic structure of Quaternary age (1.8 million years ago (mya) to present). The applicant concluded that regional geologic characteristics pose no safety issues that would impact the VEGP site. The applicant applied the information in this section towards developing a basis for evaluation of the geologic and seismic hazards covered in succeeding sections of the SSAR. Based on its review, the applicant presented the following information related to the regional geology for the ESP site.

Physiography, Geomorphology and Geologic History

SSAR Section 2.5.1.1.1 describes the regional physiography and geomorphology of the ESP site. From northwest to southeast, the site region includes parts of the Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain physiographic provinces. Figure 2.5.1-1, reproduced from SSAR Figure 2.5.1-1, illustrates these four provinces. The VEGP ESP site lies within the Coastal Plain province approximately 48 km (30 mi) southeast of the line ("fall line") separating crystalline rocks of the Piedmont province from sediments of the Coastal Plain province. The Coastal Plain province is one of low topographic relief. Depositional landforms and topography strongly modified by fluvial erosion characterize the VEGP ESP site within the Coastal Plain province. Based on published information (Soller and Mills, 1991), the applicant described Carolina Bays (shallow, elliptical landforms which commonly occur in the Coastal Plain province) as surficial, non-tectonic features resulting from erosion by southwesterly-oriented winds (eolian erosion) that have no effect on subsurface sediments. Several investigators have documented that strata are continuous and undeformed beneath both bay and interbay areas.

The applicant described the geologic history of the ESP site in SSAR Section 2.5.1.1.2. Although the ESP site is located in the Coastal Plain, all major lithotectonic (characteristically unified rock assemblage) divisions of the Appalachian mountain belt occur within the site region. The applicant stated that geologic structures and stratigraphic sequences within these lithotectonic divisions represent a complex geologic evolution ending in the modern-day, passive Atlantic continental margin. This complex evolution resulted in the deposition of Cretaceous (144 to 65 mya) and Tertiary (65 to 1.8 mya) age sediments of the Coastal Plain; Quaternary (1.8 mya to present) materials in fluvial terraces along the Savannah River and its tributaries; and colluvial (loose, heterogeneous soil material and rock fragments), alluvial (unconsolidated material deposited during relatively recent geologic time by running water) and eolian sediments, all within the site area.

Stratigraphy and Geologic Setting

In SSAR Section 2.5.1.1.3, the applicant described regional stratigraphy and geologic setting (including stratigraphy, rock type, and geologic history) for the (1) Valley and Ridge; (2) Blue Ridge; (3) Piedmont; (4) Mesozoic rift basins; and (5) Coastal Plain provinces.

1. Folded and thrust-faulted Paleozoic (543 to 248 mya) sedimentary cover rocks overlying crystalline basement represent the Valley and Ridge lithotectonic terrane, located about 290 km (180 mi) west-northwest of the VEGP ESP site. A series of northeast-southwest trending, parallel valleys, and ridges are responsible for the physiographic expression within the Valley and Ridge terrace. Most of the folding and faulting deformation is likely late Paleozoic in age (at least 248 mya).

2. A complexly folded, faulted, penetratively deformed, metamorphosed crystalline basement and cover rock sequence containing intrusive igneous rocks represents the Blue Ridge lithotectonic province, located about 225 km (140 mi) northwest of the ESP site. Multiple deformation events indicated by deformation features in the rocks relate to late Proterozoic to late Paleozoic (248 mya and older) extension and compression.

3. Variably deformed and metamorphosed igneous and sedimentary rocks ranging in age from Proterozoic to Permian (248 mya and older) represent the Piedmont Province, located about 48 km (30 mi) northwest of the ESP site. The applicant stated that Piedmont province rocks generally underlie Coastal Plain province sediments, but that the southeastern extent of the Piedmont province beneath the Coastal Plain is unknown.

4. Mesozoic Rift Basins typically consist of non-marine sandstone, conglomerate, siltstone, shale, carbonates, coal, and basaltic igneous rocks. One of these basins, the Dunbarton Triassic basin, is beneath the Coastal Plain sediments at the VEGP ESP site. Geophysical investigations, including seismic reflection, suggest that the Triassic (206 to 24 mya) section of the Dunbarton basin is at least 2 km (1.2 mi) thick. The primary fault bounding this basin on the northwest side is the Pen Branch fault, which dips to the southeast. The applicant described the Pen Branch fault to be a Paleozoic reverse fault, reactivated as an extensional normal fault during the Mesozoic (248 to 65 mya) and subsequently reactivated as a reverse fault during the Cenozoic (65 mya to present).

5. Erosion-beveled rocks of Paleozoic and Triassic age (543 to 206 mya) and unconsolidated to poorly consolidated Coastal Plain sediments deposited unconformably above the erosional surface represent the Coastal Plain province where the ESP site is located. This seaward-dipping wedge extends from the contact with crystalline rocks of the Piedmont physiographic province (the fall line) to the edge of the continental shelf. Sediment thickness increases from zero at the fall line to about 1200 m (4000 feet) at the Georgia coastline. The sediment thickness is about 335 m (1000 feet) in the center of the VEGP site area and is composed of Upper Cretaceous, Tertiary, and unconsolidated Quaternary deposits.

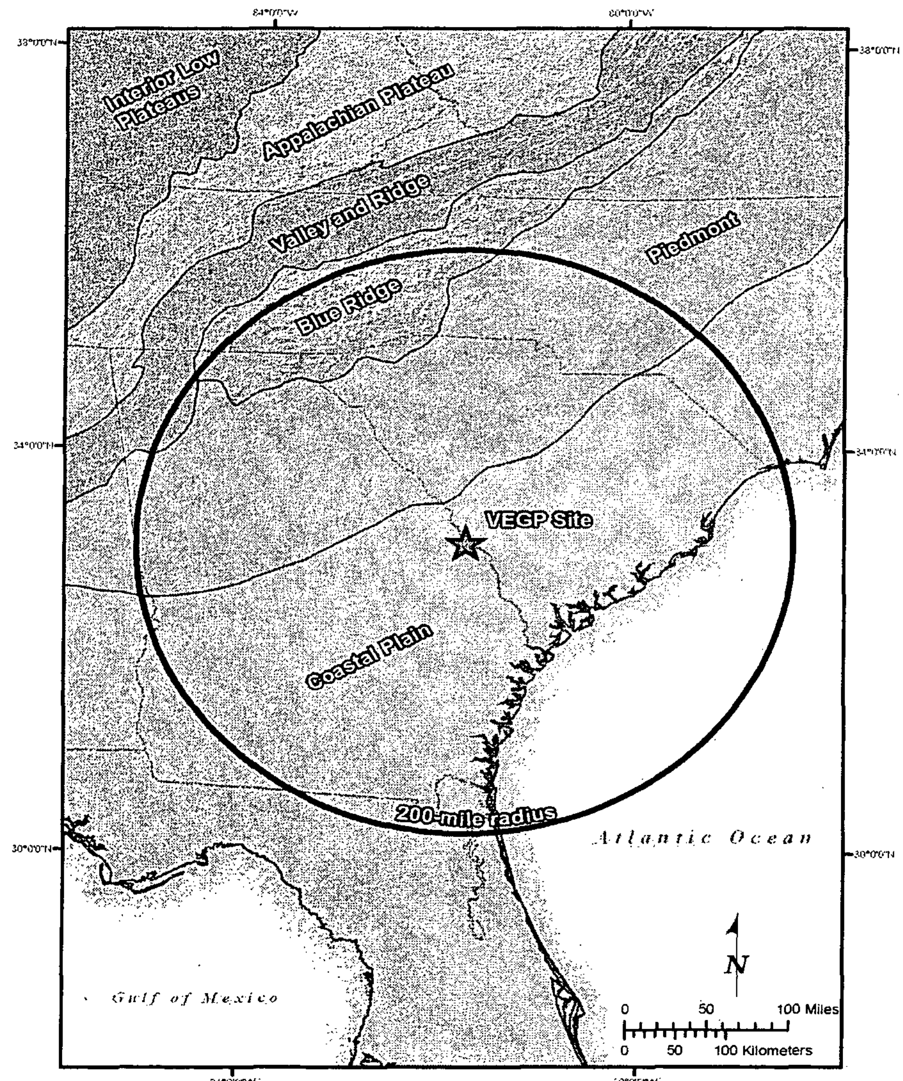


Figure 2.5.1-1 - Physiographic Provinces of the Southeastern United States

Quaternary Period (1.8 mya-present) surfaces and deposits are preserved primarily in the fluvial terraces along the Savannah River and its major tributaries, as well as in colluvium, alluvium, and eolian sediments in upland settings. Nested fluvial terraces, preserved along the east side of the Savannah River, can be used to evaluate Quaternary deformation within the Savannah River area. Major stream terraces develop as a result of sequential erosional and depositional events which may be due to tectonism, isostacy, or climatic variations. In SSAR Section 2.5.1.1.3.5, the applicant described two prominent terraces above the modern flood plain and along the east side of the Savannah River in the ESP site vicinity. The Bush Field terrace (mapped as Quaternary terrace surface "Qtb") is preserved primarily on the northeast side of the Savannah River and its surface ranges from 8 to 13 m (26 to 43 ft) above the river. Ellenton terrace surfaces (mapped as "Qte") range from 17 to 25 m (56 to 82 ft) above the river. The applicant estimated the age of the older Ellenton terrace to be 350 thousand to 1 million years old. The younger Qtb terrace is estimated to be about 90 thousand years old.

2.5.1.1.2 Regional Tectonic Description

The applicant described the tectonic setting, tectonic structures, and seismic source zones in sub-sections 2.5.1.1.4.1 through 2.5.1.1.4.6 of SSAR Section 2.5.1.1.4. The applicant discussed plate tectonic evolution of the Appalachian orogenic belt at the latitude of the ESP site, tectonic stress in the mid-continent region, principal regional tectonic structures, Charleston tectonic features, SRS tectonic features, and seismic sources defined by regional seismicity. SSAR Section 2.5.1.1.5 outlines the applicant's review of regional gravity and magnetic data, and the models used to supplement their interpretations of regional geologic and tectonic features discussed in SSAR Sections 2.5.1.1.3 and 2.5.1.1.4. The applicant concluded that (1) tectonic features in the site region are Paleozoic (> 248 mya), Mesozoic (248 to 65 mya), and Cenozoic (< 65.5 mya) in age but only the Quaternary (< than 1.8 mya) features require additional consideration for this ESP; (2) there is no significant change to the understanding of stress in the CEUS that would require updates to the currently accepted data; (3) of 11 potential Quaternary features evaluated by the applicant, only paleoliquefaction features associated with the Charleston source earthquakes clearly demonstrate the existence of a Quaternary tectonic feature; (4) based on new source geometry and earthquake recurrence information, the Charleston seismic source requires updated parameters; and (5) that there are no unexplained anomalies expressed in the gravity or magnetic data for the VEGP site region and no evidence present in the data for Cenozoic age structures or deformation. Based on published information, the applicant presented the following information related to the regional tectonic setting:

Plate Tectonic Evolution and Stress Field

The applicant discussed plate tectonic evolution of the Appalachian orogenic belt at the latitude of the site region in SSAR Section 2.5.1.1.4.1 and acknowledged the four principal tectonic elements of the Appalachian orogen: the Valley and Ridge province, Blue Ridge province, Piedmont province, and Coastal Plain province. These four tectonic elements correspond to the four physiographic provinces described in SSAR Section 2.5.1.1.1 and shown in Figure 2.5.1-1. The Appalachian orogenic belt, trending northeast-southwest and extending from southern New York State into Alabama, records the opening (between 900 to 543 mya) and closing (543 to 248 mya) of the proto-Atlantic Ocean along the eastern margin of ancestral North America. Compressional deformation due to continental collisions occurred during the Ordovician (490-443 mya), Devonian (417 to 354 mya), and Late Paleozoic (320 to 250 mya). Triassic (248 to 206 mya) basins, including the Dunbarton Basin, which occur in the Appalachian orogenic belt, represent Mesozoic rifting. Stratigraphic units of the coastal plain, the province within which the ESP site lies, record development of a passive continental margin along the east coast of the United States that followed the Mesozoic rifting and the opening of the present-day Atlantic ocean basin. The applicant concluded that, despite uncertainties in regard to origin, mode of emplacement, and boundaries of the different structural and lithologic terranes that exist in the principal tectonic provinces, there is reasonable agreement among existing tectonic models on regional structural features of the southern Appalachian orogenic belt.

In SSAR Section 2.5.1.1.4.2, the applicant discussed the regional tectonic stress acting on the mid-continent region, specifically the CEUS. The 1986 EPRI evaluation of intra-plate stresses determined that the CEUS is characterized by northeast-southwest directed horizontal compressive stress attributed mostly to ridge-push forces associated with the Mid-Atlantic ridge. The applicant concluded that based on investigations conducted since the EPRI study, which support the initial EPRI findings, there is no significant change to the understanding of stress in the CEUS and therefore it is not necessary to reevaluate the seismic potential of tectonic sources in the region based on the regional tectonic stress.

Principal Regional Tectonic Structures

In SSAR Section 2.5.1.1.4.3, the applicant defined and discussed four categories of principal regional tectonic structures occurring within a 320 km (200 mi) radius of the VEGP site based on age of formation or reactivation of the structures. These four categories included tectonic structures of (1) Paleozoic (543 to 248 mya); (2) Mesozoic (248 to 65 mya); (3) Tertiary (65 to 1.8 mya); and (4) Quaternary (1.8 mya to present) age. The applicant also discussed regional geophysical anomalies and lineaments potentially equated with tectonic features.

1. Paleozoic Tectonic Structures. The applicant indicated that rocks and structures within the physiographic provinces included in the site region are associated with thrust sheets that formed by convergent Appalachian orogenic events during the Paleozoic. In the case of the Coastal Plain province where the ESP site is located, these rocks and structures are buried beneath sedimentary cover. The majority of these structural features dip eastward into a basal, shallow dipping fault (decollement) structure. The applicant discussed two primary Paleozoic fault zones, the Augusta and the Modoc, as well as a number of other Paleozoic faults within the ESP site region, including the Hayesville Fault, the Brevard Fault, the Towaliga Fault, the Central Piedmont Suture, and the Eastern Piedmont Fault System. The applicant concluded that none of these structures are capable tectonic sources of concern for the VEGP site and that no new information has been published since 1986 on these Paleozoic faults in the site region that would result in a significant change to the EPRI seismic source model.

2. Mesozoic Tectonic Structures. The applicant recognized the broad zone of fault-bounded depositional basins associated with crustal extension and rifting in early Mesozoic time (Triassic period, 248 to 206 mya). These are relatively common features along the east coast of North America. Figure 2.5.1-2, taken from SSAR Figure 2.5.1-16, shows one of these east-northeast-trending Triassic basins, the Dunbarton Basin, which lies beneath the VEGP site and the SRS. This basin, approximately 50 km (31 mi) long and 10 to 15km (6 to 9 mi) wide, is bounded on its northwest side by the Pen Branch Fault, which experienced normal fault displacement during the Triassic. The Pen Branch fault is interpreted to have been reactivated in the Cenozoic (65 mya to present) as a reverse fault. The applicant stated that no definitive

correlation of seismicity with any Mesozoic normal fault has been conclusively demonstrated.

3. <u>Tertiary Tectonic Structures</u>. The applicant stated that only a few tectonic features were active in the Tertiary Period (65 to 1.8 mya) within the ESP site area. The applicant referred to a series of arches and embayments (topographic highs and lows) that exerted control on Coastal Plain sedimentation from late Cretaceous through Pleistocene time (144 mya to 10,000 ya) as indicative of episodic differential tectonic movement. The applicant concluded that the most prominent arches in the VEGP site region, the Cape Fear Arch on the South Carolina-North Carolina border, and the Yamacraw Arch on the Georgia-South Carolina border show no evidence of being active.

4. <u>Quaternary Tectonic Structures</u>. The applicant discussed 11 potential Quaternary features within a 320 km (200 mi) radius of the VEGP ESP site as shown in Figure 2.5.1-3, reproduced from SSAR Figure 2.5.1-17. Table 2.5.1-1, reproduced from SSAR Table 2.5.1-1, provides definitions and classes used to categorize these same potential features. The 11 potential Quaternary features discussed by the applicant include the Charleston, Georgetown, and Bluffton paleoliquefaction features, the East Coast Fault System (ECFS), the Cooke fault, the Helena Banks fault zone, the Pen Branch fault, the Belair fault, the fall lines of Weems (1998), the Cape Fear arch, and the Eastern Tennessee Seismic Zone (ETSZ). The three paleoliquefaction features are classified by Wheeler (2005) as "Class A", indicating there is geologic evidence to demonstrate the existence of Quaternary tectonic deformation related to these features. The other eight features are classified as "Class C", indicating there is insufficient geologic evidence to demonstrate the existence of Quaternary deformation associated with these features. The applicant discussed only the Belair Fault Zone and the fall lines of Weems (1998) in SSAR Section 2.5.1.1.4.3 since the other potential Quaternary features are discussed in detail in other sections of the SSAR.

The applicant documented that the Belair Fault Zone, located about 48 km (30 mi) northwest of the ESP site, occurs as a series of northeast-striking, southeast-dipping oblique-slip faults with no evidence of historic or recent associated seismicity. The applicant concluded that Quaternary slip is allowed, but not clearly demonstrated, by available data.

Weems (1998) identified numerous anomalously steep stream segments in the Blue Ridge and Piedmont physiographic provinces of North Carolina, Virginia, and Tennessee and recognized that these steep "fall zones", located north and northeast of the ESP site, are aligned from stream to stream along paths that are subparallel to the regional structural grain of the Appalachian orogenic belt. Although Weems (1998) favored a neotectonic (less than 23.8 mya) origin for these fall lines, Wheeler (2005) classified them as Class C features because he did not consider Quaternary tectonic faulting to be demonstrated by the available data.

In addition to the 11 potential Quaternary features listed above, the applicant recognized that a number of regional geophysical anomalies and lineaments occur within 320km (200 mi) of the VEGP site, including the East Coast Magnetic Anomaly (ECMA), the Blake Spur Magnetic Anomaly, the Grenville Front, the New York-Alabama Lineament (NYAL), and the Clingman and Ocoee Lineaments.

The applicant described the ECMA and the Blake Spur Magnetic Anomaly, both of which are located off the east coast of North America and interpreted to be Mesozoic in age. The applicant concluded that neither of these anomalies are associated with a regional fault or other tectonic structure and do not represent a potential seismic source for the VEGP site.

The applicant classified the NYAL as a linear feature 1600 km (1000 mi) in length defined by a series of northeast-southwest-trending magnetic gradients in the Valley and Ridge physiographic province that intersects and truncates other magnetic anomalies. King and Zietz (1978) interpreted this lineament to be a major strike-slip fault in Precambrian basement, while Shumaker (2000) equated it to a right-lateral wrench fault that formed during an initial phase of Precambrian continental rifting.

The Clingman Lineament is 1200 km (750 mi) in length and also trends northeast, showing up as an aeromagnetic linear feature passing through parts of the Blue Ridge and the eastern Valley and Ridge provinces from Alabama to Pennsylvania. The Ocoee Lineament is described as a splay that branches southwest from the Clingman Lineament approximately at latitude 36N. The Clingman-Ocoee Lineaments are subparallel to and located 50-100 km (30-60 mi) east of the NYAL.

The applicant described the "Ocoee block" as a Precambrian basement block located northwest of the ESP site and just outside of the 320 km (200 mi) site radius. The majority of southern Appalachian seismicity is interpreted to occur within the Ocoee block that coincides with the western margin of the ETSZ, as discussed in SSAR Section 2.5.1.1.4.6 "Seismic Sources Defined by Regional Seismicity". Johnston et al. (1985) interpreted seismicity within the Ocoee block as related to strike-slip displacement on faults striking north-south and east-west. More recently, Wheeler (1996) proposed that earthquakes within the Ocoee block may be related to reactivation of Precambrian normal faults as reverse or strike-slip faults in the "modern" tectonic setting.

The applicant described regional gravity and magnetic data in relation to the VEGP site region in Section 2.5.1.1.5 of the SSAR. Regional maps of North American gravity and magnetic fields were published by the Geological Society of America in 1987 as part of the Decade of North American Geology project. These maps are at a scale that allows identification and assessment of gravity and magnetic anomalies with wavelengths of about 10 km (6 mi) or greater. The applicant concluded there are no unexplained anomalies in the gravity data for the VEGP site region, and no data or gravity modeling results show evidence of Cenozoic tectonic activity or specific structures of Cenozoic age in the site region.

The applicant discussed regional magnetic signatures for the VEGP site region in Section 2.5.1.1.5.2 of the SSAR. The applicant concluded that (1) magnetic data do not have sufficient resolution to identify discrete faults such as the Pen Branch Fault; (2) there are no unexplained anomalies in the magnetic data for the VEGP site region; and (3) no data show evidence for Cenozoic structures in the VEGP site region.

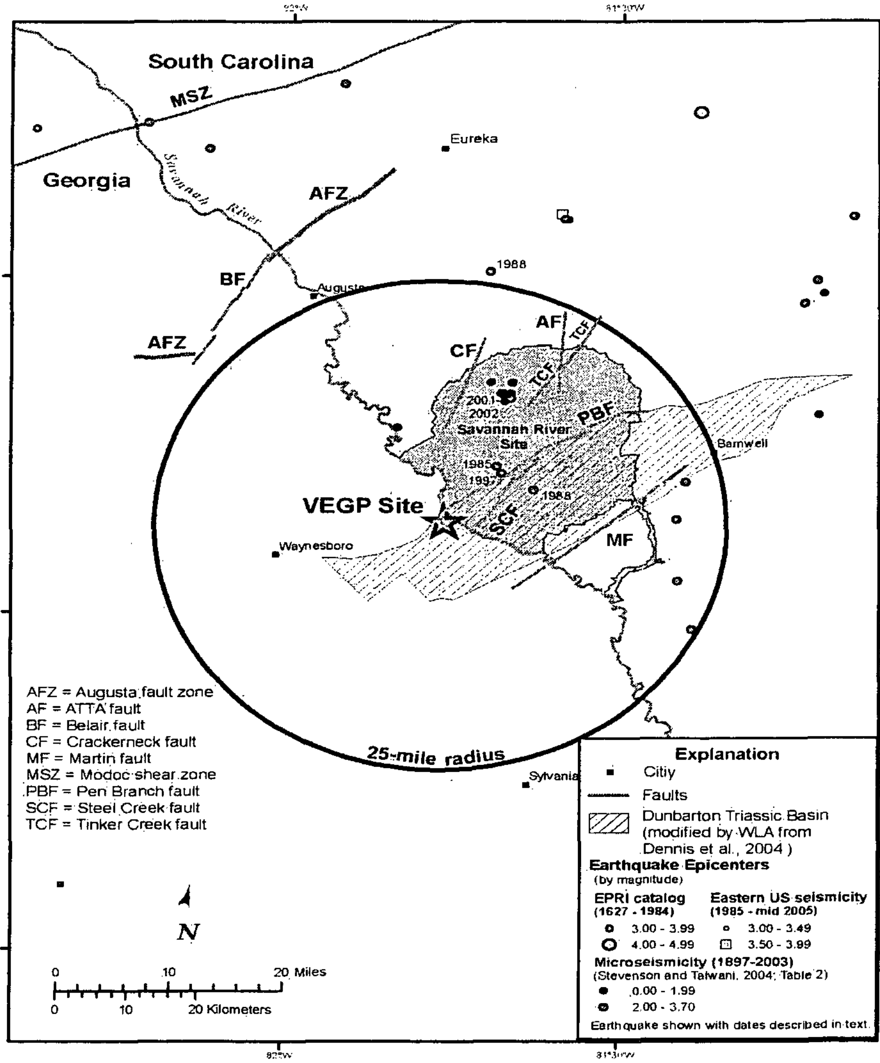


Figure 2.5.1-2 - Site Vicinity Tectonic Features and Seismicity (Reproduced from SSAR Figure 2.5.1-16)

Table 2.5.1-1 - Definitions of Classes Used in the Compilation of Quaternary Faults, Liquefaction Features, and Deformation in the Central and Eastern United States (Reproduced from SSAR Table 2.5.1-1 after Crone and Wheeler, 2000)

Class Category	Definition
Class A	Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether the fault is exposed for mapping or inferred from liquefaction to other deformational features.
Class B	Class B Geologic evidence demonstrates the existence of a fault or suggests Quaternary deformation, but either (1) the fault might not extend deeply enough to be a potential source of significant earthquakes, or (2) the currently available geologic evidence is too strong to confidently assign the feature to Class C but not strong enough to assign it to Class A.
Class C	Class C Geologic evidence is insufficient to demonstrate (1) the existence of tectonic fault, or (2) Quaternary slip or deformation associated with the feature.
Class D	Class D Geologic evidence demonstrates that the feature is not a tectonic fault or feature. This category includes features such as demonstrated joints or joint zones, landslides, erosional or fluvial scarps, or landforms resembling fault scarps, but of demonstrable non-tectonic origin.

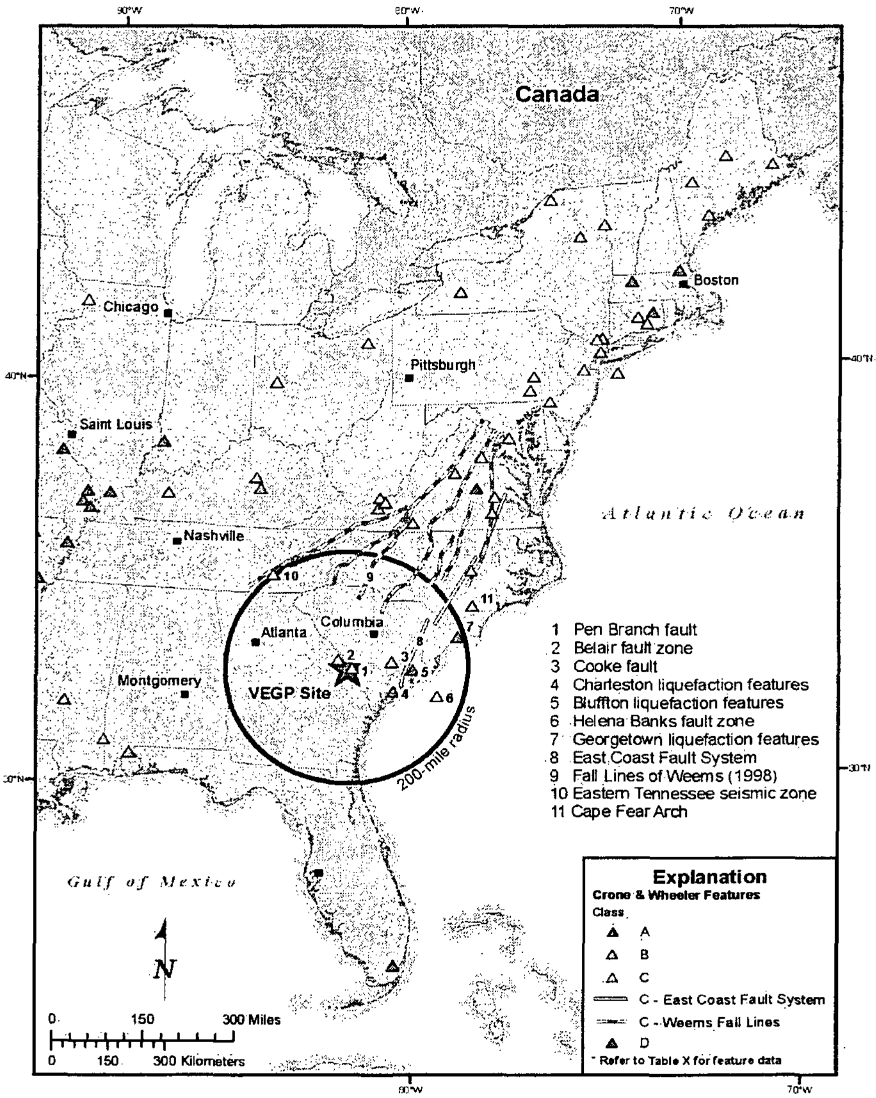


Figure 2.5.1-3 - Potential Quaternary Features Map (Reproduced from SSAR Figure 2.5.1-17)

Savannah River Site Tectonic Features

In SSAR Section 2.5.1.1.4.5, the applicant discussed faults that are interpreted to occur at the SRS on the eastern side of the Savannah River directly across from the VEGP ESP site. Locations of most of these faults are indicated on Figure 2.5.1-2. Most SRS faults are defined in the subsurface by interpretation of seismic reflection profiles, although information from seismic refraction studies and borehole studies is also used. The applicant stated that considerable uncertainty exists in regard to orientation and continuity of some of these faults. The applicant made no conclusion as to the capability of any of the SRS faults except for the Millet fault, which the applicant concluded showed no evidence of being a capable tectonic structure younger than the middle Eocene (40 mya). Four of the SRS faults occur within the VEGP site area: (1) Pen Branch, (2) Steel Creek, (3) Ellenton, and (4) Upper Three Runs faults.

1. The applicant described the northeast-trending Pen Branch fault as extending southwest off the SRS and across the Savannah River to the VEGP site location (Figure 2.5.1-2 from SSAR Figure 2.5.1–16). Since the Pen Branch is interpreted to extend beneath the VEGP site, the applicant discussed this feature in detail in SSAR Section 2.5.1.2.4.

2. The applicant described the northeast-trending Steel Creek fault, shown in Figure 2.5.1-2, as extending southwest into the VEGP site area to a point off the SRS on the west side of the Savannah River. This fault is located about 4 km (2.5 mi) east-southeast of the VEGP site location. Stieve and Stephenson (1995) considered the age of latest movement on this fault to be unresolved, but indicated that Cretaceous (144 to 65 mya) units are cut by the fault.

3. The applicant stated that the Ellenton fault strikes north-northwest, is near vertical, and extends into the VEGP site area with a location about 8 km (5 mi) northwest of the site location. However, data quality for definition of this structure is defined as poor and some researchers do not show this fault trace on their map of SRS faults.

4. The applicant stated that research indicates the Upper Three Runs fault is restricted to crystalline basement rocks, and that seismic reflection revealed no evidence for this fault offsetting Coastal Plain sediments. There is some indication that this fault extends southwest from the SRS, across the Savannah River, into the VEGP site area, and is located about 5 mi north of the site location. However, other investigators do not show this fault trace on their map of SRS faults.

Additional faults have been proposed outside the VEGP site area: (1) ATTA, (2) Crackerneck, (3) Martin, (4) Tinker Creek, (5) Lost Lake, and (6) Millet faults.

1. As described by the applicant, the ATTA fault is near vertical, strikes north-northeast, and is located about 25 km (16 mi) northeast of the VEGP site location, as shown in Figure 2.5.1-2. Research indicated a vertical separation of basement rocks by this fault of 25 m (82 ft) based on seismic reflection data, and also that penetration of the ATTA fault above basement is uncertain due to a lack of good seismic reflectors.

2. The applicant described the Crackerneck fault, which is located about 16 km (10 mi) north of the VEGP site location. Shown in Figure 2.5.1-2, this fault strikes northeast and dips steeply southeast. Research indicates that the fault exhibits a maximum vertical separation of basement rocks of about 30 m (98 ft) based on seismic reflection data, with offset decreasing upward to about 7 m (23 ft) at the top of the Upper Eocene Dry Branch formation (approximately 38.8 mya). The Middle Eocene Blue Bluff Marl (about 40 mya in age), the proposed foundation bearing unit for VEGP Units 3 and 4, underlies the Dry Branch.

3. The applicant described the Martin fault, which is located about 14.5 km (9 mi) south-southeast of the VEGP site location (based on aeromagnetic data). Shown in Figure 2.5.1-2, this fault strikes northeast with an undefined dip. Researchers estimated a vertical separation of the basement surface of about 18.5 to 31 m (60 to 100 ft) based on data from two boreholes.

4. The applicant described the Tinker Creek fault, which is located about 19 km (12 mi) north-northeast of the VEGP site location. Shown in Figure 2.5.1-2, this is interpreted to strike northeast and dips southeast. Seismic reflection data suggest a vertical separation of basement rocks by the Tinker Creek fault of 24 m (79 ft) at its northeastern extent, but the southeastern extent of the fault remains unresolved.

5. Cumbest et al (1998) defined the trace of the Lost Lake Fault based on its apparent control of groundwater flow pathways, locating it about 19 km (12 mi) north of the VEGP site location. The applicant reported that seismic and borehole data to constrain location, geometry, sense of slip, and age of latest movement are lacking.

6. The Millet fault is located about 14.5 km (9 mi) south-southeast of the VEGP site location. A study of this proposed fault by Bechtel (1982) was reviewed by the NRC staff, who concluded that there is no evidence for a capable tectonic structure as young as the Middle Eocene (40 mya) Blue Bluff Marl, which was characterized as tectonically undeformed.

Charleston Tectonic Features

In SSAR Section 2.5.1.1.4.4, the applicant discussed Charleston tectonic features, including potential source faults, area seismic zones, and area seismically-induced liquefaction features. These features, some defined since the EPRI (1986) seismic source models were developed, have been identified in or near the meizoseismal area (area of maximum damage) of the August 1886 Charleston earthquake and occur about 136 km (85 mi) east-southeast of the VEGP site.

The 1886 Charleston earthquake is recognized as one of the largest historical earthquakes to occur in the eastern United States. It produced a Modified Mercalli Intensity (MMI) X in the epicentral area near Charleston, and was felt as far away as Chicago, IL. Bakun and Hopper (2004) estimated a maximum magnitude for the 1886 Charleston earthquake ranging between M 6.4 to 7.1, a value similar to the upper-bound maximum magnitude used by EPRI (1986) for its source model. Due to a lack of observable surface deformation, the source of this earthquake has been inferred based on geology, paleoseismic features, and instrumented seismicity. The applicant recognized that, although the 1886 event was almost certainly related to a capable tectonic source, the earthquake has not been tied to any specific tectonic structure. The applicant concluded, in light of new information about source geometry and earthquake recurrence rate, that the EPRI (1986) source models for the 1886 Charleston earthquake warranted an update. The applicant presented the updated seismic source parameters in SSAR Section 2.5.2.2.2.4.

The applicant discussed the following potential causative faults for the 1886 Charleston earthquake event: (1) East Coast Fault System (ECFS), (2) Adams Run fault, (3) Ashley River fault, (4) Charleston fault, (5) Cooke fault, (6) Helena Banks fault zone, (7) Sawmill Branch fault, (8) Summerville fault, and (9) Woodstock fault. Figure 2.5.1-4, taken from SSAR Figure 2.5.1-19, shows these faults.

1. The applicant described the inferred ECFS, the southern section of which is marked by an alignment of river bends and consequently referred to as the "zone of river anomalies" (ZRA), as a northeast-trending fault system extending a total distance of about 600 km (373 mi) from Charleston, SC to southeastern Virginia. Researchers identified geomorphic anomalies (the ZRA) located along (and northwest of) the Woodstock fault and consequently defined the southern segment of the ECFS to extend the strike trend of the Woodstock fault. Data suggests that the fault system may have been active in the past 130,000 to 10,000 years and may remain active at the present time. It is further suggested that the ECFS may have been the source for the 1886 Charleston earthquake. Wheeler (2005) classified the ECFS as a Class C structure based on lack of demonstrable evidence for tectonic faulting or Quaternary slip or deformation associated with the feature.

2. The applicant described the Adams Run fault as being inferred from microseismicity and borehole data, but stated that the data were not consistent with the occurrence of fault displacement. The applicant further indicated no geomorphic evidence for the Adams Run fault and local microseismicity, as shown in Figure 2.5.1-5 from SSAR Figure 2.5.1-20, does not define a discrete structure.

3. The applicant described the Ashley River fault as being defined by a northwest-trending zone of seismicity in the meizoseismal area of the 1886 Charleston earthquake. This fault is interpreted to be a southwest-side-up reverse fault that offsets the northeast-trending Woodstock fault.

4. The applicant described the Charleston fault, also shown in Figure 2.5.1-5, as being defined by data from geologic maps and boreholes. This fault is interpreted as a major high-angle reverse fault which has been active in the Holocene (past 10,000 years). The applicant indicated that this fault has no clear geomorphic expression, nor is it clearly defined by the pattern of microseismicity in the vicinity of the fault.

5. The applicant described the Cooke fault, shown in Figure 2.5.1-5, as being defined by seismic reflection profiles in the meizoseismal area of the 1886 Charleston earthquake and interpreted as either an east-northeast-striking, northwest-dipping structure, or part of the ECFS. Crone and Wheeler (2000) classified the Cooke fault as a Class C feature based on lack of evidence for faulting younger than Eocene (54.8 to 33.7 mya).

6. The Helena Banks fault zone, located about 15 to 30 km (10 to 20 mi) off the coast of South Carolina, is clearly shown in seismic reflection lines. The applicant documented that Crone and Wheeler (2000) described this fault zone as a potential Quaternary tectonic feature, but classified it as a Class C feature since there is insufficient evidence to demonstrate Quaternary activity in the zone. The applicant stated that data suggest that the fault zone could, at a "low probability", be considered a potentially active fault. The applicant also stated that, if the Helena Banks fault zone is active, it could possibly explain distribution of paleoliquefaction features along the South Carolina coast.

7. The applicant described the Sawmill Branch fault, shown in Figure 2.5.1-5, as a northwest-trending structure defined by microseismicity and interpreted to be an extension of the Ashley River fault that offsets the Woodstock fault in a left-lateral sense. The applicant stated that microseismicity in the vicinity of the proposed Sawmill Branch fault does not clearly define a structure distinct from the Ashley River fault (the Ashley River fault was also defined based on seismicity).

8. The applicant described the Summerville fault, shown in Figure 2.5.1-5, which was initially defined by Weems et al. (1997) based on microseismicity. However, the applicant concluded that there is no geomorphic expression, borehole evidence, or microseismicity related to a discrete structure to indicate the existence of the Summerville fault.

9. The applicant described the Woodstock fault, shown in Figure 2.5.1-5, as a postulated north-northeast-trending, dextral strike-slip fault in the meizoseismal area of the 1886 Charleston earthquake defined by a linear zone of seismicity. Researchers subdivided this fault into two segments offset in a left-lateral sense across the Ashley River fault, and later included it as a part of the proposed ZRA in the southern portion of the ECFS.

__Charleston Area Seismic Zones__

The applicant discussed three zones of increased seismicity identified in the greater Charleston area, including the (1) Middleton Place-Summerville, (2) Bowman, and (3) Adams Run seismic zones. These three zones are shown in Figure 2.5.1-4. Details of the seismicity data catalog are discussed in SSAR Section 2.5.2.1.

1. The applicant described the Middleton Place-Summerville Seismic Zone as an area of elevated microseismicity located about 19 km (12 mi) northwest of Charleston. Between 1980 and 1991, 58 events with magnitudes ranging from body wave magnitude (mb) 0.8 to 3.3 and hypocentral depths ranging from 2 to 11 km (1 to 7 mi) were recorded in this zone, which lies inside the meizoseismal area of the 1886 Charleston earthquake. The elevated microseismicity in the Middleton Place-Summerville seismic zone has been attributed to stress concentrations associated with intersection of the Ashley River and Woodstock faults, and there is speculation that the 1886 Charleston earthquake had its source in this zone. Persistent foreshock activity was reported prior to the 1886 Charleston earthquake in the Middleton-Summerville seismic zone.

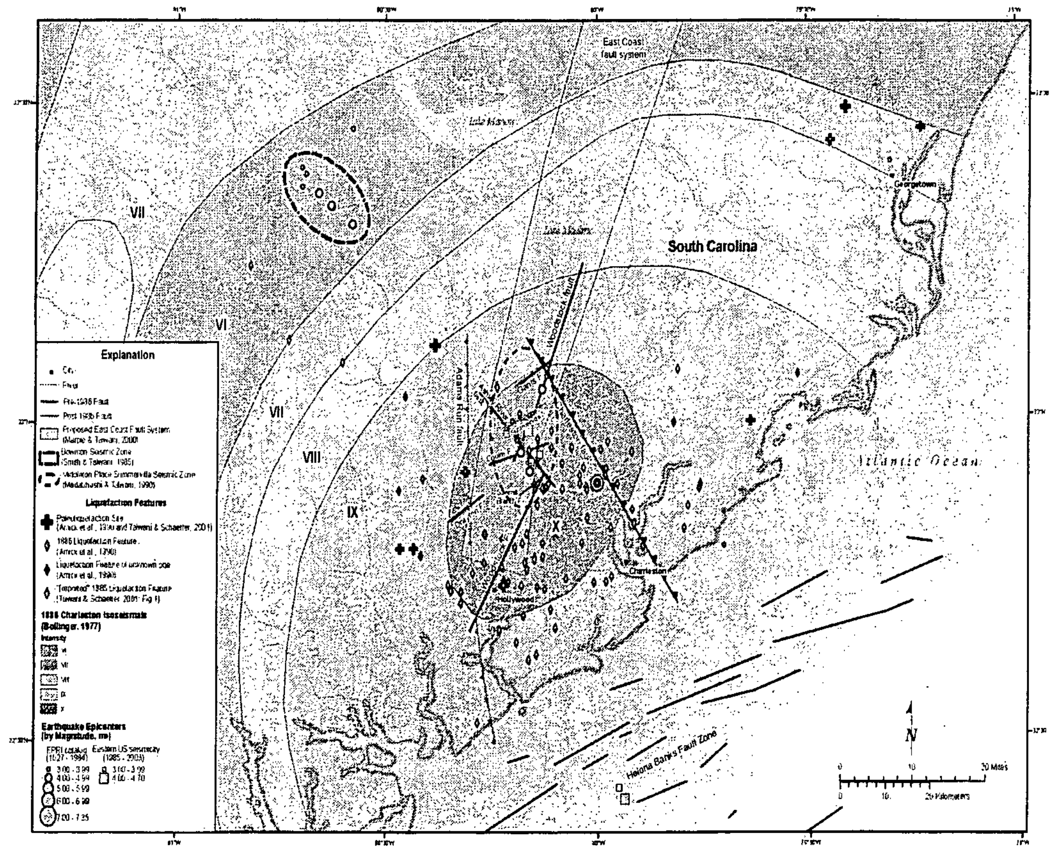


Figure 2.5.1-4 - Local Charleston Tectonic Features (Reproduced from SSAR Figure 2.5.1-19)

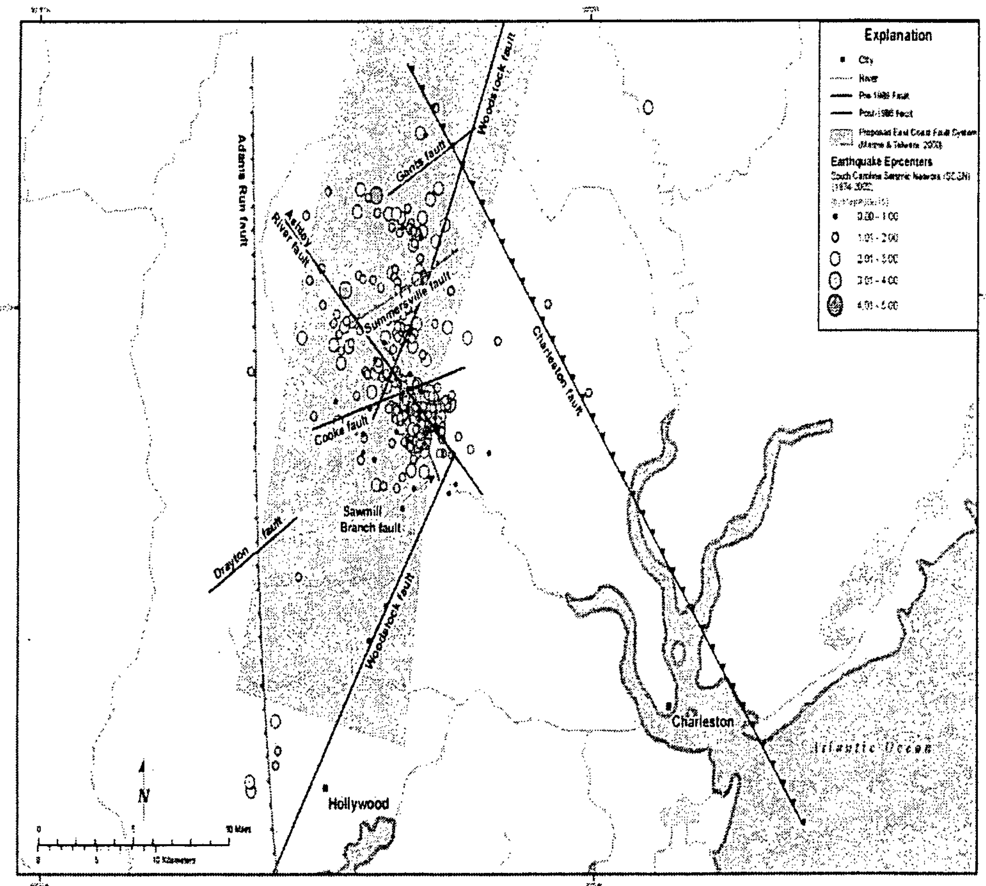


Figure 2.5.1-5 - Local Charleston Seismicity (Reproduced from SSAR Figure 2.5.1-20)

2. The applicant documented that the Bowman seismic zone lies outside the meizoseismal area of the 1886 Charleston earthquake. It is located about 80 km (50 mi) northwest of Charleston and 96 km (60 mi) east-northeast of the VEGP site as shown in Figure 2.5.1-4. The zone was identified based on a series of earthquakes with magnitudes of M3-4 which occurred in that zone between 1971-1974.

3. The applicant described the Adams Run seismic zone, located within the meizoseismal area of the 1886 Charleston earthquake as being defined by four earthquakes with magnitudes less than M2.5. Three of these four earthquakes occurred over a two day period in December 1977. This seismic zone occurs about 120 km (75 mi) east-southeast of the VEGP site and is not shown in Figure 2.5.1-4 as the text indicates.

Charleston Area Seismically-Induced Liquefaction Features

The applicant discussed Charleston area soil liquefaction in SSAR Section 2.5.1.1.4.4, which has proven to be the most broadly observable earthquake-induced phenomenon in the Charleston area. Liquefaction occurs when a mass of saturated, granular material temporarily loses its shear strength and its ability to act as a solid due to an increase in pore water pressures that exceeds overburden pressures. During an earthquake, waves are propagated upward through rock and soil, creating shear stresses that cause sediments with a high volume change capacity (saturated sediments) to compact. As pore water pressures increase, saturated materials are forced to flow in the direction of maximum principal compressive stress, typically upward through zones of weakness in dense overlying sediments. The presence of liquefaction features in the geologic record, and radiometric age dating of these features, aids in formulating an earthquake chronology with estimated magnitudes based on characteristics of the features and their geographic distribution. This extends the earthquake record back in time for defining longer-term earthquake occurrence rates.

The applicant presented data on liquefaction features observed in the South Carolina Coastal Plain and these features are shown in Figure 2.5.1-4. These liquefaction features were produced by the 1886 Charleston earthquake and earlier moderate to large earthquakes in the region. The presence of liquefaction features attributed to the 1886 Charleston earthquake and paleoliquefaction features related to earlier Quaternary earthquake events demonstrates repeated seismicity within the region and, hence, the presence of a capable tectonic source in the vicinity of Charleston. The applicant recognized that liquefaction features interpreted to have been produced by the 1886 Charleston earthquake are most heavily concentrated in the meizoseismal area for that earthquake as well as in some outlying areas. The applicant provided a description of potential Charleston earthquake sources in SSAR Section 2.5.1.1.4.4, but no definitive link has yet been made between a particular fault and the 1886 Charleston event, or any previous earthquake event. The applicant presented refinements of earthquake recurrence estimates for the Charleston area in detail in SSAR Section 2.5.2.2.2.4.

Paleoliquefaction features attributed to pre-1886 earthquakes are abundant along the South Carolina coast. These features were evaluated to estimate earthquake recurrence rates in the Charleston area. Talwani and Schaeffer (2001) proposed two earthquake scenarios: Scenario 1 assumes that some events in the paleoearthquake record were smaller in magnitude (estimated M6+) than events to the northeast of Charleston, while Scenario 2 allows all earthquakes in the record to be large events (estimated M7+) located near Charleston. Based on these two scenarios, Talwani and Schaeffer (2001) estimated recurrence intervals of about 550 years (Scenario 1) and 900-1000 years (Scenario 2).

Seismic Sources Defined by Regional Seismicity

In SSAR Section 2.5.1.1.4.6, the applicant discussed the ETSZ and three other seismogenic and capable tectonic source zones located outside the 320 km (200 mi) radius of the site region (Central Virginia, New Madrid, and Giles County seismic zones (GCSZ)). These seismic zones are shown in SER Figure 2.5.1-6 taken from SSAR Figure 2.5.1-15.

The ETSZ is a northeast-trending area of concentrated seismicity, characteristically generated by small-to-moderate earthquakes, which is located in the Valley and Ridge Physiographic province of eastern Tennessee. The applicant recognized that, although most seismic events in ETSZ have occurred more than 320 km (200 mi) from the VEGP site location and consequently outside the site region, diffuse seismicity on the southeastern margin of the zone is located just within the boundary of the site region. This zone, approximately 300 km (185 mi) long and 50 km (30 mi) wide, has produced no damaging earthquake in historical time. The zone exhibits no geologic evidence of prehistoric earthquakes larger than any historical event that has occurred within the zone. However, the ETSZ has been classified by some as the second most active seismic area in the United States east of the Rocky Mountains (after the New Madrid Seismic Zone (NMSZ)). Others have determined that this zone produced the second highest release of seismic strain energy in the CEUS during the 1980s.

Earthquakes in the ETSZ occur at depths of 5 to 26 km (3 to 16 mi) in Precambrian crystalline basement rocks that underlie exposed thrust sheets made up of Paleozoic rock units, suggesting that seismogenic structures in the zone are not related to surface geologic features of the Appalachian orogen. None of the earthquakes exceeded a moment magnitude of M4.6. Earthquakes within the ETSZ cannot be attributed to known faults and the applicant reported that no capable tectonic sources have been identified within the zone, although seismicity appears to be spatially associated with the prominent magnetic field gradient defined by the NYAL. Most seismicity in the ETSZ lies between the NYAL on the west and the Clingman and Ocoee lineaments on the east, in a "block" labeled as the Ocoee block. The applicant concluded that no new information has been developed since 1986 for the ETSZ to require a significant revision to the EPRI (1986) source model, but provided additional discussion of the ETSZ in relation to potential seismic hazard for the VEGP site location in SSAR Section 2.5.2.2.2.5.

The applicant recognized the potential for distant large earthquakes in the CEUS to contribute to the long-period ground motion hazard at the VEGP site, and consequently discussed the following three additional seismic source zones–(1) Central Virginia, (2) New Madrid, and (3) Giles County–located more than 320 km (200 mi) from the site location.

1. The Central Virginia Seismic Zone (CVSZ), shown in Figure 2.5.1-6, is an area of low-level seismicity located more than 560 km (350 mi) north-northeast of the VEGP site location, extending about 120 km (75 mi) north-south and 144 km (90 mi) east-west between Richmond and Lynchburg, VA. The largest historical earthquake to occur in the CVSZ (December 1875) had a body-wave magnitude of 5.0 and a maximum intensity of VII in its epicentral region. Wheeler and Johnston (1992) indicated that seismicity in the CVSZ ranges in depth from about 4 to 13 km (2 to 8 mi), suggesting that the events extend both above and below the Appalachian detachment zone (discussed in SSAR Section 2.5.1.1.4.1). Two paleoliquefaction sites reflecting prehistoric seismicity have been found within the CVSZ, but no capable tectonic sources have been identified. The applicant concluded that no new information has been developed since 1986 for the CVSZ to require a significant revision to the EPRI (1986) source model.

2. The NMSZ is an area defined by post-Eocene (younger than 33.7 mya) to Quaternary (1.8 mya to the present) faulting located more than 640 km (400 mi) west of the VEGP site location, extending from eastern Missouri to southwestern Tennessee (Figure 2.5.1-6 from SSAR Figure 2.5.1-15). The zone, approximately 220 km (125 mi) long and 40 km (25 mi) wide, is interpreted to be made up of three fault segments: a southern northeast-trending strike-slip fault, a middle northwest-trending reverse fault, and a northern northeast-trending strike-slip fault. Three large-magnitude historical earthquakes occurred in this zone between December 1811 and February 1812 with magnitudes ranging from M7.1 to M7.5. Since the EPRI (1986) study, estimates of maximum magnitude have generally been in the range of those used in the 1986 EPRI models. However, recent summaries of paleoseismic data suggest a mean recurrence time of 500 years, an order of magnitude less than seismicity-based recurrence estimates used in EPRI (1986).

   The applicant concluded that this estimate of recurrence time represents a significant update of source parameters for the NMSZ used by EPRI (1986).

3. The GCSZ is located in Giles County, VA, more than 250 mi from the VEGP site location, as shown in Figure 2.5.1-6. Bollinger and Wheeler (1988) reported that earthquakes in this zone occur in Precambrian crystalline basement beneath the overlying Appalachian thrust sheets at depths from 5 to 25 km (3 to 16 mi). The data on depth of earthquakes in the GCSZ imply that seismogenic structures in the zone are unrelated to surface geology of the Appalachian orogen. Shallow Late Pliocene to Early Quaternary faults near Pembroke, VA, which lie within the area defined as the GCSZ, are classified as Class B features because it is not determined if they are of tectonic origin or related to solution collapse. The applicant concluded that no new information has been developed since 1986 for the GCSZ to require a significant revision to the EPRI (1986) source model.

2.5.1.1.3 Site Area Geologic Description

Sub-sections 2.5.1.2.1 to 2.5.1.2.3 of SSAR Section 2.5.1.2 describe the geology of the site area, including physiography and geomorphology, geologic history, and stratigraphy). The applicant concluded that the physiography, geomorphology, geologic history, and stratigraphy of the site area pose no safety concerns for the ESP site. The applicant presented the following information related to site area geology.

Physiography, Geomorphology and Geologic History

In SSAR Section 2.5.1.2.1, the applicant described physiography and geomorphology of the ESP site area. The site area lies within the Upper Coastal Plain, about 48 km (30 mi) southeast of the fall line that separates the Piedmont and Coastal Plain physiographic provinces, as shown in Figure 2.5.1-1. The Savannah River, located on the east side of the ESP site, is the primary drainage system in the site area and acts as the state line boundary between Georgia and South Carolina. The Savannah River is incised into surrounding topography to form steep bluffs and a topographic relief of nearly 45 m (150 ft) from river level to the VEGP site. The surface topography, characterized by gently rolling hills, ranges from about 60 to 90 m (200 to 300 ft) above mean sea level (msl) across the site area.

The applicant reported that two types of surface depressions occur in the Coastal Plain that are both non-tectonic in origin. The first type of surface depression is referred to as "Carolina Bays", and results from eolian, surficial processes. The second type of non-tectonic surface depression most likely results from the dissolution of calcareous stratigraphic units at depth. The applicant stated that these surface depressions in the site area were noted and extensively studied during the initial site investigations for VEGP Units 1 and 2.

The applicant described the geologic history of the ESP site area in SSAR Section 2.5.1.2.2. The Upper Coastal Plain is a relatively flat-lying section of unconsolidated marine and fluvial sediments overlying a basement complex of Paleozoic (greater than 248 mya) metamorphic and igneous rocks, and Triassic (248 to 206 mya) basin sedimentary rocks. Paleozoic and Triassic rocks were beveled by erosion prior to deposition of Coastal Plain sediments. The applicant reported that this erosional surface dips southeast beneath the sediments at approximately 9.5 m/km (50 ft/mi). The Coastal Plain section consists of stratified sands, clays, limestone, and gravel deposits that dip gently seaward, with the oldest sediments in the site area being Upper Cretaceous (greater than 65 mya) units and the youngest sediments being Quaternary (1.8 mya to Present) alluvium in stream and river valleys.

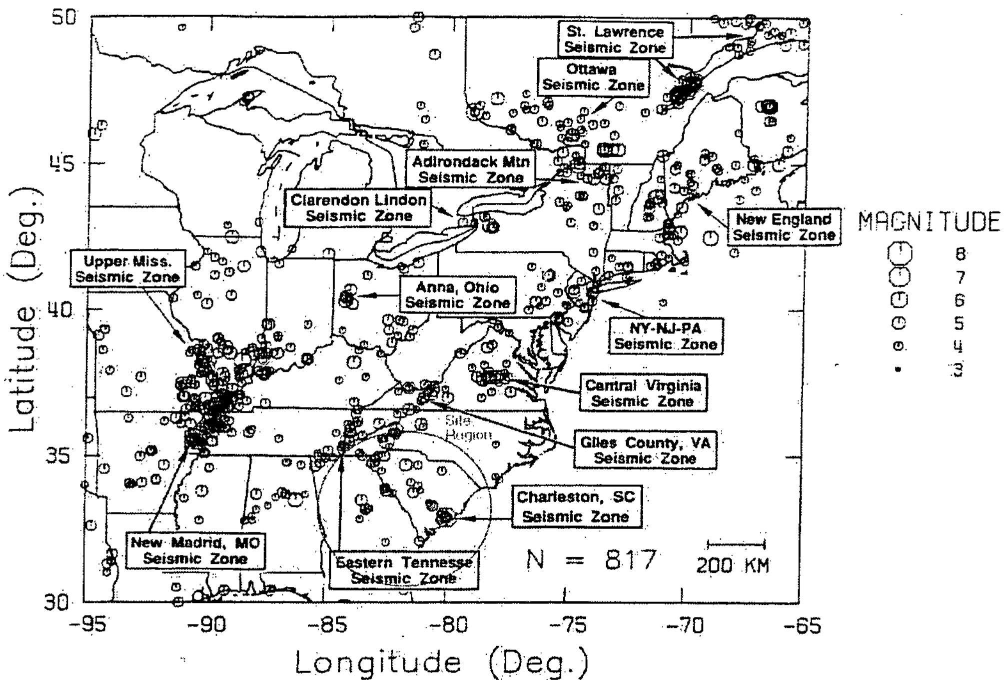


Figure 2.5.1-6 - Seismic Source Zones and Seismicity in the Central and Eastern U.S (Reproduced from SSAR Figure 2.5.1-15)

Stratigraphy

The applicant described the stratigraphy of the ESP site area in SSAR Section 2.5.1.2.3, including basement rock and coastal plain stratigraphy within the site area. The applicant based the stratigraphic descriptions on information from regional geologic maps, site area studies performed for VEGP, borehole data, and surface geophysical surveys. Figure 2.5.1-7, reproduced from SSAR Figure 2.5.1-38, shows a detailed, site-specific stratigraphic column, including sedimentary and depth-to-basement data, based on borehole B-1003, drilled within the VEGP site area.

The applicant described basement rock in the site area in SSAR Section 2.5.1.2.3.1. Basement lithologies consist of Paleozoic (543 to 248 mya) crystalline rock underlying Coastal Plain sediments in the northwestern portion of the site area, and sedimentary rock of the Dunbarton Triassic Basin beneath Coastal Plain sediments in the southeastern part. Based on logs from borehole B-1003 and inferences from seismic reflection and refraction surveys performed as part of the ESP investigation program, the applicant indicated that Triassic basement at the site occurs at a depth of 318 m (1,049 feet), or 250 m (826 ft) below mean sea level. The applicant stated that rocks of the Dunbarton Basin consist of mudstones, sandstones, and conglomerates with varying degrees of lithification based on borehole B-1003.

The applicant described site area Coastal Plain stratigraphy in SSAR Section 2.5.1.2.3.2, including the Cretaceous (144 to 65 mya), Tertiary (65 to 2 mya), and Quaternary (1.8 mya to present) stratigraphy. Weakly consolidated to unconsolidated Coastal Plain sediments that dip and thicken to the southeast unconformably (i.e., not succeeding the underlying rocks in immediate order of age and not fitting together with them as part of a continuous sequence) overlie Paleozoic (543 to 248 mya) and Triassic (248 to 206 mya) basement rocks in the site area. These units range in age from Upper Cretaceous (100 to 65 mya) to Miocene (23.8 to 5.3 mya) and are about 318 m (1,049 ft) thick in the site area.

The upper Cretaceous (100 to 65 mya) stratigraphic units logged in borehole B-1003, which unconformably overlie basement rocks, include the Cape Fear, Pio Nono, Upper Gaillard/Black Creek, and Steel Creek Formations. The applicant stated that these Upper Cretaceous units are primarily a mix of stratified sands, silts, clays, and gravels deposited in a fluvial deltaic environment.

AGE				UNIT	DEPTH (FT)	ELEVATION (FT MSL)
Cenozoic	TerTiary	Eocene	Upper	Barnwell Group	Ground surface	+223
				• Tobacco Road Sand		
				• Dry Branch Formation		
				• Clinchfield Formation		
				o Utley Limestone Member	48	+175
			Middle	Claiborne Group		
				• Lisbon Formation	86	+137
				o Blue Bluff Member / McBean Member		
				• Still Branch Sand	149	+74
				• Congaree Formation	216	+7
			Lower			
		Paleocene	Upper	• Snapp Formation	331	-108
				• Black Mingo Formation	438	-215
			Lower			
Mesozoic	Cretaceous	Upper			477	-254
				• Steel Creek Formation		
				• Gaillard Formation/ Black Creek Formation	587	-364
				• Pio Nono Formation / Unnamed Sand	798	-575
				• Cape Fear Formation	858	-635
	Triassic			Triassic (Dunbarton) basin	1049	-826
					Boring terminated at 1338	

Figure 2.5.1-7 - Site Stratigraphic Column Based on Boring B-1003 (Reproduced from SSAR Figure 2.5.1-38)

Tertiary (65 to 2 mya) sediments ranging in age from Paleocene (65 to 54.8 mya) to Miocene (23.8-5.3 mya), unconformably overlie the Upper Cretaceous (100 to 65 mya) section in the site area and include the following formations: Black Mingo, Snapp, Congaree, Still Branch Sand, Lisbon, Clinchfield, Dry Branch, Tobacco Road, and Hawthorne of the Barnwell Group, and the Pinehurst. The applicant stated that the Tobacco Road and Hawthorne Formations of the Barnwell Group and the Pinehurst Formation were not identified in any site borings but do occur in the site area. The applicant indicated that fluvial deposits at the base of the Tertiary give way to marginal marine, shallow shelf, mixed inner-tidal deposits, and to high-energy fluvial deposits.

The applicant reported that the Tertiary age (65 to 2 mya) Lisbon Formation includes the extensively mapped, shallow-shelf Blue Bluff Marl, which is the foundation-bearing stratigraphic unit for VEGP Units 1 and 2. This unit is the dominant facies in the VEGP site area and contains shell fragments suspended in a fine-grained micrite (carbonate-rich mud) matrix with occasional shell-rich zones and a carbonate unit referred to as the McBean Limestone.

The applicant reported that Quaternary age (1.8 mya to present) sediments occur as alluvium in stream and river valleys, forming terraces above the modern (Holocene age) flood plain of the Savannah River in the ESP site area. The applicant stated that these terraces are Pleistocene in age.

2.5.1.1.4 Site Area Structural Geology

In SSAR Section 2.5.1.2.4, the applicant reviewed published information to identify four faults and one monoclinal fold within a 5-mile radius of the VEGP ESP site. The four identified faults, each of which originates in basement rock underlying the Coastal Plain sediments, include the Pen Branch, Ellenton, Steel Creek and Upper Three Runs faults. The applicant interpreted the Upper Three Runs and Steel Creek faults as being incapable structures based on the fact that they are restricted to basement rock units and show no evidence that they have offset overlying Coastal Plain sediments. The Ellenton fault is no longer projected on updated fault maps and is considered by the applicant to be an incapable tectonic structure, if it does exist. The Pen Branch fault was examined in detail by the applicant and is discussed in detail below. The northeast-southwest trending monoclinal fold, located in the Blue Bluff Marl, was interpreted by the applicant to be spatially associated with the Pen Branch fault and potentially indicative of reverse fault movement on the Pen Branch.

In addition to reviewing published data, the applicant presented new information from seismic reflection and refraction surveys as well as from an evaluation of Quaternary age fluvial terraces overlying the Pen Branch Fault. The applicant collected this information for the ESP application specifically to determine whether the Pen Branch Fault is a capable tectonic feature. The applicant concluded that the structural geology of the site area poses no safety issues for the ESP site and that the Pen Branch Fault exhibits no Quaternary displacement and does not require further analysis for seismic hazard or surface faulting at the site.

Faults, Folds, Lineaments, Deformation Zones

The Pen Branch fault was first discovered in the subsurface of the SRS. Based on borehole and seismic reflection data, it is interpreted to exceed 40 km (25 mi) in length; to comprise several subparallel, northeast striking, southeast dipping segments; and to project southwestward beneath the VEGP ESP site. Although the Pen Branch fault is interpreted to be a non-capable structure from previous investigations by Bechtel (1989), Snipes et al. (1989), Geomatrix (1993), and Cumbest et al. (1998), the applicant conducted a detailed investigation of the fault based on its proximity to the VEGP site, and presented the findings from that investigation in SSAR Section 2.5.1.2.4.1.

The applicant conducted a review of previous investigations of the Pen Branch fault as a basis for conducting its own investigation. The applicant collected and processed seismic reflection and refraction data at the VEGP site to better characterize the fault parameters. Finally, the applicant undertook a focused geomorphic study to survey and interpret remnants of a Quaternary (1.8 mya to present) river terrace (the Ellenton Terrace), including mapping, collection of elevation data, and construction of a longitudinal profile of the terrace.

The applicant reviewed 17 years of previous investigations of the Pen Branch fault and provided a brief historical interpretation in SSAR Section 2.5.1.2.4.1. The Pen Branch fault is interpreted to be the western boundary fault of the Dunbarton Triassic Basin that juxtaposes Paleozoic (543 to 248 mya) crystalline rock against Triassic (248 to 206 mya) sedimentary rock. Seismic reflection data identifies a maximum vertical separation of the contact between basement rocks and Coastal plain sediments of about 28 m (92 ft), with offset decreasing upward into the Coastal Plain stratigraphic section. There is no evidence for post-Eocene (54.8 to 33.7 mya) displacement in previous subsurface investigations of the Pen Branch fault, which prompted Crone and Wheeler (2000) to assign the Pen Branch fault as a Class C feature.

In January and February 2006, the applicant collected seismic reflection and refraction data along four lines designed to image the Pen Branch fault and assess depth and character of basement rocks beneath the Coastal Plain sediments in the VEGP site area. Based on results of this survey, included in SSAR Section 2.5.1.2.4.2, the applicant concluded that the Pen Branch fault does indeed strike northeast, dips southeast, and lies beneath the site. Just as reported for the Pen Branch fault at the SRS, the strike of the fault beneath the VEGP is somewhat variable. Seismic sections indicate that the fault strikes about N34°E beneath the VEGP (southwest of the Savannah River), changing to about N45°E, then continuing southwest along the strike, and dipping 45°SE. Figure 2.5.1-8, reproduced from SSAR Figure 2.5.1-34, illustrates this interpreted change in strike from the SRS and across the VEGP site. The applicant also interpreted that, based on the new data, there is evidence that the Pen Branch fault intersects a monoclinal fold occurring in the Middle Eocene (54.8 to 33.7) Blue Bluff Marl. The Blue Bluff unit shows reverse fault displacement due to movement on the Pen Branch fault. Therefore the applicant concluded that Eocene age slip occurred on the Pen Branch fault.

In SSAR Section 2.5.1.2.4.3, the applicant described an evaluation of the Ellenton Terrace (Qte), a Quaternary age Savannah River terrace, located about 6 km (4 mi) east-northeast of the VEGP site, which overlies the Pen Branch Fault on the SRS and is estimated to be between 350 thousand and 1 mya old. Savannah River fluvial terraces represent the only significant Quaternary deposits and surfaces that straddle the trace of the Pen Branch fault. The applicant conducted this evaluation of the Qte to improve the resolution of the terrace surface elevation and to independently assess the presence or absence of any Quaternary tectonic deformation associated with the Pen Branch fault. This investigation included a review of previously published literature, aerial photographic analysis and geomorphic mapping, and field reconnaissance. The applicant surveyed about 2600 new elevation data points on the terrace surface and constructed a longitudinal profile approximately normal to the local strike of the Pen Branch Fault and parallel to the long axis of the terrace.

The applicant stated that results of a longitudinal profile of the Ellenton terrace surface in the study area provide evidence of no discernable tectonic deformation that can be attributed to the underlying Pen Branch fault within the resolution of the terrace elevation data, estimated to be about 1 m (3 ft). Based on this lack of evidentiary deformation in the Ellenton Qte, the absence of any post-Eocene (older than 33.7 mya) fault displacements interpreted in the seismic reflection and refraction study, and results of previous studies related to the Pen Branch fault, the applicant concluded that the Pen Branch fault is not a capable tectonic structure and that this conclusion is further supported by the previous results in Bechtel (1989), Snipes et al. (1989), Geomatrix (1993), and Cumbest et al. (1998 and 2000).

2.5.1.1.5 Site Area Earthquakes and Seismicity

Historical and Instrumentally Recorded Seismicity

The applicant summarized seismicity data in the VEGP ESP site vicinity (within a 40-km (25-mi) radius of the site) in SSAR Sections 2.5.3.1.4 and 2.5.3.3. The EPRI catalog of historical seismicity demonstrates that no known earthquake greater than mb 3 occurred within the site vicinity prior to 1984, while the SRS seismic recording network documents no recent microseismic activity (mb less than 3) within an 8 km (5 mi) radius of the VEGP site since 1976. The applicant stated that the nearest microseismic event to the VEGP ESP site was located on the SRS, about 11 km (7 m) northeast of the VEGP site. Figure 2.5.1-2, taken from SSAR Figure 2.5.1-16, shows diffuse microseismic activity recorded by the SRS seismic recording network since 1976, within a 40 km (25 mi) radius of the VEGP site.

Correlation of Earthquakes with Tectonic Features

The applicant described three small earthquakes that occurred between 1985 and 1997 with magnitudes ranging between 2.0 and 2.6 and depths ranging from 2.5 to 6 km (1.5 to 3.5 mi). In addition to these events, the applicant described a magnitude 3.2 event located north of the SRS in Aiken, SC, and a series of several small events (magnitudes ≤ 2.6) that occurred in 2001-2002 within the SRS boundaries. The applicant reviewed the locations of these events with respect to mapped faults in the ESP site vicinity–as well as previous studies of these events by Stevenson and Talwani (2004), Talwani et al. (1985), and Crone and Wheeler (2000)–and concluded that there is no spatial correlation of seismicity with known or postulated faults or geomorphic features.

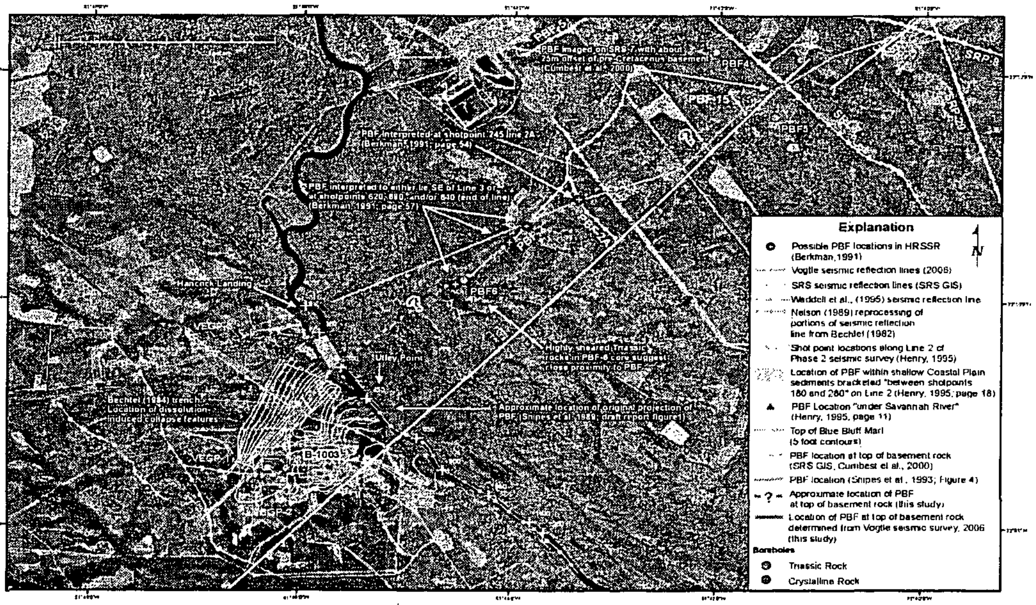


Figure 2.5.1-8 - Location of the Pen Branch Fault
(Reproduced from SSAR Figure 2.5.1-34)

2.5.1.1.6 Site Area Non-Tectonic Deformation Features

In SSAR Section 2.5.3.8, the applicant addressed the potential for the following non-tectonic deformation features at the VEGP ESP site: (1) dissolution collapse features and (2) clastic dikes.

In SSAR Section 2.5.3.8.2, the applicant discussed the potential for non-tectonic surface deformation at the ESP site, including interpretation of dissolution collapse features and "clastic dikes". Regarding dissolution collapse features discussed in SSAR Section 2.5.3.8.2.1, the applicant indicated that small-scale structures (including warped bedding, fractures, joints, minor fault offsets, and injected sand dikes) identified in the walls of a trench at the VEGP site were local features related to dissolution of the Utley Limestone (Clinchfield Formation) and subsequent collapse of overlying Tertiary sediments. The age of these features was interpreted to be younger than Eocene-Miocene host sediments and older than the overlying late-Pleistocene Pinehurst Formation. The applicant stated that no late Pleistocene or Holocene dissolution features were identified at the site. The applicant indicated that mitigation of collapse due to dissolution of the Utley Limestone, which overlies the Blue Bluff Marl at the site, could be accomplished by planned excavation and removal of the Utley to establish the foundation grade of the plant atop the Blue Bluff Marl.

In SSAR Section 2.5.3.8.2.2, the applicant addressed clastic dikes, described as relatively planar, narrow (centimeters to decimeters in width), clay-filled features that flare upwards and are decimeters to meters in length. Bechtel (1984) distinguished two types of clastic dikes in the walls of the trench on the VEGP site where dissolution collapse features were found. The first type of clastic dikes was interpreted to be "sand dikes" that resulted from injection of poorly consolidated fine sand into overlying sediments. The second type was "clastic dikes" produced by weathering and soil-formation processes that were enhanced along fractures that formed during dissolution collapse. Bechtel (1984) concluded the dikes were primarily a weathering phenomena controlled by depth of weathering and paleosol development in Coastal Plain sediments and subsequent erosion of the land surface. Clastic dike features identified by Bartholomew et al. (2002) within the site area were observed during the ESP field reconnaissance. The applicant interpreted these features to be non-tectonic in origin, although Bartholomew et al. (2002) suggested they may be evidence for paleoearthquakes associated with late Eocene to late Miocene faulting, possibly along the Pen Branch Fault.

2.5.1.1.7 Human-Induced Effects on Site Area Geologic Conditions

SSAR Section 2.5.1.2.6.5 states that no mining operation, other than borrow of surficial soils, and no excessive extraction or injection of groundwater, or impoundment of water has taken place within the site area that would impact the geologic conditions at the VEGP site.

2.5.1.1.8 Site Area Engineering Geology Evaluation

The applicant described the engineering geology evaluation of the ESP site in SSAR Section 2.5.1.2.6, including engineering soil properties and behavior of foundation materials; zones of alteration, weathering, and structural weakness; deformational zones; prior earthquake effects; and effects of human activities. In SSAR Section 2.5.1.2.6.1 for engineering soil properties and behavior of foundation materials, the applicant indicated that engineering soil properties were discussed in SSAR Section 2.5.4 and acknowledged that variability of properties in the foundation-bearing layer will be evaluated and mapped as the excavation is completed. The applicant discussed zones of alteration, weathering, and structural weakness in SSAR Section 2.5.1.2.6.2 and indicated that any desiccation, weathered zones, joints, or fractures will be mapped and evaluated as the excavation proceeds. In SSAR Section 2.5.1.2.6.4 on prior earthquake effects, the applicant stated that extensive studies of outcrops, alluvial terraces, and flood plain deposits have not shown evidence for post-Miocene (older than 5.3 mya) earthquake activity. In SSAR Section 2.5.1.2.6.5 on effects of human activities, the applicant stated that no effects resulting from human activity (e.g., mining operations, extraction or injection of groundwater, or impoundment of surface water) have occurred in the site area that affected geologic conditions at the site.

2.5.1.2 Regulatory Evaluation

The acceptance criteria for identifying basic geologic and seismic information are based on meeting the relevant requirements of 10 CFR Part 52.17 and 10 CFR Part 100.23. The staff considered the following regulatory requirements in reviewing the applicant's discussion of basic geologic and seismic information:

1. 10 CFR 52.17(a)(1)(vi), which requires that an ESP application contain a description of the geologic and seismic characteristics of the proposed site.

2. 10 CFR 100.23(c), which requires an ESP applicant to investigate geologic, seismic, and engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site; to provide sufficient information to support evaluations performed to determine the SSE Ground Motion; and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site.

3. 10 CFR 100.23(d), which requires that geologic and seismic siting factors considered for design include a determination of the SSE Ground Motion for the site; the potential for surface tectonic and non-tectonic deformation; the design bases for seismically-induced floods and water waves; and other design conditions including soil and rock stability, liquefaction potential, and natural and artificial slope stability. Siting factors and potential causes of failure to be evaluated include physical properties of materials underlying the site, ground disruption, and effects of vibratory ground motion that may affect design and operation of the proposed power plant.

The basic geologic and seismic information assembled by the applicant in compliance with the above regulatory requirements should also be sufficient to allow a determination at the COL stage of whether the proposed facility complies with the following requirements in Appendix A to 10 CFR Part 50:

1. GDC 2, which requires that SSCs important to safety be designed to withstand the effects of natural phenomena such as earthquakes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

To the extent applicable in the regulatory requirements cited above, and in accordance with RS-002, the staff applied NRC-endorsed methodologies and approaches (specified in Section 2.5.1 of NUREG-0800) for evaluation of information characterizing the geology and seismology of the proposed site as recommended in RG 1.70, Revision 3 and RG 1.165.

2.5.1.3 Technical Evaluation

This SER section presents the staff's evaluation of the geologic and seismic information submitted by the applicant in SSAR Section 2.5.1. The technical information presented in SSAR Section 2.5.1 resulted from the applicant's surface and subsurface geologic, seismic, and geotechnical investigations, which were undertaken at increasing levels of detail moving closer to the site. Through its review, the staff determined whether the applicant had complied with the applicable regulations and conducted these investigations at the appropriate levels of detail within the four circumscribed areas designated in RG 1.165, which are defined based on various distances from the site (i.e., circular areas drawn with radii of 320 km (200 mi), 40 km (25 mi), 8 km (5 m), and 1 km (0.6 mi) from the site).

SSAR Section 2.5.1 contains geologic and seismic information collected by the applicant in support of the vibratory ground motion analysis and site SSE spectrum provided in SSAR Section 2.5.2. RG 1.165 indicates that applicants may develop the SSE ground motion for a new nuclear power plant using either the EPRI or Lawrence Livermore National Laboratory (LLNL) seismic source models for the CEUS. However, RG 1.165 recommends that applicants update the geologic, seismic, and geophysical database and evaluate any new data to determine whether revisions to the EPRI or LLNL seismic source models are necessary. Consequently, the staff focused its review on geologic and seismic data published since the late 1980s to assess whether these data indicate a need for changes to the EPRI or LLNL seismic source models.

To thoroughly evaluate the geologic and seismic information presented by the applicant, the staff obtained the assistance of the USGS. The staff and its USGS advisors visited the ESP site to confirm interpretations, assumptions, and conclusions presented by the applicant related to potential geologic and seismic hazards.

2.5.1.3.1 Regional Geologic Description

In SSAR Sections 2.5.1.1.1, 2.5.1.1.2, and 2.5.1.1.3, the applicant reviewed and summarized published information related to the physiography and geomorphology (Section 2.5.1.1.1), geologic history (Section 2.5.1.1.2), and stratigraphy and geologic setting (Section 2.5.1.1.3) of the site region. Based on information presented in SSAR Sections 2.5.1.1.1, 2.5.1.1.2, and 2.5.1.1.3, the applicant concluded that the physiography, geomorphology, geologic history, stratigraphy, and geologic setting of the site region posed no safety issues for the ESP site. Consequently, the applicant considered the site suitable in regard to these specific regional features and their characteristics. The staff's evaluation of SSAR Sections 2.5.1.1.1, 2.5.1.1.2, and 2.5.1.1.3 is presented below.

<u>Physiography, Geomorphology, and Geologic History</u>

The staff focused its review of SSAR Sections 2.5.1.1.1 and 2.5.1.1.2 on the applicant's descriptions of the physiography, geomorphology, and geologic history within the site region, with an emphasis on the Quaternary Period (1.8 mya to the present). In SSAR Section 2.5.1.1.1, the applicant described each physiographic province within the site region, with emphasis on the Coastal Plain physiographic province since the ESP site is located in that province. In SSAR Section 2.5.1.1.2, the applicant described geologic history of the site region, including each episode of continental rifting and collision as well as the deposition of Coastal Plain sedimentary units found at the ESP site.

Based on its review of SSAR Sections 2.5.1.1.1 and 2.5.1.1.2, the staff concludes that the applicant presented a thorough and accurate description of the physiography, geomorphology, and geologic history of the site region in support of the ESP application as required by 10 CFR 52.17(a)(1)(vi), and 10 CFR 100.23(c), and 10 CFR 100.23(d). These two SSAR sections present well-documented geologic information, which the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined in order to ensure the accuracy of the information presented by the applicant in the SSAR.

Stratigraphy and Geologic Setting

The staff focused its review of SSAR Section 2.5.1.1.3 on the applicant's descriptions of the stratigraphy and geologic setting within the site region. The staff's review concentrated on surfaces and deposits of Quaternary age that are preserved primarily in subhorizontal fluvial terraces occurring along the Savannah River and its major tributaries. Development of fluvial terraces can be related to sequential erosion and deposition in response to faulting, climatic, isostatic (i.e., regional changes in crustal loading leading to upwarping or downwarping of portions of the earth's crust), or eustatic (i.e., global sea level changes) effects or a combination of these mechanisms. Because fluvial terrace deposits initially form as relatively level to gently inclined surfaces, the possibility exists for analyzing variations in elevations of the terrace surfaces to evaluate the potential for Quaternary deformation (i.e., tilting, warping, or offset due to fault displacement) in the site area as long as nontectonic processes, such as surficial erosion or dissolution at depth, have not strongly modified its morphology. In particular, the applicant identified a series of four abandoned fluvial terraces (Qty, Qtb, Qte, and Qto from youngest to oldest) that occur in the site area at elevations above the present-day flood plain of the Savannah River and overlie the Pen Branch fault, a structure that the applicant determined does underlie the ESP site. The applicant used these terraces to assess the presence or absence of Quaternary tectonic deformation on the Pen Branch fault.

Regarding the Pen Branch fault, the applicant analyzed seismic reflection data collected for the ESP application to determine that the fault underlies the ESP site. The fault has also been imaged beneath the SRS on the eastern side of the Savannah River, although it shows no surface expression either at the SRS or the ESP site. Although evidence from stratigraphic data discussed by the applicant in the SSAR suggests that the last motion on the Pen Branch fault was pre-Eocene (greater than 33.7 mya) in age, the applicant understood the need to analyze this fault in more detail because of its location relative to the ESP site.

In RAI 2.5.1-1, the staff asked the applicant to indicate whether the fluvial terraces (Qty, Qtb, Qte, and Qto) are regional in extent or are local features uplifted by slip along the Pen Branch fault. In response, the applicant stated that the four abandoned terraces of the Savannah River extend well beyond the vicinity of the Pen Branch fault and are regional in extent. The four terraces extend for at least 33 km (20 mi) upstream and 29 km (18 mi) downstream (i.e., straight-line distances) from the VEGP ESP site. In addition, the applicant stated that the development of a sequence of laterally extensive fluvial terraces is characteristic of other major Piedmont-draining river systems as well as the Savannah River. In conclusion, the applicant stated, "The fact that the major fluvial terrace surfaces are correlative between major Piedmont-draining river systems suggests that these terraces form in parallel response to regional climatic and/or eustatic conditions, and are not the result of local tectonic perturbations."

Based on an evaluation of the applicant's response, the staff concludes that, since the terraces are regional in extent, it is highly unlikely that they developed due to tectonic displacement along the Pen Branch fault. The trace of the fault is nearly perpendicular to the long axis of the terrace surfaces (see SSAR Figure 2.5.1-43), so the terraces are favorably oriented to register Quaternary deformation along the Pen Branch fault. Alternatively, the staff believes a more likely origin for the terraces involves regional changes in sea level relative to the continental land mass. These regional changes resulted from either climatic, isostatic, or eustatic effects or some combination of these nontectonic mechanisms. Climatic, isostatic, and eustatic perturbations alter sea level relative to the land mass on a regional scale, either by raising the sea level itself (climatic and eustatic changes) or isostatically uplifting blocks of continental crust due to regional crustal unloading (isostatic changes). The mechanism of tectonic perturbations is separate and distinct from these regional changes in sea level and would involve tectonic uplift (e.g., fault displacement) to raise a fault block and produce abandoned fluvial terraces atop that block. The staff's conclusion that the fluvial terraces developed as a result of nontectonic processes rather than by tectonic uplift is based on the staff's evaluation of the applicant's response to RAI 2.5.1-1, and subsequent RAI responses pertaining to the same subject (i.e., RAI 2.5.1-2 and RAI 2.5.1-3).

To evaluate the potential for Quaternary displacement on the Pen Branch fault, the applicant implemented a detailed investigation of fluvial terrace Qte (the Ellenton terrace) at a location approximately 6 km (4 mi) east-northeast of the ESP site. The purpose of the applicant's study was to "improve the resolution of the terrace surface elevation and independently assess the presence or absence of Quaternary tectonic deformation on the Pen Branch fault." A previous study of the fluvial terraces by Geomatrix (1993) concluded that the Pen Branch fault is not a capable tectonic source and that there is no observable deformation, within a resolution of 2-3 m (7-10 ft), of the overlying Ellenton terrace (Qte). The applicant's investigation improved on the previous investigation by surveying approximately 2600 elevation data points along the Qte terrace surface in the vicinity of the Pen Branch fault. The applicant estimated its uncertainty to be about 1 m (3 ft) and concluded that its profile of the Qte fluvial terrace surface demonstrates the absence of discernible tectonic deformation on the underlying Pen Branch fault within a 1-m (3-ft) limit of resolution for the elevation data.

In RAI 2.5.1-2, the staff asked the applicant to address whether the range in elevation of the Qtb (8 to 13 m (26 to 43 ft)) and Qte (18 to 25 m (56 to 82 ft)) terrace surfaces above the Savannah River surface can be attributed to tilting of these terrace surfaces due to Quaternary slip on the Pen Branch fault. The staff also asked the applicant to discuss the implications of the deformation detection limit of about 1 m (3 ft) for the terrace surfaces. This limit resulted from the applicant's field study. This clarification is particularly important for terrace Qte (the Ellenton terrace), which the applicant analyzed in detail to conclude that the terraces do not exhibit deformation due to Quaternary displacement along the Pen Branch fault. The applicant selected terrace surface Qte for the analysis because of its lateral extent and because it could potentially record tectonic deformation along the Pen Branch fault for up to 1 mya based on its interpreted age of 350,000 to 1 million years. The younger terraces, Qty and Qtb, covered shorter time periods, and the older terrace, Qto, exhibited too much dissection for this type of analysis. To define the best-preserved remnants of terrace surface Qte for analysis, the applicant performed geomorphic mapping and field reconnaissance studies and then surveyed approximately 2600 elevation data points on these terrace surface remnants. The applicant estimated that the overall uncertainty in elevation values of the best-preserved remnants of terrace Qte was about 1 m (3 ft) due to the presence of depressions related to dissolution collapse at depth and local deposition of alluvium and colluvium.

In response to RAI 2.5.1-2, the applicant addressed whether the terrace elevation ranges suggested tilting or warping of terrace Qte by tectonic deformation along the Pen Branch fault and the implications of the 1 m (3 ft) limit of detection for deformation. The applicant concluded that variations in elevation of the Qte terrace surface are due largely to the eroded and dissected character of terrace Qte and not from warping or tilting of the terrace by Quaternary displacement on the Pen Branch fault. The applicant cited supporting evidence that these terrace surfaces clearly exhibit a range of surface elevations resulting directly from erosion and dissection which cannot be obviously equated with displacement along the Pen Branch fault. The applicant also concluded that the deformation detection limit of 1 m (3 ft) is an improvement over that attained in previous studies and consequently acceptable for assessing the possibility of Quaternary deformation of the terrace surface due to displacement along the Pen Branch fault. The applicant stated the following:

> Work performed for the VEGP application uses the 350 ka to 1 Ma Ellenton (Qte) terrace surface as a Quaternary strain marker to assess the presence or absence of evidence for tectonic deformation across the underlying Pen Branch fault. A longitudinal profile of the Qte terrace surface in the study area provides evidence demonstrating the absence of tectonic deformation within a resolution of about 1 m (3 ft). This provides a much smaller deformation detection limit than previous studies, thereby providing greater confidence in the evidence demonstrating the lack of Quaternary deformation on the Pen Branch fault.

To completely evaluate the applicant's field study of the Qte fluvial terrace, as well as the applicant's response to RAI 2.5.1-2, the staff and its consultants visited the ESP site and examined the terrace surface. In particular, the staff focused on the adequacy of the applicant's investigations of the Qte terrace and its suitability as a strain marker to assess the presence or absence of tectonic deformation across the underlying Pen Branch fault. Based on the site visit and an examination of aerial photographs and geologic maps, the staff concludes the following:

1. The Qte fluvial terrace shows no obvious surface warping, tilting, or offset.

2. The 1 m (3 ft) detection limit is equivalent to or less than the topographic variations observed for the terrace surface.

3. The variations in elevation of the Qte terrace surface are likely the result of the eroded and dissected character of the Qte surface rather than tectonic tilting and warping due to Quaternary displacement along the Pen Branch fault.

4. The deformation detection limit of 1 m (3 ft), which the applicant achieved during the ESP-related terrace investigations, is a great improvement over previous studies and is a reasonable limit based on measured variability detected in elevation of this terrace surface due to erosion and dissection of the terrace.

SER Figure 2.5.1-9 is a photograph of the Qte fluvial terrace taken during the site visit by the NRC staff and its USGS consultants. This photograph illustrates the relatively flat terrace surface extending a considerable distance toward the horizon, and reinforces the interpretation of the applicant that this terrace surface is not offset by displacement along the Pen Branch fault.

In RAI 2.5.1-3, the staff asked the applicant to discuss the use of the youngest terrace, Qty (4,000 to 90,000 years in age), as an indicator for more recent (i.e., Holocene (10,000 years to the present in age)) potential displacement or uplift along the underlying Pen Branch fault. In response to RAI 2.5.1-3, the applicant stated the following:

> The discontinuous Qty terrace surface of late Pleistocene to possibly Holocene age does not provide constraints for evaluating the potential for Quaternary displacement on the Pen Branch fault. The significantly older and more laterally continuous remnants of the 350 ka to 1 Ma (Geomatrix, 1993) Ellenton terrace (Qte) provide a more robust datum to evaluate potential tectonic deformation on the Pen Branch fault.

The applicant concluded that the discontinuous nature of terrace Qty does not provide adequate constraint for evaluating the potential for Quaternary displacement on the Pen Branch fault. The applicant cited supporting technical evidence derived from field observations and mapping that the terrace is too discontinuous to permit construction of a longitudinal profile for properly assessing tilting and warping of the terrace surface. The applicant also concluded that terrace Qty is not developed only near the Pen Branch fault and cited evidence derived from its field observations and mapping that the Qty terrace extends outside the site area.

After review of the applicant's response to RAI 2.5.1-3, as well as geologic field maps of the area, the staff concurs with the applicant's conclusions that terrace Qty is too discontinuous to be a suitable strain marker for deformation of the terrace surface or the underlying strata. Furthermore, the terrace extends beyond the location of the Pen Branch fault. The staff also agrees with the applicant that terrace Qte provides a much more robust indicator for potential Quaternary displacement of the underlying Pen Branch fault than terrace Qty.

Based on review of SSAR Section 2.5.1.1.3, the staff concludes that the applicant presented a thorough and accurate description of the regional stratigraphy and geologic setting in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c) and 10 CFR 100.23(d). In addition, based on observations made during the site visit and review of the applicant's responses to RAI 2.5.1-1 through RAI 2.5.1-3, the staff concludes that the applicant's detailed examination of fluvial terrace surface Qte demonstrates the absence of significant Quaternary displacement on the underlying Pen Branch fault. As a result, the staff concurs with the applicant's conclusion that the Pen Branch Fault is not a capable tectonic structure (as defined by RG 1.165).

2.5.1.3.2 Regional Tectonic Description

In SSAR Sections 2.5.1.1.4 and 2.5.1.1.5, the applicant reviewed and summarized published information related to the tectonic setting (Section 2.5.1.1.4) and gravity and magnetic data (Section 2.5.1.1.5) of the site region. Based on information presented in SSAR Sections 2.5.1.1.4 and 2.5.1.1.5, the applicant concluded the following:

1. Tectonic features in the site region include structures that are Paleozoic (greater than 248 mya), Mesozoic (248 to 65 mya), Tertiary (65 to 1.8 mya), and Quaternary (1.8 mya to present) in age. Only structures of Quaternary age warrant further consideration for the ESP site with regard to the potential for surface fault displacement and seismic hazards.

2. Of the 11 regional geologic features assessed with regard to their potential for Quaternary activity, only the paleoliquefaction features associated with the 1886

Charleston earthquake clearly demonstrate the existence of a Quaternary tectonic feature.

3. Based on more recent information derived from other investigators on source geometry and earthquake recurrence rates for the Charleston seismic source, the 1986 EPRI Charleston seismic source models need to be updated.

4. All regional seismic source zones, other than the Charleston seismic source zone, have less influence on the ESP site due to their distance from the site. The Charleston seismic source model dominates the ground motion hazard for the ESP site.

5. Within the site region, there is no spatial correlation of earthquake epicenters with known or postulated faults. In general, earthquakes occurring in the South Carolina and Georgia portions of the Coastal Plain and Piedmont provinces are not concentrated or aligned with any mapped faults.

The staff's evaluation of SSAR Sections 2.5.1.1.4 (including SSAR Sections 2.5.1.1.4.1 through 2.5.1.1.4.6) and 2.5.1.1.5 (including SSAR Sections 2.5.1.1.5.1 and 2.5.1.1.5.2) is presented below.

Plate Tectonic Evolution and Stress Field

The staff focused its review of SSAR Sections 2.5.1.1.4.1 and 2.5.1.1.4.2 on the applicant's descriptions of plate tectonic evolution and tectonic stresses within the site region, with an emphasis on the Quaternary Period (1.8 mya to present). In SSAR Section 2.5.1.1.4.1, the applicant described plate tectonic evolution of the Appalachian orogenic belt at the latitude of the site region. The applicant stated that stratigraphic units of the Coastal Plain, the province within which the ESP site lies, record development of a passive continental margin along the east coast of the United States that followed Mesozoic extensional rifting and the opening of the present-day Atlantic Ocean basin. In SSAR Section 2.5.1.1.4.2, the applicant described a detailed study of the orientations and magnitudes of the principal tectonic stresses performed by Moos and Zoback (1992) for the SRS. The applicant stated that the regional stress analyses performed for the CEUS, including the study performed by Moos and Zoback (1992), which characterized a northeast-southwest orientation for the maximum principal compressive stress, did not suggest a need to alter the seismic source models developed by EPRI (1986).
Based on its review of SSAR Sections 2.5.1.1.4.1 and 2.5.1.1.4.2, the staff concludes that the applicant presented a thorough and accurate description of plate tectonic evolutionary history and tectonic stress for the site region in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23(c), and 10 CFR 100.23(d). These two SSAR sections present well-documented geologic information, which the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff used to confirm the accuracy of the information in the SSAR.

Principal Regional Tectonic Structures

The staff focused its review of SSAR Section 2.5.1.1.4.3 on the applicant's descriptions of tectonic structures (principally faults), with emphasis on the Quaternary Period. In SSAR Section 2.5.1.1.4.3, the applicant described the principal regional tectonic structures based on the age of formation or reactivation of the structures, including those of Paleozoic (greater than 248 mya), Mesozoic (248 to 65 mya), Tertiary (65 to 1.8 mya), and Quaternary (1.8 mya to the present) age. The staff's evaluation of SSAR Section 2.5.1.1.4.3 is presented below.

Figure 2.5.1-9 - Photograph of the relatively horizontal remnant of fluvial terrace Qte (the Ellenton terrace, dated at 1 Ma to 350 ka years old) which occurs on the eastern side of the Savannah River on SRS property and crosses the trace of the Pen Branch fault. This terrace surface exhibits no tilting, warping, or offset due to Quaternary (1.8 mya to the present) displacement along the Pen Branch fault.

Paleozoic Tectonic Structures. The applicant described the Paleozoic tectonic structures that are located in the site region—the Augusta fault zone, Modoc fault zone, Central Piedmont Suture, Eastern Piedmont Fault System, and the Brevard, Hayesville, and Towaliga faults. The applicant concluded that (1) there is no seismicity that can be associated with any of these Paleozoic features; (2) none of the structures are capable tectonic sources; and (3) there is no new information associated with these Paleozoic structures that would necessitate an update of the EPRI (1986) seismic source models.

In SSAR Section 2.5.1.1.4.3, the applicant described two distinct deformation fabrics that are contained in both the Augusta and Modoc fault zones. These deformation fabrics suggest that more than one phase of tectonic deformation may have occurred in these zones. Specifically, the applicant stated that a brittle deformation fabric overprinted (i.e., postdated) formation of a ductile deformation fabric in the Augusta and Modoc fault zones. In RAI 2.5.1-5, the staff asked the applicant to clarify whether the brittle fabric may have formed during a post-Alleghanian deformation event (e.g., during the Quaternary). This clarification is important to document that these two structures are old tectonic features exhibiting no evidence for reactivation during Quaternary time.

In response to RAI 2.5.1-5, the applicant addressed the timing of the development of these two deformation fabrics. The applicant concluded that the brittle deformation fabrics associated with the Augusta and Modoc fault zones, which postdate the ductile mylonitic deformation fabrics in the zones, are either late Alleghanian (greater than 248 mya, at the end of the Paleozoic) or early Mesozoic in age and do not represent Quaternary reactivation in the modern-day stress regime. The applicant cited several supporting lines of evidence for this conclusion:

1. Both the brittle and ductile fabrics exhibit similar movement directions (i.e., similar kinematic histories) during deformation.

2. The observed normal components of brittle movement are not compatible with the modern-day stress field.

3. The observed mineralization of some brittle fabrics exposed at the surface (e.g., silicification of breccias and growth of zeolite minerals and epidote) cannot form under modern-day geologic and hydrothermal conditions.

Based on its review of the applicant's response to RAI 2.5.1-5, the staff concludes that the brittle deformation fabrics do not represent Quaternary deformation, or deformation in the modern-day stress field, along the Augusta or Modoc fault zones. In particular, the staff concurs with the applicant's assertion that the normal components of the brittle movement are incompatible with the modern-day stress regime (i.e., currently a northeast to east-northeast-trending orientation of maximum principal compressive stress) indicating that these fabrics could have developed only as the result of an earlier stress field. The movement history for the brittle deformation fabrics is compatible with the stress field associated with Alleghanian orogeny at the end of the Paleozoic (greater than 248 mya), such that the brittle fabrics of both the Augusta and Modoc fault zones are considerably older than Quaternary. As the applicant stated, Maher et al. (1994) suggest Alleghanian extensional movement along the Augusta fault zone about 274 mya, and Dallmeyer et al (1986) suggest extensional movement of the Modoc fault zone from 310 to 290 mya. Based on this information, the staff also concludes that it is not necessary for the applicant to reassess the seismic hazard potential of these regional structures for the ESP site.

In RAI 2.5.1-6, the staff asked the applicant to include the Central Piedmont Suture and the Eastern Piedmont Fault System on a corrected SSAR Figure 2.5.1-14. In response to this RAI, the applicant confirmed that this correction would be made in the next revision of the ESP application. The staff confirmed that this change was made in revision 2 to the SSAR.

<u>Mesozoic Tectonic Structures</u>. The applicant discussed Mesozoic tectonic structures in SSAR Section 2.5.1.1.4.3, noting that the Dunbarton Triassic basin, an east-northeast-trending Mesozoic (i.e., Triassic (248 to 206 mya)) extensional rift basin, is located beneath both the ESP site and the SRS. The extensional Dunbarton Triassic basin is bounded on its northwest side by the Pen Branch fault, a structure determined by the applicant to underlie the ESP site and to exhibit rejuvenation as an oblique-slip reverse fault during the Cenozoic (65 mya to present) after earlier normal fault displacement during the Mesozoic (248 to 65 mya). The applicant presented a detailed assessment of the potential for Quaternary (1.8 mya to present) displacement along the Pen Branch fault in SSAR Section 2.5.1.2.4. The staff's evaluation of SSAR Section 2.5.1.2.4 is presented in SER Section 2.5.1.3.4.

With regard to regional Mesozoic extensional tectonic terranes, the applicant recognized that areas of extended crust (e.g., such as the eastern part of the Piedmont and beneath the Coastal Plain province in the southeastern United States) may host large earthquakes that are associated spatially with buried faults initially developed in response to extensional rifting. The Pen Branch fault, which forms the northwest boundary of the Dunbarton Triassic basin, is such a fault. The applicant indicated that these buried faults which bound the Triassic basins may be either listric (i.e., a fault with a dip angle that decreases with depth) or a high-angle fault. In RAI 2.5.1-9, the staff asked the applicant to discuss whether there is any evidence that these buried normal faults are listric or are high-angle faults that could extend through the crust to depths where larger magnitude earthquakes commonly nucleate. In response, the applicant stated the following:

> Data constraining the down-dip geometry of faults that bound Mesozoic basins are equivocal. Seismic reflection data, borehole studies, gravity and magnetic signatures, and geologic mapping have all been used to characterize these faults, but different studies have depicted these faults as both listric and high-angle features. The effects of these two possible geometries on hazard at the site are highly uncertain, but both geometries can produce moderate-to-large magnitude earthquakes on seismogenic structures. Because of the uncertainty regarding their geometry, the EPRI ESTs used area sources instead of individual fault sources to represent these basin-bounding faults in the PSHA.

Due to the uncertainty in the location and subsurface geometry of these faults that bound Mesozoic basins, the staff concurs with the applicant's use of area source zones. Rather than characterizing the seismic potential of each identified or postulated fault, seismic hazard studies for the CEUS generally define broad area seismic source zones. Both the EPRI and LLNL seismic source models use this approach, which is endorsed by RG 1.165. Therefore, the staff concludes that the applicant's response to RAI 2.5.1-9 is adequate and that the applicant has conservatively modeled the seismic sources in the region surrounding the ESP site by using area sources rather than individual fault sources.

Tertiary Tectonic Structures. The applicant described Tertiary tectonic structures in SSAR Section 2.5.1.1.4.3. Within 200 miles of the ESP site only a few tectonic features were active during the Tertiary Period (65 to 1.8 mya). The two most prominent Tertiary structures are the Cape Fear Arch on the South Carolina-North Carolina border and the Yamacraw Arch on the Georgia-South Carolina border. Based on Crone and Wheeler (2000), the applicant concluded that these features do not exhibit any evidence for Quaternary faulting.

Quaternary Tectonic Structures. The applicant discussed potential Quaternary tectonic structures in the region surrounding the ESP site in SSAR Section 2.5.1.1.4.3. To evaluate each of these potential Quaternary features, the applicant used the database of Quaternary tectonic features developed by Crone and Wheeler (2000) and Wheeler (2005) for the CEUS. These two studies present a compilation and description of the faults, paleoliquefaction features, seismic zones, and geomorphic features that may have been active or capable during the Quaternary period. Crone and Wheeler categorize each feature as fitting into one of four "fault classes" (Classes A, B, C, D) based on geologic evidence for Quaternary deformation. This categorization is determined from the authors' survey of the published literature rather than from direct field examination of the features. These four fault classes are defined by Crone and Wheeler (2000) and Wheeler (2005) as follows:

1. Class A—Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether mapped or inferred from liquefaction or other features.

2. Class B—Geologic evidence demonstrates the existence of Quaternary deformation, but either the fault may not cut deeply enough to be a potential earthquake source or available geologic evidence is too strong to assign the feature to Class C but not strong enough to assign it to Class A.

3. Class C—Geologic evidence is insufficient to demonstrate the existence of tectonic faulting or Quaternary deformation associated with the feature.

4. Class D—Geologic evidence demonstrates that the feature is not a tectonic fault.

Using Crone and Wheeler (2000) and Wheeler (2005), the applicant identified the following potential Quaternary tectonic features in the region surrounding the ESP site:

- Charleston, Georgetown, and Bluffton paleoliquefaction features (Class A)
- ECFS (Class C)
- Cooke fault (Class C)
- Helena Banks fault zone (Class C)
- Pen Branch fault (Class C)
- Belair fault zone (Class C)
- Fall Lines of Weems (Class C)
- Cape Fear Arch (Class C)
- ETSZ (Class C)

The applicant discussed Charleston features (including the ECFS, the Cooke fault, the Helena Banks fault zone, and the Charleston, Georgetown, and Bluffton paleoliquefaction features) in detail in SSAR Section 2.5.1.1.4.4. The applicant presented its detailed analysis of the Pen Branch fault in SSAR Section 2.5.1.2.4 and discussed the ETSZ in SSAR Section 2.5.1.1.4.6. The applicant evaluated the remaining features (i.e., the Belair fault zone, the Fall Lines of Weems, and the Cape Fear Arch) in SSAR Section 2.5.1.1.4.3. The staff's evaluation of those three remaining features is presented below.

Belair Fault Zone

As mapped, the Belair fault zone is located about 20 km (12 mi) north-northwest of the ESP site and is at least 25 km (15 mi) in length. The applicant indicated that undeformed strata overlying the disrupted stratigraphic units constrain the last episode of displacement along this fault zone between post-Late Eocene and pre-26,000 years ago, allowing for Cenozoic (i.e., 65 mya to present), including Quaternary, displacement along the fault zone. The applicant also stated that the Belair fault zone is probably a tear fault or lateral ramp in the hanging wall of the Augusta fault zone. If this association between the Augusta and Belair fault zones exists, then movement on the Belair zone may be related to displacement on the longer, regional-scale Augusta fault zone. In RAI 2.5.1-10, the staff asked the applicant to explain how the inference of Cenozoic displacement on the Belair fault zone and a possible association with the regional Augusta fault zone might affect seismic hazard for the ESP site. This clarification is important to document whether the Belair fault zone is structurally linked with the Augusta fault zone and whether it has experienced displacement during the Quaternary.

In its response to RAI 2.5.1-10, the applicant addressed the possibility of a connection between the Belair and Augusta fault zones. The applicant stated that timing and sense-of-slip for the most recent movements on the Belair and Augusta faults demonstrate that these two structures did not respond as a single tectonic element in Cenozoic or younger time. Prowell et al. (1975) and Prowell and O'Connor (1978) document brittle failure due to reverse slip on the Belair fault in the Cenozoic (65 mya to present). In contrast, the applicant stated that the latest movement on the Augusta fault, as demonstrated by brittle overprinting of ductile fabrics, exhibits a normal sense-of-slip which is constrained to late Alleghanian time (greater than 248 mya) based on Maher (1987) and Maher et al. (1994). The applicant acknowledged that Crone and Wheeler (2000) classified the Belair fault zone as Class C, suggesting Quaternary slip on the Belair fault is allowed but not demonstrated by geologic data. The applicant concluded, based on the evidence supporting different slip histories and opposite senses of dip-slip for the Belair and Augusta faults, that reactivation of these two faults as a single structure during the Cenozoic is not indicated.

Based on its review of the applicant's response to RAI 2.5.1-10, the staff concludes that the Belair and Augusta fault zones are not currently linked tectonic features. In particular, the staff concurs that there is strong field evidence for different slip histories and opposite senses of dip-slip for the Belair and Augusta faults and no indication that the structures were reactivated as a single structure during the Cenozoic.

Fall Lines of Weems (1998)

The applicant discussed a series of anomalously steep stream segments derived by Weems (1998) from a study of longitudinal profiles of streams flowing across the Blue Ridge and Piedmont physiographic provinces in North Carolina, Virginia, and Tennessee. Weems (1998) noted that these steep stream segments occurred as seven "fall zones" that were generally subparallel to the northeast-southeast regional "grain" of the Blue Ridge and Piedmont provinces as reflected by physiography, lithologic belts, and regional tectonic features. Weems (1998) suggested three hypotheses to explain this phenomenon, including climatic factors, rock characteristics, and neotectonic effects (i.e., tectonic deformation that is post-Miocene, or greater than 5.3 mya, in age). The applicant stated that the Fall Lines of Weems are classified as Class C features by Wheeler (2005) since they do not demonstrate Quaternary age deformation. Consequently, the applicant concluded that these features do not represent Quaternary faulting in the site region.

Cape Fear Arch

The Cape Fear Arch is a topographic high located on the South Carolina-North Carolina border which is bounded by the Salisbury embayment topographic low to the northeast and the Georgia embayment low to the southeast. The applicant stated that the Cape Fear Arch, a feature previously discussed under the section on tertiary tectonic structures, was classified as Class C by Crone and Wheeler (2000) based on a lack of evidence for Quaternary faulting. The applicant concluded that this feature does not exhibit evidence of Quaternary faulting in light of the Crone and Wheeler (2000) classification and that there is no existing evidence to indicate this feature is a tectonically active structure.

Based on its review of SSAR Section 2.5.1.1.4.3 related to a discussion of faults, the staff concludes that the applicant presented a thorough and accurate description of regional Paleozoic, Mesozoic, Tertiary, and Quaternary tectonic deformation features in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and

10 CFR 100.23(d). In addition, based on its review of the applicant's responses to RAI 2.5.1-5, RAI 2.5.1-6 and RAI 2.5.1-9, the staff concludes that regional Paleozoic (greater than 248 mya), Mesozoic (248–65 mya), and Tertiary (65–1.8 mya) features are older structures that do not exhibit Quaternary deformation, and no further assessment of seismic hazard potential in relation to any of these regional structures is necessary for the ESP site.

In regard to Quaternary structures discussed by the applicant in SSAR Section 2.5.1.1.4.3, the staff concurs with the applicant that there is strong field evidence for different slip histories and opposite senses of dip-slip for the Belair and Augusta faults, as the applicant qualified in the response to RAI 2.5.1-10. The staff further concurs with the applicant that these structures did not reactivate as a single, linked structure during Cenozoic time (65 mya to present, which includes the Quaternary). In addition, concerning Quaternary history for the seven Fall Lines of Weems (1998), the citation by the applicant of Wheeler (2005) as the primary basis for assessing the potential for Quaternary activity, in relation to the fall lines, is deemed insufficient by the staff. From previous analysis of these features in connection with the SER for North Anna (see NUREG-1835, "Safety Evaluation Report for an Early Site Permit (ESP) at the North Anna ESP Site," issued September 2005), the staff concludes that differential erosion resulting from variable hardness in rock units is a more plausible origin for the fall lines than Quaternary tectonism. The staff further notes that interpretation of the fall lines as Quaternary tectonic features comes solely from Weems, and no other investigators have suggested this origin. Concerning Quaternary activity for the Cape Fear Arch, the staff concurs with the applicant that there is no existing evidence to indicate that this feature is a tectonic structure exhibiting Quaternary deformation.

Furthermore, the staff concurs with the applicant that potential seismic effects of tectonic structures are fully incorporated into PSHA, because area sources, rather than individual fault sources, are used to capture tectonic features in PSHA. Therefore, the staff believes that specific regional structures need not be defined for PSHA and concludes that the applicant thoroughly evaluated the seismic potential for each of the faults in the site region to determine whether the EPRI PSHA source models require updating.

Principal Regional Tectonic Structures—Charleston

The staff focused its review of SSAR Section 2.5.1.1.4.4 on potential Charleston-area source faults, seismic zones, and liquefaction features, with emphasis on the Quaternary Period. In SSAR Section 2.5.1.1.4.4, the applicant described Charleston tectonic features, including potential source faults, seismic zones, and seismically induced liquefaction features. Analysis of Charleston tectonic features is very important in regard to a potential seismic hazard at the ESP site because the earthquake that occurred in 1886 in the Charleston area is one of the largest historical earthquakes ever to occur within the eastern United States and its source is certain to occur within the ESP site region. After a review of more recent geologic investigations in the Charleston area (some of which described liquefaction features related to the 1886 Charleston earthquake and earlier events likely generated from the same seismic source), the applicant concluded that significant new information related to source geometry and earthquake recurrence rate for the Charleston seismic source warrants an update of the EPRI (1986) source models used in the PSHA. The applicant presented and discussed these updated seismic source parameters for the 1886 Charleston earthquake in SSAR Section 2.5.2.2.2.4 The staff's evaluation of SSAR Section 2.5.1.1.4.4 is presented below.

<u>Potential Source Faults for Charleston</u>. The applicant recognized that no known tectonic source exists for the 1886 Charleston earthquake. Consequently, location of a "Charleston tectonic source" is based on historical reports of damage and occurrence of seismically induced liquefaction features to define an area rather than a specific source fault. The applicant discussed nine potential tectonic source faults for the 1886 Charleston earthquake—the ECFS, Adams Run fault, Ashley River fault, Charleston fault, Cooke fault, Helena Banks fault zone, Sawmill Branch fault, Summerville fault, and Woodstock fault. The applicant concluded that no specific linkage between any of these features and the 1886 Charleston earthquake could be proposed based on geomorphic, geologic, borehole, or seismic evidence. The applicant's discussion of potential tectonic source features for the 1886 Charleston earthquake did not include two faults shown on SSAR Figures 2.5.1-19 and 2.5.1-20 to occur in the meizoseismal area (i.e., the area of maximum damage to structures resulting from the earthquake) of the Charleston earthquake, namely the Gants and Drayton faults. The staff asked, in RAI 2.5.1-13, the applicant to acquire additional descriptive information on these two faults to enable a thorough review of all faults postulated to occur in the meizoseismal area of the 1886 Charleston earthquake.

In response to RAI 2.5.1-13, the applicant provided descriptive information for the Gants and Drayton faults. For the Drayton fault, the applicant concluded that Cenozoic (65 mya to present), and consequently Quaternary (1.8 mya to present), displacement is precluded based on interpretations of seismic reflection data (Hamilton et al., 1983) which suggest that the fault terminates at a depth of about 750 m (2500 ft) below the ground surface in a Jurassic (206 to 144 mya) basalt layer. For the Gants fault, the applicant concluded that seismic reflection data suggested that the fault may disrupt Cenozoic strata, but with decreasing displacement during Cenozoic time. The conclusions drawn by the applicant for both the Gants and Drayton faults are, therefore, supported by the evidence derived from seismic reflection data, as neither fault exhibits any surface expression.

Based on its review of the applicant's response to RAI 2.5.1-13, the staff concludes that the response provides an adequate description of the Gants and Drayton faults. The staff also concludes that neither of these two faults exhibit any obvious linkage to the 1886 Charleston earthquake in space or time. Because the applicant could not correlate this earthquake with any of the nine potential source faults discussed in SSAR Section 2.5.1.1.4.4, including the Gants and Drayton faults, and uncertainty remains in selecting a specific tectonic source, the staff considers it important that the applicant incorporate the new information on source geometry and earthquake recurrence rate for the 1886 Charleston earthquake into the seismic source models for Charleston. The applicant incorporated these new data into the analyses discussed in SSAR Section 2.5.2.2.2.4 (seismic potential for a Charleston source fault is captured in PSHA by use of a source area rather than a specific tectonic structure for the Charleston area).

<u>Potential Seismic Source Zones for Charleston</u>. Regarding seismic source zones for the 1886 Charleston earthquake, the applicant discussed three zones of increasing seismicity identified in the Charleston area. The zones include the Middleton Place-Summerville, Bowman, and Adams Run seismic zones. The characteristics of these zones are discussed in SSAR Section 2.5.1.1.4.4 and SER Section 2.5.1.1.2. The applicant reached no specific conclusions regarding these three seismic zones in SSAR Section 2.5.1.1.4.4. Details related to specific data in the seismicity catalog for these three zones are discussed in SSAR Section 2.5.2.1. The staff found the descriptions of the seismic source zones, based on published literature (provided by the applicant in SSAR Section 2.5.1.1.4.4) to be acceptable.

Charleston Area Liquefaction Features. Regarding seismically induced liquefaction features in the Charleston area, the applicant stated that such features produced by the 1886 Charleston earthquake are most heavily concentrated in the meizoseismal area for that earthquake. The applicant also reported the locations of prehistoric liquefaction features related to significant seismic events that pre-dated the 1886 Charleston earthquake, but likewise interpreted to most likely have been generated by the same tectonic source. The applicant indicated that, based on consideration of these prehistoric liquefaction data, Talwani and Schaeffer (2001) suggested a mean recurrence interval of 550 years for a Charleston-type earthquake. This interval is roughly an order of magnitude less than the seismicity-based estimates used by EPRI (1986) to characterize recurrence interval for earthquakes generated by the Charleston seismic source. Based on the identification of earthquakes pre-dating the 1886 Charleston seismic event from the prehistoric liquefaction features, the applicant refined earthquake recurrence rate estimates for a Charleston-area earthquake in SSAR Section 2.5.2.2.2.4. The applicant made no specific conclusions regarding seismically induced liquefaction features in SSAR Section 2.5.1.1.4.4.

With regard to liquefaction features in the Charleston area, the staff found that the descriptions of these features provided by the applicant in SSAR Section 2.5.1.1.4.4 needed clarification. To better correlate liquefaction features with proposed tectonic sources, in RAI 2.5.1-11, the staff asked the applicant to include new figures that clearly distinguished liquefaction features related to the 1886 Charleston earthquake from the prehistoric liquefaction events shown in SSAR Figure 2.5.1-19. In RAI 2.5.1-12, the staff asked the applicant to include an additional pertinent reference by Bollinger (1977). The applicant provided the new figures and the reference in its responses to RAI 2.5.1-11 and RAI 2.5.1-12.

The staff concludes that the applicant presented a thorough and accurate geologic description of Charleston tectonic features (including potential source faults, seismic source zones, and liquefaction features) in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). In addition, based on its review of the information presented by the applicant on Charleston tectonic features in SSAR Section 2.5.1.1.4.4, and the applicant's responses to RAI 2.5.1-11, RAI 2.5.1-12, and RAI 2.5.1-13, the staff concurs with the applicant that no specific linkage between any of the nine faults discussed and the 1886 Charleston earthquake can be proposed based on geomorphic, geologic, borehole, or seismic evidence. The staff also concludes that it is important for the applicant to incorporate new information on source geometry and earthquake recurrence rate for the Charleston seismic source into PSHA source models for the ESP site. Furthermore, with regard to seismically induced liquefaction features, the staff concurs with the applicant that liquefaction features produced by the 1886 Charleston earthquake are most heavily concentrated in the meizoseismal area. The applicant refined earthquake recurrence rate estimates for a Charleston-area earthquake in SSAR Section 2.5.2.2.2.4. The staff considers it important for the applicant to define a seismic source zone for a Charleston-area earthquake by considering all faults and liquefaction features that it deemed feasible to include for establishing reasonable geologic boundaries for the seismic source zone.

Principal Regional Tectonic Structures—Savannah River Site

The staff focused its review of SSAR Section 2.5.1.1.4.5 on the applicant's descriptions of SRS faults, with emphasis on the Quaternary Period. In SSAR Section 2.5.1.1.4.5, the applicant discussed SRS tectonic features, including the Pen Branch, Steel Creek, Ellenton, Upper Three Runs, ATTA, Crackerneck, Martin, Tinker Creek, Lost Lake, and Millet faults. The applicant indicated that four of these faults (i.e., the Pen Branch, Steel Creek, Ellenton, and Upper Three

Runs faults) are interpreted to occur within the site area. Because the Pen Branch fault underlies the ESP site, the applicant discussed this fault in great detail in SSAR Section 2.5.1.2.4 on site area structural geology. The staff's evaluation of SSAR Section 2.5.1.1.4.5 is presented below.

Descriptions of faulting at the SRS provided in the SSAR are based on published literature from technical specialists who are very knowledgeable about tectonic features at the SRS. These descriptions are as accurate as possible, based on the consideration that most of these faults are defined in the subsurface primarily from interpretation of seismic reflection profiles (i.e., none of the faults exhibit surface expression at the SRS). The staff asked, in RAI 2.5.1-14, the applicant to obtain clarification of why the density of faults at the SRS on the eastern side of the Savannah River is so much greater than for the ESP site on the western side of the river and the implication this has for the seismic hazard at the ESP site. In RAI 2.5.1-15, the staff asked for a summary of pertinent data derived from the SRS leading to the applicant's conclusion that the Pen Branch fault is not a capable tectonic structure. In RAI 2.5.1-15, the staff also asked the applicant to compare data and analyses for the SRS with data and analyses employed by the applicant to conclude that the Pen Branch fault is not a capable structure at the ESP site. Since detailed studies of faulting at the SRS have been conducted for an extended period of time, and the ESP site is adjacent to the SRS although on the opposite side of the Savannah River, information collected from and analyses performed for the SRS are very pertinent for assessing the potential for capable faults at the ESP site.

In response to RAI 2.5.1-14, the applicant stated that the SRS was the focus of several decades of subsurface exploration and research. The applicant emphasized that the availability of high-resolution seismic reflection profiles that completely traverse the ESP site from north to south (normal-to-regional structural grain) and image the complete Coastal Plain stratigraphic section from the top of the basement to shallow levels, collected as part of the VEGP ESP project, makes the existence of any unrecognized faults at the ESP site unlikely. The applicant also stated that, although the faults shown on the SRS are greater in number, considering the difference in the size of the area of investigation between the SRS and the ESP site, fault densities are comparable. The applicant indicated that resolution and signal-to-noise ratio of the seismic profile that traverses the ESP site (i.e., proposed VEGP Unit 4) are significantly better than almost all of the seismic reflection data available for SRS. Based on these lines of evidence, the applicant concluded that the absence of previously unrecognized faults in the ESP seismic reflection data indicate that faulting at the ESP site and in the site area has been adequately characterized. The applicant thus concluded that no unknown faults exist that would affect the seismic hazard at the site.

In response to RAI 2.5.1-15, the applicant summarized the evidence substantiating that the Pen Branch fault is not a capable tectonic feature as follows:

1. Faulting deforms sediments no younger than Eocene in age. The data for this conclusion are based on 18 closely-spaced SRS drill holes that allowed construction of a subsurface geologic map of a formation above the fault. Additional support for this conclusion is based on geologic mapping and data from 20 auger holes in the Long Branch, South Carolina 7.5 minute quadrangle (Nystrom et al. 1994). The auger holes are located adjacent to the SRS but along strike of the Pen Branch fault and showed no evidence for faulting.

2. Savannah River Quaternary fluvial terraces are not deformed across the fault trace, within a resolution limit of 2 to 3 m (7 to 10 ft), based on longitudinal profiles along two Savannah River terraces (Geomatrix 1993).

3. Based on data from Moos and Zoback (1992), regional principal stress orientations determined from boreholes show that the maximum horizontal stress is parallel to the regional orientation of the Pen Branch fault, making strike-slip faulting unlikely and reverse faulting essentially impossible.

4. The VEGP terrace study documented that no fault-related deformation of the 350 ka to 1 Ma Ellenton (Qte) terrace above the projected surface trace of the Pen Branch Fault occurs within a resolution of 1 m (3 ft). The resolution of this study makes it the most definitive evidence for non-capability of the Pen Branch Fault both at the SRS and the ESP site.

The conclusion stated by the applicant that the absence of previously unrecognized faults in the ESP seismic reflection data indicates that faulting at the ESP site and in the site area has been adequately characterized, as well as its conclusion that there are no unknown faults that would affect the seismic hazard at the site, is supported by the evidence from high-resolution seismic profile data. The conclusion stated by the applicant that faulting does not deform strata younger than Eocene (54.8 to 33.7 mya) is supported by the evidence from 18 drill holes at the SRS. The conclusion stated by the applicant that the analysis of the Ellenton terrace, which overlies the Pen Branch fault, revealed no fault-related deformation within a resolution limit of 1 meter (3 feet) is supported by data collected for the ESP application.

Based on its review of the applicant's responses to RAI 2.5.1-14 and RAI 2.5.1-15, the staff concludes that the applicant adequately addressed the topics of concern raised in RAI 2.5.1-14 and RAI 2.5.1-15. The staff summarizes and discusses the evidence presented by the applicant indicating that the Pen Branch fault is not a capable tectonic structure in SER Section 2.5.1.3.4.

The staff concludes that the applicant presented a thorough and accurate description of SRS tectonic features in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). In addition, based on its review of the information presented by the applicant on SRS tectonic features in SSAR Section 2.5.1.1.4.5 and the applicant's responses to RAI 2.5.1-14 and RAI 2.5.1-15, the staff concurs with the applicant that the absence of previously unrecognized faults in the ESP seismic reflection data indicate that faulting at the ESP site and in the site area has been adequately characterized. The staff also concurs with the applicant that unknown faults that would affect the seismic hazard at the site are not likely to exist, but the staff will examine all excavations for the ESP site applying regulatory guidance in RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants", to ensure that this point is true. The staff further concurs with the applicant's conclusion that faulting does not deform strata younger than Eocene (54.8 to 33.7 mya) because this conclusion is supported by evidence from 18 drill holes at the SRS. Finally, the staff concurs with the applicant's conclusion that the analysis of the Ellenton terrace, which overlies the Pen Branch fault, revealed no fault-related deformation within a resolution limit of 1 m (3 ft) because this conclusion is supported by data collected for the ESP application.

Principal Regional Tectonic Structures—Anomalies and Lineaments

The staff focused its review of SSAR Sections 2.5.1.1.4.3 and 2.5.1.1.5 on the applicant's descriptions of regional geophysical anomalies and lineations and regional gravity and magnetic data, with emphasis on the Quaternary Period. The applicant discussed these anomalies and lineaments in SSAR Section 2.5.1.1.4.3 (the East Coast Magnetic and Blake Spur anomalies and the New York-Alabama, Clingman, and Ocoee lineaments). These two SSAR sections present well-documented geologic information, which the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined to ensure the accuracy of the information in the SSAR. The staff's evaluation of SSAR Sections 2.5.1.1.4.3 and 2.5.1.1.5 is presented below.

The applicant concluded that the geophysical anomalies and lineaments discussed in SSAR Section 2.5.1.1.4.3 did not pose concerns for the ESP site in regard to seismic hazard. In SSAR Section 2.5.1.1.5, the applicant summarized regional gravity and magnetic data and concluded that no large, unexplained anomalies exist in either data set, and no evidence exists for Cenozoic (i.e., including Quaternary age) tectonic activity or features based on that data. Information that the applicant presented for these two topics is well documented in published literature.

The staff asked, in RAI 2.5.1-7, the applicant to acquire information on the Grenville Front, listed among the features occurring within the site region but not discussed in SSAR Section 2.5.1.1.4.3, to enable assessment of whether this feature should be considered as a potential seismic source for the ESP site. The staff asked, in RAI 2.5.1-8, the applicant to (1) locate the Clingman and Ocoee lineaments and the Ocoee block on the map shown in SSAR Figure 2.5.1-12; (2) indicate the age of the "modern" tectonic setting referred to by Wheeler (1996) for earthquakes within the region of the Ocoee block to aid assessment of whether faults in this region are potentially capable structures requiring consideration for the ESP site; and (3) indicate whether the New York-Alabama, Clingman, and Ocoee lineaments could be potential seismic sources for the site.

In response to RAI 2.5.1-7, the applicant indicated that the Grenville Front was incorrectly listed as a feature occurring within 320 km (200 mi) of the ESP site (i.e., within the site region) and agreed to include the feature on SSAR Figure 2.5.1-12 to eliminate any confusion about its location. The applicant described the Grenville Front in SSAR Section 2.5.1.1.4.1 as a feature developed in Precambrian time during the Grenville Orogeny (i.e., 1100 mya) and concluded in the response that it does not represent a potential seismic source based on the firm evidence that it developed in Precambrian time.

In the response to RAI 2.5.1-8, the applicant agreed to include the Clingman and Ocoee lineaments and the Ocoee block in SSAR Figure 2.5.1-12. The applicant also indicated that the "modern" tectonic setting refers to the setting for the east coast of the United States as a passive continental margin, with regional tectonic stress for the CEUS characterized by northeast-southwest horizontal compression. The applicant stated that this regional stress orientation is subparallel to the lineaments, suggesting that they are not in the most favorable orientation for failure in this regional stress field. The applicant concluded that, while the New York-Alabama, Clingman, and Ocoee lineaments bound a block (i.e., the Ocoee block) that appears responsible for earthquakes in the ETSZ, most focal mechanism nodal planes derived from fault plane solutions in the ETSZ are not parallel to the northeast-trending lineaments, suggesting that features with this orientation are not favorably oriented for accommodating fault displacement. The applicant cited evidence related to orientation of nodal planes defined in the Ocoee block, derived from Johnston et al. (1985) as stated in SSAR Section 2.5.1.1.4.3, indicating north-south and east-west faults for the Ocoee block rather than structures parallel to the northeast-southwest strike trend of the lineaments. The applicant further stated that the lineaments were known to the technical teams in the 1986 EPRI study, and no new information has been published since 1986 on the lineaments that would require a significant change in the EPRI seismic source model.

Based on its review of the applicant's responses to RAI 2.5.1-7 and RAI 2.5.1-8, the staff concludes that neither the Grenville Front nor the New York-Alabama, Clingman, and Ocoee lineaments are likely to be viable seismic sources.

The staff concludes that the applicant presented a thorough and accurate description of regional geophysical anomalies and lineations and regional gravity and magnetic data in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Furthermore, based on its review of the information presented by the applicant on regional geophysical anomalies and lineations and regional gravity and magnetic data in SSAR Sections 2.5.1.1.4.3 and 2.5.1.1.5 and the applicant's responses to RAI 2.5.1-7 and RAI 2.5.1-8, the staff concurs with the applicant that no regional anomalies or lineaments and no regional gravity or magnetic data indicated features requiring consideration for seismic hazard analysis at the ESP site. The staff further concurs with the applicant that none of the anomalies or lineaments described by the applicant in SSAR Sections 2.5.1.1.4.3 and 2.5.1.1.5 are likely to be seismic sources requiring seismic hazard consideration at the ESP site.

Seismic Source Zones

The staff focused its review of SSAR Section 2.5.1.1.4.6 on the applicant's descriptions of the seismically defined source zones, including selected seismogenic and capable tectonic sources beyond the site region, with emphasis on the Quaternary Period (1.8 mya to present). In SSAR Section 2.5.1.1.4.6, the applicant described seismic sources (defined based on regional seismicity) comprising the ETSZ within the site region and the Central Virginia, New Madrid, and GCSZs outside of the site region. This SSAR section presents well-documented geologic information which the applicant derived from published sources. The applicant provided an extensive list of references for these sources, and the staff directly examined relevant references to ensure the accuracy of the information derived from published sources and presented in the SSAR. The staff's evaluation of SSAR Section 2.5.1.1.4.6 is presented below.

In regard to seismic sources within, and selected sources outside, the site region, the applicant concluded that only the NMSZ required an update of source parameters, in particular, of the recurrence rate. This conclusion was rendered necessary by new information that the applicant reported in the SSAR, as derived from the published literature. The applicant concluded further that information for none of the other three zones (i.e., the East Tennessee, Central Virginia, and Giles County zones) required a significant revision to the 1986 EPRI source model in light of data that were also derived from the published literature. This information included interpretations from Wheeler (2005) that the East Tennessee and GCSZs are Class C features based on a lack of geologic evidence for large earthquakes associated with the zones.

The staff concludes that the applicant presented a thorough and accurate description of seismic source zones defined by seismicity within the site region, including selected sources outside the site region, in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on its review of the information presented by the applicant on seismic source zones in SSAR Section 2.5.1.1.4.6, the staff also concludes that all regional seismic source zones discussed by the applicant have less influence on the ESP site due to their distance from the site than the updated Charleston seismic source model discussed in SSAR Section 2.5.2.2.2.4. The staff concurs with the applicant that the Charleston seismic source model dominates ground motion hazard for the site. The applicant incorporated new information on source geometry and earthquake recurrence rate for this source into an updated seismic source model in SSAR Section 2.5.2.2.2.4.

Based on its review of SSAR Section 2.5.1.1.4 and the applicant's responses to RAIs as set forth above, the staff concludes that the applicant identified and properly characterized all regional tectonic features. The staff also concludes that SSAR Section 2.5.1.1.4 provides an accurate and thorough description of regional tectonic features, with an emphasis on potential Quaternary deformation, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

2.5.1.3.3 Site Area Geologic Description

In SSAR Sections 2.5.1.2.1, 2.5.1.2.2, and 2.5.1.2.3, the applicant reviewed and summarized published information related to physiography and geomorphology (Section 2.5.1.2.1), geologic history (Section 2.5.1.2.2), and stratigraphy (Section 2.5.1.2.3) of the site area. Based on information presented in SSAR Sections 2.5.1.2.1, 2.5.1.2.2, and 2.5.1.2.3, the applicant concluded that physiography, geomorphology, geologic history, and stratigraphy of the site area pose no safety issues for the ESP site. Consequently, the applicant considered the site suitable in regard to these area-specific features and their characteristics. The staff's evaluation of SSAR Sections 2.5.1.2.1, 2.5.1.2.2, and 2.5.1.2.3 is presented below.

Physiography, Geomorphology, and Geologic History

The staff focused its review of SSAR Sections 2.5.1.2.1 and 2.5.1.2.2 on the applicant's descriptions of physiography, geomorphology, and geologic history of the site area, with emphasis on the Quaternary Period. In SSAR Section 2.5.1.2.1, the applicant described the geomorphology of the Coastal Plain physiographic province within which the ESP site lies. In SSAR Section 2.5.1.2.2, the applicant described geologic history of the site area, emphasizing the Coastal Plain. These two SSAR sections present well-documented geologic information, which the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined to ensure the accuracy of the information presented by the applicant in the SSAR.

In the description of site area physiography and geomorphology presented in SSAR Section 2.5.1.2.1, the applicant indicated that the Savannah River is relatively straight and incised in the site area in the vicinity of the projected surface trace of the Pen Branch fault. Tectonic uplift, among other factors, can lower the base level to which a stream will naturally erode, resulting in active erosion by down-cutting and incision of the stream channel. The staff asked, in RAI 2.5.1-4, the applicant to address why the Savannah River is relatively straight and incised at a position that appears to correspond with the location of the Pen Branch fault. This clarification is important to enable an assessment of whether reverse or reverse-oblique slip along the Pen Branch fault occurred to uplift the hanging wall fault block; lower the base level to which the Savannah River would erode; and thus create an incised river channel.

In response to RAI 2.5.1-4, the applicant concluded that the straight, incised segment of the Savannah River is not the result of Quaternary displacement along the Pen Branch fault. The applicant cited three lines of evidence interpreted to preclude Quaternary displacement along the Pen Branch fault as being the mechanism that produced this straight, incised segment of the Savannah River channel:

1. The geomorphic surface of the 350 ka to 1 Ma Ellenton fluvial terrace along the Savannah River is undeformed to within a resolution of 1 m (3 ft). The applicant stated that this observation is the best evidence precluding late Quaternary activity of the Pen Branch fault and establishing that the Pen Branch is not a capable fault. The applicant considered it highly unlikely that changes in the modern river channel morphology at the location of the Pen Branch fault would be the result of Quaternary fault activity if the Ellenton terrace surface is preserved across the fault with no evidence of deformation.

2. Several other examples of linear or incised portions of rivers are present in the Coastal Plain within 80 km (50 mi) of the ESP site that are not associated with any mapped fault. The applicant stated that the occurrence of other linear portions of river channels demonstrates that the morphology of the Savannah River adjacent to the VEGP site is not unique, but relatively common in the region. The applicant indicated that these other linear reaches of river channels are not spatially associated with known mapped faults, strongly suggesting a nontectonic origin for this type of feature.

3. Localized remnant surfaces on the modern flood plain that formed as the result of paleochannel migration indicate that, although the river at present appears relatively straight, it has meandered across the flood plain in recent time. Therefore, the applicant stated that the apparent "straight" segment of the Savannah River channel near the ESP site appears to be an ephemeral feature that changes or evolves through geologic time in response to changes in sediment load, discharge, and eustatic base-level change.

Based on its review of the applicant's response to RAI 2.5.1-4, the staff concludes that the straight, incised channel of the Savannah River which occurs in the site area in the vicinity of the Pen Branch fault does not require a mechanism related to Quaternary displacement along the Pen Branch fault to produce this morphology along the river channel.

Based on its review of SSAR Sections 2.5.1.2.1 and 2.5.1.2.2 and the applicant's response to RAI 2.5.1-4, the staff concludes that the applicant presented a thorough and accurate description of the physiography, geomorphology, and geologic history of the site area in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

Stratigraphy

The staff focused its review of SSAR Section 2.5.1.2.3 on the applicant's description of stratigraphic units in the site area, with emphasis on sedimentary units of the Coastal Plain within which the ESP site lies. In SSAR Section 2.5.1.2.3, the applicant described Coastal Plain stratigraphy in the site area in detail and also discussed basement rocks (i.e., both Paleozoic crystalline rocks and sedimentary rocks of the Dunbarton Triassic basin) which underlie Coastal Plain sedimentary units in the site area. The applicant used information derived from borehole B-1003 drilled at the ESP site to describe stratigraphic units of the Coastal Plain that occur at the site. The staff also examined core from this specific borehole during a visit to the ESP site, and this examination of subsurface stratigraphy by the staff added credence to the accuracy of the applicant's description of site stratigraphy. The applicant's discussion of previous data on the site-specific stratigraphic units cited well-documented geologic information derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined to ensure the accuracy of the information presented in the SSAR.

Based on its review of SSAR Section 2.5.1.2.3, the staff concludes that the applicant presented a thorough and accurate description of stratigraphic relationships for the site area in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). SER Section 2.5.4 provides further discussion of the engineering properties of soil and rock materials that underlie the ESP site and the staff's complete evaluation of the applicant's description of these materials.

2.5.1.3.4 Site Area Structural Geology

In SSAR Section 2.5.1.2.4, the applicant reviewed and summarized published information related to the structural geology of the site area, including the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults. Of these four faults, the applicant determined that the Pen Branch fault underlies the ESP site and required further investigation to determine whether it is a capable tectonic feature exhibiting Quaternary displacement. Therefore, in addition to summarizing published results from previous studies of the Pen Branch fault, the applicant presented important new information from seismic reflection and refraction surveys and evaluation of Quaternary-age fluvial terraces overlying the Pen Branch fault. The applicant collected this information for the ESP application specifically to determine whether the Pen Branch fault is a capable tectonic feature. The applicant stated that the Upper Three Runs and Steel Creek faults are restricted entirely to basement rocks and do not offset Coastal Plain deposits, and the Ellenton fault no longer appears on recent maps of the SRS where it was first interpreted to occur based on seismic reflection data.

Based on information presented in SSAR Section 2.5.1.2.4, the applicant concluded that the structural geology of the site area poses no safety issues for the ESP site. With due consideration for the results of previous studies of the Pen Branch fault and the new information collected for the ESP application, the applicant concluded that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature requiring analysis for seismic hazard or surface-faulting issues at the site. The applicant also concluded that the Ellenton, Steel Creek, and Upper Three Runs faults are not capable tectonic features. Consequently, the applicant considered the site suitable in regard to area-specific geologic structures (i.e., faults) and their characteristics, including the Pen Branch fault. The staff's evaluation of SSAR Section 2.5.1.2.4 specifically in regard to the Pen Branch fault, including SSAR Sections 2.5.1.2.4.1, 2.5.1.2.4.2, and 2.5.1.2.4.3 is presented below.

Pen Branch Fault

The staff focused its review of SSAR Section 2.5.1.2.4 on the applicant's descriptions of the Pen Branch fault (SSAR Section 2.5.1.2.4.1), including new information collected for the ESP application derived from site subsurface investigation of the Pen Branch fault (SSAR Section 2.5.1.2.4.2) and evaluation of Quaternary river terrace Qte (Ellenton terrace) which overlies the Pen Branch fault (SSAR Section 2.5.1.2.4.3). The staff's review emphasized the Quaternary Period and included careful analysis of all information presented by the applicant related to determining whether the Pen Branch fault exhibited Quaternary displacement. The applicant's discussion of previous data on the Pen Branch fault cited well-documented geologic information derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined to ensure the accuracy of the information in the SSAR. However, in the extensive list of references, the applicant did not cite a publication by Hanson et al. (1993) in which the investigators suggested that possible rejuvenation of drainage along projected surface traces of the Pen Branch and Steel Creek faults on the SRS may indicate either local tectonic uplift along these faults at a very low rate of displacement (i.e., 0.002 to

0.009 mm/yr) or nontectonic geologic processes. In RAI 2.5.1-17, the staff asked the applicant to determine whether the concept presented by Hanson et al. (1993), related to the suggestion of possible Quaternary displacement along the Pen Branch fault based on their analysis of drainage morphology at the SRS, held any implications of geologic hazard for the ESP site.

In response to RAI 2.5.1-17, the applicant addressed the suggestion of Hanson et al. (1993) that stream drainage patterns along the trace of the Pen Branch fault on the SRS may suggest local Quaternary tectonic uplift. The applicant summarized results of a 1993 study by Geomatrix that concentrated on collecting and analyzing several types of information in regard to Quaternary tectonic deformation at the SRS. The applicant discussed data derived from a regional slope map, slope profiles, longitudinal stream profiles, and residual maps that Geomatrix (1993) constructed for this analysis. Based on this information, the applicant concluded that no obvious topographic or geomorphic characteristics could be equated with geologic structures or required the occurrence of Quaternary deformation along the Pen Branch fault. The applicant also reviewed data developed from evaluation of drainage basin shape, drainage density, and drainage frequency by Geomatrix (1993). The applicant likewise concluded from this information that none of these aspects of the drainage patterns indicated geologic structures or required Quaternary deformation along the Pen Branch fault. The applicant referred to fluvial terrace studies conducted by Geomatrix (1993), as well as the more refined terrace studies conducted for the ESP application discussed in SSAR Section 2.5.1.2.4.3, as the most conclusive evidence for a lack of Quaternary deformation along the Pen Branch fault.

Based on its review of the applicant's response to RAI 2.5.1-17, the staff concludes that there is no definitive evidence described by Hansen et al. (1993) indicating the existence of Quaternary displacement along the Pen Branch fault in the site area. The staff further concludes that the applicant's response to RAI 2.5.1-17 adequately qualified the conclusion presented by the applicant.

In the discussion of geometry of the Pen Branch fault presented in SSAR Section 2.5.1.2.4.2, the applicant stated that the Pen Branch fault at the ESP site is made up of two specific fault segments trending N45°E and N34°E with a dip of 45°SE. Considering the N50° to 70°E modern-day orientation of maximum principal horizontal compressive stress defined by Moos and Zoback (1992) for the site region in relation to orientations of segments of the Pen Branch fault, the staff asked, in RAI 2.5.1-18, the applicant to determine whether either fault segment is favorably oriented to experience displacement in the existing regional stress field.

In response to RAI 2.5.1-18, considering the N50° to 70°E modern-day orientation of maximum principal horizontal compressive stress defined by Moos and Zoback (1992) for the site region, the applicant chose an average orientation of the maximum horizontal stress as N60°E and determined that planes striking N45°E and N34°E and dipping 45°SE form angles to the maximum horizontal stress of approximately 10° and 20°, respectively. The applicant stated that these orientations are not parallel to the maximum horizontal stress and therefore would experience some amount of resolved shearing stress. However, based on Ramsey and Huber (1987), the applicant indicated that planes of such orientations relative to maximum principal horizontal compressive stress would not experience maximum shearing stress. The applicant pointed out that favorably oriented planes for maximum resolved shearing stress occur at 45° to the maximum horizontal compressive stress direction. Moos and Zoback (1992) further stated that stress magnitudes at shallow depths only approach the frictional strength of favorably oriented reverse faults (i.e., 45°). Therefore, the applicant concluded that stress magnitudes resolved along planes of other orientations will be well below those necessary for displacement in the modern-day stress field. The applicant also concluded that the orientation of the Pen Branch fault segments at the ESP site makes them less favorably oriented for failure in response to the intermediate-depth stress perturbation of N33°E which Moos and Zoback (1992) reported.

Based on its review of the applicant's response to RAI 2.5.1-18, the staff concurs with the applicant that neither of the segments of the Pen Branch fault occurring at the ESP site are favorably oriented to experience displacement in the modern-day stress field. As the applicant indicated, shear failure theory predicts that favorably oriented planes for maximum resolved shearing stress occur at 45° to the maximum horizontal compressive stress direction.

The staff concludes that the applicant presented a thorough and accurate description of the Pen Branch and other faults in the site area in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Furthermore, upon consideration of the information the applicant presented in SSAR Section 2.5.1.2.4, including the applicant's responses to RAI 2.5.1-17 and RAI 2.5.1-18, to support its conclusions about the noncapable nature of the Pen Branch fault, the staff concurs with the applicant that no definitive evidence exists to indicate that the Pen Branch fault (1) shows any surface expression; (2) exhibits Quaternary displacement based on analysis of fluvial terraces and age of stratigraphic units which bound the time of fault displacement; or (3) is a capable tectonic structure. SER Section 2.5.3 contains the staff's complete evaluation of surface faulting near the ESP site in regard to the potential for tectonic deformation and vibratory ground motion due to surface faulting.

The technical bases for the staff's conclusions in regard to site area structural geology, specifically that the Pen Branch fault is not a capable tectonic feature at the ESP site, are related to the evidence which the applicant presented in the SSAR and in its responses to RAIs. The evidence presented by the applicant and summarized below covers information acquired from previous investigations at the SRS and the VEGP site; geomorphic mapping and field reconnaissance, seismic reflection and refraction studies, and investigation of Quaternary fluvial terraces performed by the applicant for the ESP application; and analysis of the regional stress field.

Previous Investigations at the Savannah River Site History of and evidence from previous investigations of the Pen Branch fault conducted at the SRS, which the applicant outlined in SSAR Section 2.5.1.2.4.1, are summarized as follows:

1. Based on seismic data, Snipes et al. (1989) suggested Late Eocene (33.7 mya or older) displacement, but no younger, on the Pen Branch fault and concluded that the fault should not be considered a capable tectonic structure at the SRS.

2. Based on a seismic reflection survey designed to investigate the Pen Branch fault, Berkman (1991) reported deformation of the Cretaceous age (144 to 65 mya) Cape Fear Formation, but no younger units, and concluded that the Pen Branch fault is not a capable tectonic feature.

3. A fluvial terrace study performed by Geomatrix (1993) confirmed no tectonic deformation of terrace surfaces overlying the Pen Branch fault within a resolution of 2 to 3 m (7 to 10 ft), and Geomatrix (1993) concluded that the Pen Branch is not a capable tectonic feature.

4. Snipes et al. (1993) reported that the youngest stratigraphic horizon known from borehole studies to be deformed by fault displacement along the Pen Branch fault is the Dry Branch Formation of Late Eocene (33.7 mya or older) age, and that a Quaternary soil horizon overlying the projected trace of the Pen Branch fault at the SRS showed no offset. The applicant reported this information in SSAR Section 2.5.3.6.

5. Based on results of a drilling project designed to investigate the Pen Branch fault using 18 boreholes, Stieve et al. (1994) concluded that the Pen Branch fault is no younger than 50 mya and is not a capable tectonic feature.

6. Cumbest et al. (1998) integrated information from more than 60 boreholes and 100 miles of seismic reflection profiling and concluded that no faults on the SRS, including the Pen Branch Fault, are capable tectonic features.

7. Based on seismic reflection data, Cumbest et al. (2000) concluded that offset along the Pen Branch fault decreased upward within Coastal Plain sediments to no greater than 9 m (30 ft) at the top of Upper Cretaceous/Lower Paleocene units (i.e., about 66.4 mya).

<u>**Previous Investigations at the VEGP Site**</u>

Henry (1995) collected and interpreted 115 km (70 mi) of seismic reflection data along the Savannah River, including in the vicinity of VEGP Units 1 and 2, and crossing the projected trace of the Pen Branch fault. Henry (1995) concluded that the Pen Branch fault extended into possibly Eocene age (54.8 to 33.7 mya) sediments. The applicant summarized this information in SSAR Section 2.5.1.2.4.1.

In SSAR Section 2.5.3.8.2.1, the applicant indicated that an old garbage trench that crossed the trace of the Pen Branch fault in the ESP site area, mapped by Bechtel in 1994, contained only dissolution collapse features and no tectonic structures that resulted from displacement along the Pen Branch fault. The applicant interpreted these dissolution features to be older than Late Pleistocene (i.e., greater than 10,000 years old) based on stratigraphic units exposed in the trench, providing an upper age limit for deformation due to displacement along the Pen Branch fault. More recent investigations, as discussed in the following paragraph, indicate a minimum age for displacement along the Pen Branch fault greater than 33.7 mya.

<u>**Seismic Reflection and Refraction Data Collected for the ESP Application**</u>

The applicant discussed seismic reflection and refraction data collected for the ESP application in SSAR Section 2.5.1.2.4.2. The applicant defined orientation of the Pen Branch fault in the ESP site area and concluded that a monoclinal fold in the Blue Bluff Marl marks the up-section effects of the Pen Branch fault on stratigraphic units in the site area, indicating no displacement that is post-Eocene (i.e., older than 33.7 mya).

<u>**Geomorphic Mapping and Field Reconnaissance for the ESP Application**</u>

In SSAR Sections 2.5.1.2.4.3 and 2.5.3.6, the applicant indicated that geomorphic mapping and field reconnaissance performed for the ESP application as preparation for the terrace study showed no surface expression of Quaternary deformation along the Pen Branch fault in the site region.

Terrace Study Performed for the ESP Application

The applicant discussed results of its analysis of the Ellenton fluvial terrace (i.e., terrace Qte) at the SRS, which was performed to assess the capability of the Pen Branch fault in the site area, in detail in SSAR Section 2.5.1.2.4.3. The applicant concluded that no Quaternary deformation of the terrace is indicated due to displacement along the Pen Branch fault within a resolution limit of 1 meter (3 feet). RAIs described in SER Section 2.5.1.3.1 (i.e., RAI 2.5.1-1, RAI 2.5.1-2, and RAI 2.5.1-3) posed questions to address the conclusion that the applicant drew from the analysis of fluvial terrace Qte, since this analysis was cited by the applicant as the most important piece of evidence indicating no Quaternary displacement along the Pen Branch fault. The staff and its USGS advisors also visited the ESP site to gain firsthand knowledge about the accuracy of the terrace analysis, and observations made during the site visit added credence to the applicant's conclusion that this study indicates that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature at the ESP site.

Orientation of the Pen Branch Fault in the Modern-Day Regional Stress Field

In SSAR Section 2.5.1.1.4.2, the applicant stated, based on information from Moos and Zoback (1992), that maximum horizontal regional compressive stress in the modern-day stress field is oriented N50° to 70°E in the upper 640-meter (2100-foot) depth range. Such an orientation of regional stress (the applicant used a reasonable average of N60°E in its response to RAI 2.5.1-18) is subparallel to the measured strike of the Pen Branch fault, even when the fault is divided into segments striking N45°E and N34°E as the applicant discussed in SSAR Section 2.5.1.2.4.1. Shear failure theory predicts that maximum shear stress occurs on a surface oriented at 45° to maximum principal compressive stress; consequently, the Pen Branch fault surface is not oriented as a favorable plane for shear failure and resulting fault displacement.

2.5.1.3.5 Site Area Geologic Hazard Evaluation—Faulting, Earthquakes, and Seismicity

In SSAR Section 2.5.1.2.5, the applicant stated that no geologic hazards, effectively including any related to faulting, earthquakes, and seismicity, occur within the ESP site area. The applicant provided detailed discussions on surface faulting in SSAR Section 2.5.3 and seismic hazards in SSAR Section 2.5.2. The applicant provided results of the detailed analysis of the Pen Branch fault specifically, which demonstrate that the Pen Branch is not a capable structure in the site area, in SSAR Section 2.5.1.2.4. In SSAR Section 2.5.1.2.6.4, the applicant also stated that extensive studies of alluvial terraces and floodplain deposits showed no evidence of post-Miocene (i.e., greater than 5.3 mya) earthquake activity as discussed in SSAR Section 2.5.1.2.4. Based on information presented in SSAR Sections 2.5.1.2.4, 2.5.1.2.5, and 2.5.1.2.6.4, the applicant concluded that the ESP site exhibits no geologic hazards resulting from faulting, earthquakes, or seismicity that occur in the site area. Consequently, the applicant considered the site suitable in regard to geologic hazards related to faulting, earthquakes, and seismicity, including the Pen Branch fault, in the site area. However, the applicant does incorporate new information from other investigators on source geometry and earthquake recurrence rate for the Charleston seismic source into PSHA source models for the ESP site, as discussed in SSAR Section 2.5.2.2.2.4. The staff's evaluation of SSAR Section 2.5.1.2.5 in regard to potential hazards due to faulting, earthquakes, and seismicity is presented below.

Based on its review of the information that the applicant presented in SSAR Sections 2.5.1.2.4, 2.5.1.2.5, and 2.5.1.2.6.4, the staff concludes that the applicant presented a thorough and accurate description of faulting, earthquakes, and seismicity in the site area in support of the

ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff concurs with the applicant that the ESP site exhibits no geologic hazards resulting from faulting, earthquakes, or seismicity that occur in the site area.

2.5.1.3.6 Site Area Nontectonic Deformation Features

In SSAR Section 2.5.1.2.5, the applicant stated that nontectonic surface depressions associated with dissolution of the Utley Limestone member of the Clinchfield Formation which overlies the Blue Bluff Marl do not pose a geologic hazard at the ESP site. The applicant plans to remove this unit from the site excavation, and the Blue Bluff Marl will form the foundation-bearing layer. These units are discussed in SSAR Section 2.5.1.2.3.2, and the surface depressions are discussed in detail in SSAR Section 2.5.3.8.2.1. In SSAR Section 2.5.1.1.1, the applicant indicated that Carolina Bays, which occur in the site area, are related to eolian erosion resulting from strong, unidirectional, southwesterly winds and not from dissolution. The applicant also indicated in SSAR Section 2.5.1.2.5 that any structures founded above the Blue Bluff Marl will require subsurface exploration to define low bearing strength layers associated with dissolution in units overlying the Blue Bluff Marl. Based on information presented in SSAR Section 2.5.1.2.5, the applicant concluded that the ESP site exhibits no hazard resulting from nontectonic deformation features. Consequently, the applicant considered the site suitable in regard to geologic hazards related to these features in the site area. The staff's evaluation of SSAR Section 2.5.1.2.5 in regard to potential hazard from nontectonic deformation is presented below.

Based on its review of the information presented in SSAR Section 2.5.1.2.5 and the SSAR sections (i.e., Section 2.5.3.8.2.1 for dissolution features and 2.5.1.1.1 for Carolina Bays) in which the applicant discussed surface depressions in detail, the staff concludes that the applicant presented a thorough and accurate description of nontectonic deformation features in the ste area in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff concurs with the applicant that the ESP site exhibits no geologic hazards resulting from nontectonic deformation features.

2.5.1.3.7 Human-Induced Effects on Site Area Geologic Conditions

In SSAR Section 2.5.1.2.6.5, the applicant stated that no mining operations other than borrow of surficial soils, excessive extraction of injection of ground water, or impoundment of water exists in the site area that will detrimentally affect geologic conditions. Based on information presented in SSAR Section 2.5.1.2.6.5, the applicant concluded that the ESP site exhibits no hazard resulting from human-induced effects on site geologic conditions. Consequently, the applicant considered the site suitable in regard to geologic hazards related to human-induced effects in the site area. The staff's evaluation of SSAR Section 2.5.1.2.6.5 is presented below.

Based on its review of the information presented in SSAR Section 2.5.1.2.6.5, the staff concludes that the applicant presented an accurate description of human-induced effects in the site area in support of the ESP application, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff concurs with the applicant that the ESP site exhibits no hazard resulting from human-induced effects on site geologic conditions.

2.5.1.3.8 Site Area Engineering Geology Evaluation

In SSAR Section 2.5.1.2.6, the applicant addressed engineering soil properties and behavior of foundation materials (Section 2.5.1.2.6.1), zones of alteration, weathering, and structural weakness (Section 2.5.1.2.6.2), and deformational zones (Section 2.5.1.2.6.3). The applicant addressed ground water conditions in SSAR Section 2.5.1.2.7. Regarding engineering properties (including index properties, static and dynamic strength, and compressibility), the applicant indicated that this information is discussed in detail in SSAR Section 2.5.4. In regard to zones of alteration, weathering, and structural weakness, the applicant indicated that some desiccation of the Blue Bluff Marl is expected and that desiccation, weathered zones, and fractures will be mapped and evaluated. Regarding deformational zones, the applicant stated that none were reported from previous studies for VEGP Units 1 and 2, but the applicant will evaluate any such zones detected during excavation mapping. In regard to site ground water conditions, the applicant indicated that a detailed discussion of these conditions is provided in SSAR Section 2.4.12. The staff's evaluation of SSAR Section 2.5.1.6, including SSAR Sections 2.5.1.2.6.1, 2.5.1.2.6.2, 2.5.1.2.6.3, and 2.5.1.2.7, is presented below.

Based on its review of the information that the applicant presented in SSAR Sections 2.5.1.2.6 and 2.5.1.2.7, the staff concludes that the applicant presented an accurate description of site area engineering geology, as far as existing data will allow, in support of the ESP application, as required by 10 CFR 100.23(c). The staff's detailed analysis of engineering properties of soil and rock is presented in SER Section 2.5.4, and the analysis of site ground water conditions is presented in SER Section 2.4.12.

Based on its review of SSAR Section 2.5.1.2 and the applicant's responses to RAIs as set forth above, the staff concludes that the applicant identified and properly characterized all site area geologic features, including the Pen Branch fault. The staff also concludes that SSAR Section 2.5.1.2 provides an accurate and thorough description of site area geologic features, with an emphasis on the Quaternary Period, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

2.5.1.4 Conclusions

As discussed in SER Sections 2.5.1.1, 2.5.1.2, and 2.5.1.3, the staff carefully reviewed the basic geologic and seismic information submitted by the applicant in SSAR Section 2.5.1. The staff concurs that the data and analyses presented by the applicant in the SSAR provide an adequate basis to conclude that no capable tectonic faults exist in the plant site area that have the potential to generate surface or near-surface fault displacement.

In addition, the staff concludes that the applicant has identified and appropriately characterized all seismic sources significant for determining the SSE for the ESP site, in accordance with the guidance provided in RG 1.70, RG 1.165, and Section 2.5.1 of NUREG-0800. Because ground motion hazard at the ESP site is dominated by the Charleston seismic source, the staff concurs with the applicant's decision to update the EPRI (1986) source model for this seismic source in light of new information on source geometry and earthquake recurrence rate. No capable tectonic feature has as yet been linked to the Charleston seismic source. Based on information from the applicant's thorough review of the literature on regional geology, and the applicant's literature review and geologic, geophysical, and geotechnical investigations of the site vicinity and site area, the staff further concludes that the applicant has properly characterized regional and site lithology, stratigraphy, geologic and tectonic history, and structural geology, as well as subsurface soils and rock units at the site. The staff also concludes that there is no potential for the effects of human activity (i.e., mining activity or ground water injection or withdrawal) that will compromise the safety of the ESP site.

On the basis of the foregoing, the staff concludes that the applicant has provided a thorough and accurate characterization of the geologic and seismic characteristics of the site, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

2.5.2 Vibratory Ground Motion

2.5.2.1 Introduction/Overview/General

SSAR Section 2.5.2 describes the applicant's determination of the ground motion response spectrum (GMRS) at the Early Site Permit (ESP) site from potential earthquakes in the site area and region. SSAR Section 2.5.2.1 describes the earthquake catalog used for the ESP site, SSAR Section 2.5.2.2 summarizes the geologic structures and tectonic activity that could potentially result in ground motion at the ESP site, and SSAR Section 2.5.2.3 describes the correlation of earthquake activity with geologic structures or tectonic provinces. SSAR Section 2.5.2.4 describes the earthquake potential for seismic sources in the region surrounding the ESP site, SSAR Section 2.5.2.5 describes the seismic wave transmission characteristics of the site, SSAR Section 2.5.2.6 provides the horizontal GMRS, SSAR Section 2.5.2.7 provides the vertical GMRS, SSAR Section 2.5.2.8 discusses the operating-basis earthquake ground motion spectrum, and SSAR Section 2.5.2.9 describes the results of site response sensitivity calculations.

The applicant stated that the information provided in SSAR Section 2.5.2 of the ESP application uses the procedures recommended in RG 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," issued March 1997, for performing the Probabilistic Seismic Hazard Analysis (PSHA) for the ESP site. However, rather than using the reference-probability approach described in Regulatory Guide (RG) 1.165 for determining the SSE, the applicant developed the GMRS using the performance-based method described in RG 1.208, A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," issued March 2007. According to RG 1.208, the GMRS represents the first part of the development of the safe-shutdown earthquake (SSE) for a site. In addition, the applicant used the 1986 EPRI [Electric Power Research Institute] Project (EPRI NP-4726) seismic source model for the Central and Eastern United States (CEUS) as an input for its seismic ground motion calculations. RG 1.165 indicates that applicants may use the seismic source interpretations developed by Lawrence Livermore National Laboratory (LLNL (1993) or EPRI as inputs for a site-specific analysis. RG 1.165 also recommends a review and update, if necessary, of both the seismic source and ground motion models used to develop the SSE ground motion for the ESP site.

To determine whether an update of the seismic source and ground motion models used in the 1989 EPRI PSHA (EPRI NP-6395-D) was necessary, the applicant reviewed the literature published since the mid-to-late 1980s. This literature review identified the need for changes to the source characterization parameters of the Charleston seismic zone. In addition, the applicant determined that the ground motion models used for the 1989 EPRI PSHA needed to be updated.

2.5.2.2 Summary of Application

2.5.2.2.1 Seismicity

SSAR Section 2.5.2.1 describes the development of a current earthquake catalog for the ESP site. The applicant started with the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is complete through 1984. To update the earthquake catalog, the applicant used information from the Advanced National Seismic System (ANSS) and the South Eastern United States Seismic Network (SEUSS).

The EPRI catalog covers the time period from 1627 to 1984 and contains earthquakes that occurred within the CEUS. Earthquakes comprising the EPRI catalog are characterized by a variety of different size measures, including local magnitude (M_L), surface-wave magnitude (M_S), duration or coda magnitude (M_d or M_c), body-wave magnitude (m_{bLg}), felt area (FA), and epicentral Modified Mercalli (MM) intensity (I_o). Earthquake measures such as M_L, M_S, M_d, M_c, and m_{bLg} are based on characteristics of instrumentally recorded events. M_d and M_c are related to the duration of a recorded earthquake, while M_L, M_S, and m_{bLg} are related to the amplitude of a recorded earthquake. FA and I_o, are based on qualitative descriptions of the effects of the earthquake at a particular location (Kramer 1996).

All earthquakes comprising the EPRI catalog are described in terms of m_b. The applicant converted all earthquakes that were not originally characterized by m_b to best, or expected, estimates of m_b ($E[m_b]$) using conversion factors developed in EPRI NP-4726-A (1988). EPRI NP-4726-A (1988) developed these conversion factors from regression models relating m_b to M_L, M_S, M_d or M_c; FA; and I_o. In addition, the 1988 EPRI study calculated a uniform magnitude (m_b*) from Em_b and the variance of m_b (σ^2_{mb}) in order to account for the uncertainty in estimating m_b.

The applicant updated the EPRI historical seismicity catalog to incorporate earthquakes that have occurred within (and beyond) the site region (320-kilometer (km) or 200-mile (mi) radius) since 1984. To update the EPRI catalog, the applicant used a latitude-longitude window of 30° to 37°N, 78° to 86°, which incorporated the 320 km (200 mi) radius and all seismic sources contributing significantly to the ESP site seismic hazard. The applicant used information from the ANSS and the SEUSS for the update. Of these two catalogs, the applicant primarily used the SEUSS catalog for the period from 1985 to 2005. Events in the SEUSS and ANSS catalogs that have occurred since 1985 are primarily reported as m_{bLg}, M_L, M_c, and M_d. To be consistent with the m_b estimates provided in the EPRI catalog, the applicant converted the magnitudes given in both the SEUSS and ANSS catalogs to E[mb]. The applicant included a total of 61 events with $E[m_b]$ magnitude greater than 3.0 in the update of the EPRI NP-4726-A (1988) seismicity catalog. The applicant also calculated m_b* using $E[m_b]$ and σ^2_{mb} (estimated from the ANSS and SEUSS catalogs).

As shown in Figure 2.5.2-1 of this SER, a comparison of the geographic distribution of earthquakes contained in the EPRI catalog (1627–1984) and the earthquakes contained in the updated catalog (1985–2005) shows a very similar spatial distribution. The cluster of events along the coast of South Carolina is related to the Charleston Seismic Zone, while the cluster of events in eastern Tennessee is associated with the Eastern Tennessee Seismic Zone (ETSZ). The ETSZ extends from southwest Virginia to northeast Alabama.

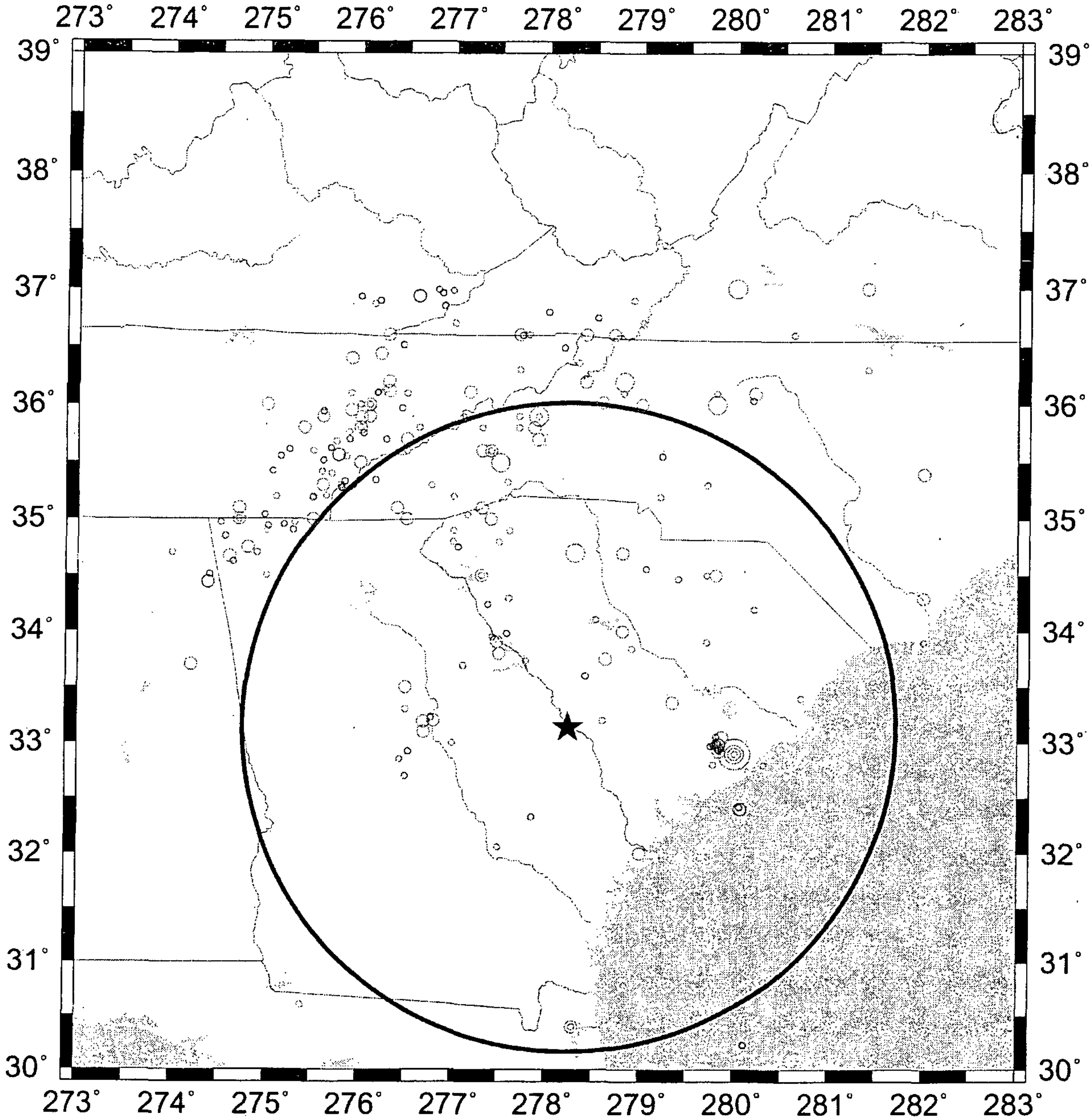


Figure 2.5.2-1 - A comparison of events (m_b greater than 3) from the EPRI historical catalog (depicted by blue circles) with events from the applicant's updated catalog (depicted by red circles). The star corresponds to the location of the ESP site and the large black circle corresponds to the 200-mi site radius.

2.5.2.2.2 Geologic and Tectonic Characteristics of the Site and Region

SSAR Section 2.5.2.2 describes the seismic sources and seismicity parameters that the applicant used to calculate the seismic ground motion hazard for the ESP site. Specifically, the applicant described the seismic source interpretations from the 1986 EPRI Project (EPRI NP-4726 1986), relevant post-EPRI seismic source characterization studies, and its updated EPRI seismic source zone for the Charleston area based on more recent data.

Summary of EPRI Seismic Sources

The applicant used the 1986 EPRI seismic source model for the CEUS as a starting point for its seismic ground motion calculations. The 1986 EPRI seismic source model is comprised of input from six independent earth science teams (ESTs), which included the Bechtel Group, Dames and Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants. Each team evaluated geological, geophysical, and seismological data to develop a model of seismic sources in the CEUS. The 1989 EPRI PSHA study (EPRI NP-6395-D 1989) subsequently incorporated each of the EST models. SSAR Sections 2.5.2.2.1.1 through 2.5.2.2.1.6 provide a summary of the primary seismic sources developed by each of the six ESTs. As stated in SSAR Section 2.5.2.2.1, the 1989 EPRI seismic hazard calculations implemented screening criteria to include only those sources with a combined hazard that exceeded 99 percent of the total hazard from all sources for two ground-motion measures (EPRI NP-6395-D 1989).

Each EST representation of seismic source zones affecting the ESP site region differs significantly in terms of total number of source zones and source characterization parameters such as geometry and maximum magnitudes (and associated weights). For example, the total number of primary source zones identified by each EST ranged from 2 (Rondout Associates team) to 15 (Law Engineering team). However, all teams identified and characterized one or more seismic source zones or background sources that accounted for seismicity in the vicinity of the ESP site. In addition, all of the ESTs identified and characterized one or more seismic source zones to account for the occurrence of Charleston-type earthquakes.

SER Table 2.5.2-1 provides the sources that account for Charleston-type earthquakes. The largest maximum magnitudes (M_{max}) assigned to the Charleston source zone by each team ranged from m_b 6.8 (Law Engineering, with a weight of 1) to m_b 7.5 (Woodward-Clyde, with a weight of 0.33). This corresponds to a moment magnitude (M) range of 6.8 to 8.0.

Table 2.5.2-1 - Summary of EPRI EST Charleston Seismic Sources (Based on Information Provided in SSAR Tables 2.5.2-2 to 2.5.2-7)

EPRI EST	Source	Description	Probability of Activity	M_{max} (mb) and Weights
Bechtel	H	Charleston Area	0.50	6.8 [0.20] 7.1 [0.40] 7.4 [0.40]
	N3	Charleston Faults	0.53	6.8 [0.20] 7.1 [0.40] 7.4 [0.40]
Dames & Moore	54	Charleston Seismic Zone	1.00	6.6 [0.75] 7.2 [0.25]
Law Engineering	35	Charleston Seismic Zone	0.45	6.8 [1.0]
Rondout	24	Charleston	1.0	6.6 [0.20] 6.8 [0.60] 7.0 [0.20]
Weston	25	Charleston Seismic Zone	0.99	6.6 [0.90] 7.2 [0.10]
Woodward-Clyde	30	Charleston (includes NOTA)	0.573	6.8 [0.33] 7.3 [0.34] 7.5 [0.33]
	29	S. Carolina Gravity Saddle (Extended)	0.122	6.7 [0.33] 7.0 [0.34] 7.4 [0.33]
	29A	S. Carolina Gravity Saddle No. 2 (Combo C3)	0.305	6.7 [0.33] 7.0 [0.34] 7.4 [0.33]

Post-EPRI Seismic Source Characterization Studies

SSAR Section 2.5.2.2.2 focuses on the Charleston seismic source zone. The applicant described several PSHA studies that were completed after the 1989 EPRI PSHA, which involved the characterization of seismic sources within the ESP site region. These PSHA studies developed models of the Charleston seismic source that differed from those used in the 1989 EPRI PSHA study because they incorporated recent paleoliquefaction data. The applicant also provided its justification for not updating the EPRI seismic source parameters for the ETSZ, which is situated at the edge of the 320-km (200-mi) site region radius.

Charleston Seismic Source Zone. SSAR Section 2.5.2.2.2 describes three post-EPRI (1989) PSHA studies that characterized the seismic sources within the ESP site region. These studies include the USGS National Seismic Hazard Mapping Project (Frankel et al. 1996, 2002) and the South Carolina DOT (SCDOT) seismic hazard mapping project (Chapman and Talwani 2002). Unlike the EPRI study, these PSHA studies developed models of the Charleston seismic source that incorporated recent paleoliquefaction data.

The applicant stated that abundant soil liquefaction features induced by the 1886 Charleston earthquake, as well as other large prehistoric earthquakes that date back to the mid-Holocene (at least 5000 years), are preserved in geologic deposits at numerous locations within the 1886 meizoseismal area and along the South Carolina coast. In 2001, Talwani and Schaeffer (2001) reevaluated all of the liquefaction data previously compiled for the Charleston area and, based on recalibrated radiocarbon dates for liquefaction features, provided an estimate of earthquake recurrence for the region. Talwani and Schaeffer (2001) reinterpreted radiocarbon dates for previously published liquefaction features documented along the coast of South Carolina. Radiocarbon dates are useful in providing contemporary, minimum, and maximum limiting ages for liquefaction features. Talwani and Schaeffer (2001) recalculated previously compiled age data to account for fluctuations in atmospheric carbon-14 over time. They used the calibrated data to correlate ages of past individual earthquakes and then to estimate earthquake recurrence. Talwani and Schaeffer (2001) also identified individual earthquake episodes based on samples with a "contemporary" age constraint that had overlapping calibrated radiocarbon ages at the 68 percent (1-sigma) confidence interval. They calculated the estimated age of each earthquake from the weighted averages of overlapping contemporary ages. Talwani and Schaeffer (2001) identified a total of eight events from the paleoliquefaction record, including the 1886 Charleston event. These events are referred to as 1886, A, B, C, D, E, F, and G (in order of increasing age).

Talwani and Schaeffer (2001) proposed two scenarios to explain the distribution and timing of paleoliquefaction features (shown in SSAR Table 2.5.2-13). In Scenario 1, they interpreted events A, B, E, and G to be large Charleston-type events, while they interpreted events C, D, and F to be smaller, moderate magnitude (~**M** 6) events. In Scenario 2, Talwani and Schaeffer (2001) interpreted all events as large, Charleston-type events. In addition, they combined events C and D into a large event C' based on the observation that the calibrated radiocarbon ages that constrain the timing of Events C and D are indistinguishable at the 95 percent (2-sigma) confidence interval.

In 2002, the USGS updated the seismic hazard maps for the contiguous United States based on new seismological, geophysical, and geologic information (Frankel et al. 2002). The 2002 USGS update included modifications to the geometry, recurrence, and M_{max} of the Charleston seismic source zone. In its update, the USGS represented Charleston-type earthquakes by two equally weighted areal sources. One of these seismic source zones envelops most of the tectonic features and liquefaction data in the greater Charleston area, while the other source envelops the southern half of the southern segment of the East Coast Fault System (ECFS). Frankel et al. (2002) adopted a mean paleoliquefaction-based recurrence interval of 550 years for Charleston-type earthquakes which ranged from **M** 6.8 to 7.5.

The SCDOT model (Chapman and Talwani 2002) characterized Charleston-type earthquakes by using a combination of three equally weighted line and area sources. The SCDOT model comprises a coastal South Carolina areal source zone that includes most of the paleoliquefaction sites, a source that captures the intersection of the Woodstock and Ashley River faults, and a source that represents the southern ECFS source zone. For Charleston-type earthquakes, which ranged from **M** 7.1 to 7.5, Chapman and Talwani (2002) also adopted a mean paleoliquefaction-based recurrence interval of 550 years.

The applicant briefly mentioned the Trial Implementation Project (TIP) study in the SSAR. However, the applicant did not explicitly include the findings of this study in the SSAR because the TIP study primarily focused on the implementation of the Senior Seismic Hazard Advisory Committee (SSHAC) methodology, rather than the actual seismic hazard estimation.

Eastern Tennessee Seismic Zone. In SSAR Section 2.5.2.2.2.5, the applicant concluded that no new information regarding the ETSZ has been developed since 1986 that would require a significant revision to the original EPRI seismic source model. The applicant noted that despite being one of the most active seismic zones in Eastern North America, no evidence for larger prehistoric earthquakes, such as paleoliquefaction features, has been discovered. The largest earthquake recorded in the ETSZ was a magnitude 4.6 and occurred in 1973. The applicant also noted that a much higher degree of uncertainty is associated with the assignment of M_{max} for the ETSZ than for other CEUS seismic source zones where values of M_{max} are constrained by paleoliquefaction data.

The 1986 EPRI seismic source model (EPRI NP-4726 1986) included various source geometries and parameters to represent the seismicity of the ETSZ. All of the EPRI ESTs, except for the Law Engineering team, represented this area of seismicity with one or more local source zones. The Law Engineering team's Eastern Basement source zone included the ETSZ seismic source zone. With the exception of the Law Engineering team's Eastern Basement source, none of the other ETSZ sources contributed more than 1 percent to the site hazard, and thus were excluded from the final 1989 EPRI PSHA hazard calculations (EPRI NP-6452-D 1989).

Upper-bound $_{max}$imum values of M_{max} developed by the EPRI teams for the ETSZ ranged from **M** 4.8 to 7.5. The applicant found that M_{max} estimates for the ETSZ in more recent studies fall within the range of magnitudes captured by the EPRI model. Bollinger (1992) estimated an M_{max} of **M** 6.3, while the USGS hazard model (Frankel et al. 2002) assigned a single M_{max} value of **M** 7.5 for the ETSZ.

Updated EPRI Seismic Sources

Based on the results of several post-EPRI PSHA studies (Frankel et al. 2002; Chapman and Talwani 2002) and the availability of paleoliquefaction data (Talwani and Schaeffer 2001), the applicant updated the EPRI characterization of the Charleston seismic source zone as part of the ESP application. SSAR Section 2.5.2.2.2.4 describes how the applicant used post-EPRI information to recharacterize the source geometry, M_{max}, and magnitude recurrence for the Charleston seismic source zone. The applicant stated that it updated the Charleston seismic source zone using the guidelines provided in RG 1.165. Specifically, the applicant performed an SSHAC Level 2 study to incorporate current literature and data and the understanding of experts into an update of the Charleston seismic source model. The applicant referred to the updated model in the SSAR as the Updated Charlestown Seismic Source (UCSS) model. Bechtel (2006) describes the development of the UCSS model in greater detail.

UCSS Geometry. To represent the Charleston seismic source, the applicant developed four mutually exclusive source zone geometries. The applicant based the geometries of these four source zones, referred to as A, B, B', and C, on the following information:

- current understanding of geologic and tectonic features in the 1886 Charleston earthquake epicentral region
- the 1886 Charleston earthquake shaking intensity
- distribution of seismicity

- geographic distribution, age, and density of liquefaction features associated with both the 1886 and prehistoric earthquakes

SER Figure 2.5.2-2, reproduced from SSAR Figure 2.5.2-9, depicts the geometries of the applicant's four source zones. As shown in SER Figure 2.5.2-2, Geometry A is an approximately 100 x 50 km, northeast-oriented area centered on the 1886 Charleston meizoseismal area and envelops the following:

- the 1886 earthquake MMI X (severe damage) isoseismal (Bollinger 1977)
- the majority of identified Charleston-area tectonic features and inferred fault intersections
- the area of ongoing concentrated seismicity
- the area of greatest density for the 1886 and prehistoric liquefaction features

Based on the available geologic and seismologic evidence, the applicant concluded that Geometry A defines the area where future Charleston-type earthquakes will most likely occur. For this reason, the applicant assigned a weight of 0.70 to Geometry A in the UCSS model. However, in order to capture the uncertainty that future events may not be entirely restricted to Geometry A, the applicant developed three additional geometries, referred to as B, B', and C, that were each assigned a weight of 0.1.

As shown in SER Figure 2.5.2-2, Geometry B is a coast-parallel source, with an area of approximately 260 x 100 kilometers (161.6 x 62.1 miles), that incorporates all of Geometry A. The elongation and orientation of Geometry B roughly parallels both the regional structural grain as well as the elongation of the 1886 isoseismals (damage contours). Paleoliquefaction features mapped by Amick (1990), Amick et al. (1990a, 1990b), and Talwani and Schaeffer (2001) define the northeastern and southwestern extents of Geometry B. In addition, Geometry B extends to the southeast to include the offshore Helena Banks fault zone; offshore earthquakes in 2002 (mb 3.5 and 4.4) suggest a possible spatial association with the mapped trace of the Helena Banks fault zone. Multiple reflection profiles clearly show the Helena Banks fault, which demonstrates late Miocene (23.8 to 5.3 million years ago (mya)) offset (Behrendt and Yuan 1987).

Geometry B' is an approximately 260 x 50-km (161.6 x 31.1-mi) source area that is identical to Geometry B with the exception that Geometry B' does not include the offshore Helena Banks fault system. The applicant excluded the Helena Banks fault system from Geometry B' because the majority of data and evaluations (e.g., Behrendt and Yuan 1987) suggest that this fault system is no longer active.

Geometry C is an approximately 200 x 30-km (124.3 x 18.6-mi), north-northeast-oriented source area that envelops the southern segment of the ECFS as depicted by Marple and Talwani (2000). Both the U. S. Geological Survey (USGS) hazard model (Frankel et al. 2002) and the SCDOT hazard model (Chapman and Talwani 2002) explicitly incorporate the southern segment of the ECFS as a source zone. However, the USGS hazard model (Frankel et al. 2002) truncated the northern extent of the southern fault segment, while the SCDOT hazard model (Chapman and Talwani 2002) extended the southern segment to include, in part, the liquefaction features in southeastern South Carolina (Chapman 2005). The applicant concluded that the liquefaction features in southeastern South Carolina are captured in source zones B and B'. The applicant further concluded that the truncation of the northern extent of the southern fault segment of the ECFS in the USGS hazard model is not supported by any available data.

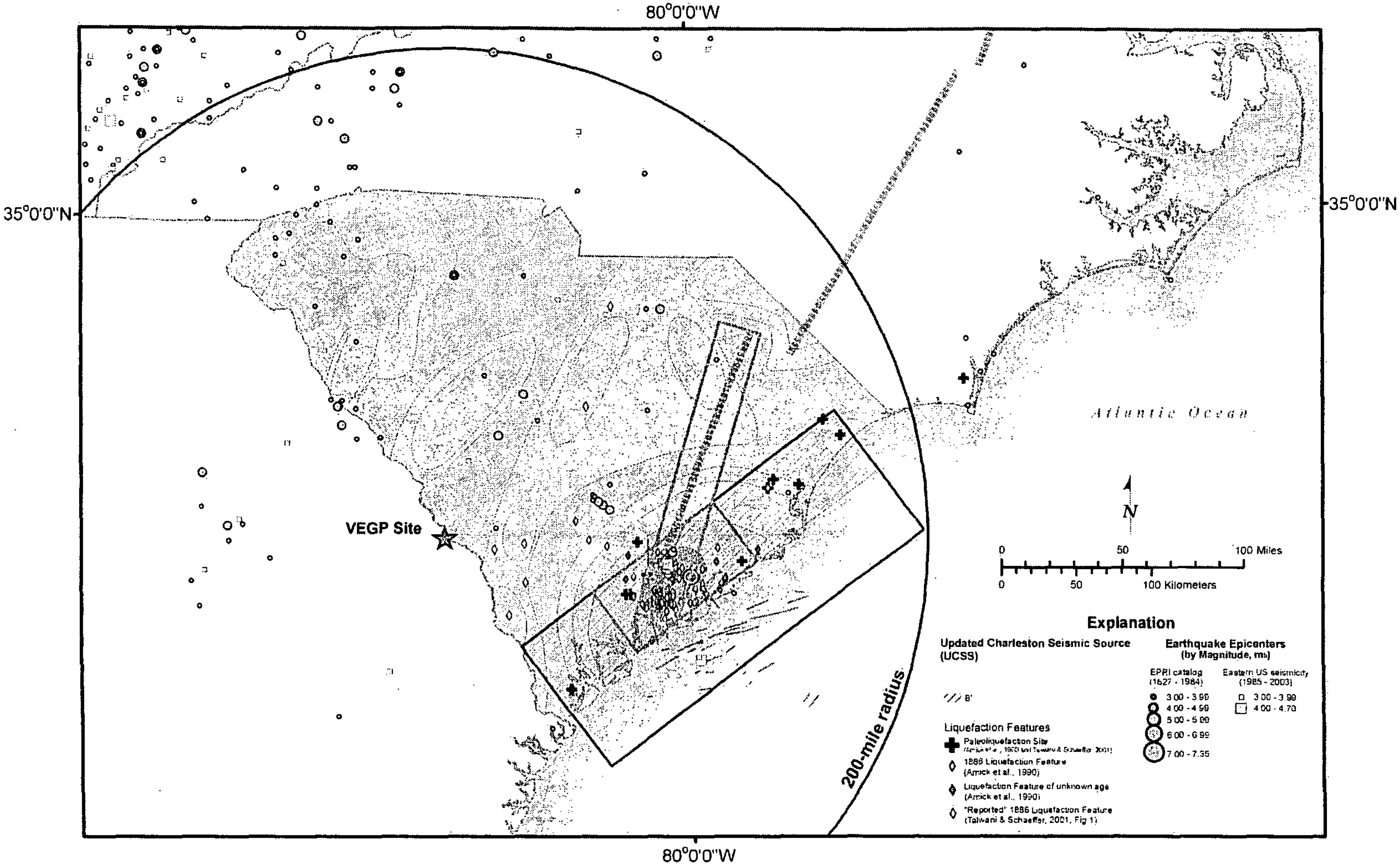


Figure 2.5.2-2 - Alternative geometries comprising the UCSS model updated Charleston seismic source (reproduced from SSAR Figure 2.5.2-9)

UCSS Maximum Magnitude. In order to define the largest earthquake that could be produced by the Charleston seismic source, the applicant stated that it developed a distribution for M_{max} based on several post-EPRI (1989) magnitude estimates for the 1886 Charleston earthquake. The applicant modified the USGS hazard model magnitude distribution (Frankel et al. 2002), shown in SER Table 2.5.2-2, to include a total of five discrete magnitude values, each separated by 0.2 M units. The applicant's M_{max} distribution included a discrete value of M 6.9 to represent the Bakun and Hopper (2004) best estimate of the 1886 Charleston earthquake magnitude, as well as a lower value of M 6.7 to capture the probability that the 1886 earthquake was smaller than the Bakun and Hopper (2004) mean estimate of M 6.9. In their study, Bakun and Hopper (2004) provide a 2-sigma range of M 6.4 to M 7.2.

Table 2.5.2-2 - Comparison of Maximum Magnitudes and Weights for the USGS and SCDOT Models with the Applicant's UCSS Model

M_{max} (M)	USGS Model Weight	SCDOT Model Weight	UCSS Model Weight
6.7	—	—	0.1
6.8	0.2	—	—
6.9	—	—	0.25
7.1	0.2	0.2	0.3
7.3	0.45	0.6	0.25
7.5	0.15	0.2	0.1

UCSS Recurrence Model. Most of the available geologic data pertaining to the recurrence of large earthquakes in the South Carolina region were published after 1990. In the absence of these data, the 1989 EPRI study (EPRI NP-6395-D) estimated the recurrence of large Charleston-type earthquakes using a truncated exponential model. The 1989 EPRI study estimated the parameters of this exponential model from historical seismicity. The recurrence of M_{max} earthquakes in the EPRI study was on the order of several thousand years, which is significantly greater than more recently published estimates of about 500 to 600 years that are based on paleoliquefaction data (Talwani and Schaeffer 2001).

To estimate recurrence for earthquakes with **M** less than 6.7, the applicant used an exponential magnitude distribution. The applicant estimated the parameters of this exponential distribution from the earthquake catalog. However, based on paleoliquefaction data, the applicant found that M_{max} earthquakes (**M** greater than 6.7) have occurred more frequently than would be implied by extrapolation of the recurrence of smaller magnitude (**M** less than 6.7) earthquakes within the UCSS. Thus, the applicant treated M_{max} events within the UCSS according to a characteristic earthquake model, which means that this source repeatedly generates earthquakes, known as characteristic earthquakes, similar in size to M_{max}. The applicant estimated the recurrence of these characteristic earthquakes from paleoliquefaction data.

The applicant stated that it further reevaluated the data presented by Talwani and Schaeffer (2001) and provided an updated estimate of earthquake recurrence. Talwani and Schaeffer (2001) used calibrated radiocarbon ages with 1-sigma error bands to define the timing of past liquefaction episodes in coastal South Carolina. However, the standard practice in paleoliquefaction studies is to use calibrated ages with 2-sigma error bands (e.g., Sieh et al. 1989; Grant and Sieh 1994; Tuttle 2001) to more accurately reflect uncertainties associated with radiocarbon dating. The applicant determined that the use of 1-sigma error bands by Talwani and Shaeffer (2001) may lead to overinterpretation of the paleoliquefaction record such that more episodes are interpreted than actually occurred. For this reason, the applicant recalibrated the radiocarbon ages presented in Talwani and Schaeffer (2001) and reported the newly recalibrated ages with 2-sigma error bands.

The applicant identified six individual paleoearthquakes, including the 1886 Charleston event, from the UCSS calibrated 2-sigma data. The applicant determined that two earthquake events (C and D) identified in the Talwani and Schaeffer (2001) 1-sigma analysis are not individually distinguishable at the 95 percent (2-sigma) confidence interval, and the applicant defined these two events as a single event, C'. The applicant also suggested that Talwani and Schaeffer (2001) events F and G likely represent a single large event, defined by the applicant as event F'. The applicant interpreted the six large paleoearthquakes (1886, A, B, C', E, and F') to represent Charleston-type events that occurred within the past ~5000 years. Furthermore, the applicant determined that results of the 2-sigma analysis suggest there have been four large earthquakes in the most recent ~2000-year (yr) portion of the earthquake record (1886, A, B, and C').

The applicant calculated two different average recurrence intervals, which represent two recurrence branches on the logic tree shown in SSAR Figure 2.5.2-11. The first average recurrence interval is based on the four events (1886, A, B, and C') that the applicant interpreted to have occurred within the past ~2000 years. The applicant concluded that this time period represents a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of the expert elicitation. The applicant assigned a weight of 0.8 to the logic tree branch representing the recurrence interval calculated for the 2000-yr record. The second average recurrence interval is based on events that the applicant interpreted to have occurred within the past ~5000 years and includes events 1886, A B, C', E, and F'. This time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). Published papers and researchers questioned suggest that the older part of the record (i.e., older than ~2000 years) may be incomplete. The applicant noted, however, that it may also be possible that the older record is complete but exhibits longer inter-event times. For this reason, the applicant assigned a weight of 0.2 to the logic tree branch representing the recurrence interval calculated for the 5000-yr record. The applicant indicated that the 0.80 and 0.20 weighting of the ~2000-yr and 5000-yr paleoliquefaction records, respectively, reflect the incomplete knowledge of both the short- and long-term recurrence behavior of the Charleston source.

The applicant used the methods of Savage (1991) and Cramer (2001) to calculate the mean recurrence interval for both the ~2000-yr and ~5000-yr records. According to the applicant, these methods describe the mean recurrence interval with best estimate mean Tave and an uncertainty described as a lognormal distribution with median T0.5 and parametric lognormal shape factor σ0.5. The average recurrence interval for the ~2000-yr record, based on the three most recent inter-event times (1886–A, A–B, B–C'), has a best estimate mean value of 548 years and an uncertainty distribution described by a median value of 531 years and a lognormal shape factor of 0.25. The average recurrence interval for the ~5000-yr record, based on five inter-event times (1886–A, A–B, B–C', C'–E, E–F'), has a best estimate mean value of 958 years and an uncertainty distribution described by a median value of 841 years and a lognormal shape factor of 0.51.

The applicant modeled earthquakes in the exponential part of the distribution as point sources uniformly distributed within the source area, with a constant depth fixed at 10 kilometers. For the characteristic model, the applicant represented source zone Geometries A, B, B', and C by a series of closely spaced, vertical, northeast-trending faults parallel to the long axis of each source zone.

2.5.2.2.3 Correlation of Earthquake Activity with Seismic Sources

SSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI seismic source model. The applicant compared the distribution of earthquake epicenters from both the original EPRI historical catalog (1627–1984) and the updated seismicity catalog (1985–2005) with the seismic sources characterized by each of the EPRI ESTs. Based on this comparison, the applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure. In addition, it concluded that there are no clusters of seismicity that would suggest a new seismic source not captured by the EPRI seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require significant revision to the geometry of any of the EPRI seismic sources. The applicant further stated that the updated catalog does not show or suggest an increase in M_{max} or a significant change in seismicity parameters (activity rate, b-value) for any of the EPRI seismic sources.

2.5.2.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

SSAR Section 2.5.2.4 presents the results of the applicant's PSHA for the ESP site. PSHA is an acceptable method to estimate the likelihood of earthquake ground motions occurring at a site (RG 1.165 and RG 1.208). The hazard curves generated by the applicant's PSHA represent generic hard rock conditions (characterized by a shear- (S-) wave velocity of 9200 feet per second (ft/s)). In SSAR Section 2.5.2.4, the applicant also described the earthquake potential for the site in terms of the most likely earthquake magnitudes and source-site distances, which are referred to as controlling earthquakes. The applicant determined the low-and high-frequency controlling earthquakes by deaggregating the PSHA at selected probability levels. Before determining the controlling earthquakes, the applicant updated the original 1989 EPRI PSHA (EPRI NP-6395 1989) using the seismic source zone adjustments, described in SER Section 2.5.2.1.2, and the new ground motion models described below.

PSHA Inputs

Before performing the PSHA, the applicant updated the original 1989 EPRI PSHA inputs using the seismic source zone adjustments described in SSAR Section 2.5.2.2. In addition, the applicant used the updated 2004 EPRI (EPRI 1009684) ground motion models instead of the EPRI NP-6395-D (1989) ground motion models, which were used in the original 1989 EPRI PSHA.

Seismic Source Model

To update the original EPRI model, the applicant removed all of the sources identified as a Charleston source from each of the six EPRI EST models. SER Table 2.5.2-1 lists these sources. The applicant then incorporated its four UCSS alternative source geometries, M_{max}, and recurrence distributions into each of the six EST models. The applicant explained that in most cases, this involved replacing a single Charleston source with four alternative Charleston sources.

The applicant used an exponential magnitude distribution to model smaller earthquakes (**M** less than 6.7) within the UCSS. To calculate the activity rate and b-value for this distribution, the applicant used the same methodology and smoothing assumptions that were used in the 1989 EPRI study. However, the applicant calculated these seismicity parameters using the new geometries of the UCSS along with the updated seismicity catalog (through April 2005). Because old and new source geometries are not coincident, the applicant allowed the portions of "old" EPRI sources that fell outside of the new UCSS source geometries to default to the existing EPRI background sources. According to the applicant, this ensured that no areas in the seismic hazard model were aseismic. For the unmodified sources of the 1989 EPRI PSHA, the applicant used the original seismicity rates from the 1988 EPRI (EPRI NP-4726-A 1988) earthquake catalog (through 1984) in its seismic hazard calculations.

To determine whether the seismicity rates used in the 1989 EPRI PSHA (EPRI NP-6395-D 1989) are appropriate for the assessment of the seismic hazard at the ESP site, the applicant assessed seismicity rates for two sources in the site region: 1) a small rectangular source around the Charleston seismicity; and (2) a triangular-shaped source representing seismicity in South Carolina and a strip of Georgia that incorporates the ESP site. The applicant stated that it selected these sources because they contribute the most to the seismic hazard at the ESP site.

The applicant investigated the seismicity rates in the two sources by running the program EQPARAM (from the EPRI EQHAZARD package) first for the original EPRI catalog and then for the updated EPRI catalog (through April 2005). The applicant used the a- and b-values obtained from EQPARAM to calculate the recurrence rates for different earthquake magnitudes. For the rectangular Charleston source, the applicant concluded that the seismicity rates remain the same when the seismicity from 1985 to April 2005 is added. For the triangular South Carolina source, the applicant concluded that the seismicity rates decrease when the seismicity from 1985 to April 2005 is added.

The applicant concluded that the seismicity recorded since 1984 does not indicate that seismic activity rates have increased in those sources contributing most to the hazard at the ESP site, under the assumptions of the 1989 EPRI PSHA. Based on the review of geological and seismological data published since the 1986 EPRI Project (EPRI NP-4726), presented in SSAR Section 2.5.2, the applicant concluded that, with the exception of the Charleston seismic source, there are no significant changes to the original EPRI M_{max} values. SSAR Section 2.5.2.2.2 discusses the applicant's modifications to M_{max} for the Charleston seismic source.

Ground Motion Models

The applicant used the ground-motion models developed by the 2004 EPRI-sponsored study (EPRI 1009684 2004) for the updated PSHA. For general area sources, the applicant combined 9 estimates of median ground motion with 4 estimates of aleatory uncertainty, which resulted in 36 combinations. For fault sources in rifted regions (which apply to the East Coast Fault System [ECFS] fault segments), the applicant combined 12 estimates of median ground motion with four estimates of aleatory uncertainty, resulting in 48 combinations.

The applicant compared the EPRI NP-6395 (1989) ground motion model with the EPRI 1009684 (2004) ground motion models. The differences between the two models are a function of magnitude, distance, and structural frequency. The applicant stated that in general, the median ground-motion amplitudes are similar at high frequencies. At low frequencies, the EPRI 1009684 (2004) models show lower median ground motions because these models incorporate the possibility of a double-corner source model. However, the applicant stated that the EPRI 1009684 standard deviations are universally higher than those of EPRI NP-6395.

PSHA Methodology and Calculation

For the PSHA calculation, the applicant used the Risk Engineering, Inc. FRISK88 seismic hazard code. The applicant first performed a PSHA using the original 1989 EPRI primary seismic sources and ground-motion models in order to validate FRISK88 against the EPRI software EQHAZARD. The applicant compared the results from FRISK88 with the original EPRI hard rock results. The applicant determined that a comparison of the mean hazard curves for peak ground acceleration (PGA) generally agrees to within 5.1 percent for amplitudes up to 1 g.

Using the updated EPRI seismic source characteristics and new ground-motion models as inputs, the applicant performed PSHA calculations for PGA and spectral acceleration at frequencies of 25, 10, 5, 2.5, 1, and 0.5 hertz (Hz). Following the guidance provided in RG 1.165, the applicant performed PSHA calculations assuming generic hard rock site conditions (i.e., an S-wave velocity of 9200 ft/s). The applicant incorporated the effects of the ESP site geology into its calculation of the SSE spectrum, which uses the hard rock PSHA results as a starting point.

PSHA Results

To determine the low- and high-frequency controlling earthquakes for the ESP site, the applicant followed the procedure outlined in Appendix C to RG 1.165. This procedure involves the deaggregation of the PSHA results at a target probability level to determine the controlling earthquake in terms of a magnitude and source-to-site distance. The applicant chose to perform the deaggregation of the mean 10^{-4}, 10^{-5}, and 10^{-6} PSHA hazard results. SER Figure 2.5.2-3 shows the results of the applicant's high-frequency (5 to 10 Hz) 10^{-4} hazard deaggregation, while SER Figure 2.5.2-4 shows the results of the low-frequency (1 to 2.5 Hz) 10^{-4} hazard deaggregation. The staff did not show the applicant's deaggregation plots for the 10^{-5} and 10^{-6} mean hazard levels because of their similarity to the 10^{-4} deaggregation plot shown in SER Figures 2.5.2-3 and 2.5.2-4.

High Frequency, 1.0e-4

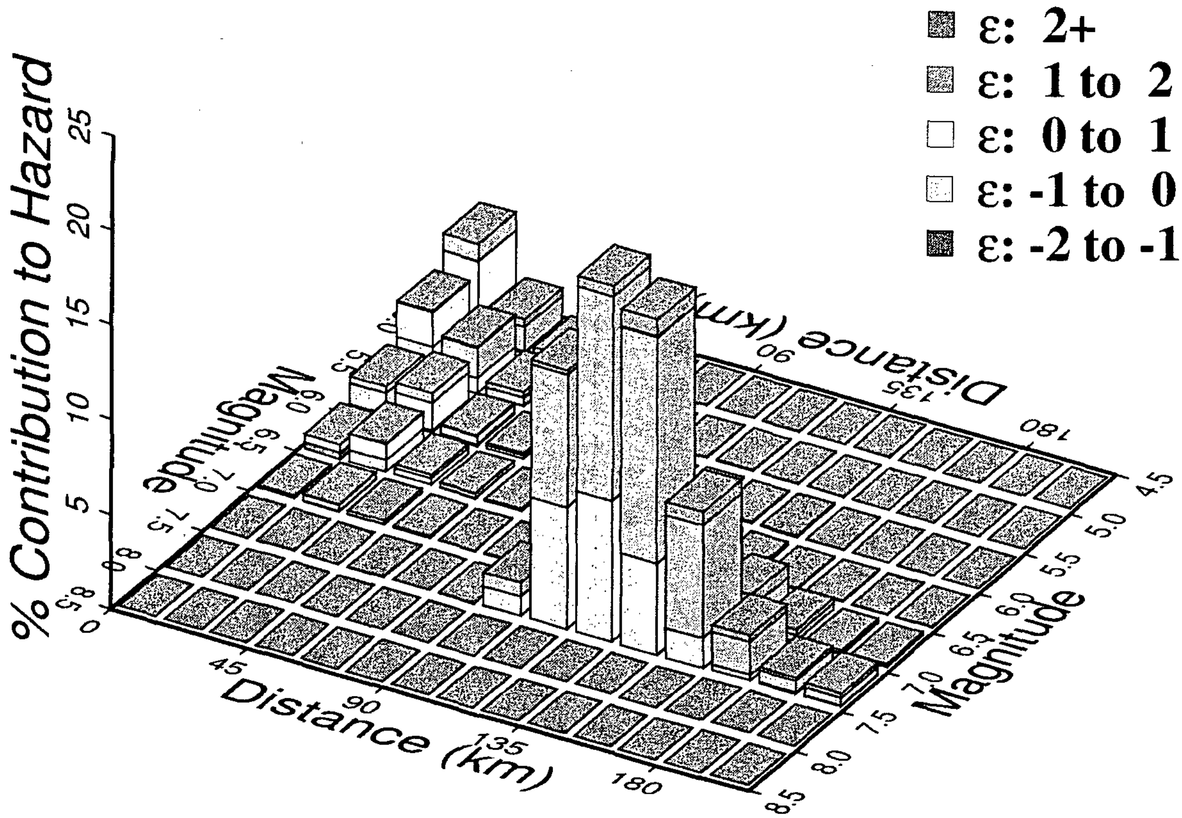


Figure 2.5.2-3 - High-frequency (5 to 10 Hz) 10^{-4} hazard deaggregation (reproduced from SSAR Figure 2.5.2-22)

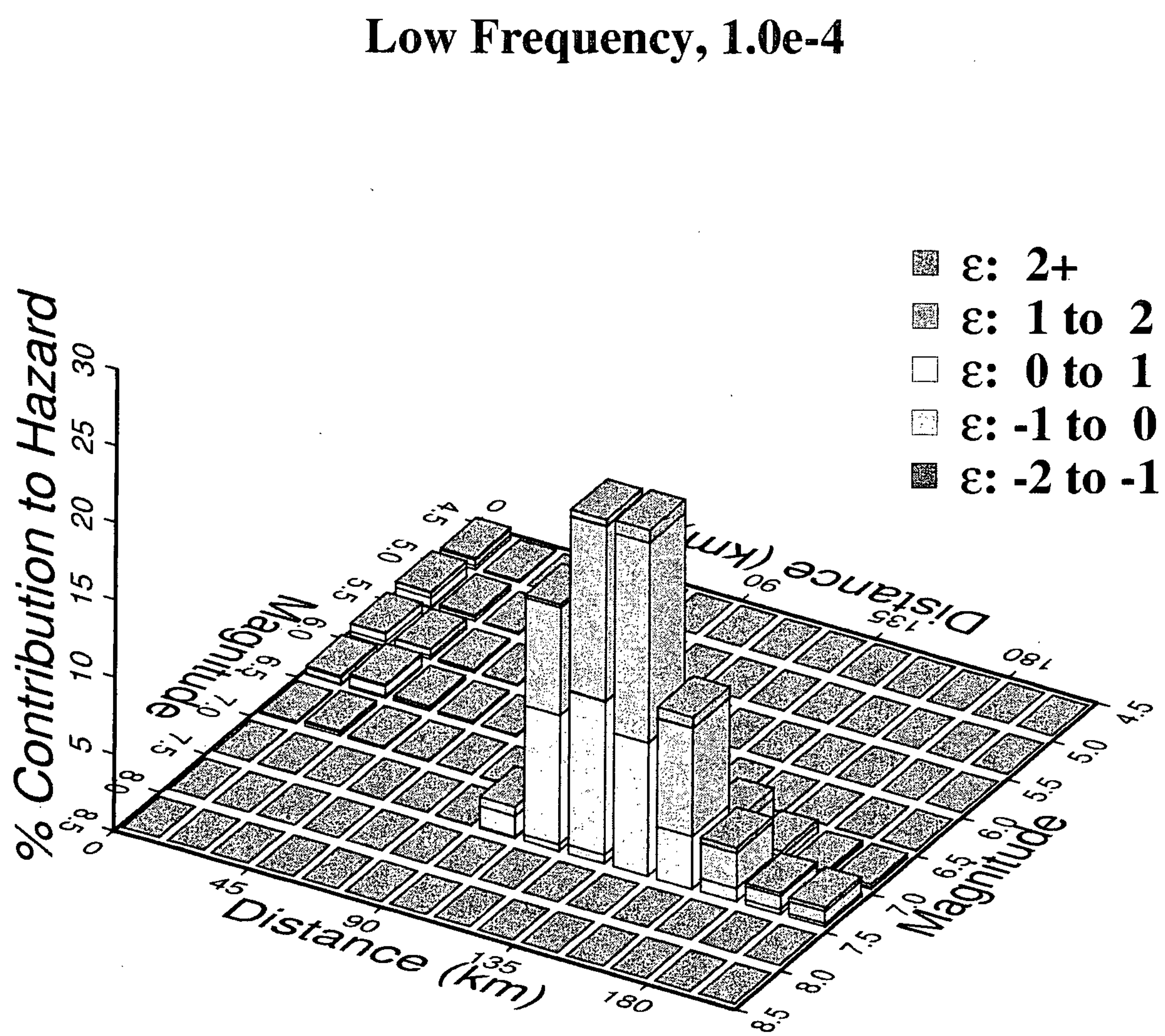


Figure 2.5.2-4 - Low-frequency (1 to 2.5 Hz) 10^{-4} hazard deaggregation (reproduced from SSAR Figure 2.5.2-23)

Because of the similarity of the mean magnitude (Mbar) and mean distance (Dbar) values for the three hazard levels, the applicant selected a single Mbar and Dbar value for each frequency range. SER Table 2.5.2-3 provides the Mbar and Dbar values for the high- and low-frequency controlling earthquakes corresponding to the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. SER Table 2.5.2-3 also provides the applicant's final Mbar and Dbar values for the high- and low-frequency controlling earthquakes. For the high-frequency mean 10^{-4}, 10^{-5}, and 10^{-6} hazard, the controlling earthquake, based on the final Mbar and Dbar pair, is an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles), corresponding to an earthquake from a local seismic source zone. For the low-frequency mean 10^{-4}, 10^{-5}, and 10^{-6} hazard, the controlling earthquake is an **M** 7.2 event and occurs at a distance of 130 kilometers (80.8 miles). This earthquake corresponds to an event in the Charleston seismic zone.

Table 2.5.2-3 - Computed and Final Mbar and Dbar Values Used for Development of High- and Low-Frequency Target Spectra (Based on the Information Provided in SSAR Table 2.5.2-17)

High Frequency (5 to 10 Hz)				
Mean Hazard Level	10^{-4}	10^{-5}	10^{-6}	Final Values
Mbar (M)	5.5	5.6	5.6	5.6
Dbar	17.7 km (11 mi)	11.5 km (7.1 mi)	9.1 km (5.7 mi)	12 km (7.5 mi)
Low Frequency (1 to 2.5 Hz)				
Mean Hazard Level	10^{-4}	10^{-5}	10^{-6}	Final Values
Mbar (M)	7.2	7.2	7.2	7.2
Dbar	136.5 km (84.8 mi)	134.3 km (83.5 mi)	132.9 km (82.6 mi)	130 km (80.8 mi)

2.5.2.2.5 Seismic Wave Transmission Characteristics of the Site

SSAR Section 2.5.2.5 describes the method used by the applicant to develop the site free-field soil uniform hazard response spectrum (UHRS). The hazard curves generated by the PSHA are defined for generic hard rock conditions (characterized by an S-wave velocity of 9200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 2000 feet below the ground surface at the ESP site. To determine the soil UHRS, the applicant: (1) developed soil/rock profile models for the ESP site; (2) selected seed earthquake time histories; and (3) performed the final site response analysis.

Site Response Model

According to the applicant, the soil profile to a depth of approximately 1049 feet at the ESP site consists of approximately 86 feet of predominantly sands, silty sands, and clayey sands, with occasional clay seams, referred to as the Upper Sand Stratum (Barnwell Group). At the base of this sand unit is a Shelly Limestone (Utley Limestone), which is characterized by solution channels, cracks, and discontinuities. Beneath the Utley limestone is the Blue Bluff Marl (Lisbon Formation), consisting of approximately 64 feet of slightly sandy, cemented calcareous clay. The Blue Bluff Marl is underlain by approximately 900 feet of fine-to-coarse sand with interbedded silty clay and clayey silt, referred to as the Lower Sand Stratum. The Lower Sand Stratum comprises the Still Branch, Congaree, Snapp, Black Mingo, Steel Creek, Gaillard/Black Creek, Pio Nono, and Cape Fear formations.

According to the applicant, the rock profile at the ESP site, below approximately 1049 feet, consists of the Dunbarton Triassic (206–24 mya) basin followed by Paleozoic (543–248 mya) crystalline rock. The Dunbarton Triassic basin rock comprises red sandstone, breccia, and mudstone and is characterized by a weathered zone in the upper 120 feet. The Paleozoic crystalline basement is characterized by a high S-wave velocity (greater than 9200 ft/s). The Pen Branch fault forms the boundary between the Dunbarton Triassic basin and the Paleozoic basement rock. As described in SSAR Section 2.5.1, the Pen Branch fault dips to the southeast at an angle of 45 degrees below the ESP site.

The soil/rock profile model used by the applicant for its site response analysis is shown in SSAR Figure 2.5.4-7 and SSAR Table 2.5.4-11. The uppermost competent in-situ layer is the Blue Bluff Marl, which is encountered at a depth of 86 feet and characterized by an average S-wave velocity of 2354 ft/sec. Note that SSAR Figure 2.5.4-7 and SSAR Table 2.5.4-11 do not show the Barnwell Group and Utley Limestone. The applicant intends to remove the incompetent Barnwell Group (and the underlying Utley Limestone) because it is susceptible to liquefaction and dissolution-related ground deformation. Furthermore, its S-wave velocity is generally below 1000 ft/s. Thus, in its site response calculations, the applicant assumes that these layers have been replaced with 86 feet of structural backfill.

SSAR Figure 2.5.4-7 shows S-wave velocities for each of the different soil and rock layers to a maximum depth of 2275 feet. The applicant based this S-wave velocity profile on the results of suspension primary and secondary (P-S) velocity and seismic cone penetrometer tests (CPTs) performed at the ESP site, as well as deep borehole S-wave velocity data from the Savannah River Site (SRS 2005). The applicant did not determine S-wave velocity for the compacted backfill as part of the ESP subsurface investigation. Instead, the applicant relied on data for existing Units 1 and 2. To represent the variability of the depth to the top of the Paleozoic crystalline basement, where the S-wave velocity is at least 9200 ft/s, the applicant developed six alternative site response profiles, which are provided in Part B of SER Table 2.5.4-11. For the six alternative profiles, the depth to the top of the Paleozoic crystalline rock ranged from 1525 feet to 2275 feet. According to the applicant, the six alternative site response profiles also accounted for the uncertainty of the S-wave velocity gradient between the top of the unweathered section of the Dunbarton Triassic basin to the top of the Paleozoic crystalline rock. In its site response model, the applicant used the PSHA rock motions at the top of the Paleozoic crystalline rock as input.

The applicant collected additional S-wave velocity data as part of the COL site investigation. This data is described in detail in SSAR Section 2.5.4.4 and is referred to as "COL" data by the applicant. The applicant used the SASW (Spectral Analysis of Surface Waves) and cross-hole methods, and the results of Resonant Column and Torsional Shear (RCTS) tests to determine the S-wave velocity of the proposed backfill. The applicant also determined the S-wave velocity of the Blue Bluff Marl and the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum using down-hole seismic CPT tests and suspension P-S velocity tests, combined these data with two ESP profiles (located in the powerblock area of Units 3 and 4) and averaged the results. The applicant then developed an S-wave velocity profile for soil (i.e. to a depth of 1059 ft). The resulting S-wave velocity profile is presented in SSAR Table 2.5.4-11a and SSAR Figure 2.5.4-7a. Because the COL S-wave velocity measurements only extended to a maximum depth of 420 feet below ground surface, the applicant incorporated the S-wave velocity data from the ESP profile (provided in SSAR Table 2.5.4-11 and SSAR Figure 2.5.4-7) below this depth.

The applicant did not use the additional COL S-wave velocity profile as input to its site response calculations. Instead, the applicant provided justification that the use of only the ESP S-wave velocity profile is adequate. In SSAR Section 2.5.4.7.5, the applicant presented a comparison of the ESP and COL S-wave velocity profiles. Based on the comparison of the two S-wave velocity profiles shown in SSAR Figure 2.5.4-7a, the applicant concluded that there is good agreement between the two data sets. Furthermore, based on the results of site response sensitivity studies presented in SSAR Section 2.5.2.9, the applicant concluded that the difference in the amplification between the ESP and COL data is small.

The strain-dependent shear modulus and damping relationships used by the applicant for the soil units at the ESP site are based on EPRI TR-102293 (1993). The applicant also used the strain-dependent shear modulus and damping relationships developed for the nearby SRS by Lee (1996). For the Dunbarton Triassic basin and Paleozoic crystalline rocks, the applicant assumed linear behavior during earthquake shaking with 1-percent damping.

As part of the COL site investigation, the applicant also developed strain-dependent shear modulus and damping relationships based on RCTS tests performed on compacted backfill, Blue Bluff Marl, and Lower Sand samples. The resulting site-specific shear modulus reduction curves are provided in SSAR Table 2.5.4-12a and SSAR Figure 2.5.4-9a, while the site specific damping curves are provided in SSAR Table 2.5.4-12a and SSAR Figure 2.5.4-11a. Although the applicant relied only on the generic EPRI and SRS strain-dependent shear modulus and damping relationships as input to its site response calculations, the applicant presented a comparison with the site-specific relationships in SSAR Figures 2.5.4-19a through 2.5.4-20c. Specifically, SSAR Figures 2.5.4-19a, 19b, and 19c compare the normalized shear modulus reduction versus shear strain curves for the compacted backfill, Blue Bluff Marl, and Lower Sands, respectively. SSAR Figures 2.5.4-20a, 20b, and 20c compare damping versus shear strain for the same units. In SSAR Section 2.5.4.7.5, the applicant stated that generally, the figures suggest that the subsurface soils behave more linearly (i.e. provide a smaller reduction in shear modulus and less damping) than both the generic EPRI and SRS relationships. However, the applicant's site response sensitivity studies, described in SSAR Section 2.5.2.9, resulted in small differences in amplification between the ESP and COL data.

The applicant stated that once it determined the appropriate soil and rock dynamic properties, it modeled the variability present in the site data by randomizing the soil and rock S-wave velocity profiles, soil shear modulus reduction and damping relationships, and rock-damping values. For each family of degradation curves (i.e., EPRI or SRS), the applicant generated 60 randomized soil/rock profiles to account for the variability in the site properties. The applicant generated the 60 randomized soil/rock profiles using the stochastic model described in EPRI TR-102293 (1993) and Toro (1996). Inputs to the applicant's stochastic model include the base-case soil and rock profiles provided in SSAR Table 2.5.4-11, as well as the depth to bedrock, which the applicant randomized to account for the range of depths associated with the Pen Branch fault. For each randomized velocity profile, the applicant developed one set of randomized shear modulus reduction and damping curves from the EPRI family of curves and another set from the SRS family of curves.

To account for the variability in soil shear strain modulus and material-damping ratio with shearing strain amplitude, the applicant randomized the shear modulus reduction and damping curves used for the site response analysis. For each of the randomized velocity profiles, the applicant developed one set of randomized shear modulus reduction and damping curves for each family of degradation curve (i.e., EPRI or SRS). Inputs to the applicant's model include the base-case shear modulus reduction and damping curves provided in SSAR Tables 2.5.4-12 and 2.5.4-13 and shown in SSAR Figures 2.5.4-9 to 2.5.4-12. The applicant stated that it also accounted for the uncertainty in damping ratio for the Dunbarton Triassic basin rock, which is represented by a 5- to 95-percentile range of 0.7 to 1.5 percent.

Site Response Input Time Histories

The applicant developed target spectra for two different frequency ranges, high-frequency (5 to 10 Hz) and low-frequency (1 to 2.5 Hz), as defined in RG 1.165. These high- and low-frequency target response spectra represent the Mbar and Dbar values from the deaggregation of the 10^{-4}, 10^{-5}, and 10^{-6} hazard curves. For the high-frequency cases, the applicant considered only those sources within 105 kilometers of the site to compute the Mbar and Dbar values. To compute the low-frequency Mbar and Dbar values, the applicant only considered sources at distances greater than 105 kilometers from the site. The applicant noted that this distinction was made based on the dominance of the Charleston source for low frequencies and long return periods.

Because of the similarity of the calculated Mbar and Dbar values for the three hazard levels, the applicant selected a single Mbar and Dbar pair to represent the high-frequency controlling earthquake and a single Mbar and Dbar pair to represent the low-frequency controlling earthquake. SER Table 2.5.2-3 provides the final Mbar and Dbar values used for the development of the high- and low-frequency target spectra.

Using the final high- and low-frequency Mbar and Dbar values, described above, the applicant developed target response spectra using the log-average of the single and double corner CEUS spectral shape models of NUREG/CR-6728 (Technical Basis for Revision of Regulatory Guidance of Design Ground Motions: Hazard- and Risk- Consistent Ground Motion Spectra Guidelines). The applicant scaled the low-frequency spectral shape to the corresponding UHRS (i.e., 10^{-4}, 10^{-5} or 10^{-6}) at 1.75 and scaled the high-frequency spectral shape to the corresponding UHRS at 7.5 Hz. SER Figure 2.5.2-5 shows the resulting high- and low-frequency target response spectra for the 10^{-4} mean hazard level. The applicant also developed target response spectra for the 10^{-5} and 10^{-6} hazard levels.

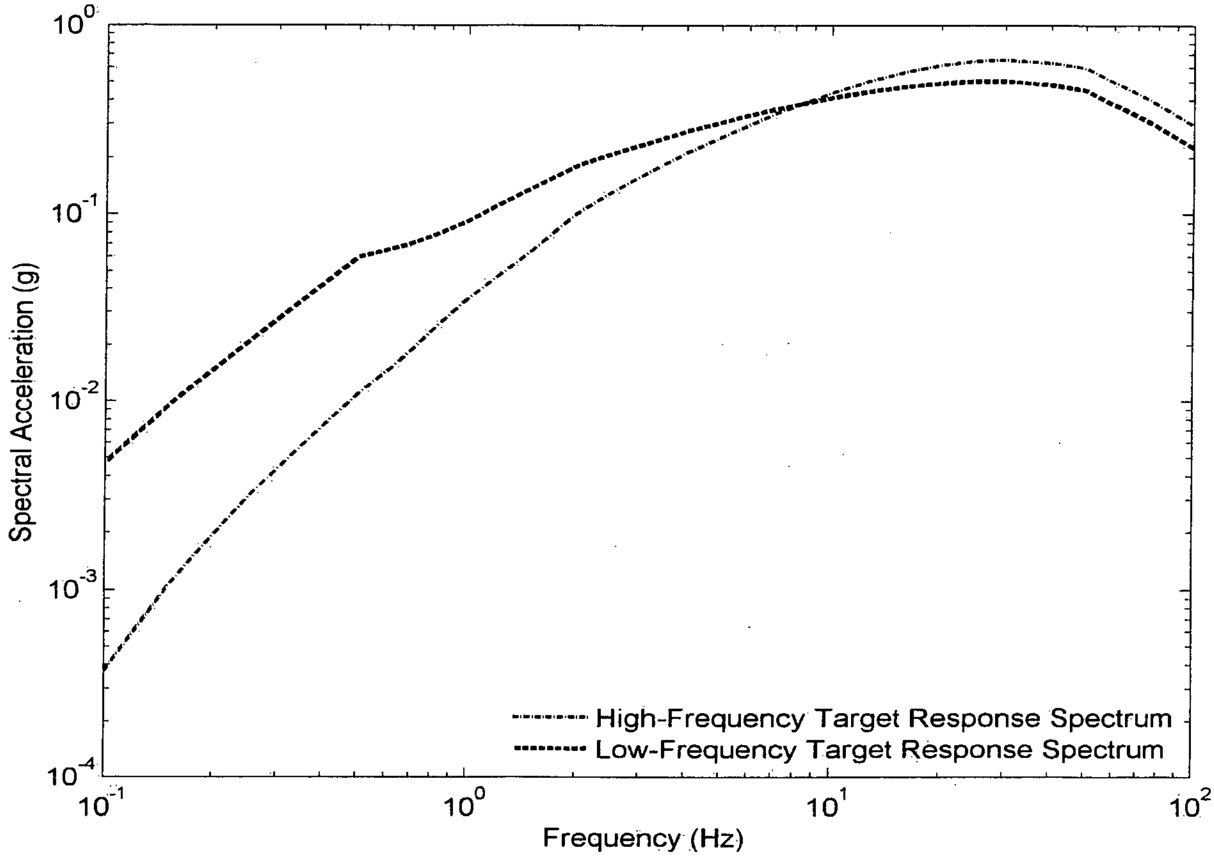


Figure 2.5.2-5 - Low- and high-frequency target response spectra representing the 10^{-4} hazard level (based on the information provided in SSAR Tables 2.5.2-20a, and 2.5.2-20b).

To determine the ESP dynamic site response, the applicant spectrally matched a suite of acceleration time histories to the six target response spectra described above. The applicant selected strong motion acceleration time histories that were recorded at rock-site locations in the Western United States (WUS), Eastern Canada, Turkey, and Japan. Specifically, the applicant selected time histories recorded at sites characterized by S-wave velocities greater than 600 meters per second (m/s) (1968.5 ft/s) in the upper 30 meters (98.4 feet) and similar magnitudes and distances to the final high- and low-frequency Mbar and Dbar values.

The applicant spectrally matched a total of 30 seed time histories to the low-frequency target response spectra corresponding to the 10^{-4}, 10^{-5}, and 10^{-6} mean hazard levels. The applicant spectrally matched a different group of 30 seed time histories to the high-frequency target response spectra representing the 10^{-4}, 10^{-5}, and 10^{-6} mean hazard levels. The applicant used the spectral matching criteria recommended in NUREG/CR-6728 to check the average spectrum from the 30 spectrally matched time histories for a given frequency range and mean hazard level.

Site Response Methodology and Calculation

To determine the final site response, the applicant used the program SHAKE to compute the site amplification functions (AFs) for each of the spectrally matched time histories. As shown in SER Table 2.5.2-4, for each hazard level (10^{-4}, 10^{-5}, and 10^{-6}) and for each deaggregation earthquake (high- and low-frequency), the applicant paired the 60 randomized soil profiles corresponding to the EPRI curves and the 60 randomized soil profiles representing the SRS curves with the 30 spectrally matched time histories. The applicant applied each time history to two of the randomized soil/rock profiles, which resulted in a total of 240 AFs for each of the three mean hazard levels.

Table 2.5.2-4 - Site Response Analyses Performed (Based on the Information Provided in SSAR Table 2.5.2-19)

Mean Hazard Level	10^{-4}		10^{-5}		10^{-6}		Total Number of Analyses
Deaggregation Earthquake	High Freq.	Low Freq.	High Freq.	Low Freq.	High Freq.	Low Freq.	
Number of Input Time Histories	30	30	30	30	30	30	
Number of Randomized Soil Profiles (EPRI)	60	60	60	60	60	60	360
Number of Randomized Soil Profiles (SRS)	60	60	60	60	60	60	360
							720

Site Response Results

To obtain the final site AFs, the applicant divided the output response spectrum (defined at the top of the backfill) by the hard rock input response spectrum for each of the cases shown in SER Table 2.5.2-4. For the 10^{-4} mean hazard level, the applicant computed the mean of the 60 individual AFs corresponding to the high-frequency input time histories and the EPRI-based randomized soil profiles. The applicant repeated this process for the SRS-based randomized soil profiles. The applicant's final high-frequency AF (shown in the lower plot of SER Figure 2.5.2-6) corresponds to the mean of these two results. The applicant developed the final low-frequency AF in a similar manner and this is also shown in SER Figure 2.5.2-6 (upper plot). According to the applicant's results, the ESP site subsurface amplifies the high-frequency input hard rock motion over the fairly wide frequency range of 0.1 to ~25 Hz, with the maximum amplification of 3.8 at a frequency of 0.6 Hz. The applicant's results also show that the low-frequency input hard rock motion is amplified over the frequency range of 0.1 to ~20 Hz, with the maximum amplification of 4.0 at a frequency of 0.6 Hz.

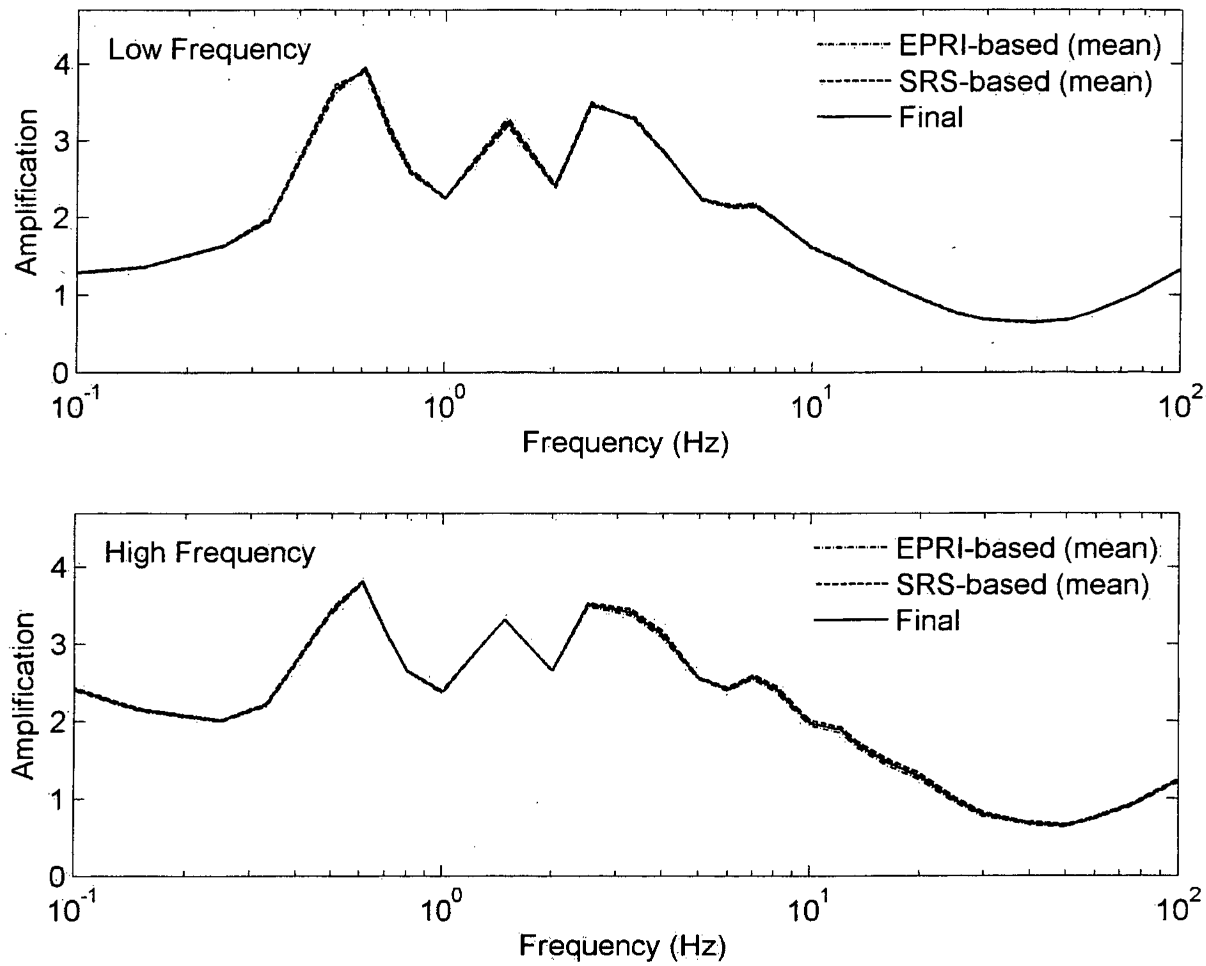


Figure 2.5.2-6 - Final EPRI and SRS high- and low-frequency AFs for the 10^{-4} hazard level (based on the information provided in SSAR Tables 2.5.2-20e and 2.5.2-20f)

The applicant determined the final 10-4 soil surface spectrum for the ESP site by scaling the hard rock UHRS (shown in SER Figure 2.5.2-5) by the final AFs (shown in SER Figure 2.5.2-6). The applicant defined each of the AFs at a total of 300 frequencies, but only defined the hard rock UHRS at 7 structural frequencies. For this reason, the applicant interpolated the hard rock UHRS at values between the 7 structural frequencies using the high- and low-frequency spectral shapes for hard rock from NUREG/CR-6728. The applicant's choice of the high- or low-frequency spectral shape for the interpolation depended on the envelope motion. The applicant defined the envelope motion as the envelope of the high- and low-frequency mean output response spectra (defined at the top of the soil column). The applicant noted that at frequencies above 8 Hz, this is always the HF motion and at frequencies below 2 Hz, this is always the LF motion. The applicant further noted that at frequencies between 2 and 8 Hz, the envelope motion depended on the frequency.

Next, the applicant multiplied the hard rock UHRS (now defined at 300 structural frequencies) by either the high- or low-frequency final amplification factors (shown in SER Figure 2.5.2-6). The applicant multiplied the hard rock UHRS by the low-frequency mean amplification factor if it used low-frequency spectral shape to interpolate the hazard rock UHRS at that structural frequency. If the applicant used the high-frequency spectral shape to interpolate the hard rock

UHRS at that frequency, then it multiplied the hard rock UHRS by the high-frequency mean AF. The applicant stated that at some intermediate frequencies between 2 and 8 Hz, the high- and low- frequency AFs are weighted in order to achieve a smooth transition between HF and LF spectra.

The applicant repeated the above process for the 10^{-5} hazard level to determine the final 10^{-5} soil UHRS. SER Figure 2.5.2-7 provides the final soil UHRS for the 10^{-4} and 10^{-5} hazard levels.

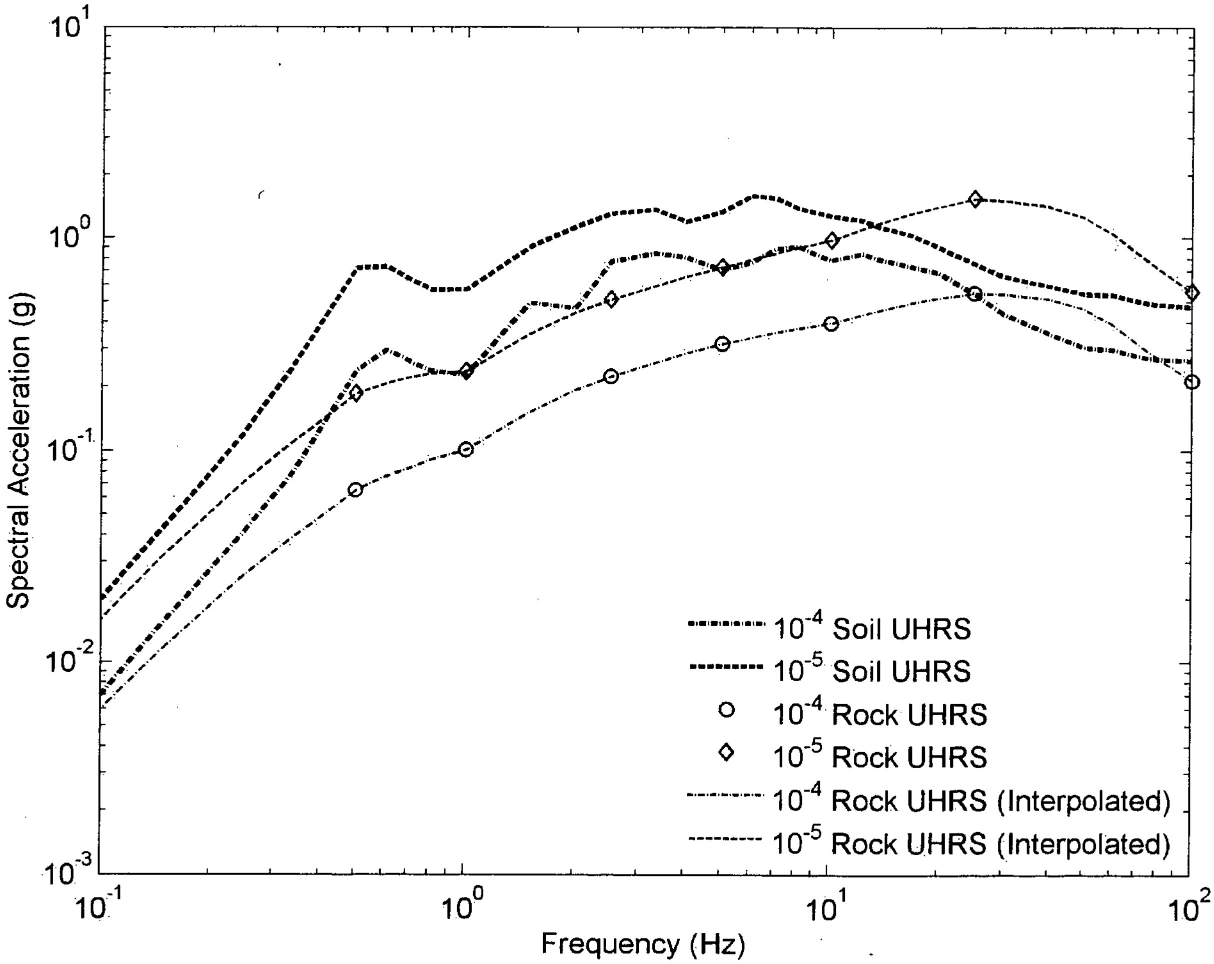


Figure 2.5.2-7 - Horizontal soil-based UHRS for the 10^{-4} and 10^{-5} hazard levels (based on the information provided in SSAR Tables 2.5.2-16 and 2.5.2-21b)

2.5.2.2.6 Ground Motion Response Spectra

SSAR Section 2.5.2.6 describes the method used by the applicant to develop the horizontal and vertical site-specific ground motion response spectra (GMRS). To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and in ASCE/SEI Standard 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary." The applicant developed the vertical GMRS by applying vertical-to-horizontal response spectral (V/H) ratios, based on NUREG/CR-6728 and Lee (2001), to the horizontal GMRS.

Horizontal Ground Motion Response Spectrum

The applicant developed a horizontal, site-specific, performance-based GMRS using the method described in ASCE/SEI Standard 43-05 and RG 1.208. The performance-based method achieves the annual target performance goal (PF) of 10^{-5} per year for frequency of onset of significant inelastic deformation. This damage state represents a minimum structural damage state, or essentially elastic behavior, and falls well short of the damage state that would interfere with functionality. The horizontal GMRS, which meets the PF, is obtained by scaling the site-specific mean 10^{-4} UHRS by a design factor (DF):

$$DF = \max\{1.0, 0.6(A_R)^{0.8}\} \qquad \text{Equation (1)}$$

where the amplitude ratio, AR, is given by the ratio of the 10^{-5} UHRS and the 10^{-4} UHRS spectral accelerations for each spectral frequency.

The applicant determined the horizontal performance-based GMRS by scaling the 10^{-4} soil UHRS, shown in SER Figure 2.5.2-7, by the DF defined by Equation (1). The applicant's horizontal GMRS is shown in SER Figure 2.5.2-8, which is defined at the top of the structural backfill. The applicant smoothed the GMRS using a running average filter (above 1 Hz) constrained to go through the seven structural frequencies that define the original rock UHRS (SER Figure 2.5.2-5). The applicant made an exception for the 5-Hz structural frequency because of the trough observed in the 10^{-4} soil UHRS (refer to SER Figure 2.5.2-8) at this frequency. The smoothed 5-Hz GMRS value is based on amplitudes at adjacent frequencies. SER Figure 2.5.2-8 also shows the soil UHRS for both the 10^{-4} and 10^{-5} mean hazard levels for comparison.

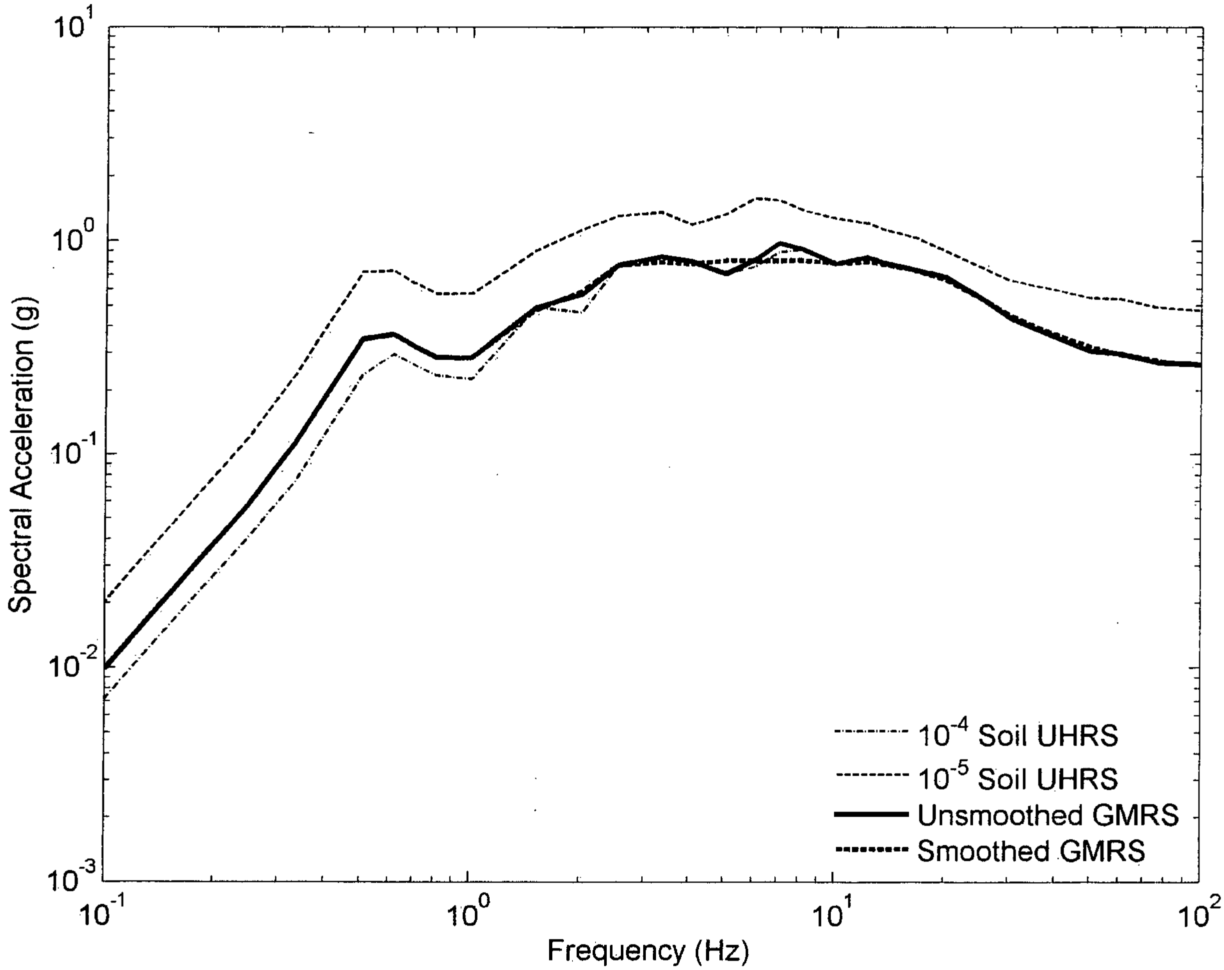


Figure 2.5.2-8 - Horizontal raw and smoothed GMRS (based on the information provided in SSAR Table 2.5.2-22b)

Vertical GMRS

To determine the vertical GMRS, the applicant applied V/H ratios, based on NUREG/CR-6728 and Lee (2001), to the horizontal smoothed GMRS shown in SER Figure 2.5.2-8. Since the V/H ratios presented in NUREG/CR-6728 and Lee (2001) are functions of magnitude, source distance, and local site conditions, the applicant developed V/H ratios corresponding to the final low- and high-frequency controlling earthquakes shown in SER Table 2.5.2-3. The low-frequency controlling earthquake corresponds to an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles), while the high-frequency controlling earthquake is represented by an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles).

NUREG/CR-6728 presents V/H ratios for soft rock WUS sites and hard rock CEUS sites. The WUS rock V/H ratios provided in NUREG/CR-6728 are based on an empirical database of WUS strong-motion records. Due to the limited number of available CEUS ground motion recordings, NUREG/CR-6728 uses the WUS ratios and modifies them based on the results of modeling studies to obtain CEUS rock ratios. In addition, Appendix J to NUREG/CR-6728 provides a formula to develop V/H ratios for CEUS soil sites:

$$V/H_{CEUS,Soil} = V/H_{WUS,Soil,Empirical} * \left[V/H_{CEUS,Soil,Model} / V/H_{WUS,Soil,Model} \right] \qquad \text{Equation 2}$$

Because the ESP site is a soil site, the applicant used Equation (2) to determine V/H ratios. The applicant obtained the first term of Equation (2), $V/H_{WUS,Soil,Empirical}$, from the ground motion model of Abrahamson and Silva (1997) which provides horizontal and vertical ground motion relationships for deep soil sites. In NUREG/CR-6728, generic soil columns were used to determine $V/H_{WUS,Soil,Model}$ and $V/H_{CEUS,Soil,Model}$ ratios, which provided results for **M** 6.5 and distances of 1, 5, 10, 20, and 40 kilometers. The applicant obtained the second term of Equation (2) using $V/H_{CEUS,Soil,Model}$ and $V/H_{WUS,Soil,Model}$ ratios corresponding to **M** 6.5 and 20 kilometers to represent the high-frequency (**M** 5.6, 12 km) controlling earthquake. In addition, the applicant used the $V/H_{CEUS,Soil,Model}$ and $V/H_{WUS,Soil,Model}$ ratios corresponding to **M** 6.5 and 40 kilometers to represent the low-frequency (**M** 7.2, 130 km) controlling earthquake. The applicant considered these magnitude and distance substitutions to be conservative because V/H ratios are observed to decrease with distance for a given magnitude. The applicant assigned a weight of approximately 1:3 to the results representing the high- and low-frequency controlling earthquakes, respectively.

Lee (2001) used the methodology outlined in NUREG/CR-6728 to develop V/H ratios for the MOX Fuel Fabrication Facility at the SRS. However, Lee (2001) developed $V/H_{CEUS,Soil,Model}$ ratios using a site-specific soil model for the SRS, rather than the generic CEUS profile used in Appendix J to NUREG/CR-6728. To obtain V/H ratios corresponding to the high-frequency controlling earthquake (**M** 5.6, 12 km), the applicant interpolated the results provided in Lee (2001) between **M** 5.5 at 10 kilometers and 20 kilometers and **M** 6.0 at 10 kilometers and 20 kilometers. Similarly, to obtain V/H ratios corresponding to the **M** 7.2, 130-km earthquake, the applicant interpolated the results provided in Lee (2001) between **M** 7.0 at 100 kilometers and **M** 7.2 at 100 kilometers. The distance of 100 kilometers was the largest distance considered in Lee (2001). However, the applicant considered the distance substitution of 100 kilometers for 130 kilometers to be conservative because V/H ratios are observed to decrease with distance for a given magnitude. The applicant assigned a weight of approximately 1:3 to the results representing the high- and low-frequency controlling earthquakes, respectively.

SER Figure 2.5.2-9 plots the resulting V/H ratios obtained from NUREG/CR-6728 and Lee (2001), as well as the final V/H ratios. The V/H ratios from Lee (2001) are higher than those derived from the NUREG/CR-6728 results for frequencies greater than about 0.7 Hz. To develop the final V/H ratios, the applicant used an approximate envelope of the two results. The applicant assigned a greater weight to the V/H ratios from Lee (2001) because this study used a site-specific soil model for the nearby SRS. SER Figure 2.5.2-7 also plots V/H ratios from RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, issued December 1973. The final V/H ratios are slightly less than those provided in RG 1.60 at all frequencies.

To obtain the vertical GMRS, the applicant scaled the horizontal smoothed GMRS, shown in SER Figure 2.5.2-8, by the final V/H ratio (shown in SER Figure 2.5.2-9).

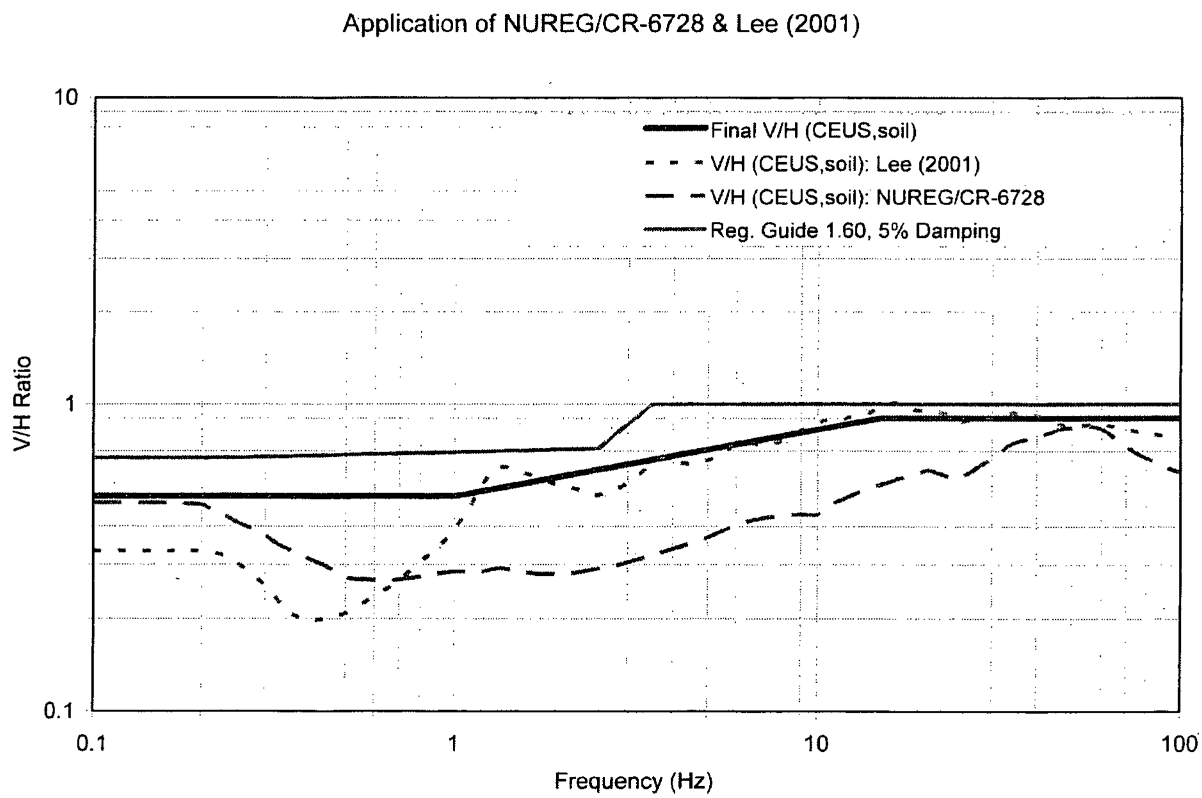


Figure 2.5.2-9 - Final V/HCEUS,Soil ratios (reproduced from SSAR Figure 2.5.2-43)

2.5.2.2.7 Operating Basis Ground Motion

The applicant did not determine the Operating Basis Earthquake (OBE) as part of the Vogtle ESP and stated that the OBE will be determined during the COL stage.

2.5.2.2.8 Sensitivity Studies

As part of its COL site investigation, the applicant collected additional S-wave velocity data and developed site-specific strain-dependent shear modulus and damping relationships based on RCTS test results. However, the applicant did not use any of this additional COL data as input to its site response calculations. Instead, the applicant relied on the SRS and generic EPRI strain-dependent shear modulus and damping curves and S-wave velocity profiles developed as part of the ESP. Rather than recalculating site amplification factors that also account for additional COL data, the applicant performed site response sensitivity calculations for a select number of cases in order to demonstrate that use of the ESP S-wave velocity data and SRS and generic EPRI strain-dependent shear modulus and damping curves is appropriate.

The applicant conducted three sets of sensitivity calculations in order to evaluate: (1) the sensitivity of the AP1000 nuclear island responses to changes in the backfill S-wave velocity; (2) the effects of the backfill geometry on the site response and on the SSI response of the nuclear island; and (3) the effects of additional COL data on site response.

In the first set of calculations, the applicant evaluated the effects of changes in the backfill S-wave velocity. A comparison of the ESP S-wave velocity profile (used for the GMRS and FIRS [foundation input response spectra] computation in SSAR Section 2.5.2.5.1.5) with the S-wave velocity profile used in the sensitivity study is provided in SSAR Figure 2.5.2-51. The staff notes that the S-wave velocity profile used in the sensitivity study did not correspond to the COL backfill data because the applicant performed the sensitivity study before conducting the Phase I test pad program. The S-wave velocity values of the sensitivity study median S-wave velocity profile are larger than both the ESP and COL profiles, which are provided in SSAR Tables 2.5.4-10 and 2.5.4-10a, respectively. The applicant's analysis involved the randomization of the entire soil column with new backfill properties and development of the new outcrop motion at the foundation level of the AP1000 nuclear island. The applicant then used the new time-history and associated strain-compatible soil properties in the SSI analysis of the AP1000. The results of this sensitivity study are provided in Appendix 2.5E (Vogtle Site Specific Seismic Evaluation Report) to the SSAR. The applicant concluded that, even with significant variation of the backfill S-wave velocity, the AP1000 design is applicable to the Vogtle site with a large margin.

In the second sensitivity study, the applicant evaluated the effects of the backfill geometry. Due to the large volume of excavation and the large lateral extent of the backfill at the Vogtle site, the applicant modeled the backfill layers as free-field soil layers for both the soil amplification for development of the ground motion (GMRS and FIRS) and the site-specific seismic SSI analysis of the AP1000. To confirm this assumption, the applicant performed a two-dimensional site response analysis (Part I) followed by a two-dimensional SSI analysis (Part II) of the AP1000 model in order to evaluate the extent of backfill on the site response and on the SSI response of the Nuclear Island. For the 2D analysis, the applicant used the cross section shown in the East-West direction provided in SSAR Figure 2.5.2-53. In Part I of the analysis, the applicant performed a 2D site response analysis. The applicant's 2D model for the site response analysis is provided in SSAR Figure 2.5.2-54, which is based on the cross section shown in SSAR Figure 2.5.2-53. The applicant used the same properties for backfill, Blue Bluff Marl, the lower sand layers and layers extending to the rock at the base as those that it used to develop the GMRS and FIRS. The computation of the GMRS and FIRS (described in SSAR Section 2.5.2.5), however, involved 60 randomized soil profiles, 30 high-frequency and 30 low-frequency input time histories. Thus, for its 2D analysis, the applicant only considered a subset of the soil profiles (i.e. the upper, mean, and lower bound soil profiles) and input time histories (i.e. three high-frequency and three low-frequency input time histories). The applicant compared the resulting site amplification factors with those calculated from the 1D SHAKE results for the same set of input motions and soil properties, which are shown in SSAR Figures 2.5.2-55, 2.5.2-56, and 2.5.2-57 for locations (presented as "in-column" motions) at depths of 0 ft (GMRS), 40 ft (FIRS horizon), and at 86 ft depth (Top of Blue Bluff Marl), respectively, at the centerline of the backfill (shown in SSAR Figure 2.5.2-54). The applicant concluded that the differences are very small. The applicant further concluded that the geometry of the backfill has an insignificant effect on GMRS and FIRS. In addition, the applicant compared transfer functions for the 1D SHAKE and 2D SASSI analyses, which determine how the soil profile amplifies or deamplifies each frequency in the input motion (Kramer, 1996). In SSAR Figure 2.5.2-55a, the applicant compared the transfer function that relates the motion at a depth of 0 ft to the bedrock input motion, while the transfer function that relates the motion at a depth of 40 ft to the bedrock input motion is shown in SSAR Figure 2.5.2-56a. In both cases the applicant used one high-frequency input motion and the mean soil profile. The applicant stated that this additional comparison also confirmed that the use of a 1D SHAKE analysis is adequate given the geometry of the backfill at the site.

In Part II, the applicant developed a Vogtle 2D SASSI model of the nuclear island (NI) to include the backfill as part of the structural model shown in Figure 2.5.2-58. This model is similar to the model in Part I except that the applicant included the AP1000 NI model using only the mean soil profile and a single time history from the analysis performed in Part I (i.e. the input motions for the two SSI analyses are obtained from the respective 1D SHAKE analysis in Part I). The applicant compared the SSI responses for the 2D SASSI NI model (referred to as Bathtub Model-d5) at key locations in the NI are compared with the SSI results of the 2D SASSI (referred to as 2D-AP-d5) that assumes backfill extends to infinity in lateral directions. These comparisons are shown in SSAR Figure 2.5.2-59 through 2.5.2-64. The applicant concluded that the response spectra are similar and it considered the differences to be negligible. The applicant also plotted the generic AP1000 standard design response spectra for comparison for the purpose of demonstrating that a significant margin exists between the AP1000 generic response and the Vogtle 2D results. The applicant stated that a detailed discussion of the 2D SASSI NI model and a comparison of transfer functions are documented in more detail in Appendix A of Appendix 2.5E.

Finally, the applicant performed sensitivity studies to evaluate the effects of the additional COL S-wave velocity and the strain dependent shear modulus and damping relationships based on RCTS test results. As input, the applicant selected three high-frequency and three low-frequency rock time histories representing the 10^{-4} annual exceedance frequency level from the suite of motions used for the GMRS computation in SSAR Section 2.5.2.5. The applicant also used three soil profiles representing the best estimate COL velocity profile (shown in SSAR Figure 2.5.4-7a) as well as the upper and lower bounds. In addition, the applicant used the associated COL strain-dependent soil properties presented in SSAR Figures 2.5.4-9a and 2.5.4-11a and in SSAR Table 2.5.4-12a. The applicant performed two sets of analyses in order to consider the high and low PI (Plasticity Index) cases of the Blue Bluff Marl as illustrated in SSAR Figures 2.5.4-9a and 2.5.4-11a. The applicant then averaged the results using the three high-frequency input time histories, three soil profiles, and the high and low PI cases of the Blue Bluff Marl, then divided this average response spectrum (corresponding to a depth of 40 ft) by the 10^{-4} high-frequency input response spectrum to obtain site amplification factors. The applicant repeated this process for the low-frequency input time histories. The applicant then enveloped the resulting high-frequency and low-frequency amplification factors, which is represented by the green dashed curve in SSAR Figure 2.5.2-65c. The blue solid curve in SSAR Figure 2.5.2-65c corresponds to the amplification factors based on a limited number of ESP soil profiles. From the ESP set of runs described in SSAR Section 2.5.2.5.1, the applicant used the strain compatible velocity and damping profiles to obtain the median and upper bound profiles (using one standard deviation as the variation) to use as input to the analysis. The applicant used the same three high-frequency and three low-frequency time histories used for the analysis of the COL data above. In SSAR Figure 2.5.2-65c, the applicant also plotted (depicted by the red dashed curve) the amplification factors resulting from the fully randomized ESP soil profiles and entire group of input time histories (described in SSAR Section 2.5.2.5). The applicant concluded that the comparison of the two sets of results based on the ESP data shows good agreement and thus that the limited number of profiles and time histories are adequate for the purpose of the evaluation of the inpact of the COL data. Furthermore, the applicant concluded that the difference in amplification between the ESP and COL data is small.

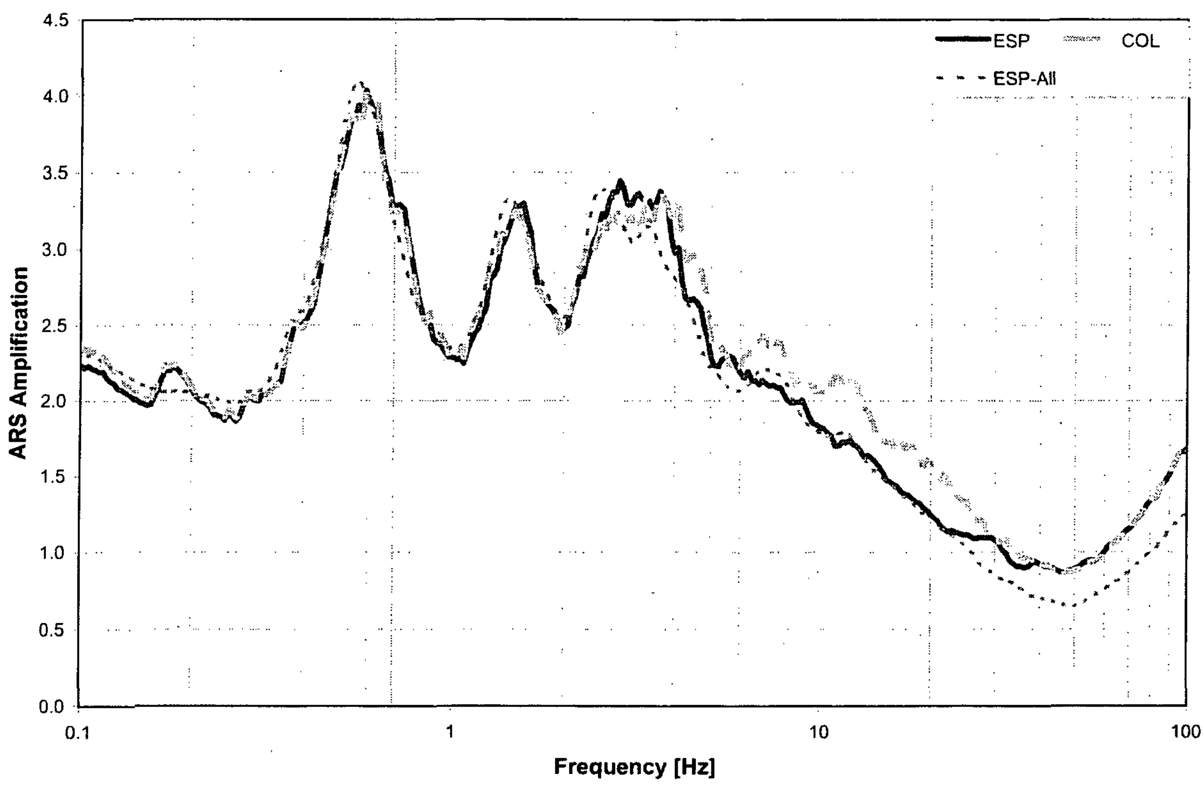


Figure 2.5.2-10 – Comparison of Amplification Factors from Sensitivity Analyses (reproduced from SSAR Figure 2.5.2-65c)

2.5.2.3 Regulatory Basis

SSAR Section 2.5.2 presents the applicant's determination of ground motion at the ESP site from possible earthquakes that might occur in the site region and beyond. In SSAR Section 1.8, the applicant stated that it had developed the geological and seismological information used to determine the seismic hazard in accordance with regulations listed in SSAR Table 1-2, which includes 10 CFR 50.34; Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50; and 10 CFR 100.23. The applicant further stated in SSAR Table 1-2 that it developed this information in accordance with the guidance presented in Section 2.5.2 of Revision 3 of NUREG-0800 and RG 1.165. The staff reviewed this portion of the application for conformance with the regulatory requirements and guidance applicable to the determination of the SSE ground motion for the ESP site, as identified below. The staff notes that the application of Appendix S to 10 CFR Part 50 in an ESP review, as referenced in 10 CFR 100.23(d)(1), is limited to defining the minimum SSE for design.

In its application review, the staff considered the regulatory requirements of 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23(c) and (d), which require that the applicant for an ESP describe the seismic and geologic characteristics of the proposed site. In particular, 10 CFR 100.23(c) requires that an ESP applicant investigate the geological, seismological, and engineering characteristics of the proposed site and its environs with sufficient scope and detail to support estimates of the SSE ground motion and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. In addition, 10 CFR 100.23(d) states that the SSE ground motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface. Section 2.5.2 of Revision 3 of NUREG-0800 and RG 1.208 provide guidance concerning the evaluation of the proposed SSE ground motion, and RGs 1.165 and 1.208 provide guidance regarding the use of PSHA to address the uncertainties inherent in the estimation of ground motion at the ESP site.

2.5.2.4 Technical Evaluation

This section of the SER provides the staff's evaluation of the seismological, geological, and geotechnical investigations that the applicant conducted to determine the GMRS for the ESP site. The technical information presented in SSAR Section 2.5.2 resulted from the applicant's surface and subsurface geological, seismological, and geotechnical investigations performed in progressively greater detail as distance to the ESP site decreases. The GMRS is based upon a detailed evaluation of earthquake potential, taking into account regional and local geology, Quaternary (1.8 mya–present) tectonics, seismicity, and specific geotechnical characteristics of the site's subsurface materials.

SSAR Section 2.5.2 characterizes the ground motions at the ESP site from possible earthquakes that might occur in the site region and beyond to determine the site GMRS. According to RG 1.208, applicants may develop the GMRS for a new nuclear power plant using either the EPRI or LLNL PSHAs for the CEUS. However, RG 1.208 recommends that applicants perform geological, seismological, and geophysical investigations and evaluate any relevant research to determine whether revisions to the EPRI or LLNL PSHA databases are necessary. As a result, the staff focused its review on geologic and seismic data published since the late 1980s that could indicate a need for changes to the EPRI or LLNL PSHAs.

2.5.2.4.1 Seismicity

SSAR Section 2.5.2.1 describes the development of a current earthquake catalog for the ESP site. The applicant started with the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is complete though 1984. To update the earthquake catalog, the applicant used information from the ANSS and SEUSS.

The staff focused its review of SSAR Section 2.5.2.1 on the adequacy of the applicant's description of the historical record of earthquakes in the site region. In Request for Additional Information (RAI) 2.5.2-1, the staff asked the applicant to provide electronic versions of the EPRI seismicity catalog (EPRI NP-4726-A 1988) for the region of interest (30 degrees to 37 degrees N, 78 degrees to 86 degrees W), as well as its updated EPRI seismicity catalog. The staff used the catalog data that the applicant provided in response to RAI 2.5.2-1 to compare with its own compilation of recent earthquakes for the site region. The applicant's updated catalog consisted of a total of 61 events. Of these 61 events, there were 56 m_b 3 events and 5 mb 4 events. In comparison, the staff's list of earthquakes, based entirely on the ANSS earthquake catalog, consisted of 50 m_b 3 events and 3 m_b 4 events.

Because the applicant used the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is part of the 1989 EPRI seismic hazard study that the NRC endorsed in RG 1.165, the staff concludes that the seismicity catalog used by the applicant is complete and accurate for the time period 1777–1985. The staff compared the applicant's update of the regional seismicity catalog with its own listing of recent earthquakes and, as a result, concludes that the earthquake catalog used by the applicant is complete and provides a conservative estimate of earthquake magnitudes and locations for the ESP site region.

To determine whether the seismicity rates used in the EPRI study (EPRI NP-6395-D 1989) are appropriate for the assessment of the seismic hazard at the ESP site, the applicant used two areas in the site region: (1) a small rectangular area around the Charleston seismicity; and (2) a triangular-shaped area that envelops the seismicity in South Carolina and a strip of Georgia. The applicant concluded that, for the rectangular Charleston source, the updated catalog indicates that the seismicity rates are the same. For the triangular South Carolina source, the updated catalog indicated that seismicity rates decreased when the seismicity from 1985 to April 2005 was added. In RAI 2.5.2-18, the staff asked the applicant to provide a justification for the selection of the geometries used to represent the Charleston source and the South Carolina source. In response to RAI 2.5.2-18, the applicant assessed the seismicity in two additional areas within the site region. The applicant concluded that any region in South Carolina that would affect the seismic hazard at the ESP site would have estimated activity rates that stay constant or decrease, if the new regional earthquake catalog were added to the analysis.

Based on the applicant's evaluation of multiple areas and its determination that seismicity rates in the region have not increased since 1985 for any of these selected areas, the staff concludes that the applicant's use of the EPRI seismicity rates is appropriate and that these rates are appropriate for the assessment of the seismic hazard at the ESP site.

2.5.2.4.2 Geologic and Tectonic Characteristics of the Site and Region

SSAR Section 2.5.2.2 describes the seismic sources and seismicity parameters used by the applicant to calculate the seismic ground motion hazard for the ESP site. Specifically, the applicant described the seismic source interpretations from the 1986 EPRI Project (EPRI

NP-4726), relevant post-EPRI seismic source characterization studies, and its updated EPRI seismic source zone for the Charleston area. The staff focused its review of SSAR Section 2.5.2.2 on the applicant's update of the Charleston seismic source zone. The staff also reviewed the applicant's basis for not updating the other EPRI source zones that contribute to the seismic hazard at the ESP site.

Summary of EPRI Seismic Sources

Section 2.5.2.2.1 summarizes the seismic sources and seismicity parameters used in the 1986 EPRI Project and subsequently implemented in the 1989 PSHA (EPRI NP-D 1989). The 1989 EPRI PSHA study expressed M_{max} values in terms of m_b. The applicant noted that most modern seismic hazard analyses describe M_{max} in terms of **M** and used the arithmetic average of the conversion relations presented in Atkinson and Boore (1995), Frankel et al. (1996), and EPRI TR-102293 (1993) to convert from mb to **M**. In RAI 2.5.2-5, the staff asked the applicant to provide its converted **M** values. In response to RAI 2.5.2-5, the applicant provided a table that listed a range of m_b values and the corresponding converted **M** values.

To confirm the applicant's magnitude conversions, the staff compared the applicant's converted **M** values with the **M** values it obtained using the conversion relations of Frankel et al. (1996) and Johnston (1994), which were provided in Chapman and Talwani (2002). The staff found that the conversion provided in Chapman and Talwani (2002) yields slightly larger **M** values in the m_b 4.0 to 7.5 range. However, based on the uncertainties associated with magnitude conversions and the applicant's use of the average of three different conversion relations to account for this uncertainty, the staff concludes that the applicant's converted **M** values are adequate.

SSAR Sections 2.5.2.2.1.1 through 2.5.2.2.1.6 provide a summary of the primary seismic sources developed in the 1980s by each of the six EPRI ESTs. Each EST described its set of seismic source zones for the CEUS in terms of source geometry, probability of activity, recurrence, and M_{max}. Each EPRI EST identified one or more seismic source zones that include the ESP site. Although some of the ERPI ESTs assigned M_{max} values as high as **M** 7.5 for the source zones that make up the Atlantic coastal region, the M_{max} values for the seismic source zones that include the site have a weighted mean of about **M** 6.0. In RAI 2.5.2-6, the staff asked the applicant to explain whether it considered more recent studies on large worldwide earthquakes by Johnston (1994) and Kanter (1994) as possible updates of the earlier EPRI seismic source models.

In response to RAI 2.5.2-6, the applicant stated that the final versions of the Johnston (1994) and Kanter (1994) assessments (included in Volume 1 of the Johnston et al. 1994 study) do not constitute new information that would require an update of the M_{max} values used for the EPRI seismic source models. In its response, the applicant stated that the initial results of the Johnston et al. (1994) study were available to the EPRI ESTs, and that the final results of the Johnston et al. (1994) study generally support the initial findings of the study.

The staff reviewed the applicant's response to RAI 2.5.2-6 and concluded that, although many of the EPRI ESTs assigned M_{max} values that reflect the studies of Johnston and Kanter, the applicant did not provide an adequate justification to support the low weights for some of the larger M_{max} values. In particular, the Dames and Moore EST gave fairly low weights to some of its seismic source zones. For example, the two M_{max} values assigned by the Dames and Moore EST for the "Southern Appalachian Mobile Belt" are mb 5.6 with a weight of 0.8 and 7.2 with a weight of 0.2. These two M_{max} values and weights are similar to those for the other ESTs for the

Atlantic coastal margin; however, the Dames and Moore EST also assigned a probability of activity of only 0.26 for this source. Similarly, for its "Southern Cratonic Margin," the Dames and Moore EST assigned a probability of activity of only 0.12. The combined effect of these low probabilities of activity and low weights for the larger magnitudes results in a lower hazard for the ESP site. This result is shown in SER Figures 2.5.2-17 and 2.5.2-18, which are plots of the 1- and 10-Hz PSHA hazard curves for each of the EPRI ESTs. As shown in these two figures, the Dames and Moore seismic hazard curves are substantially lower than those for the other ESTs.

In response to RAI 2.5.2-6, the applicant also stated that the North Anna site is located within Kanter's (1994) Piedmont domain 223 in nonextended crust and, as a result, large magnitude earthquakes are not expected in this domain. The staff, however, notes that the Vogtle ESP site is located within the Mesozoic passive margin. Specifically, the site is on the hanging wall of the southeast-dipping Pen Branch fault (SSAR Figures 2.5.1-2, 2.5.1-29, and 2.5.1-34), which is the main border fault of the Dunbarton Triassic basin (SSAR Figures 2.5.1-2 and 2.5.1-10). In turn, the Dunbarton Triassic basin is a subbasin within the much larger South Georgia basin complex (SSAR Figures 2.5.1-2 and 2.5.1-7). Therefore, the site is in Kanter's Eastern Seaboard domain 218. The rocks beneath the site are Triassic strata of domain 218's rift basins (SSAR Figures 2.5.1-34 and 2.5.1-38). Beneath the Triassic rocks is the Piedmont domain, but the Piedmont rocks have been cut by the Mesozoic extensional faults that bound the rifts. The distinction between the Eastern Seaboard and Piedmont domains depends on the presence or absence of Mesozoic extensional faults, rather than the age of the rocks cut by those faults. Accordingly, the staff believes that the site is subject to the higher M_{max} of the Eastern Seaboard domain of Kanter (1994). The site is in one of the regions that Johnston et al. (1994) found to have hosted all earthquakes of **M** 7.0 and larger in the world's stable continental regions (SCRs).

SER Figure 2.5.2-11 shows a histogram of magnitudes of the 30 earthquakes that had **M** 6.5 and larger in the world's extended margin, which is based on the compilation of the largest earthquakes in the world's SCRs by Johnston et al. (1994). The histogram has a large peak at **M** 6.6 and 6.7. The earthquakes making up the peak come from various SCRs, continents, and plate tectonic settings, indicating that values of 6.6 and 6.7 occur widely in diverse geologic and tectonic settings. This implies that M_{max} is unlikely to be less than these values anywhere in the extended margin of North America. As such, the low weights and low probability of activities assigned by the Dames and Moore EST to larger M_{max} values do not reflect worldwide earthquake activity in extended margins.

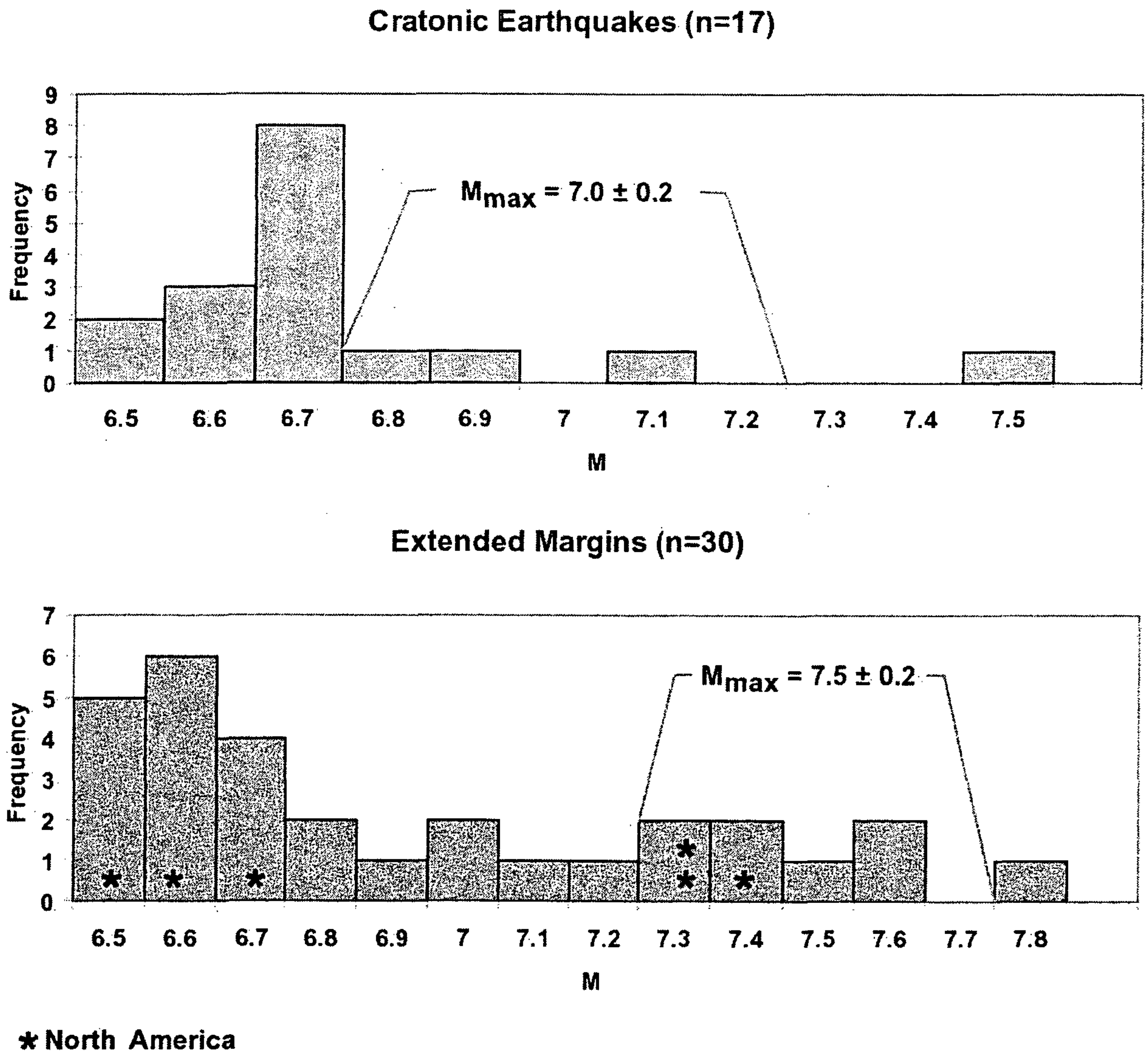


Figure 2.5.2-11 - Histogram showing magnitudes of the 30 earthquakes that had M 6.5 and larger in the world's extended margins (Source: USGS)

In summary, the staff concluded that the applicant did not provide an adequate justification to support the low weights for the larger M_{max} values for the EPRI source zones that include the site. In particular, the staff was concerned that the low weights and low probability of activities assigned by the Dames and Moore EST to some of its seismic source zones result in hazard curves for the ESP site that may not adequately characterize the regional seismic hazard. In addition, the staff concluded that the site is located within the Mesozoic passive margin, rather than the Piedmont unextended province as stated in the applicant's response. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-1.

As noted above, Open Item 2.5-1 related to the staff's concern that the low weights and low probability of activities assigned by the Dames and Moore EST to some of its seismic source zones resulted in hazard curves for the ESP site that may not adequately characterize regional seismic hazard. In response to Open Item 2.5-1, the applicant stated the following:

As pointed out in the DSER, the Dames & Moore team assigned low probabilities of activity (PA) to some of its sources, such as source zones 41 and 53. Zone 53 (Southern Appalachian Mobile Belt) is a default zone for several Triassic rift basin sources, represents a host zone for the Vogtle site, and has a PA = 0.26. The lack of a background zone beneath the region covered by source 53 results in a source-less area when 53 is "turned off." While the implementation of this aspect of the Dames & Moore source model has been the subject of debate, this is not an "error" or misinterpretation in their model. Statements in both the Dames & Moore EPRI report (1986) as well as recent discussions with James McWhorter, an original member of the Dames & Moore EST, indicate that Dames & Moore intended to represent the earthquake process in this fashion.

The applicant provided the following discussion from page 5-3 of the Dames and Moore report (1986, Volume 6), which indicates that Dames and Moore believes earthquake occurrence can be explained by tectonic reasons and that they do not use background zones as in other traditional seismic hazard assessments:

"In our model, uniform seismicity is a consequence of a reasonable tectonic explanation for earthquake occurrence in the zone. To avoid muddling the tectonic aspect, our team does not use backgrounds. There is either a tectonic reason for a block of the earth's crust to be seismically active or there is not. So what we formerly called a "global background" no longer exists; the sources replacing it have a PA reflecting our confidence in a tectonic reason for earthquake activity there."

The applicant stated that although the Dames and Moore seismic source zone implementation is different from the other ESTs, it still represents the range of expert opinion in the EPRI SSHAC Level 4 study. The applicant further stated that "from a process standpoint, it is not the responsibility of the applicant to defend the original rationale or implementation of the EPRI study, which has been approved by the NRC in Regulatory Guide 1.165 and forms the basis for evaluating sites across the CEUS. The individual teams were given latitude as to how to model seismic hazard in order to capture the full range of opinion for the poorly understood earthquake process in the CEUS. Without new data to invalidate the model, an individual team or model should not be reinterpreted or disregarded simply because their resultant hazard is less than the other EST source models."

In addition, the applicant subsequently provided supplemental information regarding Open Item 2.5-1 in a letter dated December 11, 2007. This letter addressed additional concerns that the staff had about the Dames and Moore model regarding a quotation in the 1992 DOE Standard "Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites for Department of Energy Facilities" (DOE-STD-1024-92). The purpose of the DOE Standard was to provide guidance in the use of the seismic hazard curves developed by the LLNL and the EPRI. The Standard based its recommendations on the evaluations of the LLNL and EPRI seismic hazard method performed by LLNL, Jack Benjamin and Associates, and Risk Engineering Inc. The following quotation is from one of the issues identified by Risk Engineering, Inc.:

"Risk Engineering, Inc. has also found that the EPRI team of Dames and Moore does not fully account for historic seismicity near the Savannah River Site (SRS). One reason for this is the fact that the SRS host source zone was given a low probability of activity. Risk Engineering, Inc. recommended that the Dames and Moore seismic source input not be used to calculate the seismic hazard at SRS."

The applicant's December 11, 2007 supplemental information contained a letter enclosure from Dr. Robin K. McGuire of Risk Engineering, Inc., which provided additional background regarding the above quotation. In his letter, Dr. McGuire stated that "the quote from my 1991 report was taken from a study that had the purpose of reconciling hazard curves from the EPRI and LLNL reports. In my role as a seismic-hazard analyst in that project (rather than an expert in seismic source characterization), I achieved the project goal by giving credibility only to those interpretations that were consistent with historical seismicity at all magnitude levels. Interpretations that were high or low relative to historical seismicity were given zero weight. The remaining interpretations gave hazard that was relatively consistent (as one would expect), which achieved the purpose of the study. Thus the down-weighting of the Dames & Moore source model was not made on the basis of its geologic or technical merits."

With respect to the quotation in the DOE report, Dr. McGuire stated the following:

> "Examining historical earthquakes from the EPRI catalog in Dames & Moore source 53, one event occurred in 1966 with m_b=4.7, and all other historical earthquakes had m_b≤4.3. A search of the PDE and ISC catalogs indicates that the 1966 event was an offshore explosion, and if so the largest historical earthquake in source 53 was m_b~4.3. In any case the quotation in the 1st paragraph is accurate relative to historical earthquakes with m_b≤4.7, but the Dames & Moore interpretation is not inconsistent with the occurrence of earthquakes with m_b>5. Stated another way, no earthquakes with m_b>5 have occurred historically in the Dames & Moore source 53, and Dames & Moore said there is a 26 percent chance that earthquakes with m_b>5 will occur there in the future."

In its supplemental response, the applicant also provided a letter from Dr. Robert Kennedy, which demonstrated that the Dames and Moore model contribution is not significant at the Vogtle ESP site. Dr. Kennedy looked at the 10 Hz total mean hazard curve together with the contributing mean hazard curves from the updated Charleston source and each of the six ESTs source models. He noted that at any spectral acceleration, the total mean annual frequency of exceedance, H, is given by combining the Charleston source mean annual frequency of exceedance with the mean of the 6 ESTs mean annual frequency of exceedance:

$$H = H_C + (H_R + H_{WC} + H_{We} + H_L + H_B + H_{DM})/6 \qquad \text{Equation (3)}$$

Where HC is the mean annual frequency of exceedance from the updated Charleston source, and HR, HWC, HWe, HL, HB, HDM, are the mean annual frequencies of exceedance from the Rondout, Woodward-Clyde, Weston, Law, Bechtel, and Dames and Moore teams, respectively. At a spectral acceleration of 0.42 g, Dr. Kennedy found that deleting the Dames and Moore input (HDM) increased the total mean annual frequency of exceedance by only approximately 5 percent. He further concluded that similar results exist at a spectral acceleration corresponding to a mean annual frequency of exceedance of 10^{-5}.

In reviewing the response to Open Item 2.5-1 and supplemental information provided by the applicant, the staff concluded that the applicant did not provide adequate justification for the low probabilities of activity that Dames and Moore team assigned to several of its source zones.

The staff is concerned because the Dames and Moore model states that there is only a 26 percent and 12 percent chance that earthquakes larger than mb 5.0 can occur in source zones 53 and 42, respectively. The Dames and Moore team's interpretation differs significantly from the other ESTs interpretations as well as other recent seismic hazard studies including USGS, SCDOT, and TIP studies. The staff, however, agrees with the applicant's determination that the Dames and Moore team does not contribute significantly to the hazard at the Vogtle site. The staff performed a similar comparison to the one performed by Dr. Kennedy, but instead compared percentage changes in spectral acceleration rather than annual exceedance frequency. The results showed that the percentage increase in the 10 Hz total mean hazard spectral acceleration at the 10^{-4} annual exceedance frequency is 2.07 percent if the Dames and Moore team's contribution is removed. At the 10^{-5} annual exceedance frequency, the percentage increase in spectral acceleration is 3.44 percent. The staff concludes that the percentage increase is even less for the 1 Hz hazard curve. The percentage increase in spectral acceleration at the 10^{-4} annual exceedance frequency is 0.39 percent when the Dames and Moore team's contribution is removed. At the 10^{-5} annual exceedance frequency, the percentage increase in spectral acceleration is 0.38 percent. Thus, in spite of the staff's concerns that the Dames and Moore team did not adequately characterize the regional seismic hazard, the staff considers open Item 2.5-1 to be resolved because the Dames and Moore team's contribution to the total mean hazard at the Vogtle ESP site is not significant.

Post-EPRI Seismic Source Characterization Studies

SSAR Section 2.5.2.2.2 describes three PSHA studies that were completed after the 1989 EPRI PSHA and which involved the characterization of seismic sources within the ESP site region. These three studies include the USGS National Seismic Hazard Mapping Project (Frankel et al. 1996, 2002), the SCDOT seismic hazard mapping project (Chapman and Talwani 2002), and the NRC TIP study (NUREG/CR-6607, "Guidance for Performing Probabilistic Seismic Hazard Analysis for a Nuclear Plant Site: Example Application to the Southeastern United States"). The applicant provided a description of both the USGS and SCDOT [South Carolina Department of Transportation] models, as well as the impact of these more recent studies on the EPRI PSHA models. The applicant did not, however, consider the TIP study to be a relevant source of information. The TIP study implemented the PSHA guidelines developed by the SSHAC (NUREG/CR-6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts") and focused on the development of seismic zonation and earthquake recurrence models for the Watts Bar (Tennessee), and Vogtle sites. The applicant stated that it did not explicitly incorporate the results of the TIP study into the SSAR because "the study was more of a test of the methodology rather than a real estimate of the seismic hazard." Because part of the TIP study focused on the Vogtle site, the staff, in RAI 2.5.2-7, asked the applicant to explain why it concluded that the TIP study was more of a test of the methodology rather than a real estimate of the seismic hazard and why it did not use the TIP study results. In response, the applicant stated the following:

> The TIP study focuses primarily on implementing the Senior Seismic Hazard Advisory Committee (SSHAC) PSHA methodology (SSHAC 1997), however, and was designed to be as much of a test of the methodology as a calculation of seismic hazard. For example, as part of the test of the methodology, Committee members were asked to present opposing arguments, regardless of whether they agreed with the position they were asked to present. As a disclaimer, Kevin Coppersmith prefaced his discussion of the Pen Branch fault with the following statement:

> The following white paper—much like a lawyers (sic) legal argument—presents a particular position and seeks only to support that position. I have intentionally tried to present an unbalanced case, giving only lip service to counter arguments...Further, I have done a poor job of citing references and providing supporting data to many of my arguments (p. A-51).
>
> The TIP study provides useful discussions, including speculations regarding the Charleston seismic source, seismic hazards of the South Carolina–Georgia region, and Eastern Tennessee. However, the TIP study focuses primarily on methodology. The process-oriented focus of the TIP study is also illustrated in the report presentation, which is very thorough on methodology, but significantly lacking in presenting a summary of seismic source model parameters. For these reasons, the TIP study results are not explicitly incorporated into the VEGP ESP application.

The staff reviewed the applicant's response to RAI 2.5.2-7, as well as the TIP report, and disagrees with the applicant's conclusion that the TIP report was more of a test of the methodology rather than a real estimate of the seismic hazard.

The disclaimer provided in the applicant's response to RAI 2.5.2-7 accompanied a white paper titled, "Include the Pen Branch and Other Local Faults in the PSHA," written by Kevin Coppersmith after the first TIP workshop, which involved a panel of five expert evaluators, the technical facilitator/integrator (TFI) team, and expert proponents and presenters. The workshop comprised a series of technical sessions, which included presentations of recent research and interpretations by the presenters. Each of the technical sessions was followed by a discussion moderated by the TFI team in which key outstanding technical issues were defined. These key issues were then assigned to evaluators as the topics of "white papers" to be written after the workshop. For example, Kevin Coppersmith was assigned to write the white paper in support of "Discrete local fault sources for Vogtle," while Pradeep Talwani was assigned to present a case against "Discrete local fault sources for Vogtle." The TIP report states that "the objective of these papers is to clarify the arguments for and against key interpretations having direct bearing on seismic source characterization in a way that will stimulate interaction among the evaluators." The TIP report also states that "the experts were asked to act as proponents of a certain scientific position and since the issues selected involved dichotomous positions they had to argue for a position that they do not necessarily defend. This has an advantage of forcing the experts, and all the participants, into discovering the positive aspects of scientific concepts other than their own." Thus, Kevin Coppersmith's disclaimer that accompanied his white paper merely reflects his assigned role to provide supporting arguments for a key workshop issue.

The staff concludes that, while the primary objective of the TIP study was to implement the SSHAC PSHA methodology, there is nothing to suggest that the project's final hazard results are not valid. In fact, the seismic hazard results from the TIP triggered a followup NRC-sponsored study, documented in Appendix G to NUREG/CR-6607, which involved a comparison of the TIP hazard results with NUREG-1488, "Revised Livermore Seismic Hazard Estimates for 69 Sites East of the Rocky Mountains." Therefore, although portions of the TIP report may have been focused on implementing the SSHAC methodology, much of the data and results contained in the report are applicable to the ESP site. Thus, in the SER with open items, the staff did not concur with the applicant's disposition of the TIP study. The staff requested that the applicant provide an evaluation of any information contained in the TIP study that is relevant to the seismic source characterization of the ESP site. The staff considered this information necessary in order to determine whether the applicant provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-2.

In response to Open Item 2.5-2, the applicant reiterated its position that the Trial Implementation Project (TIP) study was primarily an exercise in implementation of the SSHAC process. The applicant also stated the following:

> The fact that all final seismic source model parameters and weights are not presented in the TIP report also support that this study focused primarily on implementation of the SSHAC process as opposed to the development and publication of a new source model for the southeastern US. The absence of a complete set of parameters and weights in the TIP study also makes it difficult to replicate the entire source model and directly compare with some of the specific EPRI model parameters. The TIP report provides tables and figures that illustrate how the individual EVA's (experts) evaluated or weighted certain issues or parameters, but the report does not provide a final tabulation of all source parameters and weights that were used in the computation of hazard in the TIP study.

The applicant noted, however, that "the TIP report does present logic trees, tables, and plots that summarize different aspects of their seismic source characterization and uncertainty in several key parameters". The applicant also stated the following in support of the TIP study:

> However, the TIP study does address some key issues and provides assessments of these issues by the five experts assembled (Bollinger, Chapman, Coppersmith, Jacob, and Talwani) that can be evaluated and compared, in a more general sense, to the EPRI EST source model parameters. The TIP study included multiple workshops to define, clarify, and elicit expert opinion on several critical issues relating to the source characterization process and specific technical questions on seismic sources that were judged to be significant to the hazard at the Vogtle and Watts Bar sites.

As requested by the staff in Open Item 2.5-2, the applicant also presented an evaluation of information in the TIP study relevant to the seismic source characterization of the ESP site, including the ETSZ. The applicant stated that "Several of the key issues addressed in the TIP study support the wide range of uncertainty expressed in the EPRI EST seismic source characterizations for the ESP site."

In summary, the applicant acknowledged that the TIP study is a valid study and also provided an evaluation of information relevant to the seismic source characterization of the ESP site (see Open Item 2.5-3 for the applicant's discussion of the TIP study report with respect to the ETSZ). Therefore, the staff considers Open Item 2.5-2 to be closed.

Northwest of the ESP site, at a distance just beyond 200 miles, is the ETSZ zone. As shown in SER Figure 2.5.2-1, the ETSZ covers a cluster of earthquakes in eastern Tennessee. In SSAR Section 2.5.2.2.2.5, the applicant stated that, despite being one of the most active seismic zones in Eastern North America, the largest recorded earthquake recorded in the ETSZ is a magnitude 4.6, and no evidence for larger prehistoric earthquakes, such as paleoliquefaction features, has been discovered. The applicant also stated that, with the exception of the Law source 17 (Eastern Basement), none of the EPRI EST sources that included the ETSZ contributed more than 1 percent of the total hazard at the ESP site. For this reason, the applicant's hazard calculations did not include the sources that accounted for ETSZ seismicity, with the exception of Law source 17. The applicant also concluded that no new information regarding the ETSZ has been developed since 1986 that would require a significant revision to the original EPRI seismic source model, specifically with regards to M_{max} for the ETSZ.

In RAI 2.5.2-16, the staff asked the applicant to provide the M_{max} distributions and geographic coordinates defining the geometry of each EST-identified ETSZ. In response to RAI 2.5.2-16, the applicant provided the staff with the requested information and also stated the following:

> None of the EPRI-SOG teams specifically defined a zone identified as "Eastern Tennessee Seismic Zone." Each EPRI-SOG team did define one or more zones that encompass seismicity in eastern Tennessee and, in most cases, the surrounding regions.

The staff concludes that the information provided by the applicant, in response to RAI 2.5.2-16, is complete. SER Table 2.5.2-5 shows the M_{max} distributions for the EPRI EST seismic sources that encompass seismicity in eastern Tennessee, provided by the applicant in its response to RAI 2.5.2-16.

Table 2.5.2-5 - M_{max} Values Corresponding to the EPRI EST Seismic Source Zones That Encompass Seismicity in Eastern Tennessee (Provided by the Applicant In Response to RAI 2.5.2-5)

EPRI EST	Source	Description	Probability of Activity	M_{max} (M) and Weights
Bechtel	24	Bristol Trends	0.25	5.31 [0.10] 5.66 [0.40] 6.06 [0.40] 6.49 [0.10]
	25	NY-AL Lineament	0.3	4.97 [0.10] 5.31 [0.40] 5.66 [0.40] 6.49 [0.10]
	25A	NY-AL Lineament (Alternative)	0.45	4.97 [0.10] 5.31 [0.40] 5.66 [0.40] 6.49 [0.10]
Dames & Moore	4	Appalachian Fold Belt	0.35	5.66 [0.80] 7.51 [0.20]
	4A	Kinks in Appalachian Fold Belt	0.65	6.82 [0.80] 7.51 [0.20]
Law Engineering	17	Eastern Basement	0.62	5.31 [0.20] 6.82 [0.80]
Rondout	13	Southern NY-AL Lineament	1	4.78 [0.30] 6.06 [0.55] 6.34 [0.15]
	24	Southern Appalachians	0.99	6.49 [0.30] 6.82 [0.60] 7.16 [0.10]
	27	TN-VA Border	0.99	4.78 [0.30] 6.06 [0.55] 6.34 [0.15]
Weston	24	NY-AL Clingman	0.9	4.97 [0.26] 5.66 [0.58] 6.49 [0.16]
Woodward-Clyde	31	Blue Ridge Combo	0.024	5.54 [0.33] 6.06 [0.34] 7.16 [0.33]
	31A	Blue Ridge Combo (Alternative)	0.211	5.54 [0.33] 6.06 [0.34] 7.16 [0.33]

In RAI 2.5.2-17, the staff asked the applicant to justify its rationale for not updating the ETSZ as characterized by the EPRI ESTs and to discuss how the M_{max} distributions developed by each EST compare with more recent M_{max} estimates for the ETSZ included in the USGS hazard model (Frankel et al. 2002) and Bollinger (1992). In addition, the staff asked the applicant to explain whether the contribution to the hazard would change if the EST source zones representing the ETSZ were assigned a single M_{max} of M 7.5, or alternatively, to explain why it believes an M_{max} value of M 7.5 with a weight of 0.5 or higher is not warranted for the ETSZ.

In response, the applicant concluded that the majority of the seismicity that defines the ETSZ is beyond the 200-mi site region. The applicant also noted that its update of the Charleston seismic source model (based on recent paleoliquefaction studies) has increased the relative contribution of the Charleston source to the ESP site and thus served to decrease the relative contribution of more distant sources such as the ETSZ. Furthermore, the applicant stated that there is no historic or prehistoric evidence for large magnitude events occurring in the eastern Tennessee area. In support of the low weights assigned by the EPRI ESTs for this region, the applicant stated the following:

> While the lack of evidence for past large events in ETSZ does not preclude large events from occurring in the future, this fact should influence the weighting of the M_{max} distribution. It is therefore logical that the M_{max} distribution for the ETSZ should have lower weights assigned to the largest magnitudes, in contrast to the Charleston and New Madrid sources, where there is a high confidence that those sources are capable of producing large events since they have occurred in the past.

In response to RAI 2.5.2-17, the applicant concluded that the EPRI EST maximum magnitude distributions for the ETSZ span the range of more recent assessments. The applicant's discussion focused on Bollinger's (1992) source model for the SRS. The applicant stated that Bollinger's (1992) M_{max} of **M** 6.3, which was given a weight of 95 percent, is close to the mean $_{max}$imum magnitude of ~**M** 6.2 of the EPRI study. The applicant also noted that Bollinger (1992) assigned a low weight of 5 percent to an M_{max} of **M** 7.8, which was calculated based on a low probability that the dimensions of seismogenic structures within the zone may extend along the entire 300-km northeast-trending axis of the zone. The applicant also concluded that the TIP study (NUREG/CR-6607) provided a similarly broad M_{max} magnitude distribution as did the EPRI distribution of M 4.8 to M 7.5 for the ETSZ. The applicant stated that the magnitude distributions for all TIP Study ETSZ source zone representations ranged from as low as **M** 4.5 to as high as **M** 7.5, with the mode of about **M** 6.5 for almost each distribution (NUREG/CR-6607, pages F-12 to F-19 of Appendix F).

In summary, the applicant concluded the following in its response to RAI 2.5.2-17:

> The ETSZ is characterized by abundant seismicity, but has yet to produce a recorded event greater than **M** 5, which is about the minimum magnitude used to characterize seismic sources in modern PSHA studies. In our opinion, we believe that there is sufficient uncertainty in the M_{max} potential of the ETSZ that a broad range of magnitudes is appropriate and that the EPRI model sufficiently captures the range of more recent M_{max} distributions for this source. While the ETSZ may be capable of producing a **M** 7.5, we do not believe that a weight of 0.5 to 1.0 for this magnitude represents the range of expert opinion reflected in the post-EPRI studies by Bollinger (1992) and Savy et al. (2002). The exception, of course, is the USGS model that assigns a single magnitude of **M** 7.5.

The staff reviewed the applicant's response to RAI 2.5.2-17 and disagrees with the applicant that the ETSZ EPRI EST M_{max} values adequately represent the ETSZ. Rather, the staff concludes that even though these EPRI EST sources have M_{max} values as large as **M** 7.5, the corresponding weights are very low. In addition, the probabilities of activities of many of the ETSZ EPRI EST sources are also low. For example, in SER Table 2.5.2-5, the Dames and Moore Appalachian Fold Belt source has an M_{max} value of **M** 7.5 and a weight of 0.20, and the probability of activity of this source is only 0.35.

SER Table 2.5.2-6 shows recent M_{max} values for the ETSZ including Frankel et al. (2002), Chapman and Talwani (2002), and Bollinger (1992). A comparison of the two results shows that the EPRI M_{max} values shown in SER Table 2.5.2-5 are significantly lower than more recent studies, as shown in SER Table 2.5.2-6. For example, Chapman and Talwani (2002) assigned a single M_{max} of **M** 7.0 to the ETSZ. They noted that epicentral locations of the earthquakes define a major northeast-trending seismic zone, over 300 kilometers in length, suggesting the possibility of a major shock, if the zone is viewed as defining a through-going basement fault. Chapman and Talwani (2002) also stated that "focal mechanisms and the spatial locations of seismicity have revealed much information concerning this important issue, but the seismic hazard posed by this seismic zone remains uncertain."

Table 2.5.2-6 - M_{max} Values for the ETSZ for Recent Studies

Study	M_{max} (M) and Weights
Bollinger (1989)	6.2 [1.0]
Johnston and Chiu (1989)	7.2 [1.0]
Bollinger (1992)	5.7 [0.158] 6.1 [0.158] 6.2 [0.317] 6.5 [0.158] 7.2 [0.158] 7.8 [0.050]
Frankel et al. (2002)	7.5 [1.0]
Chapman and Talwani (2002)	7.0 [1.0]

Furthermore, as stated in the applicant's response above, none of the EPRI ESTs specifically defined a zone identified as the "Eastern Tennessee Seismic Zone." Each EPRI EST did define one or more zones that encompass seismicity in eastern Tennessee and, in most cases, the surrounding regions. In more recent studies, the seismicity within the ETSZ is explicitly developed into source geometries to account for the ETSZ (e.g., Frankel et al. 2002; Chapman and Talwani 2002; Bollinger 1992; and NUREG/CR-6607).

To validate the applicant's claim that the ETSZ hazard results are insignificant compared to the Charleston seismic source, the staff did a confirmatory analysis. The staff performed hazard calculations using maximum magnitudes for the ETSZ that ranged from **M** 6.0 to **M** 7.8. This magnitude range reflects more recent M_{max} values assigned to the ETSZ, as shown in SER Table 2.2.5-6. SER Figure 2.5.2-12 shows the staff's 1-Hz hazard curves for the ETSZ using this range of M_{max} values. SER Figure 2.5.2-12 also shows the applicant's total mean hazard curve and the Charleston seismic source zone contribution for comparison. The staff's results show that, although the Charleston seismic source zone clearly dominates the 1-Hz hazard, the contribution from the ETSZ for some of the larger M_{max} values (greater than 7.0) may contribute significantly more than 1 percent to the total hazard for the ESP site.

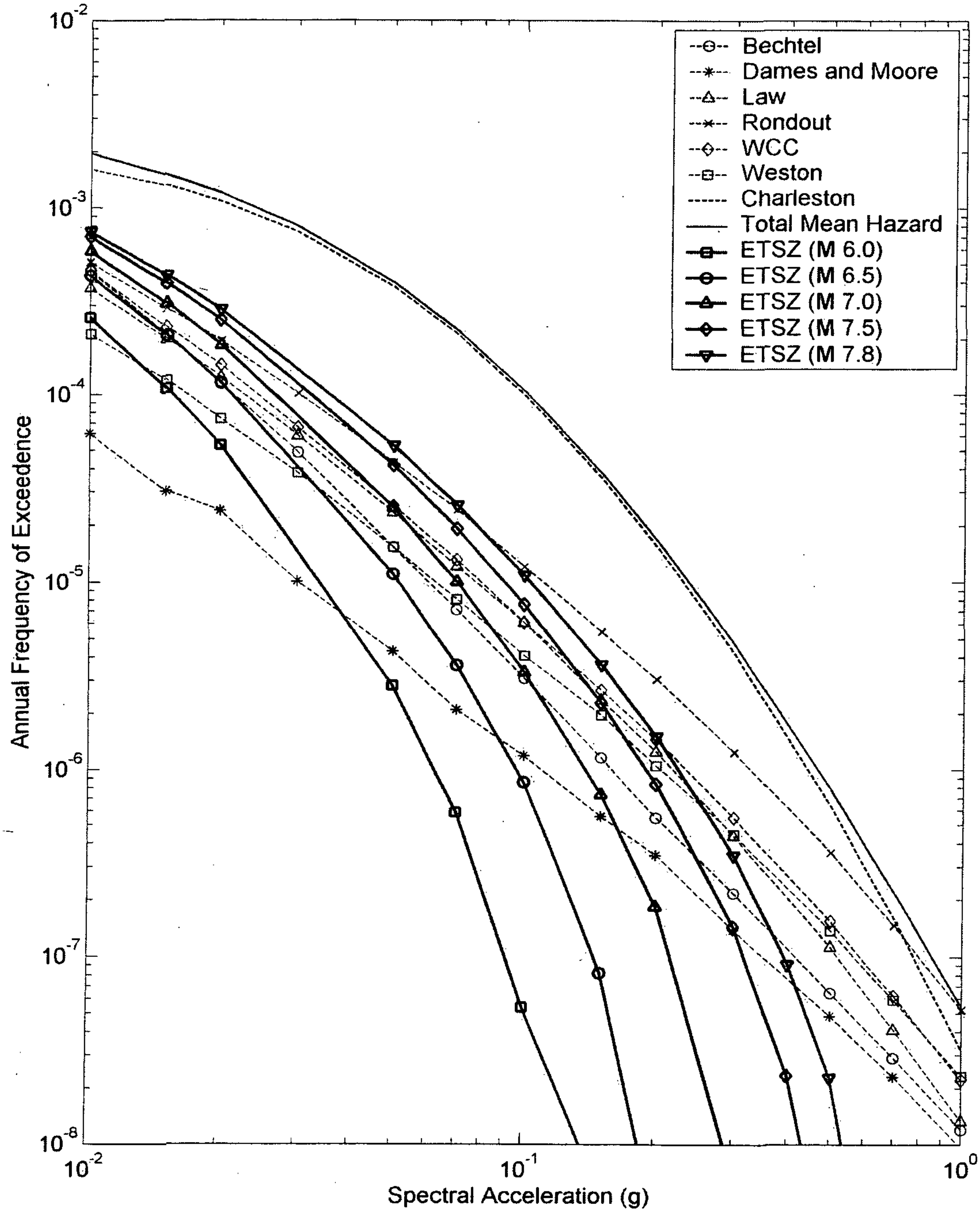


Figure 2.5.2-12 - Comparison of the staff's 1-Hz hazard curves for the ETSZ for magnitudes ranging from M 6.0 to M 7.8

The staff concluded that, despite the uncertainty regarding the potential for large earthquakes within the ETSZ, the results of post-EPRI source characterizations for the ETSZ suggest that the EPRI EST characterization of the ETSZ needs to be updated. The results of the staff's confirmatory analysis confirmed the applicant's assertion that the Charleston seismic source dominates the 1-Hz hazard. However, the staff concluded in the SER with open items that the contribution of the ETSZ at the ESP site may be significant enough to warrant inclusion in the applicant's PSHA, if larger M_{max} values are considered. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-3.

In response to Open Item 2.5-3, the applicant stated the following:

> The Eastern Tennessee seismic zone (ETSZ) lies between the New York-Alabama and Ocoee aeromagnetic anomalies in what Kanter (1994) has classified as non-extended crust. Wheeler (1995; 1996) has defined this region associated with Eastern Tennessee seismicity as Late Proterozoic/early Paleozoic lapetan extended crust. Based on the Johnston et al. (1994) study of stable continental cratons, the global seismicity database indicates that the largest historic earthquakes (**M**>7) are limited to Mesozoic extended crust. The Johnston et al. (1994) data base shows that Paleozoic non-extended crust has a mean M_{max} of **M**6.4. Therefore, based on the global database, there is no analog to suggest that the ETSZ portion of the crust should fail in large (**M**>7) events.

As requested by the staff in Open Item 2.5-2, the applicant also provided an evaluation of the TIP study relevant to the seismic source characterization of the ESP site. In response to Open Item 2.5-3 (as well as in response to the staff's request in Open Item 2.5-2) the applicant provided the following evaluation of the ETSZ based on the TIP study:

> The Trial Implementation Project (TIP) study (Savy et al., 2002) identified the ETSZ as a key issue in assessing hazard for the Watts Bar site in Tennessee. While this study was primarily a trial implementation of the SSHAC process, the NRC has requested in Open Item 2.5-2 that we more closely examine information contained in the TIP study that is relevant to the seismic source characterization of the ESP site. The TIP study defined eight source zones to represent uncertainty in the geometry of the ETSZ and defined composite M_{max} distributions for each source zone using the weighting schemes from each of the five experts. The composite M_{max} distributions are presented graphically (pages F-12 through F-19 of the TIP study) for each of the ETSZ source zones, and are summarized in the table below with values of the minimum, maximum, and mode of the distributions.

Source Zone	Min	Mode	Max
4a1	4.5	6.5	7.5
4a1+2	5.0	6.5	7.5
4a1+2+3	5.0	6.5	7.5
4b1	5.0	6.5	7.5
4b2	5.0	6.5	7.5
4c	5.0	6.5	7.5
4d	5.0	6.5	7.5
4e	5.0	6.5	7.5

The magnitude distributions for all ETSZ source zone representations in the TIP study ranged from as low as **M**4.5 to as high as **M**7.5, with a mode of either **M**6.3 or **M**6.5 for each distribution. The modal values represent the greatest weight of the distributions, indicating that the experts participating in the trial implementation of the SSHAC Level 4 process felt that the majority of the weight belonged in the moderate magnitude events as opposed to the largest magnitudes. The broad distribution of the TIP study is similar to the distribution of **M**4.8 to **M**7.5 in the EPRI source zones.

The modal M_{max} value for each of the TIP characterizations of the ETSZ is either **M**6.3 or **M**6.5. Even though the TIP study does not present discrete magnitudes and weights, the modal magnitudes suggest a mean magnitude on the order of ~**M**6.5 or less for the ETSZ.

In summary, the applicant concluded that "Since no new data or evidence has been developed to imply large magnitude earthquakes in the ETSZ since the EPRI study, there is no basis for rejecting the M_{max} interpretations of the EPRI teams, which cover the range of M_{max} employed in more recent seismic source characterizations. Therefore additional calculations of seismic hazard with larger M_{max} values for the ETSZ would be purely speculative and could not form a basis for conclusions."

The staff disagrees with the applicant's conclusions that additional calculations of seismic hazard with larger M_{max} for the ETSZ are not warranted. The staff notes that there are more recent seismic hazard studies, such as the LLNL TIP study and the Geomatrix TVA Dam safety study, which provide new information on the seismic hazard of the area. Furthermore, the staff does not agree with the applicant's conclusion that the EPRI team's M_{max} composite distribution for the ESTZ is similar to that of more recent studies. The applicant only compared the range of the M_{max} values of the EPRI study rather than the actual weighted values. SER Figure 2.5.2-13 clearly shows that more recent studies place a significantly higher probability on larger maximum magnitude earthquakes than the EPRI study. The mean M_{max} for the TIP (i.e. Savy et al., 2002) and Geomatrix studies are approximately **M**6.55 and **M**6.58, respectively.

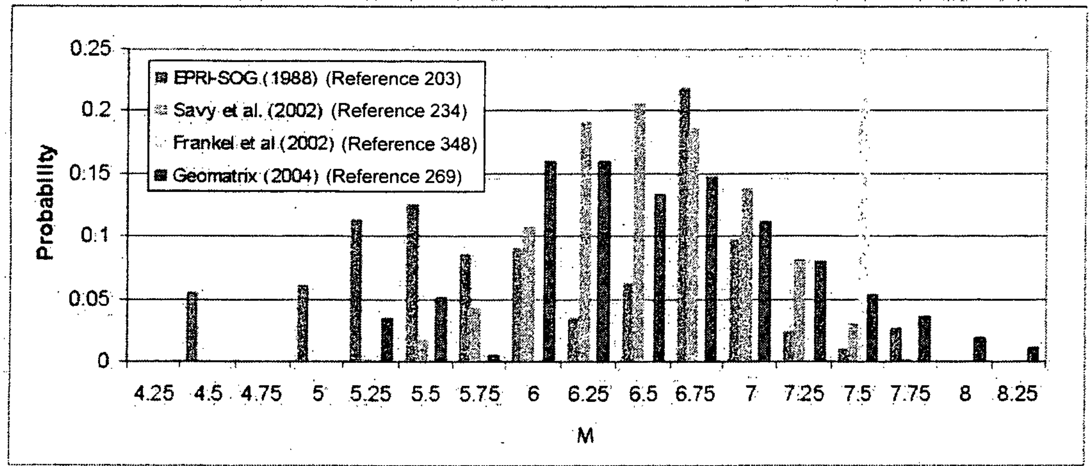


Figure 2.5.2-13. Composite EPRI-SOG distribution in terms of M compared to more recent assessments (reproduced from the Bellefonte RCOL application)

The staff concludes, however, that the contribution of the ESTZ at the Vogtle ESP site is insignificant, even when M_{max} values comparable to the mean M_{max} values for the TIP and Geomatrix studies are considered. Based on the staff's sensitivity study, presented in SER Figure 2.5.2-12, a mean magnitude of **M**6.5 for the ETSZ contributes to less than 1 percent of the total hazard at 1 Hz for ground motions critical for design levels (0.1 g and higher). Therefore, the staff considers Open Item 2.5-3 to be resolved.

Updated EPRI Seismic Sources

Based on the results of several post-EPRI PSHA studies (Frankel et al. 2002; Chapman and Talwani 2002) and the recent availability of paleoliquefaction data (Talwani and Schaeffer 2001) for the Charleston source zone, the applicant updated the EPRI characterization of the Charleston seismic source zone as part of the ESP application. The applicant referred to its update as the UCSS model. The staff focused its review on the applicant's UCSS geometry, M_{max} values, and recurrence model. The staff also reviewed the methodology that the applicant used to perform this update.

SSHAC Update of the Charleston Seismic Source. In SSAR Section 2.5.2.2.2.4, the applicant noted that the UCSS model is described in detail in a 2006 Bechtel engineering study report. In order to review the applicant's UCSS model, the staff, in RAI 2.5.2-2, requested a copy of the Bechtel (2006) report. In response to RAI 2.5.2-2, the applicant provided the staff with a copy of Bechtel (2006). Based on its review of the Bechtel (2006) report, the staff gained additional insight regarding the applicant's UCSS model.

As described in Bechtel (2006), the applicant performed an SSHAC Level 2 study to incorporate current literature and data, as well as the understanding of experts, into an update of the Charleston seismic source model. An SSHAC Level 2 study uses an individual, team, or company to act as a Technical Integrator (TI), who is responsible for reviewing data and literature and contacting experts who have developed interpretations of or who have specific knowledge about the seismic source. The TI for the update of the Charleston seismic source model consisted of a team of six William Lettis & Associates, Inc. (WLA) personnel (Scott

Lindvall, Ross Hartleb, William Lettis, Jeff Unruh, Keith Kelson, and Steve Thompson). The WLA TI team first compiled and reviewed all new information developed since 1986 regarding the 1886 Charleston earthquake and the seismic source that may have produced this earthquake and then compared this new information with the 1986 EPRI EST assessments of the Charleston seismic source. Following the literature review, the TI conducted interviews with experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. The TI consulted the following seismic and geologic experts:

- Dr. David Amick, Science Applications International Corporation
- Dr. Martin Chapman, Virginia Polytechnic Institute
- Dr. Chris Cramer, U.S. Geological Survey
- Dr. Art Frankel, U.S. Geological Survey
- Dr. Arch Johnston, Center for Earthquake Research and Information, University of Memphis
- Dr. Richard Lee, Los Alamos National Laboratory
- Dr. Joe Litehiser, Bechtel Corporation (original team leader of the 1986 Bechtel EST)
- Dr. Stephen Obermeier, U.S. Geological Survey (retired)
- Dr. Pradeep Talwani, University of South Carolina
- Dr. Robert Weems, U.S. Geological Survey

The TI next integrated this information to develop an updated characterization of the Charleston seismic source that captures the composite representation of the informed technical community.

In RAI 2.5.2-4, the staff asked the applicant to justify its rationale for selecting an SSHAC Level 2 methodology for the UCSS update, as opposed to a higher level update. To support its rationale for using the SSHAC Level 2 methodology, the applicant stated the following:

> SSHAC (1997) describes four levels of study (Levels 1 through 4), in increasing order of sophistication and effort. The choice of the level of a PSHA is driven by two factors: (1) the degree of uncertainty and contention associated with the particular project, and (2) the amount of resources available for the study (SSHAC 1997). SSHAC (1997, Table 3-1) suggests that a Level 2 study is appropriate for issues with "significant uncertainty and diversity," and for issues that are "controversial" and "complex." In a SSHAC Level 2 study, a Technical Integrator (TI) is responsible for reviewing data and literature and contacting experts who have developed interpretations or who have specific knowledge of the seismic source. The TI interacts with experts to identify issues and interpretations, and to assess the range of informed expert opinion. In Level 3 studies, the TI goes a step further by bringing together experts and focusing dialog and interaction between them in order to evaluate relevant issues. In Level 4 studies, a Technical Facilitator/Integrator (TFI) is responsible for aggregating the judgments of a panel of experts to develop a composite distribution of the informed technical community. In a meeting held on July 7, 2005, VEGP ESP Technical Advisory Group (TAG) members Dr. Martin Chapman, Dr. Robert Kennedy, Dr. Carl Stepp, and Dr. Robert Youngs agreed that a Level 2 study is appropriate for updating the Charleston seismic source model.

In RAI 2.5.2-4, the staff also asked the applicant to describe its implementation of the SSHAC Level 2 methodology. Specifically, the staff asked the applicant to describe in more detail how the expert's opinions were integrated into the development of the final UCSS model, how any conflicting opinions between the experts were dealt with, and how the final source model represents the informed consensus of the community beyond those queried for the UCSS update. In response, the applicant stated that, as part of the SSHAC process, the TI contacted 10 experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. The applicant stated the following:

> These experts were asked a series of questions pertaining to key issues regarding the Charleston seismic source. This was not a formal process of expert interrogation to obtain from each expert all of the specific parameters and weights to be used in the model. Instead, we allowed the experts to speak to their own areas of expertise. It was then the TI's responsibility to combine these responses with data from the published literature to capture the range of expert opinion and judgment regarding parameters and weights to be used in the UCSS model.

Regarding the TI integration of the expert's opinion into the development of the final UCSS model, the applicant provided the following information:

> This activity included a two-day workshop held on September 13–14, 2005 to develop the UCSS model at the WLA office in Valencia, California after several weeks of literature and data review. The workshop included the TI team, who integrated Charleston area data and expert interpretations, discussed uncertainties and conflicting expert interpretations, and developed UCSS geometries and the logic tree.

The applicant also stated the following regarding the review of the UCSS model by the TAG panel:

> A Technical Advisory Group (TAG) panel was convened in April 2006 in Frederick, Maryland to critically review the UCSS model and to provide feedback regarding the process and the results of the study. TAG members Chapman, Kennedy, Stepp, and Youngs were in attendance. In addition, Dr. Carl Stepp and Dr. Martin Chapman reviewed written copies of the Engineering Report describing the UCSS and provided written comments on, and approval of, the document.

With regard to how the final source model represents the informed consensus of the community beyond those queried for the UCSS update, the applicant stated, "for the VEGP ESP study, a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 study was performed to incorporate current literature and data and the understanding of experts into an update of the Charleston seismic source model," and that "the intent of the SSHAC process is to represent the range of current understanding of seismic source parameters by the informed technical community."

Based on its review of SSHAC (1997) and the Bechtel (2006) report provided by the applicant in response to RAI 2.5.2-2, as well as the applicant's response to RAI 2.5.2-4, the staff concludes that the applicant's overall implementation of the SSHAC Level 2 process is adequate. In accordance with an SSHAC Level 2 study, the applicant established a TI, comprising six WLA personnel, to conduct a literature review and contact experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. As defined in the SSHAC report, a TI is "a single entity (individual, team, or company, etc.) who is responsible for ultimately developing the composite representation of the informed technical community." Also in accordance with SSHAC, the applicant selected a peer review panel to "critically review the UCSS model and to provide feedback regarding the process and results of the study." The applicant referred to its peer review panel as the VEGP ESP TAG. The TAG consisted of Dr. Martin Chapman, Dr. Robert Kennedy, Dr. Carl Stepp, and Dr. Robert Youngs. According to the 1997 SSHAC report, the purpose of the peer review panel is to "assure that the process followed was adequate and to ensure that the results provide a reasonable representation of the diversity of views of the technical community."

The staff also concludes that the applicant's selection of an SSHAC Level 2 study is appropriate for the update of the Charleston seismic source zone. As shown in SER Table 2.5.2-7 (reproduced from Table 3-1 of the 1997 SSHAC report), the SSHAC criteria for deciding on the level of the study is rather subjective. The 1997 SSHAC report suggests that Level 2 studies are appropriate for issues with "significant uncertainty and diversity," and for issues that are "controversial" and "complex," while Level 3 and 4 studies are appropriate for issues that are "highly contentious; significant to hazard; and highly complex." SSHAC (1997) also states that Level 3 and 4 studies "are resource-intensive and are, therefore, most appropriate for large-scale studies for critical facilities." Thus, based on the guidance provided in SSHAC (1997), and because the applicant's study involved the update of a single seismic source zone, the staff agrees with the applicant's decision to use an SSHAC Level 2 study.

Table 2.5.2-7 - Degrees of PSHA Issues and Levels of Study (from SSHAC (1997), Table 3-1, p. 23)

ISSUE DEGREE	DECISION FACTORS	STUDY LEVEL
A Noncontroversial and/or insignificant to hazard		1 TI evaluates/weights models based on literature review and experience; estimates community distribution
B Significant uncertainty and diversity; controversial; and complex		2 TI interacts with proponents and resource experts to identify issues and interpretations; estimates community distribution
C Highly contentious; significant to hazard; and highly complex	Regulatory concern Resources available Public perception	3 TI brings together proponents and resource experts for debate and interaction; TI focuses debate and evaluates alternative interpretations; estimates community distribution
		4 TFI organizes panel of experts to interpret and evaluate; focuses discussions; avoids inappropriate behavior on part of evaluators; draws picture of evaluators' estimate of the community's composite distribution; has ultimate responsibility for project

Although the staff concurs with the applicant's selection and overall implementation of an SSHAC Level 2 method to update the Charleston seismic source model, its review of Bechtel (2006) resulted in several additional questions. For example, the staff was unable to determine the actual questions that each of the experts involved in the SSHAC Level 2 study were asked, the range of expert opinions related to key aspects of the UCSS model (i.e., recurrence, geometry, and maximum magnitude), or the specific process used to combine the expert's opinions and resolve any differing opinions. On June 18, 2007, the applicant supplemented its response to RAI 2.5.2-4 with additional information regarding its SSHAC Level 2 study. Because the staff received this information late in the review process, the staff identified this as Open Item 2.5-4 in the SER with open items, to allow additional time to complete the review. The staff also requested the applicant to explain why only two of the four members of the TAG panel reviewed and approved written copies of the engineering report describing the UCSS, as stated in response to RAI 2.5.2-4.

In its supplemental response to RAI 2.5.2-4, the applicant provided the staff with the list of questions that the technical integrator developed and used as its basis for communicating with researchers by telephone. These questions covered the main issues involving the Charleston earthquake process, geometry, maximum magnitude (M_{max}), and recurrence. The applicant also provided the responses given by each of the experts. The applicant noted that some of the experts limited their responses to their own specific area of expertise. For example, Stephen Obermeier (USGS, retired) provided comments and insight on paleoliquefaction data, but did not wish to comment on specific questions regarding source geometry modeling and other parameters. In addition, the applicant also stated that in some interviews, selected questions were not asked if the topic was outside the expert's research area or if the interview was limited on time.

The applicant's supplemental response to RAI 2.5.2-4 also describes how the expert's opinions were integrated into the development of the final UCSS model, and how any conflicting opinions between the experts were dealt with. The applicant stated that "because the SSHAC Level 2 process does not involve bringing the experts together, there was not a forum for experts to directly question or challenge each other's assumptions or results and formally resolve any conflicting opinions." The applicant noted that "in the compilation of literature and expert opinions, there were instances where one expert's opinions differed from others." The applicant further noted that "in these cases, it is the responsibility of the Technical Integrator (TI) to "evaluate the viability and credibility of the various hypotheses with an eye toward capturing the range of interpretations, their credibilities, and uncertainties" (SSHAC 1997). The applicant stated that "conflicting opinions were included in the model parameters in an effort to capture the range of opinion and uncertainty."

In Open Item 2.5-4, the staff also requested the applicant to explain why only two of the four members of the Technical Advisory Group (TAG) panel reviewed and approved written copies of the engineering report describing the Updated Charleston Seismic Source (UCSS), as stated in its response to RAI 2.5.2-4. In response to Open Item 2.5-4, the applicant stated the following:

> The Updated Charleston Seismic Source (UCSS) model was presented to the entire Technical Advisory Group (TAG) panel in meetings on April 12-13, 2006. As such, the TAG performed participatory peer review of the UCSS, including reviewing the approach (i.e., SSHAC Level 2), data, and results of the updated model. The TAG panel consisted of three seismologists and one structural engineer. It was decided that it would be in the best interest of the project to also have a detailed review of UCSS engineering report by members of the TAG. The two seismologists most familiar with the tectonics and seismicity of the southeastern US, Dr. Martin Chapman and Dr. Carl Stepp, were requested to review written copies of the engineering report and provide comments.

The staff reviewed the applicant's responses to RAI 2.5.2-4 and Open Item 2.5-4. Based on its review, the staff concludes that the applicant adequately performed a SSHAC Level 2 study to update the Charleston seismic source zone. The staff concludes that the list of questions that the TI asked the experts generally addressed the key aspects of the UCSS model, and that the applicant's UCSS adequately captured the range of expert's input, when provided. The staff further concludes that the TI adequately integrated the range of expert's responses, where appropriate, into the final UCSS along with its findings based on its review of current literature and paleoliquefaction data. In addition, the staff considers it appropriate that only two of the TAG panel members performed a detailed review the final UCSS because these members had the most familiarity with the tectonics and seismicity of the southeastern US.

<u>Paleoliquefaction features of the Charleston seismic source zone</u>. Abundant soil liquefaction features induced by the 1886 Charleston earthquake, in addition to other large prehistoric earthquakes (dating back to the mid-Holocene), are preserved in geologic deposits at numerous

locations within the 1886 meizoseismal area and along the South Carolina coast. SSAR Section 2.5.2.2.2.4.1 states that the characteristics of the 1886 Charleston earthquake, combined with the greatest density of prehistoric liquefaction features, "show that future earthquakes having magnitudes comparable to the Charleston earthquake of 1886 most likely will occur within the area defined by Geometry A. A weight of 0.7 is assigned to Geometry A". Additionally, SSAR Figure 2.5.2-9 indicates no likelihood that an 1886-sized earthquake has occurred inland from the coastal region, except along Geometry C, and then only with a probability of 0.1. In RAI 2.5.2-8, the staff asked the applicant to summarize the age, liquefaction susceptibility, and geographic distribution of liquefiable deposits in the zone that is 50 to 150 kilometers (31 to 93 miles) inland from the coast and explain whether this information supports a negligible probability of large inland earthquakes. In addition, in RAI 2.5.2-8, the staff requested that the applicant reconcile the negligible probability of large inland earthquakes, as indicated in SSAR Figure 2.5.2-9, with the discovery of prehistoric liquefaction features as much as 100 kilometers (62 miles) inland in fluvial deposits of the Edisto River (Obermeier 1996). In response to RAI 2.5.2-8, the applicant stated the following:

> Liquefaction susceptibility is a function of numerous variables including, but not limited, to, sediment grain size and sorting, degree of compaction and/or cementation, deposit thickness, depth below ground surface, degree of saturation, and sediment age. Obermeier (1996) suggested that South Carolina Coastal Plain deposits older than about 250 ka have negligible potential for liquefaction due to the effects of chemical weathering. Obermeier (1996) observed that, in general, the region within 30 mi (~50 km) of the coast is highly susceptible to liquefaction. The liquefiable deposits of the about 100 ka Princess Anne Formation, however, are mapped greater than 65 mi inland (McCartan et al. 1984).
>
> Numerous liquefaction features caused by the 1886 Charleston earthquake and paleoliquefaction features from prehistoric Events A, B, C', E and F' are distributed along a 115 mi stretch of coastal South Carolina from Bluffton in the south to Georgetown in the north. The inland extent of 1886 liquefaction is less well-constrained.
>
> There is no structural, geomorphic, paleoseismic (other than the cited sparse liquefaction data), or historic (i.e., 1886) evidence to suggest a source zone geometry that trends northwest-southeast or extends significantly inland from the 1886 meizoseismal area. The sparse liquefaction features along the Edisto River cited by Seeber and Armbruster (1981), Amick et al. (1990), and Obermeier (1996) likely reflect strong ground shaking in deposits susceptible to liquefaction, and not a localized, inland source.

The staff agrees that the applicant's response adequately summarized the age, liquefaction susceptibility, and geographic distribution of liquefiable deposits in the zone 50–150 kilometers (31–93 miles) inland from the South Carolina coast. However, it is the staff's opinion that the applicant, in its RAI response, did not provide substantial evidence to rule out the occurrence of large inland earthquakes, especially given the presence of liquefiable deposits greater than 100 kilometers (65 miles) inland from the coast. The occurrence of a large earthquake inland from the coast would necessitate a different Charleston source zone model. Accordingly, in the SER with open items, the staff identified this issue as Open Item 2.5-5. In Open Item 2.5-5, the staff asked the applicant to provide supporting evidence to rule out the occurrence of large inland earthquakes.

In response to Open Item 2.5-5, the applicant explained that it would be difficult to provide direct evidence that large earthquakes have not occurred inland from Charleston. The applicant described liquefaction and paleoliquefaction features that have been documented by a number of researchers along the Edisto River as far as 70 km (45 mi) inland from the coast. The applicant considered these sites to represent liquefaction and paleoliquefaction features documented farthest inland from the coast. The applicant explained that most researchers do not document negative findings for inland liquefaction features and provided the following statement:

> Various researchers (e.g., Amick et al. 1990, Obermeier 1996) have published maps depicting the geographic distribution of 1886 liquefaction and paleoliquefaction sites in coastal South Carolina and along the eastern seaboard. These researchers do not, however, thoroughly document their reconnaissance of the rivers and drainage ditches that lack features indicative of strong ground shaking inland from the Charleston meizoseismal area, other than to say none was observed inland.

The applicant also provided additional supporting information in the form of documented expert opinion regarding the likelihood of large inland earthquakes. The following statement by the applicant details the opinions of Stephen Obermeier (U.S. Geological Survey, retired), an expert in eastern U.S. liquefaction and paleoliquefaction:

> Obermeier discussed the areas reconnoitered as part of his and others' research into South Carolina coastal plain liquefaction sites. There are no published maps that show in detail those areas studied but in which no liquefaction features were recognized. According to Obermeier, Figure 7.6 from Obermeier (1996) represents the best published approximation of the areas of investigation. This figure indicates that, with the exception of the Edisto River, the search for liquefaction features extended roughly 12 to 30 mi (20 to 50 km) inland throughout South Carolina. Reconnaissance along the Edisto River extended to roughly 45 mi (70 km) from the coast and represents the inland-most extent of the search for liquefaction features. Reconnaissance was conducted inland along the Edisto River in part because the banks of this river and its associated drainage ditches, more so than most in South Carolina, provide relatively good geologic exposure in which liquefaction features may be recognized.

The applicant compared the geographic distribution of the inland Edisto River liquefaction features to those found along the coast and made the following statement:

> It is instructive to note that these Edisto River liquefaction sites are closer to the Charleston meizoseismal area (<40 miles) than are the liquefaction sites up and down the coast that experienced liquefaction during the 1886 event (~100 miles). These observations indicate that the local Charleston source is capable of producing the observed inland liquefaction features along the Edisto River.

The applicant also provided the following statement contained in the TIP study (Savy et al., 2002) to further support a local Charleston source rather than an inland source for producing large earthquakes:

> The hazard at the Vogtle plant will be sensitive to the northwestern and western extents of the Charleston source. There appears to be no compelling reason to extend the source to the northwest from the 1886 epicentral area by connecting the Summerville-Middleton Place and Bowman zones of microseismicity. Dave Amick has found no paleoliquefaction evidence for strong ground shaking in the Bowman area, and the microseismicity there is much shallower than in the epicentral area. (p.19)

The applicant stated that while it is difficult to provide conclusive evidence that a large earthquake would not occur inland from the coast, many large areal source zones contained in the EPRI source model allow for potential large earthquakes to occur throughout the southeastern U.S. and thus would account for the possibility of a large inland earthquake outside of the local Charleston source.

While the applicant's position for supporting a negligible probability of large inland earthquakes does not rule out the potential for large inland earthquakes to occur, the staff believes that the applicant provided adequate documentation to support the likelihood of a local Charleston source rather than a source inland from the coast. The staff found the applicant's submittal of expert opinion regarding previous documentation of inland historic and prehistoric liquefaction features to be sufficient to support the applicant's evaluation. Only a handful of sites inland from Charleston along the Edisto River provide evidence for earthquake-induced liquefaction and most researchers do not document a lack of evidence in their observations. While numerous factors contribute to the liquefaction susceptibility at a site, liquefiable sediments are known to be present greater than 100 km (65 mi) inland from the coast, with minimal evidence for liquefaction observed.

The lack of more abundant earthquake-induced liquefaction features observed farther inland coupled with the presence of features extending more than 100 miles along the coast, and mostly equidistant from Charleston, does not prove large inland earthquakes have not occurred but rather suggests a more likely centralized earthquake source closer to Charleston. The staff concurs with the applicant that it would be difficult to provide direct evidence against the occurrence of large inland earthquakes. Furthermore, the staff concludes that the information provided by the applicant in support of a localized Charleston earthquake source, rather than an inland earthquake source, is adequate based on evidence in the existing literature as well as expert opinion regarding actual observed liquefaction features. Therefore, the staff considers Open Item 2.5-5 to be resolved.

With regard to the size and quantity of earthquakes that produced the Charleston area liquefaction features, SSAR Section 2.5.2.2.2.4.3 suggests that the liquefaction features attributed by researchers to a single large, prehistoric earthquake might actually have been produced by several moderate magnitude earthquakes that are closely spaced in time (SSAR, page 2.5.2-26). In RAI 2.5.2-9, the staff asked the applicant to determine whether Talwani or Obermeier, two recognized experts, have data on the sizes of prehistoric liquefaction craters and whether these or any related data might constrain the possible magnitudes of the prehistoric earthquakes.

In response to RAI 2.5.2-9, the applicant explained that it is possible to compare the 1886 earthquake liquefaction features with liquefaction features attributed to pre-1886 events. The applicant further explained that some pre-1886 features suggest an earthquake magnitude similar to the 1886 Charleston earthquake. The applicant provided the following evidence:

> Obermeier (1996) noted "almost all craters that predate 1886 have a morphology and size comparable to the 1886 craters" (p.345). Moreover, the sizes of individual craters formed during the 600 and 1,250 years BP events are at least as large as those formed during the 1886 earthquake, both in the vicinity of Charleston and farther away (Obermeier 1996). These observations suggest that some prehistoric earthquakes have been at least as large as the 1886 earthquake.

The applicant cited a number of references, including Talwani and Schaeffer (2001), Hu et al. (2002a, 2002b), Leon (2003), and Leon et al. (2005), each of which attempted in some degree to estimate earthquake magnitudes associated with liquefaction features over the extended, as well as more limited, areas in the Charleston vicinity. According to the applicant, the magnitude estimates based on these studies vary widely, from **M** 7+ (Talwani and Schaeffer 2001) to **M** 6.8–7.8 (Hu et al. 2002b) to M 6.9–7.1 and **M** 5.6–7.2 (Leon et al. 2005) for earthquakes associated with widespread liquefaction features. Magnitude estimates for earthquakes producing liquefaction features over more limited areas vary similarly from M 6+ (Talwani and Schaeffer 2001) to **M** 5.5–7.0 (Hu et al. 2002b) to **M** 5.7–6.3 and **M** 4.3–6.4.

The applicant concluded that, even with the large uncertainties attached to estimating magnitudes from paleoliquefaction data, and in turn reflecting broad magnitude estimates for prehistoric earthquake events, the studies cited suggest that at least some of the prehistoric earthquakes have been similar in magnitude to the 1886 Charleston earthquake. Specifically, the applicant's response indicates that pre-1886 liquefaction craters "have a morphology and size comparable to the 1886 craters." This statement indicates that 1886 and pre-1886 liquefaction craters have similar maximum sizes, with ground conditions and hypocentral depths being similar, which implies similar historic and prehistoric earthquake magnitudes.

While the applicant's reasoning does not rule out the occurrence of numerous smaller earthquakes, the staff believes that the applicant made an accurate assumption that earthquake magnitudes for pre-1886 earthquakes in the Charleston area are similar to the magnitude range attributed to the 1886 event based on the documentation of large liquefaction craters induced by both 1886 and pre-1886 earthquakes. As such, the staff concludes that the applicant conservatively assumed that the pre-1886 earthquakes were similar in magnitude to the 1886 event.

In RAI 2.5.2-10, the staff asked the applicant to summarize, for each of the pre-1886 events, the number of liquefaction features and sites that have been documented, the areal extent of liquefaction (i.e., the number of square kilometers affected), the number of dates that have been collected, and how well the features correlate from one site to the next.

In response to RAI 2.5.2-10, the applicant summarized the methods used in the application to constrain the timing of liquefaction-inducing earthquakes and referenced SSAR Table 2.5.2-13 to provide an age comparison of Charleston liquefaction events (Talwani and Schaeffer 2001). The applicant provided the following background information:

> Talwani and Schaeffer (2001) used calibrated radiocarbon ages with 1-sigma error bands in order to define the timing of past liquefaction episodes in coastal South Carolina. The standard in paleoseismology, however, is to use calibrated ages with 2-sigma (95.4 percent confidence interval) error bands (e.g., Sieh et al. 1989; Grant and Sieh 1994). Likewise, in paleoliquefaction studies, in order to more accurately reflect the uncertainties in radiocarbon dating, the use of

radiocarbon dates with 2-sigma error bands (as opposed to narrower 1-sigma error bands) is advisable (Tuttle 2001).

Because Talwani and Schaeffer used calibrated ages with 1-sigma error bands, the applicant recalibrated Talwani and Schaeffer's (2001) radiocarbon data using 2-sigma error bands and presented the new data in the application. The applicant stated that the use of 1-sigma error bands by Talwani and Schaeffer (2001) possibly led to an overinterpretation of the paleoliquefaction record such that Talwani and Shaeffer (2001) may have interpreted more episodes than what actually occurred. The applicant used the 2-sigma recalibrated data to obtain broader age ranges for pre-1886 earthquake-induced liquefaction events. The applicant provided the following additional information:

> Paleoearthquakes were distinguished based on grouping paleoliquefaction features that have contemporary radiocarbon samples with overlapping calibrated ages. The event ages were then defined by selecting the age range common to each of the samples. For example, an event defined by overlapping 2-sigma sample ages of 100 to 200 cal yr BP and 50 to 150 cal yr BP would have an event age of 100 to 150 cal yr BP. We consider the "trimmed" ages to represent the ~ 95 percent confidence interval, with a "best estimate" event age as the midpoint between the ~ 95 percent age range.
>
> The 2-sigma analysis identified six earthquakes (including 1886) in the data presented by Talwani and Schaeffer (2001). As noted by that study, events C and D are indistinguishable at the 95 percent confidence interval, and together they compose Event C'. Additionally, our 2-sigma analysis suggests that Talwani and Schaeffer's (2001) events F and G may have been a single, large event, which we name Event F'.

The applicant provided a summary of the approximate number of documented liquefaction features, the areal extent of those features, and the number of radiocarbon dates collected for each of the prehistoric earthquake events (A, B, C', E, F') as well as for the 1886 event. SER Figure 2.5.1-11, in response to RAI 2.5.1-10, provides a means of visually correlating liquefaction features from one site location to the next and from one event to another.

Based on its review of the applicant's response to RAI 2.5.1-10, the staff concludes that the applicant adequately summarized the documented liquefaction features associated with 1886 and pre-1886 earthquake events. The data provided by the applicant are useful in evaluating the uncertainty associated with each of the prehistoric earthquake events and in correlating similarities between events in order to better estimate possible magnitudes and source location.

SSAR Section 2.5.2.2.2.4.3 states that paleoliquefaction Event C is defined by features north of Charleston, while Event D is defined by sites south of Charleston. Events C and D are combined into a single large event, C'. In RAI 2.5.2-11, the staff requested the applicant to provide any information on liquefaction features, geographically located between these two areas, that have similar radiocarbon ages, which would support the characterization of these events as a single large event rather than two separate events. The staff also asked the applicant to provide justification that there is enough paleoliquefaction data to support a single large event C' from a single source.

In response to RAI 2.5.2-11, the applicant stated that using 2-sigma calibration for evaluating radiocarbon dates associated with Talwani and Schaeffer (2001) events C and D, based on timing alone, provides evidence that these events are indistinguishable at the 95 percent confidence interval. The applicant combined the two events into a single event, C'. Talwani and Schaeffer (2001) themselves interpreted an alternate scenario for these two events, also based on 2-sigma calibration of the data, and referred to a possible single event, C'.

The applicant provided a visual depiction of this information (SER Figure 2.5.2-14) to allow a comparison of liquefaction features associated with Talwani and Schaeffer (2001) events C and D to determine any overlap that could provide further evidence that these two events should be combined into a single event, C'. The applicant stated that liquefaction features associated with events C and D are localized and do not show any spatial overlap and "therefore do not provide definitive geographic evidence for combining these events into a single, large event C'." However, the applicant chose to include a single, large event C' (as opposed to two smaller events C and D) into the updated Charleston seismic source model based on the following three reasons:

1. The two-sigma reanalysis of Talwani and Schaeffer's (2001) age data performed for the VEGP ESP application indicates that the age data constraining the timing of Events C and D overlap one another and therefore the two events are indistinguishable. This observation is consistent with the interpretation of a single, large Event C'.

2. The incorporation of a single, large Event C' into the updated Charleston seismic source model is, in effect, a conservative approach. In developing a recurrence interval for large, characteristic earthquakes in the updated Charleston seismic source model, it was desirable to include the possibility that Events C and D represent a single, large earthquake. Talwani and Schaeffer's (2001) moderate-magnitude (~M 6) earthquakes C and D would be eliminated from the record of large (M_{max}) earthquakes in the updated Charleston seismic source model, thereby increasing the calculated M_{max} recurrence interval and lowering the hazard without sufficient justification.

3. The distribution of paleoliquefaction sites for Event C' is very similar to the coastal extent of liquefaction features from the 1886 earthquake. Moreover, the distribution and number of paleoliquefaction sites for Event C' are very similar to those for Events A and B, the two best documented prehistoric events (SER Figure 2.5.2-15).

Based on its review of the applicant's response to RAI 2.5.2-11, the staff acknowledges that recalibration of radiocarbon ages shows that the ages of events C and D are indistinguishable at a 95.4 percent confidence interval and that the applicant's decision to combine the two events into a single larger event, C', is justified. Geographic distribution of liquefaction features associated with a single large event C' is comparable to distribution of features associated with the 1886 Charleston earthquake and prehistoric earthquake events A, B, E and F'. The effect is to decrease the average recurrence interval of 1886-sized earthquakes from what the interval would be if events C and D were two moderate earthquakes. Thus, combining C and D is conservative with respect to seismic hazard.

Charleston Seismic Source Zone Geometries. For its update of the Charleston seismic source zone, the applicant developed new source zone boundaries. Specifically, as described in SSAR Section 2.5.2.2.4, the applicant developed four, mutually exclusive source zone geometries, referred to as A, B, B', and C, to represent the Charleston seismic source. These four source zones are shown in SER Figure 2.5.2-2 (reproduced from SSAR Figure 2.5.2-9). SSAR Section 2.5.2.2.2.4.1 states that the width of Geometry B is 80 kilometers (50 miles). However, SSAR Figure 2.5.2-9 (and SER Figure 2.5.2-2) show that the width of Geometry B is

100 kilometers (62 miles). In RAI 2.5.2-14, the staff asked the applicant to provide the actual dimensions of Geometry B used for the UCSS. In response, the applicant stated that the width of UCSS Geometry B is 100 kilometers and not 80 kilometers, as stated in SSAR Section 2.5.2.2.2.4.1. Based on the applicant's clarification of the width of source zone B, the staff concludes that the source referred to as Geometry B in SSAR Figure 2.5.2-9 is accurate.

SSAR Section 2.5.2.4.4 states that "the new interpretation of the Charleston source indicates that a source of the large earthquakes in the Charleston area exists with weight 1.0...." Although the UCSS update of the Charleston source zone covers a fairly large area, the weighting and source geometries give the largest hazard only inside Zone A (either 0.9 (A, B, B') or 1.0 (A, B, B', C)), which is a relatively small zone. In view of this result, the staff asked the applicant, in RAI 2.5.2-13, to provide justification for the UCSS source geometries and weighting scheme and define what is meant by the "Charleston area." In its response, the applicant concluded that the Charleston source area is "stationary in space and is confined to a relatively restricted area," which it referred to as Geometry A. The applicant provided the following information to support its conclusion that the source area that produced 1886 Charleston-type large magnitude earthquakes is likely relatively restricted in area:

> The updated Charleston seismic source model includes four potential geometries (A, B, B', and C) to represent the source area for the Charleston seismic source zone. The greatest weight is given to a localized zone (Geometry A) that completely incorporates the 1886 earthquake Modified Mercalli Intensity (MMI) X isoseismal (Bollinger 1977), the majority of identified Charleston meizoseismal-area tectonic features and inferred fault intersections, and the majority of reported 1886 liquefaction features. Outlying liquefaction features are excluded because liquefaction occurs as a result of strong ground shaking that may extend well beyond the areal extent of the tectonic source. Data describing the size and spatial distribution of paleoliquefaction features suggest prehistoric earthquakes (Events A, B, C', E, and F') were of similar magnitude and location to the 1886 Charleston earthquake, which produced liquefaction at significant distances northeast and southwest from the meizoseismal area. Lower weights are given for source geometries that envelop specific postulated tectonic features (i.e., Geometry C for the southern segment of the East Coast fault system), or for broader areal distributions that also envelop the localized zone to allow for greater uncertainty in the location and lateral extent of a fault that may have produced the 1886 Charleston earthquake.

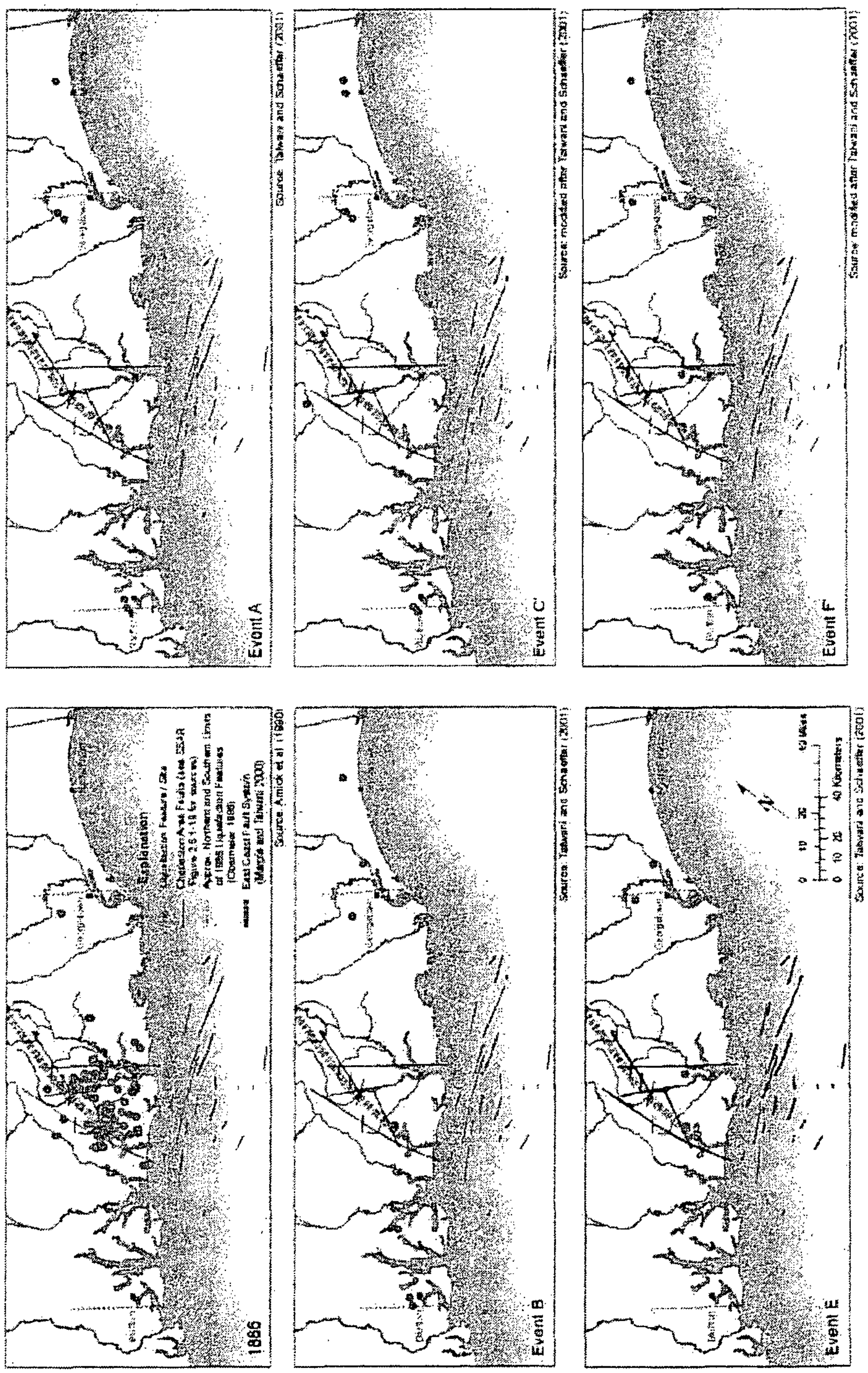


Figure 2.5.2-14 – Geographic Distribution of Liquefaction Features Associated with Charleston Earthquakes (SSAR Figure 2.5.2-12a)

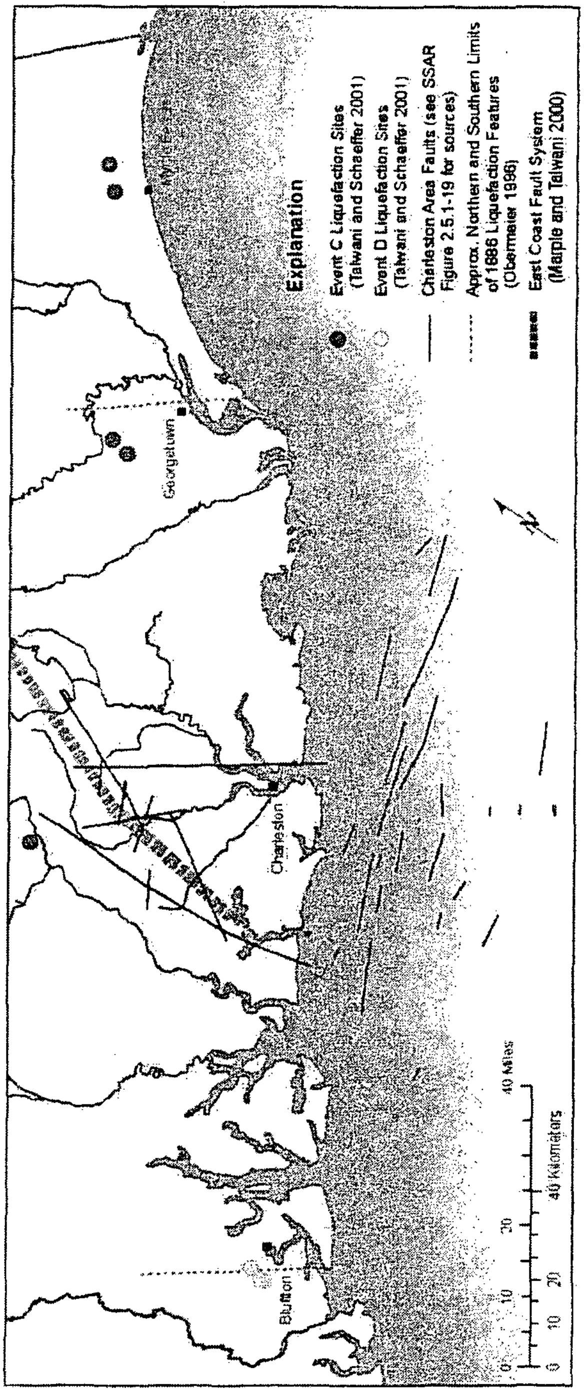


Figure 2.5.2-15 – Liquefaction Sites for Events C, C, and D (Applicant Response to RAI 2.5.2-11, Figure 2.5.2-11)

The applicant provided the following revision for the term “Charleston area” as used in the third sentence of the first paragraph of SSAR Section 2.5.2.4.4:

> The new interpretation of the Charleston source (see Section 2.5.2.2.2) indicates that a unique source of large earthquakes exists with weight 1.0 and that large magnitude events occur with a rate of occurrence unrelated to the rate of smaller magnitudes.

The applicant's response states that the SSHAC Level 2 TI concluded that the Charleston source area is stationary in space and is confined to a relatively restricted area. Geometry A represents the preferred small source area and it is given a high weight of 0.7 (SSAR 2.5.2.2.2.4.1). According to the applicant geometry A is based on (1) the 1886 meizoseismal area and greatest density of liquefaction features; (2) the concentration of known and hypothesized tectonic features, mainly faults; (3) the concentration of historical seismicity, chiefly in the Middleton Place-Summerville seismic zone; and (4) the greatest density of prehistoric liquefaction features.

The staff focused its review on the density of prehistoric liquefaction features in relation to Geometry A because the use of a small source area to represent the sources of the 1886 and all previous large earthquakes depends crucially on a demonstration that the largest liquefaction craters of all ages concentrate near Charleston. The staff also reviewed the information presented in Bechtel (2006). Bechtel (2006) briefly references recent studies regarding the geographic distribution, density, and size of liquefaction features produced by the 1886 and prehistoric earthquakes in the Charleston region, specifically Obermeier et al. (1989, 1990, 2001) and Amick et al. (1990).

The staff also reviewed the study of Obermeier et al. (1989). Obermeier et al. (1989) conclude that, "Both the size and relative abundance of pre-1886 craters are greater in the vicinity of Charleston (particularly in the 1886 meizoseismal zone) than elsewhere, even though the susceptibility to earthquake-induced liquefaction is approximately the same at many places throughout this coastal region." Figure 4 of Obermeier et al. (1989), reproduced as SER Figure 2.5.2-16, depicts the sizes of various prehistoric liquefaction features and demonstrates that the largest craters of all ages concentrate near Charleston. The staff notes that the figure cannot exclude the possibility that one (or more) of the large prehistoric earthquakes created its (or their) largest liquefaction features elsewhere. However, Obermeier's (1989) figure shows four size classes of craters, with the largest prehistoric craters (wider than 3 meters) present only in the 1886 meizoseismal area. Only smaller craters are known farther south and north. Obermeier (1989) favors attributing some of these distant, small-to-medium-sized craters to infrequent moderate earthquakes at two separate sources far north and south of Charleston. The epicentral regions of 1886-sized earthquakes should have abundant craters wider than 3 meters, and they have been found only near Charleston. Sparse exposures preclude saying much about crater sizes between Beaufort and the Edisto River, south of Charleston (Obermeier et al. 1989) and south of Geometry A. Thus, it is unlikely, but possible, that the paleoliquefaction record of a large earthquake's meizoseismal region could be concealed south of Geometry A. However, this small probability is accounted for by Geometries B and B', which span most of the length of South Carolina's coast. The absence of known abundant paleoliquefaction features in North Carolina and Georgia, despite searches there (Amick and Gelinas 1991), suggests that Geometries B and B' need not extend beyond South Carolina.

Accordingly, the staff concludes that the applicant's use of a small area to represent the sources of the 1886 and all previous large earthquakes is adequate. Available evidence suggests it is likely that 1886-sized earthquakes occurred mostly or entirely within a small area like Geometry A. Evidence provided by the applicant in response to previous Open Item 2.5-5, further supports a localized source contained within Geometry A.

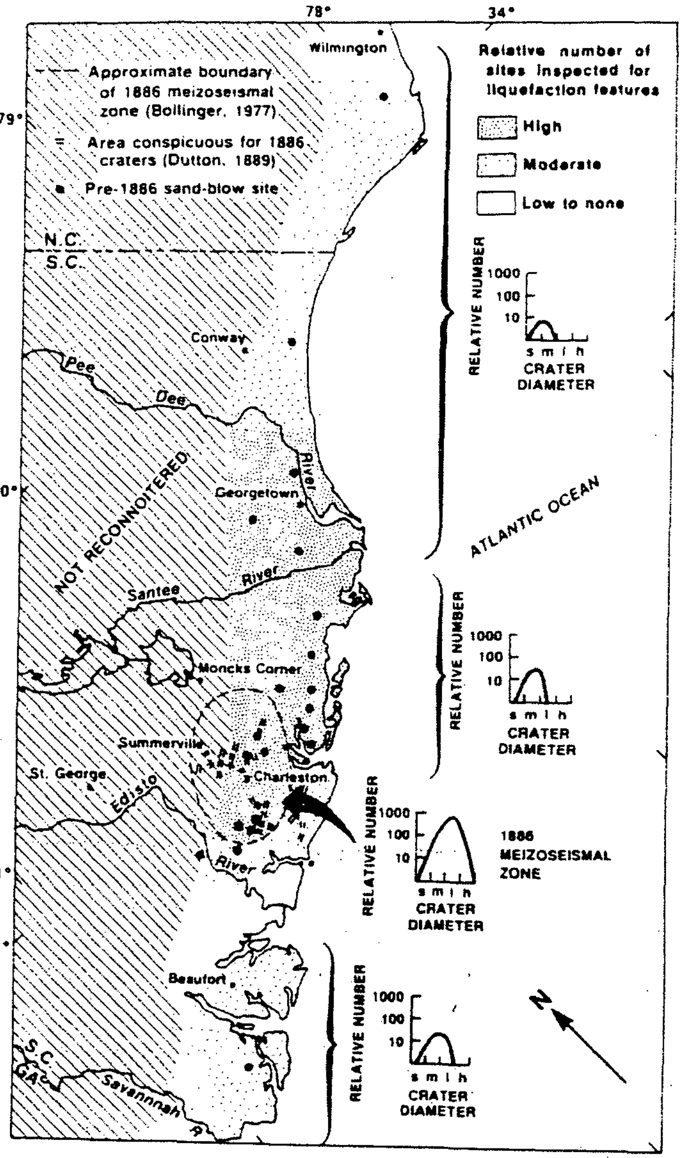


Figure 2.5.2-16 - Relative number of filled craters and crater diameters for pre-1886 sand blows at sites on marine-related sediments. The relative number is a scaling based on comparison with the abundance of craters in the 1886 meizoseismal zone, which has an arbitrary value of 1000. Crater diameters are small (s, less than 1 m), medium (m, 1–2 m), large (l, greater than 3 m) (reproduced from Obermeier et al. 1989).

Offshore of the South Carolina coast in the Charleston area there are several smaller faults (SER Figure 2.5.2-2). These faults correspond to the Helena Banks fault zone. In SSAR Section 2.5.2.2.2.4.1, the applicant concluded that, although the Helena Banks fault zone is clearly shown by multiple seismic reflection profiles and has demonstrable Late Miocene offset (Behrendt and Yuan 1987), there is no evidence to demonstrate the activity of this fault zone. In RAI 2.5.2-15, the staff asked the applicant to explain why the two seismic events (mb 3.5 and 4.4) in 2002, which occurred in the vicinity of the Helena Bank fault zone, cannot be positively correlated with the fault zone. The association of these two events with the Helena Banks fault zone would indicate that this fault zone is currently active. In response, the applicant stated that it could not positively correlate the two earthquakes with the Helena Banks fault zone for the following reasons:

The lack of detailed information on these two 2002 offshore earthquakes (poor location, no focal mechanisms) and the lack of additional seismic activity in this offshore area, make it difficult to assign the Helena Banks fault zone as the causative fault. It is possible that the two 2002 earthquakes indicate reactivation of the Helena Banks fault zone, but the fact that these events cannot be positively correlated to the fault suggests otherwise. There are numerous faults in the central and eastern United States located close to a few or more poorly located, small earthquakes, but this simple and very limited spatial association has not typically led researchers to positively correlate them to specific faults and classify these faults as reactivated seismogenic structures.

Based on its review of the applicant's response to RAI 2.5.2-15, the staff concurs with the applicant's conclusion that it could not positively correlate the recent offshore earthquakes with the Helena Banks fault zone because of the uncertainties regarding the exact locations of these two events. However, even though these two events cannot be directly correlated with the Helena Banks fault zone, the applicant's UCSS source zone Geometry B encompasses both the Helena Banks fault zone and the epicenters of these two events.

<u>Recurrence intervals for the Charleston seismic source</u>. In SSAR Section 2.5.2.2.2.4.3, the applicant describes its calculation of recurrence intervals for the updated Charleston seismic source, which is largely based on paleoliquefaction data compiled by Talwani and Schaeffer (2001). The applicant calculated two different average recurrence intervals, which represent two recurrence branches on the logic tree. The first average recurrence interval is based on the four events (1886, A, B, and C') that the applicant interpreted to have occurred within the past ~2000 years. The applicant considered this time period to represent a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of its expert elicitation. This branch of the logic tree was given a weight of 0.8. The applicant's second average recurrence interval is based on events that the applicant interpreted to have occurred within the past ~5000 years and includes events 1886, A, B, C', E, and F'. This time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). Published papers and researchers questioned by the applicant suggest that the older part of the record (i.e., older than ~2000 years) may be incomplete. The applicant noted, however, that it may also be possible that the older record is complete and exhibits longer inter-event times. For this reason, the average recurrence interval calculated for the ~5000-yr record (six events) is given a weight of 0.20 on the logic tree.

In RAI 2.5.2-12, the staff asked the applicant to provide more detail regarding its rationale for the weighting of the two recurrence branches on the logic tree. The staff also asked the applicant to justify its use of these two scenarios rather than another case study (e.g., 10 large-magnitude earthquakes occurring at approximately regular intervals during the past 5000 years), including its impact on the hazard calculation. The applicant provided the following response to justify its weighting of the 2000-yr and 5000-yr logic tree branches:

> The relative weighting of these two branches of the logic tree is based on a SSHAC level 2 assessment of completeness of the geologic record of paleoliquefaction events over these two time intervals. Earthquakes in the paleoliquefaction record do not occur at regular intervals, and this may be the result of "temporal clustering of seismicity, fluctuation of water levels, or their

evidence having been obliterated" (Talwani and Schaeffer 2001; p. 6640). Talwani and Schaeffer (2001) consider the paleoliquefaction record to be complete for the past 2,000 yrs. Moreover, Prof. Pradeep Talwani (University of South Carolina, pers. comm. 9/8/05) and Dr. Steve Obermeier (U.S. Geological Survey [retired], pers. comm. 9/2/05) consider the 2,000-yr record to represent a complete portion of the paleoseismic record. For these reasons, the average recurrence interval calculated for the most-recent ~2,000 yr portion of the paleoseismologic record is given a relatively high weight of 0.80.

The degree of completeness for the entire ~5,000-yr record of paleoliquefaction events is uncertain. It is possible that all paleoliquefaction events in this time period have been preserved and recognized in the geologic record. Alternatively, it is possible that events are missing from the ~5,000-yr record. Average M_{max} recurrence interval calculated from the entire ~5,000-yr record is greater (i.e., larger average interevent time) than that calculated for the ~2,000-yr record. The decision to give less weight (0.20) to this recurrence estimate is therefore conservative.

Regarding its use of these two scenarios rather than another case study (e.g., 10 large-magnitude earthquakes occurring at approximately regular intervals during the past 5000 years), the applicant stated the following:

> We also considered other scenarios from which to calculate earthquake recurrence, but ultimately decided not to incorporate those that included non-conservative assumptions. For example, Talwani and Schaeffer (2001) include a scenario in which their events C and D are moderate-magnitude, local earthquakes. These moderate-magnitude earthquakes would be eliminated from the record of large (M_{max}) earthquakes, thereby increasing the calculated recurrence interval. This and other permutations of the paleoliquefaction record (and resulting recurrence intervals) could be included, but, if based on nonconservative assumptions, would increase the recurrence interval and lower the hazard without sufficient justification. The given example of "ten large-magnitude earthquakes occurring at approximately regular intervals during the past 5,000 years" was not included in the model because: (1) it is permissible only if events are assumed to be missing from the geologic record; and (2) the resulting recurrence interval would be very similar to the branch of the logic tree using the ~2,000-yr paleoliquefaction record.

In summary, the applicant assigned the largest weight of 0.8 to the average recurrence interval calculated for the most recent ~2000-yr portion of the paleoseismologic record. The applicant considered this time period to represent a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of the expert elicitation. The applicant stated that the 5000-yr time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). However, the applicant only assigned a weight of 0.2 to the 5000-yr branch of the logic tree because the completeness of the ~5000-yr paleoseismic record is uncertain.

Based on its review of the applicant's response to RAI 2.5.2-12, and the information presented by the applicant in SSAR Section 2.5.2.2, the staff concurs with the applicant's logic tree weighting for earthquake recurrence because it reflects all of the available data and uncertainties. Specifically, the applicant assigned the largest weight of 0.8 to the 2000-yr logic tree branch because there is a greater certainty that this portion of the paleoseismologic record is complete. The applicant also used the entire ~5000-yr record to calculate earthquake recurrence. The applicant calculated a recurrence interval of 958 years from the ~5000-yr record. This value is less conservative than the mean recurrence interval of 548 years calculated from the ~2000-yr record. However, the applicant assigned a significantly lower weight of 0.2 to this logic tree branch because there is a greater uncertainty that the ~5000-yr record is complete.

In summary, the staff focused its review of SSAR Section 2.5.2.2 on the applicant's update of the Charleston seismic source model and its basis for not updating the other EPRI seismic source zones that contribute to the seismic hazard at the ESP site. The staff concludes that the applicant's update of the 1986 EPRI PSHA sources adequately characterizes the seismic hazard in the region surrounding the site.

2.5.2.4.3 Correlation of Earthquake Activity with Seismic Sources

SSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI seismic source model. The applicant compared the distribution of earthquake epicenters from both the original EPRI historical catalog (1627–1984) and the updated seismicity catalog (1985–2005) with the seismic sources characterized by each of the EPRI ESTs. The applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure and that there are no clusters of seismicity suggesting a new seismic source not captured by the EPRI seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require significant revision to the geometry of any of the EPRI seismic sources. The applicant further concluded that the updated catalog does not show or suggest an increase in M_{max} or a significant change in seismicity parameters (activity rate, b-value) for any of the EPRI seismic sources. The applicant based its conclusions on a comparison of the distribution of earthquake epicenters from both the original EPRI historical catalog and from its updated seismicity catalog with the seismic sources characterized by each of the EPRI ESTs.

In Parts A and B of RAI 2.5.2-1, the staff requested electronic versions of the EPRI seismicity catalog and the applicant's updated EPRI seismicity catalog for the region of interest. In Part C of RAI 2.5.2-1, the staff requested the geographic coordinates of the primary source zones developed by each of the six EPRI ESTs. The staff used the information provided in response to Parts A and B of RAI 2.5.2-1 to compare the applicant's update of the regional seismicity catalog with its own listing of recent earthquakes. Based on this comparison, the staff concurs with the applicant's assertion that the rate of seismic activity has not increased in the ESP region since 1985. Using the information provided in response to Part C of RAI 2.5.2-1, the staff compared the updated earthquake catalog with each of the primary seismic sources developed by each EPRI EST. Based on the comparision of earthquakes in the updated catalog with each of the EPRI EST seismic sources, the staff concurs with the applicant's conclusion that revisions to the existing EPRI sources are not warranted. However, additional worldwide earthquake data may indicate the need for an update of some of the EPRI seismic source models. In addition, recent paleoliquefaction studies predict shorter recurrence intervals for large Charleston-type earthquakes compared to predictions based on the historical seismicity catalog. These paleoliquefaction data also provide information regarding the locations of large prehistoric Charleston-type earthquakes. SER Section 2.5.2.3.2 describes the staff's conclusions with respect to the applicant's update of the Charleston seismic source.

2.5.2.4.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

SSAR Section 2.5.2.4 presents the earthquake potential for the ESP site in terms of the controlling earthquakes. The applicant determined the high- and low-frequency controlling earthquakes by deaggregating the PSHA results at selected probability levels. Before determining the controlling earthquakes, the applicant updated the 1989 EPRI PSHA using the seismic source zone adjustments described in SER Section 2.5.2.1.2 and the new ground motion models described in SER Section 2.5.2.1.4.

The staff focused its review of SSAR Section 2.5.2.4 on the applicant's updated PSHA and the ESP site controlling earthquakes determined by the applicant after completion of its PSHA. While the staff's review of the applicant's update of the EPRI seismic source model is described in SER Section 2.5.2.3.2, this SER section focuses on the review of the application of the updated seismic source model to the hazard calculation at the ESP site.

PSHA Inputs

As input to its PSHA, the applicant used its updated version of the 1989 EPRI seismic source model. The staff's evaluation of the applicant's update is described in SER Section 2.5.2.3.2. The applicant also used the ground motion models developed by the 2004 EPRI-sponsored study (EPRI 1009684 2004) as input to its PSHA. The ESP applications for the Clinton (Illinois), Grand Gulf (Mississippi) and North Anna (Virginia) sites also used the updated EPRI ground motion models. The staff's final SERs for Clinton (ADAMS Accession No. ML0612204890), Grand Gulf (ADAMS Accession No. ML061070443), and North Anna (ADAMS Accession No. ML063170371) provide an extensive review of the EPRI 2004 ground motion models. Thus, the staff considers the applicant's use of the EPRI 2004 ground motion model to be appropriate.

PSHA Results

In order to determine the adequacy of the PSHA results, the staff, in RAI 2.5.2-1, requested that the applicant to provide the 1- and 10-Hz mean hazard curves for each of the six EPRI ESTs, as well as the 1- and 10-Hz mean hazard curves for the UCSS model. In response to RAI 2.5.2-1, the applicant provided the requested hazard curves. SER Figures 2.5.2-17 and 2.5.2-18 show the applicant's 1-Hz and 10-Hz total mean hazard curves, as well as the hazard curves corresponding to each of the six EPRI EST seismic source model inputs. Both figures also show the hazard curves corresponding to the applicant's UCSS model.

The total mean hazard curves, shown in SER Figures 2.5.2-17 and 2.5.2-18, comprise the mean of the six EPRI EST total hazard curves plus the contribution of the UCSS.

As shown in SER Figure 2.5.2-17, for the 1-Hz hazard curves, the Charleston source dominates the overall hazard at the ESP site. In SER Figure 2.5.2-18, for the 10-Hz hazard curves, the contributions from each of the six ERPI seismic source models have a more significant contribution to the overall hazard.

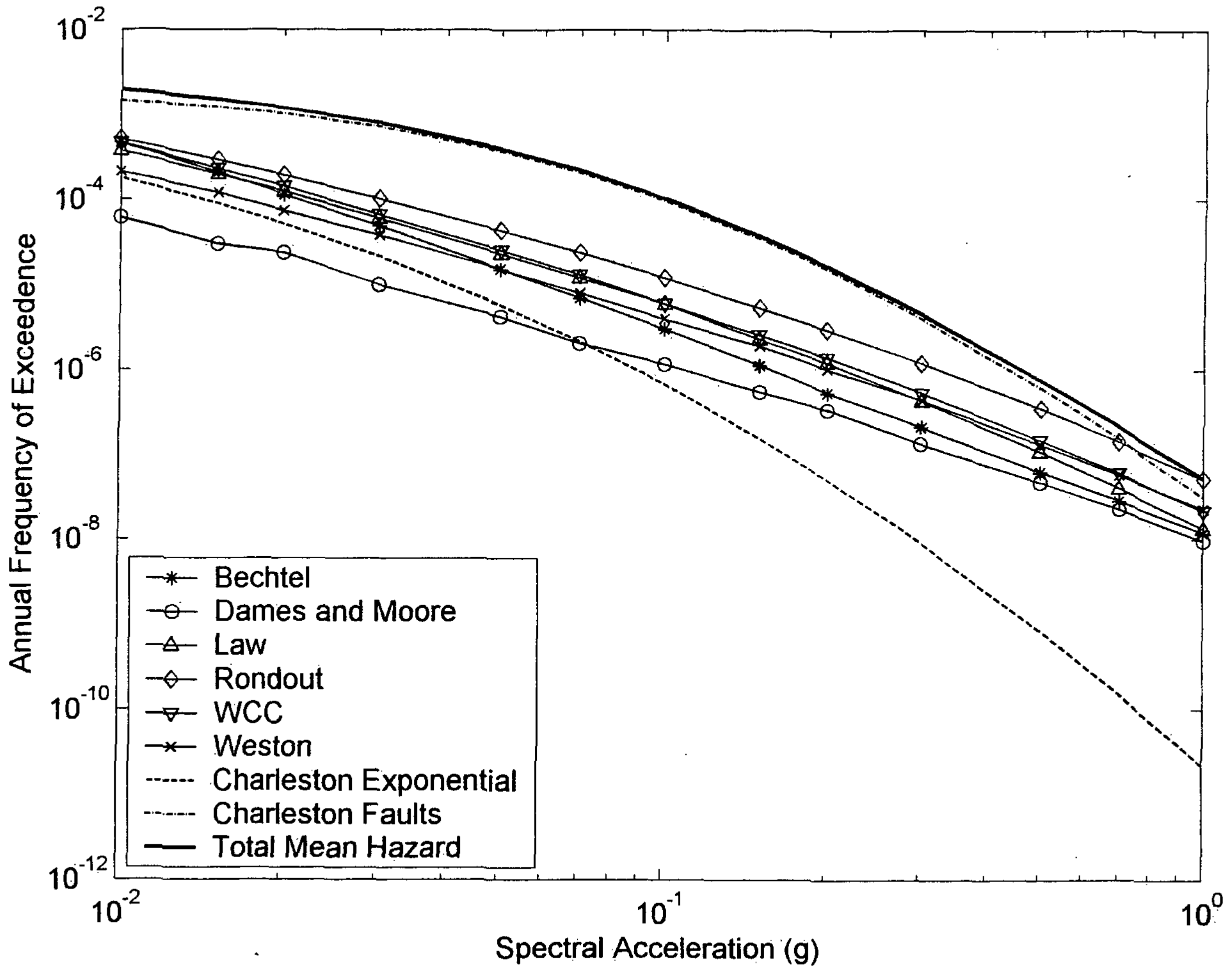


Figure 2.5.2-17 - Plot showing the applicant's 1-Hz total mean hazard curve for the ESP site. This figure also shows the contributions of the applicant's UCSS model, which consists of "Charleston Faults" and "Charleston Exponential," as well as the contributions from each of the six EPRI EST seismic source models.

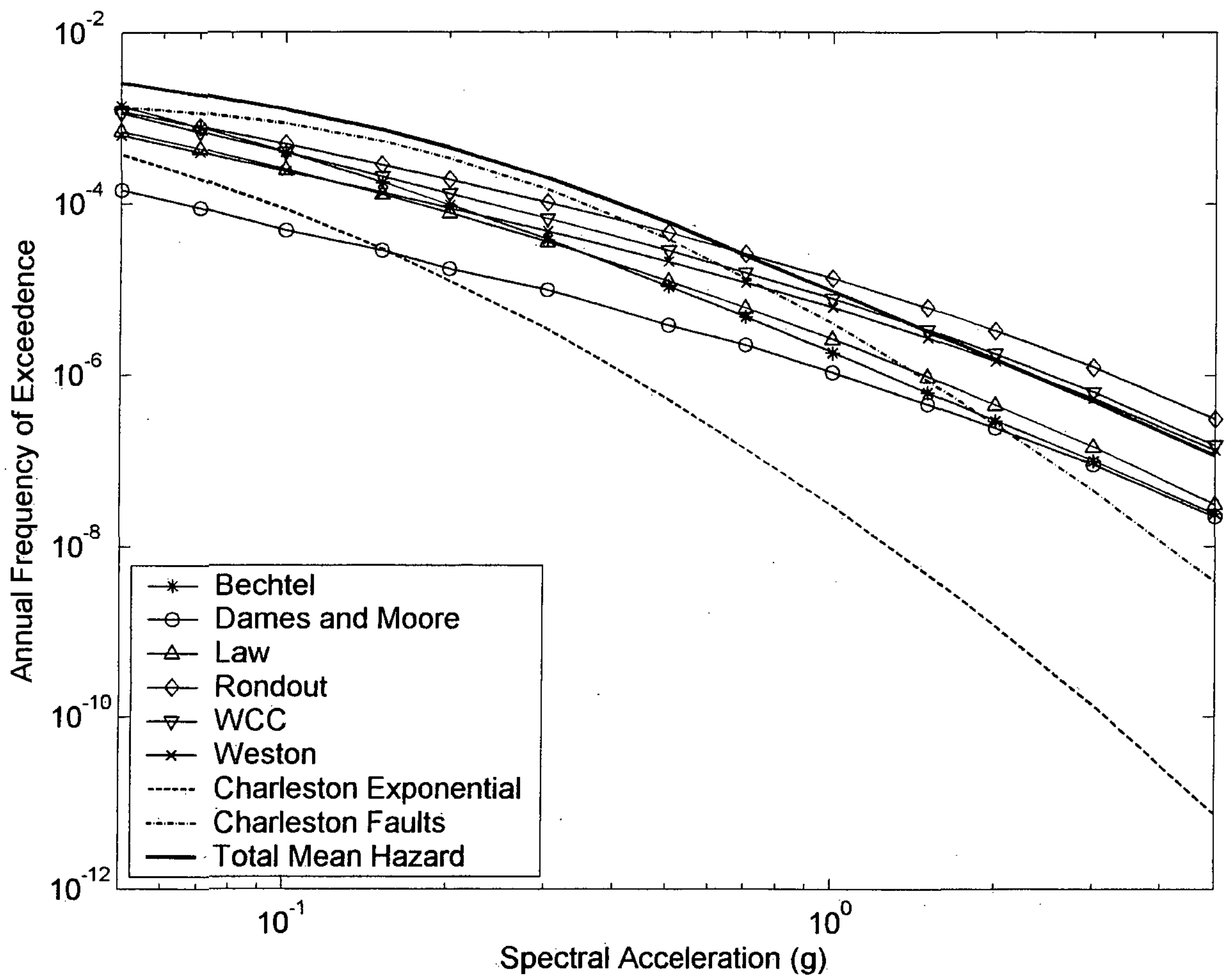


Figure 2.5.2-18 - Plot showing the applicant's 10-Hz total mean hazard curve for the ESP site. This figure also shows the contributions of the applicant's UCSS model, which consists of "Charleston Faults" and "Charleston Exponential," as well as the contributions from each of the six EPRI EST seismic source models.

<u>Controlling Earthquakes</u>. To determine the low- and high-frequency controlling earthquakes for the ESP site, the applicant followed the procedure outlined in Appendix C to RG 1.165. This procedure involves the deaggregation of the PSHA results at a target probability level to determine the controlling earthquakes in terms of magnitude and source-to-site distance. The applicant chose to perform the deaggregation of the mean 10^{-4}, 10^{-5}, and 10^{-6} PSHA results. SER Table 2.5.2-8 shows the low- and high-frequency controlling earthquakes. Because of the similarity of Mbar and Dbar values for the three hazard levels, the applicant selected a single recommended Mbar and Dbar value for each frequency range. For the high-frequency mean 10^{-4} and 10^{-5} and 10^{-6} hazard levels, the controlling earthquake has a magnitude of M 5.6 event occurring at a distance of 9.0 kilometers (5.6 miles), corresponding to an earthquake from a local seismic source zone. In contrast, for the low-frequency mean 10^{-4} and 10^{-5} and 10^{-6} hazard levels, the controlling earthquake has a magnitude of M 7.2 at a distance of 130 kilometers (80.8 miles). This controlling earthquake corresponds to an event in the Charleston seismic source zone.

Table 2.5.2-8 - Computed and Final Mbar and Dbar Values Used for Development of High-and Low-Frequency Target Spectra (Based on Information Provided In SSAR Table 2.5.2-17)

High Frequency (5 to 10 Hz)				
Mean Hazard Level	**10^{-4}**	**10^{-5}**	**10^{-6}**	**Final Values**
Mbar (M)	5.6	5.6	5.7	5.6
Dbar	17.6 km (10.9 mi)	11.4 km (7.1 mi)	9.0 km (5.6 mi)	9.0 km (5.6 mi)
Low Frequency (1 to 2.5 Hz)				
Mean Hazard Level	**10^{-4}**	**10^{-5}**	**10^{-6}**	**Final Values**
Mbar (M)	7.2	7.2	7.2	7.2
Dbar	136.5 km (84.8 mi)	134.3 km (83.5 mi)	133.0 km (82.6)	130 km (80.8 mi)

In RAI 2.5.2-21, the staff asked the applicant to explain how it calculated the final Dbar and Mbar values. In its response to RAI 2.5.2-21, the applicant stated that the final low-frequency distance value of 130 kilometers (80.8 miles) is based on the source-to-site distance for the Charleston source, while the final high-frequency value of 9 kilometers (5.6 miles) is equal to the log-average of the three computed values rounded to the nearest kilometer. The applicant also stated that the final magnitude values for the respective high- and low-frequency cases are equal to the linear average of the three magnitude values rounded to the nearest tenth of a magnitude unit. In addition, the applicant provided a comparison between the high-frequency spectral shape using the final magnitude and distance values and the computed magnitude and distance values. The applicant also provided a comparison between the low-frequency spectral shape using the final magnitude and distance values and the computed magnitude and distance values. Based on its comparison, the applicant concluded that the use of the recommended magnitude and distance values in place of the computed magnitude and distance values for each of the three annual probability levels would not significantly change the results of the site response analysis.

The staff concurs with the applicant's final high- and low-frequency Mbar and Dbar values because these final values, and the corresponding spectral shapes, are very similar to the calculated values for the three annual probability levels.

Based on its review of the ESP site controlling earthquake magnitudes and distances as discussed above, the staff concludes that the applicant's PSHA adequately characterized the overall seismic hazard of the ESP site. The staff also concludes that the applicant's controlling earthquakes for the ESP site (**M** 5.6 at 9 km (5.6 miles), **M** 7.2 at 130 km (80.8 miles)) are generally consistent with both the historical earthquake record and paleoliquefaction studies in the Charleston seismic source zone. In addition, the staff finds that the ground motions developed by the applicant from the controlling earthquakes are consistent with the most recent CEUS ground motion evaluations. Accordingly, the staff concludes that the applicant followed the guidance in RG 1.165 and RG 1.208 for evaluating regional earthquake potential and determining the ground motion resulting from controlling earthquakes.

2.5.2.4.5 Seismic Wave Transmission Characteristics of the Site

SSAR Section 2.5.2.5 describes the method used by the applicant to develop the ESP site free-field ground motion spectrum. The seismic hazard curves generated by the applicant's

PSHA are defined for generic hard rock conditions (characterized by a S-wave velocity of 9200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 2000 feet below the ground surface at the ESP site. To determine the site free-field ground motion, the applicant performed a site response analysis. The output of the applicant's site response analysis is site AFs, which are then used to determine the UHRS for three hazard levels (10^{-4}, 10^{-5}, and 10^{-6}). The 10^{-4} and 10^{-5} UHRS are then used to calculate the GMRS for the site.

In SSAR Section 2.5.2.5.1.1, the applicant describes the methodology it used to develop the soil UHRS for the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. The applicant's site free-field soil UHRS is defined at the top of the Blue Bluff Marl. According to the applicant, the top of the Blue Bluff Marl is characterized by an average S-wave velocity of 2354 ft/s. In RAI 2.5.2-19, the staff asked the applicant to provide a detailed step-by-step description of the methodology it used to develop the site AFs and the 10^{-4} and 10^{-5} soil UHRS. In response to RAI 2.5.2-19, the applicant more completely explained Steps 1 through 6. However, after reviewing the applicant's response, the staff concluded that the applicant's description of Steps 5 and 6 did not provide sufficient detail for the staff to completely evaluate the site response method. In particular, the staff was not clear on the enveloping motion used in Step 5, and the applicant's description in Step 6 appeared to differ from that described in SSAR Section 2.5.2.5.1.1. On June 18, 2007, the applicant supplemented its RAI response with additional detail on each of the steps used in the site response analysis; however, the staff had not been able to completely evaluate the applicant's supplemental information. As such, the staff was not able to reach a conclusion in the SER with open items on the adequacy of the applicant's methodology. Accordingly, in the SER with open items, the staff identified Open Item 2.5-6 to reflect the additional review time needed by the staff to review the applicant's supplemental response to RAI 2.5.2-19, as well as the staff's request for further clarification of Step 6 of the applicant's site response methodology.

Based on the applicant's response to RAI 2.5.2-19 and Open Item 2.5-6, a summary of the applicant's site response methodology is provided below:

The applicant determined the final 10^{-4} soil surface spectrum for the ESP site by scaling the hard rock UHRS (shown in SER Figure 2.5.2-5) by the final AFs (shown in SER Figure 2.5.2-6). The applicant defined each of the AFs at a total of 300 frequencies, but only defined the hard rock UHRS at 7 structural frequencies. For this reason, the applicant interpolated the hard rock UHRS at values between the 7 structural frequencies using the high- and low-frequency spectral shapes (from NUREG/CR-6728) for hard rock. This resulted in two rock spectra: a high-frequency spectrum and a low-frequency spectrum that are both constrained to equal the spectral amplitudes for the 7 PSHA structural frequencies at which the PSHA was calculated. From the high-frequency and low-frequency rock spectra, a single spectrum was then derived using the high-frequency rock spectrum for high frequencies and the low-frequency rock spectrum for low frequencies.

In order to determine the 10^{-4} soil spectrum (UHRS), the applicant multiplied the hard rock UHRS (now defined at 300 structural frequencies) by either the high- or low-frequency final amplification factors, which are shown in SER Figure 2.5.2-6. The applicant multiplied the hard rock UHRS by the high-frequency final amplification factors for frequencies above 8 Hz. For frequencies below 5 Hz, the applicant multiplied the hard rock UHRS by the low-frequency final amplification factors. In between 8 Hz and 5 Hz, the applicant interpolated the soil spectrum to achieve a smooth transition between the high-frequency and low-frequency controlled parts.

The applicant repeated the above process for the 10^{-5} hazard level to determine the final 10^{-5} soil UHRS. SER Figure 2.5.2-7 provides the final soil UHRS for the 10^{-4} and 10^{-5} hazard levels.

Upon completing its review of the supplemental response to RAI 2.5.2-19 as well as the applicant's additional response to Open Item 2.5-6, summarized above, the staff concludes that the applicant provided sufficient information for the staff to perform its review of the methodology. The staff also concludes that the supplemental information is generally consistent with what the applicant provided in SSAR Section 2.5.2.5. Furthermore, the staff concludes that the applicant's site response methodology is adequate because it follows the guidance provided in RG 1.208.

SSAR Section 2.5.2.5.1.3 describes the development of low- and high-frequency target spectra based on the low- and high-frequency controlling earthquake magnitudes and distances. To determine the target low- and high-frequency spectra, the applicant used the average of the single and double corner source models provided in NUREG/CR-6728. In RAI 2.5.2-20, the staff asked the applicant why it did not use the EPRI ground motion models (EPRI 1009684 2004) to develop the high- and low-frequency target response spectra since the applicant used these ground motion models for its PSHA. In response to RAI 2.5.2-20, the applicant provided the following information:

> The 2004 EPRI ground motion report (EPRI 1009684) gives equations to estimate spectral acceleration at 7 structural frequencies (100, 25, 10, 5, 2.5, 1, and 0.5 Hz). To properly represent rock motion for input to a site response analysis, it is necessary to interpolate between these 7 structural frequencies to obtain a realistic spectral shape, rather than using linear interpolation. For this task, NUREG/CR-6728 was used, because one of its goals was specifically to develop realistic spectral shapes for the eastern U.S. to use in earthquake ground motion analyses.

The staff concurs with the applicant's use of NUREG/CR-6728 spectral models for the CEUS, since the EPRI 2004 ground motion models only provide 7 structural frequencies. Because the applicant used the NUREG/CR-6728 source models, it was able to avoid using linear interpolation and, subsequently, obtained a more accurate estimate of the site response.

A key step in the site response analysis is the selection of actual earthquake records that closely match the low- and high-frequency controlling earthquake magnitude and distance values. The response spectra from these earthquake records, which are generally from the WUS, are matched to the CEUS spectral shapes described in the preceding paragraph. SSAR Section 2.5.2.5.1.4 describes the spectral matching of the selected seed time histories to the target response spectra and states that "the spectral matching criteria given in NUREG/CR-6728 were used to check the average spectrum from the 30 time histories for a given frequency range (high- or low-frequency) and annual probability level. This is the recommended procedure in NUREG/CR-6728 when multiple time histories are being generated and used." In RAI 2.5.2-22, the staff asked the applicant to verify that it satisfied the NUREG/CR-6728 matching criteria for each individual earthquake time history. In response to RAI 2.5.2-22, the applicant pointed out that item (e) of the NUREG/CR-6728 matching criteria provides guidance for the use of a suite of ground motion records as well as for an individual record. In addition, the applicant stated that it matched the other relevant criteria for both the low-frequency and high-frequency spectra. Since the applicant followed the guidance specified in NUREG/CR-6728 for multiple time histories and also matched the other relevant criteria, the staff concludes that the applicant adequately matched the seed time histories to the CEUS spectral shapes.

In addition to the seed time histories, another important part of the site response analysis is the model of the site subsurface soil and rock properties. In particular, the applicant's site response analysis should incorporate the uncertainty in these properties. Key properties include the shear wave velocities, material damping, and the strain-dependent behavior of each of the soil layers underlying the site. To model the strain-dependent behavior of the soil, the applicant used shear modulus and damping curves developed by EPRI (EPRI TR-102293 1993), as well as curves developed for the SRS (Lee 1996). Besides these soil properties, in RAI 2.5.2-23, the staff asked the applicant to discuss results of its site response calculations in terms of the following:

1. the effects of the six alternative site response profiles in terms of the different depths to the top of the Paleozoic crystalline rocks
2. the possible effects of the Pen Branch fault zone (i.e., as a low-velocity zone or weak zone)
3. the effects of the low-velocity zones within the Blue Bluff Marl and Lower Sand Stratum

In response to RAI 2.5.2-23, the applicant performed additional sensitivity calculations to examine the effects of the different depths to the top of the Paleozoic crystalline rocks using the six base case profiles shown in SSAR Table 2.5.4-11, Part B. In order to represent the Pen Branch fault as a low-velocity zone, the applicant modified the rock S-wave velocities of the six base profiles to include a low-velocity zone and to represent the Pen Branch fault. The applicant concluded that the depth to the Pen Branch fault, and a lower velocity layer for the Pen Branch, does not affect the site response. The applicant observed very small differences between the results. Regarding the effects of the low-velocity zones within the Blue Bluff Marl and Lower Sand Stratum, the applicant stated the following:

> The low velocity zones in the Blue Bluff Marl and in the Lower Sand Stratum were incorporated in the site response calculations, i.e., the site response calculation results inherently reflect the inclusion of these low velocity zones. The calculations were performed using the base case shear wave velocity profile that is based on field measurements, and randomized profiles.

The staff reviewed the applicant's response to RAI 2.5.2-23, as well as the results of its sensitivity calculations, and concludes that the applicant adequately captured the site variability in its site response calculations. The applicant generated randomized soil and rock S-wave velocity profiles and randomly paired them with 60 sets of shear modulus degradation and damping curves. According to RG 1.208, the use of 60 randomized profiles is generally adequate to determine a reliable estimate of the mean and standard deviation of the site response.

To determine the adequacy of the applicant's site response calculations, the staff performed its own confirmatory site response calculations. The staff used a site response methodology similar to that used by the applicant and, like the applicant, the staff used the program SHAKE. The main difference between the two sets of calculations is that the staff did not use as many input time histories as the applicant used for its analysis. In addition, the staff did not use randomized soil and rock S-wave velocity profiles, soil shear modulus reduction and damping relationships, and rock damping values. Instead, as inputs to its confirmatory analysis, the staff used the applicant's base case S-wave velocity profiles (given in SSAR Table 2.5.4-11) and shear modulus reduction and damping relationships (given in SSAR Tables 2.5.4-12 and 2.5.4-13).

SER Figures 2.5.2-19 to 2.5.2-22 show the mean AFs resulting from the staff's confirmatory site response calculations. Each figure plots the mean results of the six alternative subsurface profiles for both the EPRI and SRS shear modulus and damping curves. SER Figures 2.5.2-19 and 2.5.2-20 show the results corresponding to the 10^{-4} hazard levels for the respective high- and low-frequency input motions, while SER Figures 2.5.2-21 and 2.5.2-22 plot the results corresponding to the 10^{-5} hazard levels for the respective high- and low-frequency input motions. SER Figures 2.5.2-18 to 2.5.2-22 also show the applicant's mean AFs for comparison. The applicant's results are similar overall. For each case, the amplification peaks are very similar, and in all cases, the peaks occur at approximately 0.6 Hz. The differences between the results are likely due to the greater variability that the applicant incorporated into its model through the use of randomized profiles and material properties, as well as the use of multiple time histories. This variability is illustrated in SER Figure 2.5.2-23 (reproduced from SSAR Figure 2.5.2-37). As a result of its analysis, the staff was able to confirm the applicant's overall site response results.

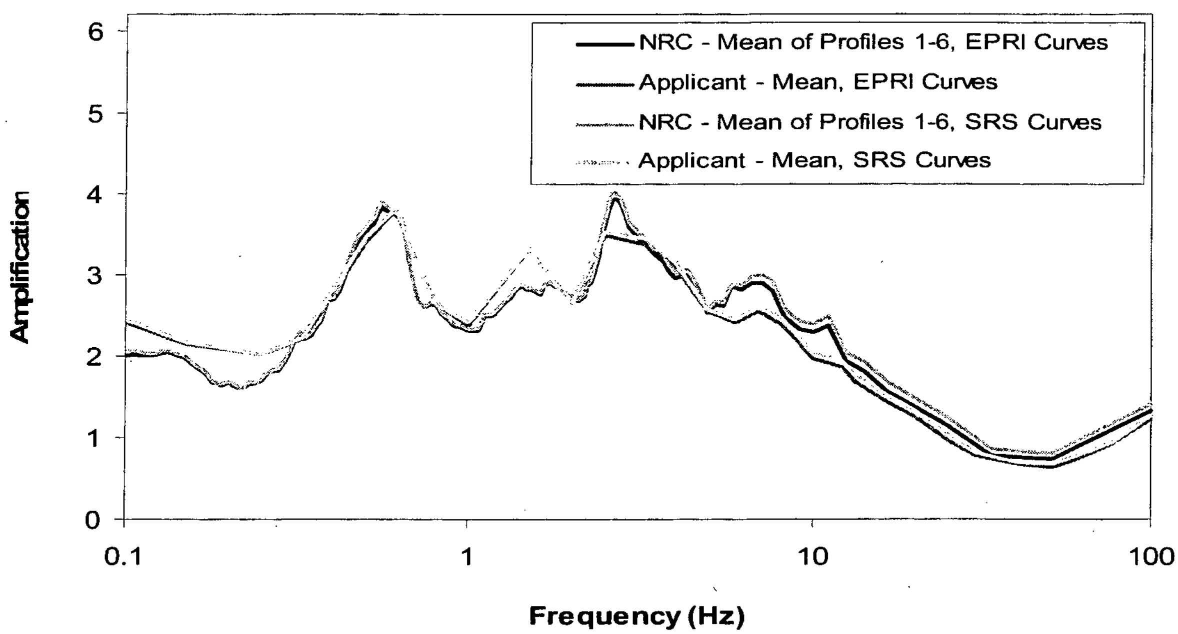


Figure 2.5.2-19 - Results of the staff's site response calculations for high-frequency rock motions for the 10^{-4} hazard level. The applicant's mean results are shown for comparison.

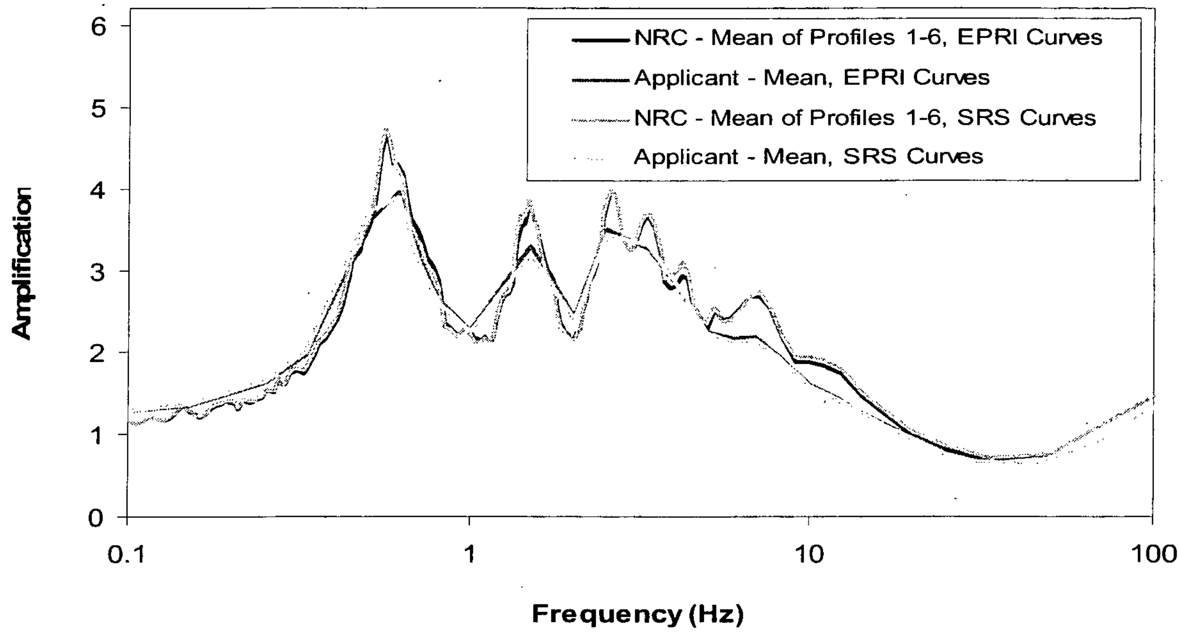


Figure 2.5.2-20 - Results of the staff's site response calculations for low-frequency rock motions for the 10^{-4} hazard level. The applicant's mean results are shown for comparison.

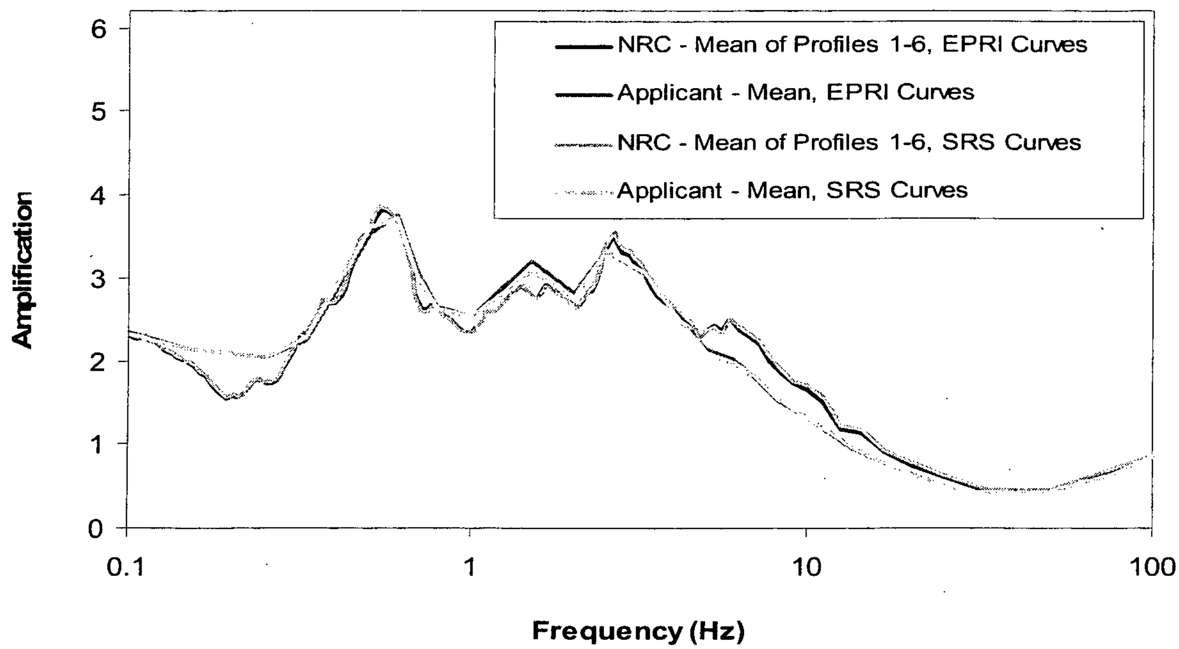


Figure 2.5.2-21 - Results of the staff's site response calculations for high-frequency rock motions for the 10^{-5} hazard level. The applicant's mean results are shown for comparison.

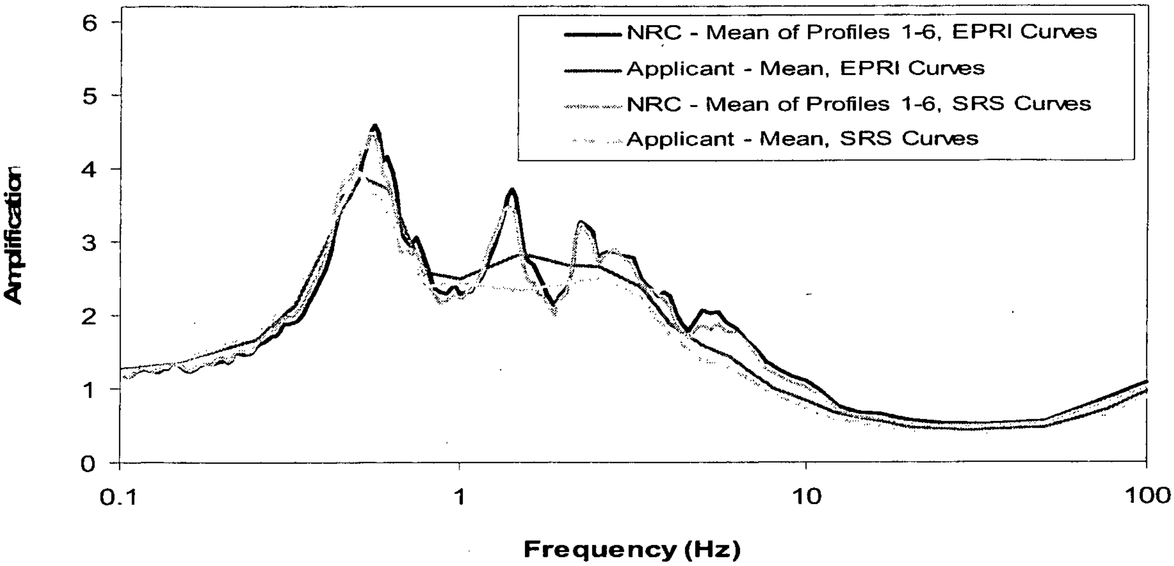


Figure 2.5.2-22 - Results of the staff's site response calculations for low-frequency rock motions for the 10^{-5} hazard level. The applicant's mean results are shown for comparison.

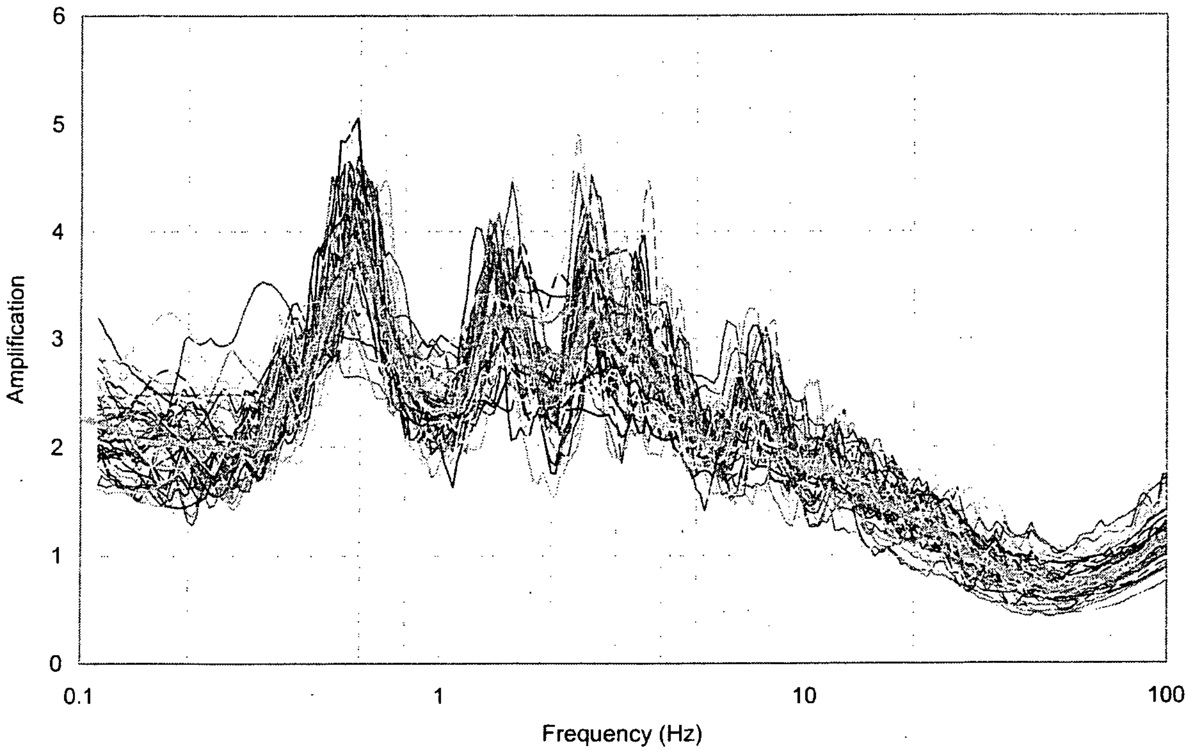


Figure 2.5.2-23 - Results of the applicant's site response calculations for high-frequency rock motions for the 10^{-4} hazard level using the EPRI degradation curves (reproduced from SSAR Figure 2.5.2-37).

In RAI 2.5.2-23, the staff asked the applicant to justify its use of an equivalent-linear approach rather than a nonlinear approach to model the soil nonlinearity at the ESP site. In response, the applicant provided a table containing the maximum shear strains obtained from its SHAKE analyses of the randomized profiles. The applicant's table is reproduced as SER Table 2.5.2-9. In reference to SER Table 2.5.2-9, the applicant stated, "The table shows that the maximum soil strain remained below 0.6 percent. The equivalent-linear approach is adequate for this low level of soil strain."

Table 2.5.2-9 - Applicant's Maximum Shear Strain Values Provided In Response to RAI 2.5.2-23

Earthquake Probability Level	EPRI Randomized Profiles		SRS Randomized Profiles	
	Low-Frequency Earthquake	High-Frequency Earthquake	Low-Frequency Earthquake	High-Frequency Earthquake
10^{-4}	0.078 percent	0.067 percent	0.082 percent	0.068 percent
10^{-5}	0.592 percent	0.300 percent	0.287 percent	0.353 percent

The staff believed that further justification was necessary in order for it to concur with the applicant's assertion that the equivalent-linear approach is suitable for strain levels as high as those for the 10^{-5} probability level. The equivalent-linear modeling approach produces a systematic shift in resonance peaks toward lower frequencies as the level of strain increases and also may predict a more dramatic reduction in AFs at higher frequencies. Accordingly, in the SER with open items, the staff identified Open Item 2.5-7, which requested that the applicant provide further justification for its claim that the equivalent-linear approach is suitable for higher strain levels.

In response to Open Item 2.5-7, the applicant referred to the 1993 EPRI study (EPRI TR102293), which presents a comprehensive study comparing the equivalent-linear method with nonlinear methods for seismic site response analysis. The applicant stated that the study involved a comparison using the equivalent-linear method using RASCAL/SHAKE and nonlinear methods with the programs SUMDES and TESS for three sites (Gilroy 2, Treasure Island, and Lotung, Taiwan). The study compared the actual recorded motion at each of the three sites with the solution from each method of analysis. The sites included soil layers ranging from sands and gravels to soft silts and stiff clays and had both high- and low- strain ground motion recordings. A comparison of the results showed reasonably good agreement between the different methods. In addition, the study analyzed higher ground motions (maximum input accelerations ranged from 0.5 g to 1.25 g) using a generic soil profile for Eastern North America using the same three programs. The applicant noted that the study also confirmed that the amplification factors obtained from the equivalent-linear method are in general agreement with those of the fully nonlinear methods. Furthermore, according to the EPRI study, the predicted peaks at the resonance frequency tend to be conservative using the equivalent-linear method.

With respect to the Vogtle site, the applicant stated that "the input motion is low compared to the range of motions used in the EPRI study and the site is generally stiffer. Therefore, the conclusion of the EPRI study applies, confirming the equivalent-linear method is adequate for the site response analysis at the Vogtle site."

The staff concludes that the applicant, in its response to Open Item 2.5-7, provided an adequate justification for using the equivalent-linear approach to perform site response calculations for the Vogtle ESP site. The applicant referred to the 1993 EPRI study (EPRI TR-102293), which showed that equivalent-linear method is in general agreement with fully nonlinear methods for the case studies considered. The EPRI study is also applicable to the Vogtle site because the study considered a generic soil profile for Eastern North America. In addition, the maximum input peak accelerations ranged from 0.5 g to 1.25 g, which are larger than the expected ground motions at the Vogtle site. Futhermore, since the expected ground motions at the Vogtle site are less than, and the soil profile is generally stiffer than, the soil profiles considered in the EPRI study, the resulting soil nonlinearity is expected to be less at the Vogtle site.

In addition to Open Items 2.5-6 and 2.5-7, the staff noted in the SER with open items that the applicant did not perform any laboratory dynamic testing of the ESP soils, as specified in RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 2, issued December 2003. Instead, as inputs to its site response calculations, the applicant relied on the EPRI and SRS shear modulus degradation and damping curves and assigned equal weights to the results for both sets of curves. This issue is discussed in greater detail in SER Section 2.5.4.4. Accordingly, in the SER with open items, the staff identified Open Item 2.5-19, in which the staff requested that the applicant justify its use of the EPRI and SRS shear modulus and damping curves in the absence of any dynamic testing of the ESP soils. In response to Open Item 2.5-19, the applicant submitted this information as Revision 4 of the SSAR. As part of its COL site investigation, the applicant developed site-specific strain-dependent shear modulus and damping relationships based on RCTS test results (performed on compacted backfill, Blue Bluff Marl, and Lower Sand samples), which are described in SSAR Section 2.5.4.7.5. Rather than recalculating site amplification factors using the site-specific strain-dependent shear modulus reduction and damping relationships, the applicant performed site response sensitivity calculations for a select number of cases in order to demonstrate that use of the SRS and generic EPRI strain-dependent shear modulus and damping curves are appropriate. The results of the applicant's sensitivity calculations are described in SSAR Section 2.5.2.9.3. The applicant evaluated the effects of the additional COL S-wave velocity and the strain dependent shear modulus and damping relationships based on RCTS test results, and compared these results to similar calculations performed using only ESP S-wave velocity data as well as the EPRI and SRS shear modulus degradation and damping curves. SER Figure 2.5.2-10 shows the applicant's results. The applicant concluded that the difference in amplification between the ESP and COL data is small.

In SSAR Section 2.5.2.9, the applicant conducted three sets of sensitivity calculations in order to evaluate: (1) the sensitivity of the AP1000 nuclear island responses to changes in the backfill S-wave velocity; (2) the effects of the backfill geometry on the site response and on the SSI response of the Nuclear Island; and (3) the effects of additional COL data on site response. In SER Section 2.5.2, the staff focused its review on the applicant's evaluation of the effects of the additional COL data on site response, which is described in SSAR Section 2.5.2.9.3. The staff reviewed the applicant's calculations to evaluate the sensitivity of the AP1000 nuclear island responses to changes in backfill S-wave velocity and the effects of the backfill geometry on the site response and on the SSI response of the Nuclear Island as part of SER Section 3.8.5.

The staff reviewed the results of the applicant's site response sensitivity calculations described in SSAR Section 2.5.2.9.3 and agrees with the applicant's conclusion that the differences between the applicant's original analysis using the ESP data and its analysis incorporating the additional COL data are insignificant. Thus, the staff concludes that the applicant's use of the

SRS and generic EPRI strain-dependent shear modulus and damping curves is appropriate. Therefore, the staff considers Open Item 2.5-19 to be resolved.

For the reasons stated above, the staff concludes that, overall, the applicant's site response methodology and results are acceptable. The applicant followed the general guidance provided in RG 1.208, and the results of the confirmatory site response calculations performed by the staff are similar to the applicant's results.

2.5.2.4.6 Ground Motion Response Spectra

SSAR Section 2.5.2.6 describes the method used by the applicant to develop the horizontal and vertical site-specific GMRS. To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and ASCE/SEI Standard 43-05. The applicant developed the vertical GMRS by applying V/H ratios to the horizontal GMRS. The applicant based these V/H ratios on the information provided in NUREG/CR-6728 and Lee (2001).

Following the guidance in RG 1.208, the staff has recently adopted new terminology to differentiate between the different types of site and design ground motion response spectra. The staff now refers to the performance-based SSE as the site-specific GMRS. The GMRS represents the first part of the development of the SSE for a site as a characterization of the regional and local seismic hazard and must satisfy the requirements of 10 CFR 100.23. In accordance with Appendix S to 10 CFR Part 50, during the combined license phase, an additional check of the ground motion is required at the foundation level. Specifically, Appendix S to 10 CFR Part 50 states that the free-field foundation level ground motion must be represented by an appropriate response spectrum with a peak acceleration of at least 0.1 g. The GMRS becomes the site SSE if it exceeds the minimum requirements of Appendix S to 10 CFR Part 50. Otherwise, if any portion of the GMRS falls below the minimum response spectrum, then the site SSE becomes the ground motion spectrum that envelops the GMRS and the minimum response spectrum. As such, the final SSE must satisfy the requirements of both 10 CFR 100.23 and Appendix S to 10 CFR Part 50.

The staff reviewed the applicant's GMRS in terms of meeting the requirements of 10 CFR 100.23 with respect to the development of the SSE.

Horizontal GMRS

The ESP applicant for the Clinton, Illinois, site also used the performance-based approach to determine the horizontal GMRS. The staff's final SER for Clinton (ADAMS Accession No.ML0612204890) provides an extensive review and derivation of the performance-based approach. As described in RG 1.208, the performance-based approach combines a conservative characterization of the ground motion hazard with equipment/structure performance (fragility characteristics) to establish a risk-consistent GMRS. The performance-based GMRS is obtained by modifying the 10^{-4} UHRS at the free-field ground surface by a DF. The resulting GMRS meets the target performance goal of 10^{-5} per year for the mean annual probability of systems, structures, and components reaching the limit state of inelastic response. The performance-based approach achieves a relatively consistent annual probability of plant component failure across the range of plant locations and structural frequencies. It does this by accounting for the slope of the seismic hazard curve, which changes with structural frequency and site location.

To verify the adequacy of the applicant's GMRS, the staff, in RAI 2.5.2-3, requested six PSHA hazard curves (1, 2.5, 5, 10, 25, and 100 Hz). The staff received the requested information from the applicant on June 18, 2007 (as supplemental information to RAI 2.5.2-3). Because the information was provided late in the review process, the staff identified this as Open Item 2.5-8 in the SER with open items. This was done to allow the staff additional time to complete its review of the applicant's response to RAI 2.5.2-3.

In response to RAI 2.5.2-3, the applicant provided the staff with soil hazard curves (corresponding to the top of the Blue Bluff Marl) at annual exceedance frequency levels of 10^{-4}, 10^{-5}, and 10^{-6}. The applicant obtained these hazard curves from its site response analysis described in SSAR Section 2.5.2.5. The applicant defined each hazard curve at a total of seven frequencies (0.5, 1, 2.5, 5, 10, 25, and 100 Hz). The applicant also obtained hazard curves at intermediate annual exceedance frequencies by performing interpolation. For each of the seven frequencies, the applicant fit a quadratic equation to the log (base 10) of the spectral ratios as a function of annual exceedance frequency.

Since the issuance of the SER with open items, the applicant changed the location of its GMRS from the top of the Blue Bluff Marl to the top of the structural backfill. At a public meeting on February 28, 2008, it was brought to the attention of the staff that the applicant's GMRS accounted for the effects of the material above the Blue Bluff Marl, which is contrary to the definition of the GMRS in RG 1.208. The applicant subsequently re-defined its GMRS and provided the updated soil hazard curves that corresponded to the top of the structural backfill.

The staff performed a confirmatory analysis in order determine the GMRS via the risk equation (Equation 1) as opposed to the direct convolution of the risk integral (Equation 3). The staff performed this confirmatory analysis in order to verify the acceptability of assuming a linear hazard curve in log-log space.

$$P_{FT} = \int_0^{\infty} H(a) f_a(a) da \qquad \text{Equation (4)}$$

Since the seismic hazard curves have a slight downward curvature, assuming a linear fit results in slightly higher exceedance values and, as a result, slightly higher GMRS values, as illustrated in Table 2.5.2-10. Therefore, the staff concludes that the applicant's use of the approximate power law hazard curve is slightly conservative and therefore acceptable.

Table 2.5.2-10. Comparison of Site-Specific GMRS Values

Natural Frequency (Hz)	GMRS	
	Risk Integral (g)	Risk Equation (g)
1.0	0.276	0.285
2.5	0.714	0.775
5.0	0.693	0.709
10.0	0.702	0.789

The DF recommended in ASCE/SEI 43-05 (Equation 1) is slightly unconservative for β=0.3 and conservative for β of 0.4 to 0.6. To evaluate the significance of the range of β values on the DF, the staff determined the unacceptable performance frequency values (PFT) for the GMRS values for four natural frequency values 1, 2.5, 5, and 10 Hz. The applicant determined the four GMRS values shown in Table 2.5.2-10 using the performance-based approach as described in ASCE/SEI 43-05, which assumes a β value of 0.4 and a target performance goal of $1x10^{-5}$/yr. The staff used the four hazard curves provided by the applicant to determine PFT via direct integration of the risk integral (Equation 3) for β ranging from 0.3 to 0.6. As shown below in Table 2.5.2-11, the PFT values for β=0.3 are only slightly larger than the target value of $1x10^{-5}$/yr (with the exception of frequencies of 2.5 and 10 Hz, which are less than the target value of $1x10^{-5}$/yr). Since the PFT values for β=0.3 are only slightly larger (at frequencies of 1.0 and 5.0 Hz) than the target performance goal of 10^{-5}/yr and fragility β values of 0.3 are not common for SSCs, the staff concludes that the applicant's assumption that β=0.4 for determining the GMRS is acceptable.

Table 2.5.2-11. Unacceptable Performance Frequency Values for β Ranging from 0.3 to 0.6

Frequency (Hz)	GMRS (g)	PFT*10^{-5}/yr			
		β = 0.3	β = 0.4	β = 0.5	β = 0.6
1.0	0.285	1.073	0.925	0.661	0.506
2.5	0.775	0.689	0.706	0.539	0.500
5.0	0.709	0.950	0.920	0.668	0.539
10.0	0.789	0.518	0.579	0.500	0.500

As determined by the staff in its final SER for the Clinton Early Site Permit, essentially elastic behavior (or OSID (onset of significant inelastic deformation)) is just beyond the occurrence of insignificant (or localized) inelastic deformation, and in this way corresponds to essentially elastic behavior. As such, OSID of an SSC can be expected to occur well before seismically-induced core damage, resulting in much larger frequencies of OSID than seismic core damage frequency (SCDF (seismic core damage frequencies)) values. To further demonstrate that the frequency of OSID is larger than the SCDF, the staff used the four Vogtle ESP hazard curves (1, 2.5, 5, and 10 Hz) to calculate SCDF values each of the GMRS values. In performing this calculation of SCDF, the staff used the risk integral (Equation 3) with the complete range of β values (0.3 to 0.6) and assumed that the seismic margin (Ms) against core damage is 1.67 for new standard plant designs as specified in the staff requirements memorandum (SRM), dated July 21, 1993, on SECY 93-087. As shown in Table 2.5.2-12 below, SCDF values for the four natural frequencies and β values vary from $0.022x10^{-5}$/yr to $0.318x10^{-5}$/yr.

Table 2.5.2-12. SCDF Values for Vogtle GMRS

Frequency (Hz)	GMRS (g)	SCDF*10^{-5}/yr			
		β = 0.3	β = 0.4	β = 0.5	β = 0.6
1.0	0.285	0.318	0.210	0.152	0.120
2.5	0.775	0.072	0.055	0.052	0.055
5.0	0.709	0.156	0.105	0.086	0.079
10.0	0.789	0.022	0.027	0.033	0.040

For comparison, NUREG-1742 shows, based on the results of seismic PRAs of 25 nuclear power plants, that the median value for mean core damage frequency is $1.2x10^{-5}$/yr. Therefore, by setting the target performance goal, PFT, to be a frequency of onset of inelastic deformation

(FOSID) value of 1×10^{-5}/yr, the resulting GMRS computed using the ASCE/SEI 43-05 methodology provides SCDF values that are substantially lower than those for most of the 25 nuclear power plants provided in NUREG-1742. For the natural frequencies of 5 and 10 Hz and for β values of 0.4 and 0.5, SCDF is about 1×10^{-6}/yr to 3×10^{-7}/yr for the Vogtle ESP performance-based SSE, which is about 12 to 40 times lower than the median of the mean SCDF for the 25 nuclear power plants considered in NUREG-1742.

In summary, the staff concludes that the applicant provided sufficient information in response to RAI 2.5.2-3 in order for the staff to verify the adequacy of the applicant's GMRS. Based on the results of the confirmatory analyses described above, the staff concludes that the applicant's use of the approximate power law hazard curve to determine the GMRS is slightly conservative and therefore acceptable. In addition, the staff concludes that the applicant's assumption that β=0.4 for determining the GMRS is acceptable. Furthermore, the staff concludes that the applicant targeted a sufficiently low structural performance frequency value (1×10^{-5}/yr), which is set equivalent to FOSID (frequency of onset of significant inelastic deformation), such that the resulting performance-based GMRS achieves an SCDF which is approximately 12 to 40 times smaller than the median of the mean SCDF for the 25 nuclear power plants considered in NUREG-1742. Therefore, the staff considers Open Item 2.5.2-8 to be resolved.

Vertical GMRS

To compute the vertical GMRS, the applicant used a combination of V/H ratios obtained from NUREG/CR-6728 and Lee (2001). Since the V/H ratios presented in NUREG/CR-6728 and Lee (2001) are functions of magnitude, source distance, and local site conditions, the applicant developed V/H ratios corresponding to the final high-frequency (**M** 7.2, 130 km) and low-frequency (**M** 5.6, 12 km) controlling earthquakes described in SSAR Section 2.5.2.4. The applicant referred to these high- and low-frequency controlling earthquakes as "near" and "far" events, respectively.

In Part A of RAI 2.5.2-24, the staff asked the applicant to justify its rationale for assigning the approximate weights of 1:3 to the V/H ratios corresponding to the respective "near" and "far" events. In response to Part A of RAI 2.5.2-24, the applicant concluded that it developed this weighting based on a review of the high- and low-frequency distance deaggregations as well as the relative contributions of the 10^{-4} and 10^{-5} hazard levels to the GMRS. Based on its review of the high-frequency distance deaggregation at the 10^{-4} hazard level (shown in SSAR Figure 2.5.2-30), the applicant concluded that approximately three-fourths of the area under the 10^{-4} hazard probability density curve corresponds to the "far" event, while about one-fourth of the area under the curve corresponds to the "near" event. In comparison, the applicant found that the relative contribution of the "near" and "far" events at the 10^{-5} hazard level is approximately the same. The applicant also reviewed the low-frequency distance deaggregation (shown in SSAR Figure 2.5.2-31) at both the 10^{-4} and 10^{-5} hazard levels and concluded that the hazard is dominated by the "far" event.

As stated in its response to Part A of RAI 2.5.2-24, the applicant focused on the 10^{-4} high-frequency distance deaggregation and the associated weights of 1:3 to determine the relative contributions of the respective "near" and "far" events because the GMRS is generally only slightly higher than the 10^{-4} ground motion. The applicant used the high-frequency distance deaggregation, rather than the low-frequency distance deaggregation, because it concluded "the low-frequency end of the spectrum is not as sensitive to magnitude and distance nor, therefore, to the distinction between 'near' and 'far' events."

The staff concludes that the applicant's use of NUREG/CR-6728 to develop V/H ratios is acceptable because the report considers the effects of magnitude and distance on spectral ratios and is applicable to CEUS soil sites. Previous regulatory guidance (RG 1.60 and NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants") recommended that the V/H ratio be fixed at two-thirds, independent of ground motion frequency, earthquake magnitude, distance, and local site conditions. More recent regulatory guidance (RG 1.208) recommends the use of V/H ratios that incorporate magnitude, distance, and local site conditions, such as those found in NUREG/CR-6728. Because of the observed similarity between the GMRS to the 10^{-4} soil UHRS, and because V/H ratios are observed to be higher in the near-field region and in the high-frequency range of the response spectrum (e.g., NUREG/CR-6728), the staff concurs with the applicant's rationale for weighting the relative contributions of the "near" and "far" events based on the 10^{-4} high-frequency distance deaggregation.

In Part B of RAI 2.5.2-24, the staff asked the applicant to discuss the similarities and differences between the site-specific soil profile used by Lee (2001) and the VEGP soil profile. In response to Part B of RAI 2.5.2-24, the applicant stated that the SRS site-specific soil profile is not published in Lee (2001) so that a comparison with the ESP profile could not be made. The applicant also stated that given the proximity of the ESP site to the SRS, it assumed that the site conditions at the SRS are more comparable to those at the ESP site than to the generic CEUS profile used in NUREG/CR-6728.

In Part C of RAI 2.5.2-24, the staff asked the applicant to provide justification for the relative weights assigned to the NUREG/CR-6728 and Lee (2001) results and final smoothing to develop the final V/H ratios for the ESP site. In response, the applicant stated that it used an approximate envelope of the two results. For frequencies between 1 and 100 Hz, the applicant approximated the V/H ratios of Lee (2001) by two log-log line segments. For frequencies less than 1 Hz, the applicant used a constant ratio of 0.5, which is greater than both Lee (2001) and NUREG/CR-6728, and more closely resembles the V/H values in RG 1.60.

For CEUS soil sites, RG 1.208 endorses the procedure provided in NUREG/CR-6728 to determine a WUS-to-CEUS transfer function to modify the WUS V/H ratios. The staff, therefore, concludes that the applicant's use of the formula provided in Appendix J to NUREG/CR-6728 to determine the ESP site V/H ratios is acceptable. However, the formula in Appendix J, shown in Equation (2) in SER Section 2.5.2.6, requires the input of site-specific V/H ratios, V/HCEUS,Soil,Model, based on ground motion modeling. For this site-specific V/H ratio, the applicant used the results of Lee (2001), which are applicable to the SRS soil profile, and NUREG/CR-6728, based on a generic CEUS soil profile. SER Figure 2.5.2-9 shows the applicant's final V/H ratios as a function of frequency. At frequencies above approximately 1 Hz, the applicant estimated the V/H ratios of Lee (2001) by two log-log line segments. At frequencies between 1–2 Hz and 10–20 Hz, this log-log line segment is less that the V/H ratios of Lee (2001). In the SER with open items, the staff concluded that the applicant did not provide adequate justification to support the applicability of either the Lee (2001) or the NUREG/CR-6728 soil V/H ratios at the ESP site. The staff further concluded that the applicant's approximate envelope was arbitrary. For example, the applicant did not provide its rationale for excluding the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10-20 Hz frequency ranges. Accordingly, in Open Item 2.5-9, in the SER with open items, the staff requested that the applicant provide more detail regarding the applicability of the Lee (2001) and the NUREG/CR-6728 V/H ratios to the ESP site. In addition, the staff requested that the applicant provide its justification for the use of an approximate envelope of the Lee (2001) and NUREG/CR-6728 V/H ratios.

In response to the staff's request to provide more detail regarding the applicability of the Lee (2001) and the NUREG/CR-6728 V/H ratios to the ESP site, the applicant stated that it considered the implementation of the NUREG/CR-6728 approach in two cases as a guide for recommending a V/H for the Vogtle ESP site. In the first case, the applicant relied on the transfer functions presented in Appendix J to NUREG/CR-6728, where the CEUS soil model corresponds to a generic "deep soil" (500 ft) site. In the second case, the applicant relied on an evaluation of V/H for the nearby SRS (Lee, 2001), which also followed the NUREG/CR-6728 approach. The applicant stated that the subsurface S-wave velocity profiles and depths to water table are similar at Vogtle and at the SRS. The applicant also stated that "while the results from the SRS (first case) may seem arguably the most applicable for the Vogtle site, the generic nature of the first case is consistent with the generic character of the rock V/H recommendation of the NUREG. Therefore both results are considered in the SSAR."

Regarding additional justification for the use of an approximate envelope of the Lee (2001) and NUREG/CR-6728 V/H ratios, the applicant stated that, similar to the RG 1.60 V/H ratios and the recommended V/H functions for rock sites in NUREG/CR-6728, it intended to derive a V/H (based on both Lee and NUREG/CR-6728 soil) that is relatively simple and smooth, yet also reflects the following general features:

- Similar to RG 1.60 and the NUREG/CR-6728 V/H for rock, V/H is higher at high frequencies than at low frequencies;
- The two cases evaluated suggest that a relatively flat V/H value at high frequencies, slightly lower (~0.9) than that given by RG 1.60 (1.0);
- both cases suggest a lower V/H value (0.5) than that given by RG 1.60 (2/3) in the lowest frequencies;
- the envelope of the two cases suggests that the transition between the higher V/H at high frequencies and the lower V/H at low frequencies is more gradual than the relatively abrupt transition in Reg. Guide 1.60; and
- the V/H at high frequencies is sustained at its high value longer toward lower frequencies (flatter) than suggested by CEUS rock V/H from the NUREG/CR-6728.

In Open Item 2.5.2-9, the staff also requested that the applicant provide its rationale for excluding the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10–20 Hz frequency ranges. In response, the applicant stated that given the complexities, assumptions, and uncertainties of developing CEUS, deep soil V/H for the Vogtle site, as well as the desire to develop a smooth function, it developed a conservative envelope of the V/H for the two cases. The applicant further stated that three discrete acceleration intervals (≤0.2g, 0.2 – 0.5 g, and >0.5 g) for which the rock V/H is defined in NUREG/CR-6728 also suggests approximate evaluations of V/H. For this reason, some peaks are cut (e.g., 1.3 Hz) and valleys are filled (e.g., 2.5 Hz) by the applicant. However, the applicant stated that it did not consider this to be significant relative to other uncertainties in ground motion evaluations.

Based on the applicant's response to Open Item 2.5-9, the staff concludes that the applicant's use of the generic CEUS soil profile V/H ratios provided in Appendix J is acceptable because the applicant also considered the V/H ratios developed for the adjacent SRS, which has a similar S-wave velocity profile to the Vogtle site and is therefore more applicable. Furthermore, the V/H ratios for the SRS soil profile are always larger than the generic CEUS soil profile, and the applicant used an approximate envelope of the two results, with the exception of the peaks excluded in the 1–2 Hz and 10–20 Hz frequency ranges. The staff, however, concludes that the applicant's exclusion of the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10-20 Hz frequency ranges is not significant. As observed in SER Figure 2.5.2-9, the peak excluded in the 10–20 Hz frequency range is approximately 10 percent larger than the approximate envelope, while the peak in the narrow 1–2 Hz frequency range is less than 20 percent larger. Furthermore, the valleys on either side of the narrow peak at 1–2 Hz have also been filled. The staff notes that the applicant's final vertical GMRS is not changed significantly as a result of this smoothing. In addition, the staff notes that the applicant's use of 100 km instead of 130 km distance to obtain V/H corresponding to the **M** 7.2, 130-km earthquake from the Lee (2001) results is conservative because V/H decreases with distance for a given magnitude. This would effectively increase the final V/H based on the Lee (2001) results for the SRS shown in SER Figure 2.5.2-9. Therefore, the staff considers Open Item 2.5-9 to be resolved.

Based on its review of SSAR Section 2.5.2.6, the staff thus concludes that, overall, the applicant's horizontal and vertical GMRS, which are shown in SSAR Figure 2.5.2-44b, are acceptable. The applicant followed the general guidance provided in RG 1.208 to develop both the horizontal and vertical GMRS for the Vogtle site. In SSAR Table 1-1, the applicant identified the GMRS as an ESP site characteristic[11]. For the reasons set forth above, the staff agrees with the applicant's GMRS as a site characteristic, which appears as SER figure 2.5.2-25 and in Appendix A of the SER.

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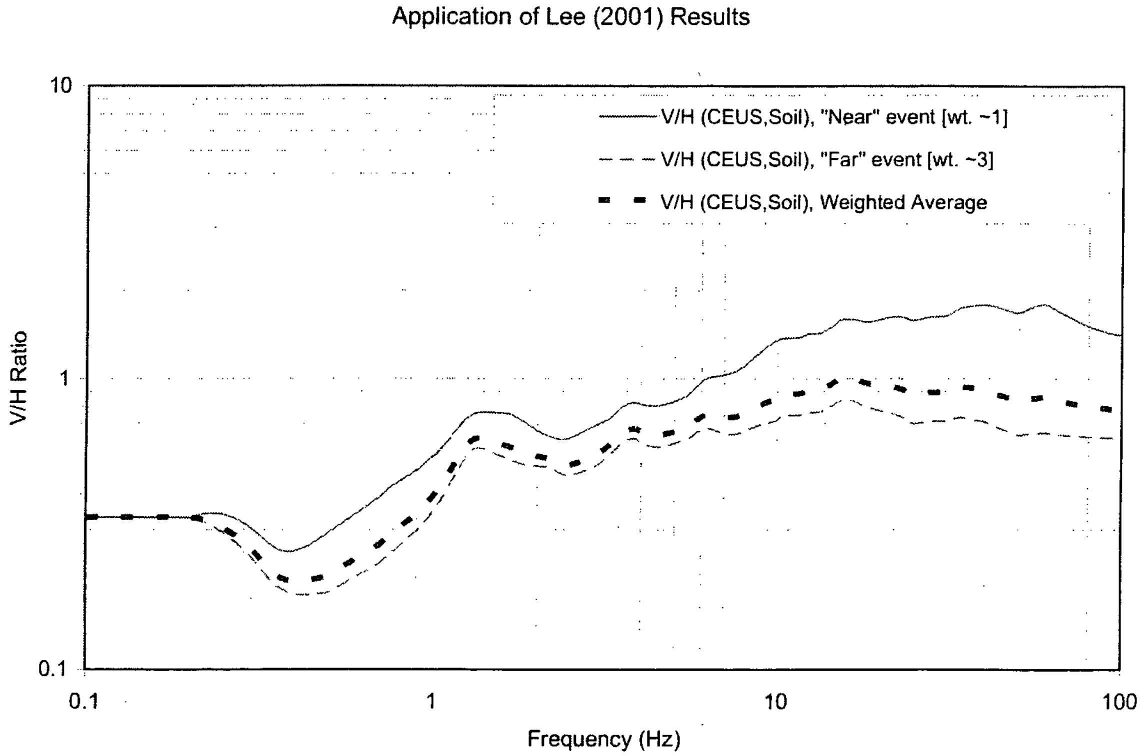
11 The staff notes that this site characteristic for the GMRS is not bounded by the AP1000 certified design response spectrum (CSDRS). However, the staff considers the GMRS value determined for the Vogtle site to be within the range of values that new reactor designs generally are engineered to withstand. Accordingly, the staff considers the approval of this site characteristic to be consistent with the staff's determination that, from a geologic and seismologic perspective, the ESP site is suitable and meets the applicable requirements of Part 52 and Part 100. Whether the reactor design ultimately chosen for the site bounds the GMRS site characteristic will be determined at the COL stage.



Note: Considering the relative contribution of the "near" and "far" events to the horizontal SSE design response spectrum, the approximately 1:3 weighted average is shown.

Figure 2.5.2-24. Plots of recommended V/H CEUS,Soil ratios using the results from Lee (2001) for the SRS (reproduced from SSAR Figure 2.5.2-41).

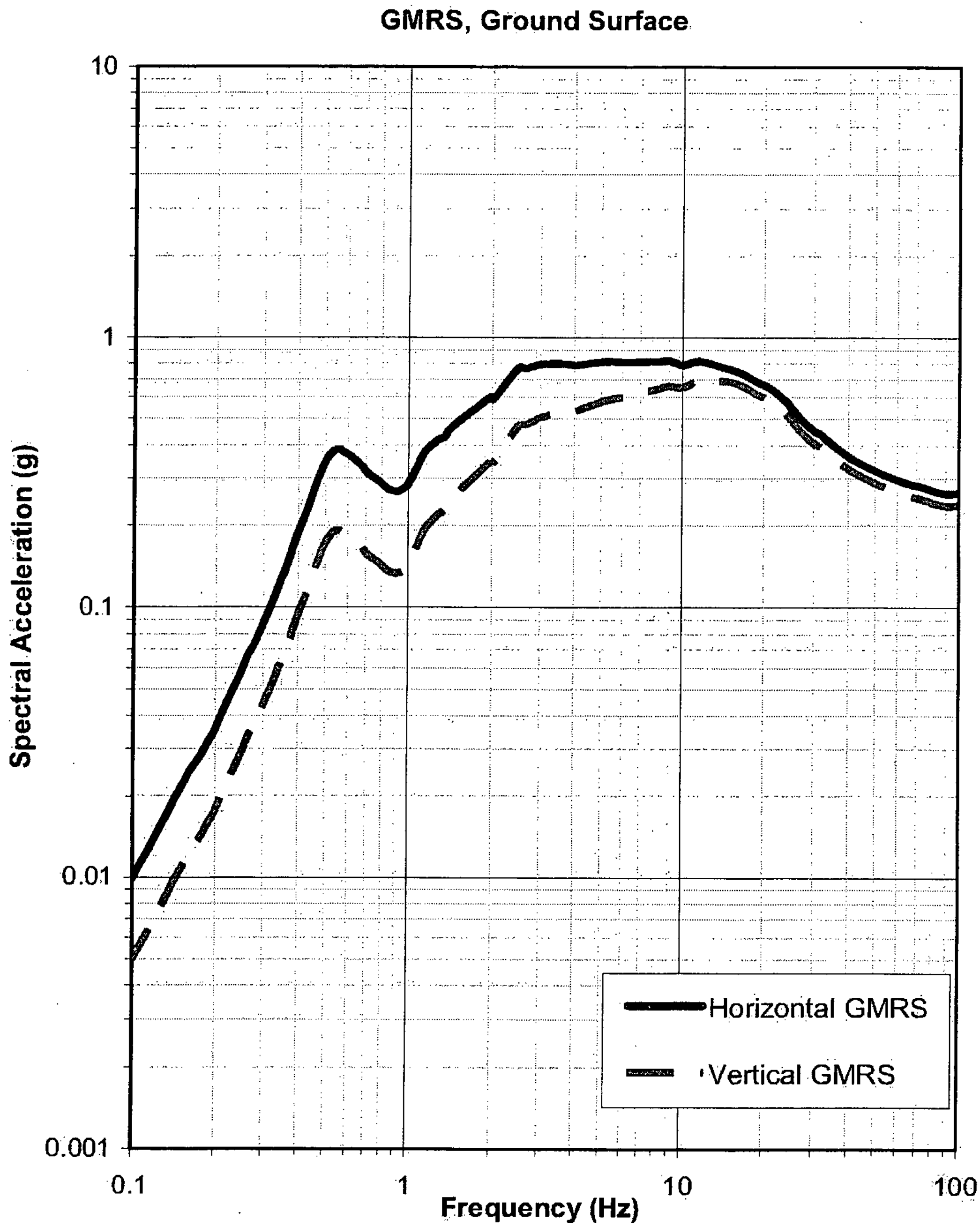


Figure 2.5.2-25. Plots of the horizontal and vertical GMRS (reproduced from SSAR Figure 2.5.2-44b).

2.5.2.5 Conclusions

As set forth above, the staff reviewed the seismological information submitted by the applicant in SSAR Section 2.5.2. On the basis of its review of SSAR Section 2.5.2, the staff finds that the applicant has provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. In addition, the staff finds that the applicant has adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA, and this PSHA follows the guidance provided in RGs 1.165 and 1.208. The staff concludes that the controlling earthquakes and associated ground motion derived from the applicant's PSHA are consistent with the seismogenic region surrounding the ESP site. In addition, the staff finds that the applicant's GMRS, which was developed using the performance-based approach, adequately represents the regional and local seismic hazards and accurately includes the effects of the local ESP subsurface properties. The staff concludes that the proposed ESP site is suitable with respect to the vibratory ground motion criteria for new nuclear power plants and meets the applicable requirements of 10 CFR 100.23.

2.5.3 Surface Faulting

In SSAR Section 2.5.3, the applicant evaluated the potential for tectonic and nontectonic surface and near-surface deformation at the VEGP ESP site. The applicant included a review of geologic, seismic, and geophysical investigations in SSAR Section 2.5.3.1.1 to assess the potential for surface deformation that could impact the ESP site. In SSAR Sections 2.5.3.1.2 and 2.5.3.1.4, the applicant assessed geologic evidence, or the absence of evidence, for surface deformation by evaluating known geologic structures in the VEGP site vicinity. SSAR Section 2.5.3.3 provides a review of seismicity within the site vicinity (a 40 km (25 mi) radius of the VEGP site) and addresses any correlation between the seismicity and capable tectonic structures. SSAR Sections 2.5.3.1.4 and 2.5.3.1.5 evaluate the tectonic structures in the site area, how these structures relate to the regional tectonics, and any ages of deformation associated with these structures. The applicant discussed the potential for tectonic and/or nontectonic deformation at the VEGP site in SSAR Section 2.5.3.1.8. On the basis of this evaluation, the applicant concluded that: (1) no capable tectonic sources exist within the VEGP site area (within an 8 km (5 mi) radius); (2) the potential for tectonic fault displacement is negligible; (3) only limited potential exists for nontectonic surface deformation within the site area; and (4) the potential for nontectonic surface deformation can be mitigated by excavation of materials.

2.5.3.1 Technical Information in the Application

2.5.3.1.1 Geologic, Seismic, and Geophysical Investigations

In SSAR Section 2.5.3.1, the applicant described the geologic, seismic, and geophysical investigations performed to assess the potential for tectonic and nontectonic surface and near-surface deformation at and within an 8 km (5 mi) radius of the VEGP site. The applicant reviewed previous VEGP site investigations, published geologic mapping, previous SRS investigations, previous seismicity data, previous seismic reflection data, current seismic reflection studies, and current aerial and field reconnaissance. The applicant stated that geologic and geomorphic investigations within and beyond the site vicinity (a 40 km (25 mi) radius) and interpretation of aerial photographs taken within the site area (an 8 km (5 mi) radius) were used to supplement existing information for documenting the presence or absence of features indicative of potential Quaternary (1.8 million years ago (mya) to present) fault activity at or near the site.

Data from Previous Investigations

SSAR Section 2.5.3.1.1 describes previous site area investigations conducted for VEGP Units 1 and 2. Section 2.5.3.1.2 describes the applicant's review of published geologic maps for analyzing surface deformation within the site area. The applicant reviewed previous SRS investigations (SSAR Section 2.5.3.1.3), including geologic, seismic, hydrologic, and geophysical investigations, and concluded that the Pen Branch fault does not exhibit surface deformation, is not a capable tectonic structure, and is not favorably oriented in the modern-day stress regime to experience displacement. In SSAR Section 2.5.3.1.4, the applicant reviewed historical seismicity and microseismicity data for the site vicinity (within a 40 km (25 mi) radius) and the site area (within an 8 km (5 mi) radius). The applicant stated that no recent earthquake activity has occurred within the site area and that the closest microearthquake to the ESP site is located on the SRS, about 11 km (7 mi) to the northeast of the VEGP. In SSAR Section 2.5.3.1.5, the applicant discussed previous seismic reflection studies and again concluded that the Pen Branch fault is not a capable tectonic structure.

Data from Current Investigations

The applicant described current seismic reflection studies in SSAR Section 2.5.3.1.6 and current aerial and field reconnaissance studies in SSAR Section 2.5.3.1.7. These investigations were performed for the ESP application in order to image the Pen Branch fault beneath the surface and to check for evidence of surface faulting within the ESP site vicinity. The applicant stated that the Pen Branch fault was clearly imaged beneath the ESP site area in the seismic reflection data. The applicant concluded that, based on aerial and field reconnaissance data, no geomorphic features within the site vicinity display evidence for surface rupture, surface warping, or fault offset.

2.5.3.1.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

In SSAR Section 2.5.3.2, the applicant stated that four bedrock faults are mapped within a 5-mile radius of the VEGP ESP site. These faults, interpreted from seismic reflection, borehole, gravity, and magnetic and/or ground water data, include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults. Of these four faults, only the Pen Branch fault is interpreted to extend beneath the VEGP ESP site area, motivating the applicant to perform a detailed investigation of the Pen Branch fault as it relates to the ESP site. A complete description of the applicant's investigation of the Pen Branch fault is included in SSAR Section 2.5.1.2.4.1. The remaining three faults, mapped in relation to the SRS, are located within a 5-mile radius of the VEGP site, but are not interpreted to extend beneath the site. The applicant concluded that none of the four faults mapped within the site area displays evidence of surface rupture and that none of these faults is a capable tectonic structure.

Pen Branch Fault

The applicant presented its conclusions regarding the Pen Branch fault in SSAR Sections 2.5.3.2.1 and 2.5.3.5.1. The Pen Branch fault is more than 30 km (greater than 20 mi) in length along its northeastern strike direction and forms the northwest boundary of the Dunbarton Triassic basin. The fault initially accommodated regional crustal extension during the Mesozoic (248 to 65 mya) by normal slip during the Triassic (248 to 206 mya) period to form the

Dunbarton Basin, and was reactivated in the Cretaceous (144 to 65 mya) and Tertiary (65 to 2 mya) as a reverse fault. The applicant stated that the Pen Branch fault is not exposed or geomorphically expressed at the surface, and borehole and seismic reflection data collected at the SRS show no evidence for post-Eocene slip on the fault. According to the applicant, the Ellenton Quaternary terrace (Qte) at the SRS, dated between 350,000 and 1 mya in age, was evaluated for the ESP application and demonstrates no Quaternary tectonic deformation of the terrace surface within a resolution of about 1 m (3 ft). The applicant stated that both previous and more recent investigations to define the presence or absence of surface deformation related to displacement on the Pen Branch fault indicate no evidence of Quaternary (1.8 mya to present) deformation. Based on these findings, the applicant concluded that the Pen Branch fault is not interpreted as a capable tectonic source.

Ellenton Fault

In SSAR Sections 2.5.3.2.2 and 2.5.3.5.2, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Ellenton fault, located about 7.4 km (4.6 mi) from the VEGP site. As initially mapped by Stieve and Stephenson (1995), the Ellenton fault was a north-northwest striking fault located in the Dunbarton Basin between the Upper Three Runs and Pen Branch faults. The applicant stated that the Ellenton fault likely does not exist because the data used to suggest the existence of this potential structure were acknowledged to be of poor quality, there is no geomorphic expression of this fault at the surface, and the fault does not appear on the most recent SRS fault maps by Cumbest et al. (2000). Therefore, the applicant concluded that this fault could not represent a capable tectonic structure within the site area.

Steel Creek Fault

In SSAR Sections 2.5.3.2.3 and 2.5.3.5.3, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Steel Creek fault, located about 4.8 km (3 mi) from the VEGP site. This fault is interpreted to be more than 17.7 km (greater than 11 mi) in length, with a northeast strike and a northwest dip, and exhibits reverse slip movement. The Steel Creek fault cuts upward into Cretaceous units, but its uppermost extension remains unresolved. According to the applicant, longitudinal profiles along Quaternary fluvial terraces overlying the surface projection of the fault, with a resolution of 2-3 m (7-10 ft), show no evidence of warping or faulting of the terrace surfaces and therefore provides no evidence for Quaternary (1.8 mya) deformation. Based on a lack of geomorphic surface expression, the applicant concluded that the Steel Creek fault is not a capable tectonic structure within the site area.

Upper Three Runs Fault

In SSAR Sections 2.5.3.2.4 and 2.5.3.5.4, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Upper Three Runs fault, located about 8 km (5 mi) from the VEGP site. The fault is not included on the more recent fault map of the SRS by Cumbest et al. (2000), but its northernmost trace is roughly parallel to the Tinker Creek fault that is shown on the Cumbest et al. (2000) fault map. According to the applicant, seismic profiles show that Coastal Plain sediments are not offset or deformed by this structure, and the fault is interpreted to be confined to basement rocks. Based on these findings and the fact that there is no geomorphic surface expression of this fault, the applicant concluded that it is not a capable tectonic structure within the site area.

2.5.3.1.3 Correlation of Earthquakes with Capable Tectonic Sources

The applicant summarized seismicity data for the VEGP ESP site vicinity in SSAR Sections 2.5.3.3 and 2.5.3.1.4 in order to determine whether any correlation exists between seismicity and capable tectonic structures. Figure 2.5.3-1 of this SER, taken from SSAR Figure 2.5.1-16, shows diffuse microseismic activity recorded by the SRS seismic recording network since 1976 within a 40 km (25 mi) radius of the VEGP site.

Based on the data shown in this figure, the applicant concluded that there is no spatial correlation of earthquake epicenters with known or postulated faults. The applicant reviewed published literature to further conclude that there are no known historical earthquake epicenters associated with bedrock faults or known tectonic structures in the site vicinity. The EPRI catalog of historical seismicity demonstrates that no known earthquake greater than body wave magnitude (mb) 3 has occurred within the site vicinity, while the SRS seismic recording network documents no recent microseismic activity (mb less than 3) within an 8 km (5 mi) radius of the VEGP site since 1976. The applicant stated that the nearest microearthquake event to the VEGP ESP site was located about 11 km (7 mi) northeast of the VEGP site on the SRS.

The applicant described three small earthquakes that occurred between 1985 and 1997 with magnitudes ranging between 2.0 and 2.6 and depths ranging from 2.5 to 6 km (1.5 to 3.5 mi). In addition to these events, the applicant described a magnitude 3.2 event located north of the SRS in Aiken, South Carolina, and a series of several small events (magnitudes less than or equal to 2.6) that occurred in 2001–2002 within the SRS boundaries. The applicant reviewed the locations of these events with respect to mapped faults in the ESP site vicinity, as well as previous studies of these events by Stevenson and Talwani (2004), Talwani et al. (1985), and Crone and Wheeler (2000), and concluded that there is no spatial correlation of seismicity with known or postulated faults or geomorphic features.

2.5.3.1.4 Ages of Most Recent Deformations

In SSAR Section 2.5.3.4, the applicant stated that, based on information presented in SSAR Section 2.5.3.2, none of the four faults (Pen Branch, Ellenton, Steel Creek, or Upper Three Runs) exhibits Quaternary (1.8 mya to present) displacement. Thus, the applicant concluded that none is considered a capable tectonic structure. In particular, the applicant stated that the Pen Branch fault exhibits no post-Eocene (33.7 mya to present) displacement.

2.5.3.1.5 Relationship of Site Area Tectonic Structures to Regional Tectonic Structures

SSAR Section 2.5.3.5 discusses the four faults identified within the site area and previously discussed in SER Section 2.5.3.1.2. Of these four faults, the applicant stated that only the Pen Branch fault occurs west of the SRS and within the ESP site area. The applicant concluded that, based on a review of all available data, none of these four faults is considered a capable tectonic structure and none is associated with any capable regional tectonic structure.

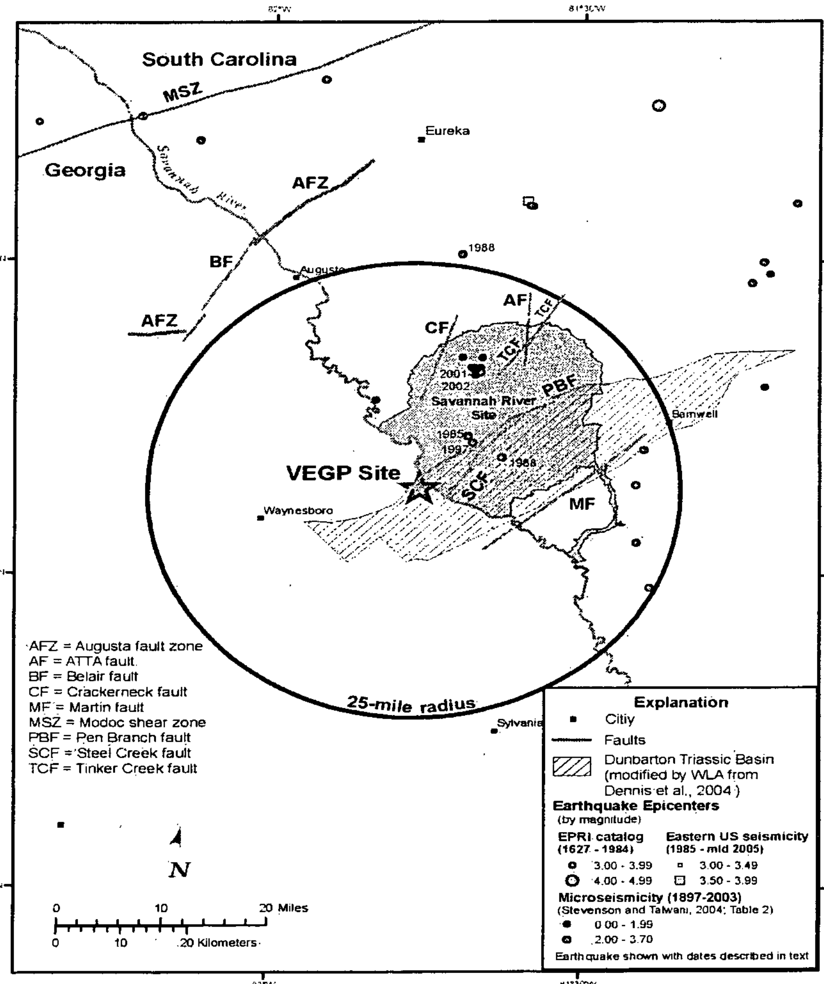


Figure 2.5.3-1 - Site Vicinity Tectonic Features and Seismicity (Reproduced from SSAR Figure 2.5.1-16)

2.5.3.1.6 Characterization of Capable Tectonic Sources

The applicant described characterization of capable tectonic sources in SSAR Section 2.5.3.6 and reiterated that no capable tectonic structures occur within 8 km (5 mi) of the VEGP site based on the following geologic evidence:

1. The Pen Branch fault is not exposed or expressed at the surface. Field reconnaissance and aerial photograph interpretations performed for the ESP study confirmed that there is no surface exposure of the fault or geomorphic expression indicative of Quaternary deformation.

2. Snipes et al. (1993) indicated that there was no displacement of a Quaternary soil horizon overlying the projected trace of the Pen Branch at the SRS, and the youngest horizon offset by fault displacement on the Pen Branch was the Dry Branch Formation of late Eocene age.

3. Geomatrix (1993) evaluated longitudinal profiles of Quaternary fluvial river terraces on the SRS and concluded that no evidence for warping or faulting of the terraces existed within a resolution limit of 2 to 3 m (7 to 10 ft).

4. Longitudinal terrace profiles across the now well-located Pen Branch fault also indicated no deformation of the Ellenton terrace (estimated to be 350,000 to 1 million years old) within a resolution limit of 1 m (3 ft).

5. Also as part of the ESP study, geomorphic analysis of the Ellenton terrace, which overlies the surface projection of the Pen Branch, demonstrates a lack of tectonic deformation of this Quaternary surface within a resolution limit of 1 m (3 ft). Details of this ESP study are presented in SSAR Section 2.5.1.2.4.3.

2.5.3.1.7 Designation of Quaternary Deformation Zones Requiring Detailed Investigation

In SSAR Section 2.5.3.7, the applicant concluded that no zones of Quaternary deformation requiring detailed fault investigation exist within the VEGP site area based on the absence of any Quaternary deformation features in the ESP site area.

2.5.3.1.8 Potential for Tectonic or Nontectonic Deformation at the Site

In SSAR Section 2.5.3.8.1, the applicant concluded that the potential for tectonic deformation at the ESP site is negligible and stated that no new information has been reported since the original site studies for VEGP Units 1 and 2 in the early 1970s to suggest the existence of Quaternary surface deformation. Also in SSAR Section 2.5.3.8, the applicant addressed the potential for nontectonic deformation features at the VEGP ESP site, including dissolution collapse features and clastic dikes.

In SSAR Section 2.5.3.8.2, the applicant specifically discussed the potential for nontectonic surface deformation at the ESP site, including interpretation of dissolution collapse features and clastic dikes. Regarding dissolution collapse features, which are discussed in SSAR Section 2.5.3.8.2.1, the applicant indicated that small-scale structures, including warped bedding, fractures, joints, minor fault offsets, and injected sand dikes, identified in the walls of a trench at the VEGP site were local features related to dissolution of the Utley Limestone and subsequent collapse of overlying Tertiary sediments. Age of these features was interpreted to be younger than Eocene-Miocene host sediments and older than the overlying late-Pleistocene Pinehurst Formation. The applicant stated that no late Pleistocene or Holocene dissolution features were identified at the site. The applicant indicated that mitigation of collapse due to dissolution of the Utley Limestone, which overlies the Blue Bluff Marl (BBM) at the site, could be accomplished by planned excavation and removal of the Utley Limestone to establish the foundation grade of the plant atop the BBM.

In SSAR Section 2.5.3.8.2.2, the applicant addressed clastic dikes, described as relatively planar, narrow (centimeter-to-decimeter wide) clay-filled features that flare upwards and are decimeters to meters in length. The applicant stated that Bechtel (1984) distinguished two types of clastic dikes in the walls of the trench on the VEGP site where dissolution collapse features were found. The first type of clastic dikes was interpreted to be sand dikes that resulted from injection of poorly consolidated fine sand into overlying sediments; the second type was clastic dikes produced by weathering and soil formation processes that were enhanced along fractures that formed during dissolution collapse. Bechtel (1984) concluded that the dikes were primarily a weathering phenomenon controlled by depth of weathering and paleosol development in Coastal Plain sediments and subsequent erosion of the land surface. According to the applicant, clastic dike features identified by Bartholomew et al. (2002) within the site area were observed during the ESP field reconnaissance. The applicant interpreted these features to be nontectonic in origin, although Bartholomew et al. (2002) suggested that they might be evidence for paleoearthquakes associated with late-Eocene to late-Miocene faulting, possibly along the Pen Branch fault.

2.5.3.2 Regulatory Evaluation

The acceptance criteria for evaluating the potential for surface or near-surface tectonic and nontectonic deformation are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100.23. The staff considered the following regulatory requirements in reviewing the applicant's discussion of information on surface faulting:

1. 10 CFR 53.17(a)(1)(vi), which requires that an ESP application contain a description of the geologic and seismic characteristics of the proposed site.

2. 10 CFR 100.23(c), which requires an ESP applicant to investigate geologic, seismic, and engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site, to provide sufficient information to support evaluations performed to determine the SSE Ground Motion, and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site.

3. 10 CFR 100.23(d), which requires that geologic and seismic siting factors considered for design include a determination of the SSE Ground Motion for the site, the potential for surface tectonic and non-tectonic deformation, the design bases for seismically-induced floods and water waves, and other design conditions including soil and rock stability, liquefaction potential, and natural and artificial slope stability. Siting factors and potential causes of failure to be evaluated include physical properties of materials underlying the site, ground disruption, and effects of vibratory ground motion that may affect design and operation of the proposed power plant.

The basic geologic and seismic information assembled by the applicant in compliance with the above regulatory requirements should also be sufficient to allow a determination at the COL stage of whether the proposed facility complies with the following requirements in Appendix A to 10 CFR Part 50:

1. 10 CFR Part 50, Appendix A, GDC 2, which requires that SSCs important to safety be designed to withstand the effects of natural phenomena such as earthquakes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

2. 10 CFR Part 50, Appendix S - IV, "Application to Engineered Design", which requires that vibratory ground motion (including the Safe Shutdown Earthquake Ground Motion and the Operating Basis Earthquake Ground Motion) and surface deformation be considered in the design of a nuclear power plant.

To the extent applicable in the regulatory requirements cited above, and in accordance with RS-002, the staff applied NRC-endorsed methodologies and approaches (specified in Section 2.5.3 of NUREG-0800) for evaluation of information characterizing the potential for surface or near-surface tectonic and nontectonic deformation at the proposed site as recommended in RG 1.165.

Section 2.5.3 of NUREG-0800 and RG 1.165 provide specific guidance concerning the evaluation of information characterizing the potential for surface and near-surface deformation, including the geologic, seismic, and geophysical data that the applicant needs to provide to establish the potential for surface deformation.

2.5.3.3 Technical Evaluation

This SER section presents the staff's evaluation of the geologic, seismic, and geophysical information submitted by the applicant in SSAR Section 2.5.3 to address the potential for surface or near-surface tectonic and nontectonic deformation within an 8 km (5 mi) radius of the ESP site (i.e., the "site area" as defined in RG 1.165). The technical information presented in SSAR Section 2.5.3 resulted from the applicant's surface and subsurface geologic, seismic, and geophysical investigations performed within the site area, supplemented by aerial and field reconnaissance studies undertaken within a 40 km (25 mi) radius of the site (i.e., the "site vicinity" as defined in RG 1.165). Through its review, the staff determined whether the applicant complied with the applicable regulations and conducted its investigations with an appropriate level of detail in accordance with RG 1.165.

To thoroughly evaluate the geologic, seismic, and geophysical information presented by the applicant, the staff obtained the assistance of the USGS. The staff and its USGS advisors visited the ESP site to confirm interpretations, assumptions, and conclusions presented by the applicant and related to the potential for surface or near-surface faulting and nontectonic deformation.

2.5.3.3.1 Geologic, Seismic, and Geophysical Investigations

In SSAR Sections 2.5.3.1.1 through 2.5.3.1.7, the applicant reviewed and summarized information related to previous VEGP site investigations (Section 2.5.3.1.1), published geologic mapping (Section 2.5.3.1.2), previous SRS investigations (Section 2.5.3.1.3), previous seismicity data (Section 2.5.3.1.4), previous seismic reflection data (Section 2.5.3.1.5), current seismic reflection studies (Section 2.5.3.1.6), and current aerial and field reconnaissance (Section 2.5.3.1.7).

Based on the information presented in SSAR Sections 2.5.3.1.1 through 2.5.3.1.7, the applicant concluded that no capable tectonic sources occur within the site area and that there is negligible potential for surface or near-surface fault rupture. Consequently, the applicant considered the site to be suitable in regard to the potential for surface or near-surface faulting. The staff's review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.7 is presented below.

Data from Previous Investigations

The staff focused its review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.5 on the applicant's descriptions of previous studies and data collected within the site area in order to assess the potential for surface tectonic deformation at the ESP site. In SSAR Section 2.5.3.1.1, the applicant described the results of previous investigations conducted for VEGP Units 1 and 2, which support the concepts that the Pen Branch fault (known to underlie the ESP site) exhibits no surface displacement and is a noncapable tectonic structure and that nontectonic deformation features occur in the site area. In SSAR Section 2.5.3.1.2, the applicant discussed information from published geologic maps documenting the existence of nontectonic deformation features in the site area. SER Section 2.5.3.3.9 provides a more detailed discussion of nontectonic features in the site area. The applicant also stated in SSAR Section 2.5.3.1.2 that Crone and Wheeler (2000) and Wheeler (2005) classified the Pen Branch fault as a Class C feature based on insufficient geologic evidence to document Quaternary displacement along the fault. In SSAR Section 2.5.3.1.3, the applicant cited evidence collected from the SRS that the Pen Branch fault does not exhibit surface displacement, is not a capable tectonic structure, and is not favorably oriented in the modern-day stress field to experience displacement. In SSAR Section 2.5.3.1.4, the applicant stated that no recent earthquake activity has occurred within the site area based on microseismicity data. In SSAR Section 2.5.3.1.5, the applicant discussed previous seismic reflection studies supporting the interpretation that the Pen Branch fault is not a capable tectonic structure.

Based on a review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.5, the staff concludes that the applicant presented thorough and accurate descriptions of previous studies and data collected within the site area. The applicant used this information to assess the potential for tectonic deformation at the ESP site, which is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). These five SSAR sections present well-documented geologic information that the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined in order to ensure the accuracy of the information presented by the applicant in the SSAR.

Data from Current Investigations

The staff focused its review of SSAR Sections 2.5.3.1.6 and 2.5.3.1.7 on the applicant's descriptions of the investigations performed to image the Pen Branch fault at the ESP site using seismic reflection and to look for evidence of surface faulting in the site vicinity using field and aerial reconnaissance. In SSAR Section 2.5.3.1.6, the applicant stated that the Pen Branch fault is clearly imaged beneath the ESP site in the seismic reflection data. In SSAR Section 2.5.3.1.7, the applicant indicated that no geomorphic evidence exists for surface rupture, surface warping, or fault offset. The applicant also reported its reinterpretation of features observed within the site vicinity and initially considered as possible evidence for tectonic activity. The applicant reinterpreted these features as nontectonic in origin.

Based on its review of SSAR Sections 2.5.3.1.6 and 2.5.3.1.7, the staff concludes that the applicant presented thorough and accurate descriptions of data from current investigations within the site area in order to assess the potential for tectonic deformation at the ESP site. This information supports the requirements set forth in 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff further concludes that the applicant presented adequate evidence to support the conclusions that the Pen Branch fault underlies the ESP site. The staff believes that the applicant also provided adequate evidence that no surface rupture due to displacement along the Pen Branch fault exists in the site area or site vicinity. SER Section 2.5.1.3.4 presents the staff's evaluations and conclusions regarding all new information that was collected by the applicant to assess the Pen Branch fault. This information was used to support the applicant's conclusions that the Pen Branch fault does not exhibit surface rupture or Quaternary (1.8 mya to present) displacement and is not a capable tectonic feature at the ESP site.

2.5.3.3.2 Geologic Evidence for Surface Deformation

In SSAR Section 2.5.3.2, the applicant described four bedrock faults identified within the site area. These structures include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults, which the applicant discussed in SSAR Sections 2.5.3.2.1, 2.5.3.2.2, 2.5.3.2.3, and 2.5.3.2.4, respectively. Based on information presented in SSAR Sections 2.5.3.2 and 2.5.1.2.4, the applicant concluded that none of the four faults mapped within the site area shows any evidence of surface rupture and that none of the faults is a capable tectonic source. The staff's evaluation of SSAR Section 2.5.3.2, including Sections 2.5.3.2.1, 2.5.3.2.2, 2.5.3.2.3, and 2.5.3.2.4, is presented below.

The staff focused its review of SSAR Section 2.5.3.2 on the applicant's descriptions of the four bedrock faults mapped within the site area. The staff concludes that the applicant presented accurate descriptions of these four faults to enable assessment of the potential for tectonic surface deformation within the site area. This assessment is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.2, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant that none of these four faults exhibits surface displacement and none is a capable tectonic feature.

The rationale for the staff's conclusions in regard to the existence of surface faulting in the site vicinity and at the site, particularly in relation to the Pen Branch fault, is presented in detail in SER Section 2.5.1.3.4, which discusses geology of the site area. Also in SER section 2.5.1.3.4, the staff presents a summary of the lines of evidence cited by the applicant in the SSAR to indicate that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.3 Correlation of Earthquakes with Capable Tectonic Sources

In SSAR Section 2.5.3.3, the applicant described the distribution of epicenters for instrumentally recorded earthquakes that have occurred in the site vicinity (within an 8-km (5-mi) radius). The applicant stated that neither historical nor instrumentally recorded earthquake epicenters show a correlation with known or postulated faults in the site vicinity. Based on information presented in

SSAR Section 2.5.3.3, as well as in SSAR Section 2.5.1.1.4.3 and SSAR Figure 2.5.1-16, the applicant concluded that no spatial correlation exists between earthquake epicenters and known or postulated faults in the site vicinity or site area. The staff's evaluation of SSAR Section 2.5.3.3 is presented below.

The staff focused its review of SSAR Section 2.5.3.3 on the applicant's description of historical and instrumentally recorded earthquake epicenters and faults that have occurred within the site vicinity. The staff concludes that the applicant presented convincing data and logical interpretations related to a lack of correlation between earthquakes and tectonic sources in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.3, as well as information presented by the applicant in SSAR Section 2.5.1.1.4.3 and SSAR Figure 2.5.1-16, the staff concurs with the applicant's conclusion that no spatial correlation exists between earthquake epicenters and faults in the site vicinity or site area.

2.5.3.3.4 Ages of Most Recent Deformations

In SSAR Section 2.5.3.4, the applicant discussed information related to ages of the most recent deformations indicated for the four bedrock faults identified within the site area (i.e., the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults). Based on information presented in SSAR Sections 2.5.3.4 and 2.5.1.2.4, the applicant concluded that none of these four faults exhibits Quaternary displacement and none is considered a capable tectonic structures. For the Pen Branch fault, the applicant stated that there is no evidence indicating this fault has experienced displacement younger than Eocene (i.e., less than 33.7 mya). The Pen Branch fault is of particular interest to the staff because it underlies the ESP site. The staff's evaluation of SSAR Section 2.5.3.4 is presented below.

The staff focused its review of SSAR Section 2.5.3.4 on the applicant's discussion of the ages of most recent deformations indicated for the four bedrock faults mapped within the site area. The staff concludes that the applicant presented accurate descriptions of the ages of deformation for these four faults in order to enable an accurate assessment of Quaternary displacement along faults within the ESP site area and at the ESP site. This assessment is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.4, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant's conclusion that none of these four faults exhibits Quaternary displacement.

The rationale for the staff's conclusions in regard to the ages of most recent deformation, specifically for the Pen Branch fault, is presented in detail in SER Section 2.5.1.3.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence used by the applicant in the SSAR indicating that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.5 Relationship of Site Area Tectonic Features to Regional Tectonic Structures

In SSAR Section 2.5.3.5, the applicant discussed the four faults identified within the site area. These structures include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults, which the applicant discussed in SSAR Sections 2.5.3.5.1, 2.5.3.5.2, 2.5.3.5.3, and 2.5.3.5.4, respectively. Of these four faults, the applicant indicated that only the Pen Branch fault occurs west of the SRS on the ESP site. Based on information presented in SSAR Section 2.5.3.5, the applicant concluded that none of the four faults is considered a capable tectonic feature within the site area, effectively concluding that none is linked with any capable regional tectonic structure. The staff's evaluation of SSAR Section 2.5.3.5 is presented below.

The staff focused its review of SSAR Section 2.5.3.5 on the applicant's descriptions of these four faults identified within the site area. The staff concludes that the applicant presented accurate descriptions of these four faults to enable assessment of possible linkage with regional tectonic structures in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.5, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the conclusions of the applicant that none of the four faults is a capable tectonic feature and none is linked with a capable regional tectonic structure.

2.5.3.3.6 Characterization of Capable Tectonic Sources

In SSAR Section 2.5.3.6, the applicant stated that no capable tectonic sources occur within the site area. The applicant summarized the data supporting a noncapable status for the Pen Branch fault. Based on information presented in SSAR Section 2.5.3.6, the applicant concluded that no capable tectonic sources exist in the site area that would require characterization. The staff's evaluation of SSAR Section 2.5.3.6 is presented below.

The staff focused its review of SSAR Section 2.5.3.6 on the applicant's description of the Pen Branch fault. The staff concludes that the applicant presented an accurate summary to enable assessment of the capability of the Pen Branch fault in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), and 10 CFR 100.23(c), 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.6, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant's conclusion that no capable tectonic sources exist in the site area requiring characterization, including the Pen Branch fault.

The rationale for the staff's conclusions in regard to the noncapability of the Pen Branch fault is presented in detail in SER Section 2.5.1.2.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence used by the applicant in the SSAR indicating that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.7 Designation of Zones of Quaternary Deformation for Detailed Investigation

In SSAR Section 2.5.3.7, the applicant concluded that there are no zones of Quaternary deformation within the site area which require detailed investigation. The applicant based its conclusion on data presented in SSAR Sections 2.5.1.2.4, 2.5.3.2, 2.5.3.4, and 2.5.3.5. The staff's evaluation of SSAR Section 2.5.3.7 is presented below.

The staff focused its review of SSAR Section 2.5.3.7 on the applicant's descriptions of faults identified in the site area and discussed in SSAR Sections 2.5.1.2.4, 2.5.3.2, 2.5.3.4, and 2.5.3.5. The staff concludes that the applicant presented accurate descriptions of faults identified in the site area to enable an assessment of Quaternary deformation within the site area and at the ESP site in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of this information, the staff concurs with the applicant's conclusion that there are no zones of Quaternary deformation within the site area that require a detailed investigation.

The rationale for the staff's conclusions in regard to a lack of Quaternary deformation in the site area is presented in detail in SER Section 2.5.1.3.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence cited by the applicant in the SSAR to indicate that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.8 Potential for Surface Tectonic Deformation

In SSAR Section 2.5.3.8.1, the applicant stated that the Pen Branch fault is noncapable and will not cause surface rupture in the future. The applicant also stated that the nonbrittle folding of the Blue Bluff Marl, interpreted to result from displacement along the Pen Branch fault, indicates near-surface tectonic deformation that is not younger than Eocene (i.e., less than 33.7 mya). Based on information summarized in SSAR Section 2.5.3.8.1, which is discussed in more detail by the applicant in SSAR Section 2.5.1.2.4.2, the applicant concluded that the potential for tectonic deformation at the site is negligible. The staff's evaluation of SSAR Section 2.5.3.8.1 is presented below.

The staff focused its review of SSAR Section 2.5.3.8.1 on the applicant's discussion of near-surface tectonic deformation interpreted by the applicant to result from displacement along the Pen Branch fault more than 33.7 mya. The staff concludes that the applicant presented an accurate discussion of the field data indicating no displacement younger than Eocene along the Pen Branch fault in the site area. This assessment is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Sections 2.5.3.8.1 and 2.5.1.2.4.2, the staff concurs with the conclusion of the applicant that the potential for tectonic deformation at the site is negligible.

2.5.3.3.9 Potential for Nontectonic Deformation

In SSAR Section 2.5.3.8.2, the applicant discussed dissolution collapse features (SSAR Section 2.5.3.8.2.1) and "clastic" dikes (SSAR Section 2.5.3.8.2.2). Based on information presented in SSAR Section 2.5.3.8.2.1, the applicant stated that dissolution collapse features are not considered to be tectonic structures or paleoseismic features, and concluded that they do not represent a safety issue for the ESP site in regard to nontectonic surface deformation. Based on information presented in SSAR Section 2.5.3.8.2.2, the applicant indicated that two types of so-called "clastic" dikes occur in the site area: (1) sand dikes that resulted from injection of poorly-consolidated, liquefied fine sand into overlying sediments; and (2) pedogenic clastic dikes related to weathering and soil formation (i.e., pedogenic) processes that were enhanced along fractures. The applicant stated that these two types of dikes are also not tectonic structures or paleoseismic features and likewise concluded that they do not represent a safety issue for the ESP site in regard to nontectonic surface deformation. The staff's evaluation of SSAR Section 2.5.3.8.2 is presented below.

The staff focused its review of SSAR Section 2.5.3.8.2 on the applicant's descriptions of the modes of formation of the dissolution collapse features and "clastic" dikes (i.e., both the injection type and the pedogenic clastic type) because the applicant used this descriptive information to conclude that these features resulted from nontectonic deformation. The applicant also referred to "small-scale deformation features" in SSAR Sections 2.5.3.1.2 and 2.5.3.1.7, considered by

McDowell and Houser (1983) and Bartholomew et al. (2002) to be possible evidence of tectonic activity. The applicant stated in SSAR Sections 2.5.3.1.2, 2.5.3.1.7, and 2.5.3.8.2.2 that these small-scale features are considered to be nontectonic in origin based on observations made by the applicant during field reconnaissance studies performed for the ESP application. However, the applicant did not fully discuss the field observations and reasoning used to conclude that these small-scale deformation features are nontectonic in origin, and did not provide adequate information about the origin of the injection sand dikes or the pedogenic clastic dikes.

In RAI 2.5.3-1, the staff asked the applicant to more clearly describe its logic for concluding that the deformation features mapped and described by McDowell and Houser (1983) and Bartholomew et al. (2002) are nontectonic in origin. In RAI 2.5.3-2, the staff asked the applicant for additional information on field data used by the applicant to conclude that both the injection sand dikes and the pedogenic clastic dikes are nontectonic in origin. This clarification is important because paleoliquefaction features related to the 1886 Charleston earthquake or other previous seismic events are known to occur in the region, and the staff must ensure that none of the features described by the applicant in SSAR Sections 2.5.3.1.2, 2.5.3.1.7, and 2.5.3.8.2.2 are related to Quaternary tectonic deformation.

In response to RAI 2.5.3-1, the applicant stated that, based on reconnaissance of exposures in the site area, certain primary characteristics of the pedogenic type of clastic dikes suggested an origin consistent with weathering and soil forming processes for these features. Specifically, (1) the dikes are widely distributed in deeply weathered clayey and silty sands of the Hawthorne Formation and the Barnwell Group formations; (2) the dikes occur in nearly all exposures of the weathered profile, but are generally absent in exposures of stratigraphically lower, less weathered sedimentary units; (3) the dikes contain a central zone of bleached host rock bounded by a cemented zone of iron oxide and may contain a clay core; (4) grain-size analyses indicate that the dikes contain the same grain-size distribution as the host sediment, but with more silt and clay; and (5) the dikes decrease downward in width and density, usually tapering and pinching out over a distance of 5 to 15 feet. The applicant indicated that the "clastic" dikes identified by Bartholomew et al. (2002) are syndepositional, as indicated by the presence of marine animal burrows crossing the dikes, and that they developed in a subaqueous marine environment during the Late Eocene (i.e., more than 33.7 mya). Based on these lines of evidence, the applicant concluded that the clastic dikes observed in the site area are pedogenic, and not tectonic, in origin. The applicant also concluded that the clastic dikes described by Bartholomew et al. (2002), whether their origin is tectonic or nontectonic, developed more than 33.7 mya.

Based on its review of the applicant's response to RAI 2.5.3-1, the staff concurs with the applicant's conclusion that the clastic dikes described by Bartholomew et al. (2002) are older than 33.7 mya. The staff further concludes, in agreement with the applicant, that the clastic dikes observed in the site area are the result of pedogenic processes and are nontectonic in origin.

In response to RAI 2.5.3-2, the applicant indicated that the deformation features (i.e., warped bedding, fractures, small-scale faults, injection sand dikes, and clastic dikes), interpreted by the applicant to be nontectonic in origin, occurred in a garbage trench on the VEGP site mapped by the Bechtel staff in 1984. The trench (now filled but illustrated in SSAR Figures 2.5.3-1 and 2.5.3-2, as well as in Figure 2.5.3-2A accompanying the applicant's RAI response) contained a monocline in the Blue Bluff Marl that is interpreted by the applicant as related to Eocene displacement along the Pen Branch fault. The monocline is positioned above the subsurface line of intersection of the Pen Branch fault with the contact of basement rock and Coastal Plain sediments.

In response to RAI 2.5.3-2, the applicant also stated that the local spatial relationships of warped bedding, fractures, and small-scale faults with the margins of dissolution depressions clearly demonstrate a nontectonic, dissolution collapse origin for these features. The applicant cited the trench map produced by Bechtel (1984), illustrated in Figure 2.5.3-2A, which accompanied its response to RAI 2.5.3-2, as conclusive evidence for this statement. The applicant reiterated the five primary characteristics of clastic dikes presented in its response to RAI 2.5.3-1, which suggested an origin consistent with a pedogenic origin for these features. In response to RAI 2.5.3-2, the applicant further indicated that the injection sand dikes likely were formed by fluid or plastic injection of an underlying source sand and that the close spatial association of the injection dikes with the sides of dissolution collapse depressions suggests that this type of dike is also related to a nontectonic, dissolution collapse origin. The applicant also stated that the injection sand dikes likely formed prior to an erosional event that occurred at the end of the Miocene (i.e., more than 5.3 mya), but did not discuss the basis for this statement in detail in the RAI response. The applicant stated that clastic dikes developed during a weathering event that is older than Late Pleistocene (i.e., more than 10,000 years ago). Based on its review of the applicant's response to RAI 2.5.3-1, the staff concurs with the applicant that the clastic dikes described by Bartholomew et al. (2002) are older than 33.7 mya. The staff further concludes, in agreement with the applicant, that the clastic dikes observed in the site area are the result of pedogenic processes and are nontectonic in origin. Based on its review of the applicant's response to RAI 2.5.3-2, the staff concludes that the response qualifies timing of the development of warped bedding, fractures, small-scale faults, clastic dikes, and injection sand dikes. The timing of that development as suggested by information presented by the applicant is as follows:

1. Deposition of Tertiary (i.e., a range of 65 to 1.8 mya in age) sedimentary units, including at least Eocene (54.8 to 33.7 mya) and Miocene (23.8 to 5.3 mya) sediments, with some periods of subaerial (i.e., above water in open air) erosion.

2. Initiation of dissolution of the Utley Limestone (Late Eocene in age) at the base of the Eocene Barnwell Group, with development of incipient depressions and formation of injected sand dikes in Barnwell Unit "D" above the Utley Limestone as illustrated in Figure 2.5.3-2A of the applicant's response to RAI 2.5.3-2. The initiation of dissolution and development of the injected sand dikes occurred after deposition of the sedimentary units in which they are found, and the applicant reported Late Pleistocene (more than 10,000 years in age) to Holocene (less than 10,000 years in age) sands that do not appear to be deformed overlying the warped bedding, fractures, small-scale faults, clastic dikes, and injection sand dikes in the trench mapped by Bechtel (1984).

3. Continued and increasing dissolution of the Utley Limestone, with numerous nontectonic dissolution collapse features developed in overlying units, including collapse-generated faults that cut, and consequently postdate, the injected sand dikes. Consequently, the injected sand dikes are the oldest of the deformation features mapped that the applicant equated with a response to nontectonic near-surface deformation.

4. Development of nontectonic clastic dikes above the sedimentary units that experienced dissolution collapse, many in the Miocene-age Hawthorne Formation based on Figure 2.5.3-2A of the applicant's response to RAI 2.5.3-2. The clastic dikes do not extend into

Late Pleistocene to Holocene-age sands, indicating that the clastic dikes are at least 10,000 years old.

The staff concludes that the evidence presented by the applicant in the response to RAI 2.5.3-2 clearly documents a nontectonic origin for the warped bedding, fractures, small-scale faults, and clastic dikes.

In regard to the origin of the injection sand dikes, the applicant made the case that these features are the oldest structures generated by nontectonic deformation in the site area. That is, the applicant considered that the injection sand dikes are not related to paleoliquifaction resulting from Quaternary tectonic deformation and seismic shaking in the site area. From information presented by the applicant in the SSAR and its response to RAI 2.5.3-2, the staff concludes that the injection sand dikes are the oldest of the observed features, and the age constraints discussed by the applicant appear to limit the youngest timing for development of these features to earlier than Late Pleistocene (i.e., more than 10,000 years in age) and possibly Pliocene (5.3 to 1.8 mya). This upper age limit for the injection sand dikes is supported by information provided by the applicant in the response to RAI 2.5.3-2, suggesting that the dikes pre-date an erosional event at or near the end of the Miocene (23.8 to 5.3 mya). Consequently, even if the injection sand dikes were the result of seismically-induced paleoliquefaction, the features are not Holocene (10,000 years to present) in age. However, a Pleistocene age (1.8 mya to 10,000 years) is not precluded for the injection sand dikes based on information provided by the applicant in the response to RAI 2.5.3-2.

The staff concurs with the applicant that no evidence exists to indicate that any of these features represent a safety issue for the ESP site in regard to nontectonic surface or near-surface deformation. However, in developing the SER with open items, the staff considered that the applicant's response to RAI 2.5.3-2 in regard to the injection sand dikes did not provide adequate information to bracket the pre-Miocene upper age limit for development of this feature as suggested by the applicant. Furthermore, the staff considered that the applicant did not clearly show that the injection sand dikes are spatially related to what must have been incipient dissolution depressions (i.e., much of the dissolution must have occurred after development of the injection sand dikes since, as the applicant pointed out, nontectonic small-scale faults associated with dissolution collapse cross-cut the injection dikes). Since the mechanism described by the applicant as responsible for the sand injection (i.e., fluid or plastic injection of the liquefied source sand) could be associated with seismic shaking and liquefaction of the sand materials, the staff formulated Open Item 2.5-10 to request that the applicant provide a more detailed description of geometry and physical characteristics of the injection sand dikes and their spatial association with dissolution depressions. The applicant's response and the staff's evaluation in regard to this open item are presented below.

In response to Open Item 2.5-10, the applicant cited all available field evidence used to interpret the injection sand dikes as nontectonic in origin (i.e., unrelated to seismic shaking and resultant liquefaction of materials) and pre-Quaternary in age. The applicant presented the following field evidence and logic to support its conclusions in regard to the injection sand dikes:

1. All injection sand dikes were found at a single location at the site and occurred within stratigraphic horizon "Unit D" of the Upper Eocene (more than 33.7 mya) Barnwell Group in the Coastal Plain sedimentary sequence.

2. The dikes registered upward movement of liquefied sands from a sand source in stratigraphic Unit C of the Barnwell Group, which directly underlies Unit D. The dikes,

which penetrated and were confined to Unit D, clearly flattened along the base of Barnwell stratigraphic Unit E, which directly overlies Unit D. Since Units C, D, and E are Upper Eocene Barnwell Group stratigraphic horizons that sequentially overlie each other from C to E, all units involved are older than 33.7 mya.

3. The injection sand dikes appear to be spatially related to areas of localized dissolution at depth in the Utley Limestone, as shown by location of the sand dikes in relation to surface morphology of Unit F (Upper Eocene Barnwell Group) in Figure 2.5-10B which accompanied the applicant's response to Open Item 2.5-10. The surface of Unit F clearly reflects a dissolution-related morphology of generally circular to elongated depressions due to the collapse of overlying sediments as dissolution of the underlying Utley Limestone occurred.

4. Based on the three field observations stated above, the applicant proposed a sand injection mechanism related to the response of sands in Unit C to increased overburden pressure associated with an early phase of collapse of sedimentary units overlying dissolution depressions in the Utley Limestone.

5. The Hawthorne Formation (Miocene, 23.8 to 5.3 mya) is the youngest unit showing effects of dissolution at depth (i.e., the "dissolution-related morphology" described above in Item 3). An erosion surface/relict paleosol (i.e., an earlier soil horizon that has persisted without major alteration of its morphology) overlying the Hawthorne does not show these effects. The applicant interpreted the erosion surface/paleosol to be Late Miocene to Pliocene in age (i.e., Late Tertiary, more than 1.8 mya, and therefore pre-Quaternary).

6. The erosion surface/paleosol is in turn overlain by Pleistocene-Holocene (less than 1.8 mya) eolian sands, which the applicant also reported showed no morphological effects of dissolution at depth.

7. Based on stratigraphic ages of units reflecting the dissolution-related morphology, the applicant interpreted the dissolution to be no younger than Late Miocene-Pliocene (i.e., more than 1.8 mya). By association, the injected sand dikes are also interpreted by the applicant to be no younger than Late Miocene-Pliocene.

The staff considers that the applicant used all available field evidence as cited above to conclude that the injected sand dikes formed in response to movement of liquefied sands resulting from collapse of overlying sediments related to dissolution at depth, rather than in response to liquefaction of saturated sands resulting from seismic shaking, and are most likely no younger than Late Miocene-Pliocene (i.e., more than 1.8 mya, so pre-Quaternary). Although the staff was not able to examine the injected sand dikes in the field because the trench in which they occurred is now filled, the applicant did show that the dikes are spatially related to areas of localized dissolution at depth in the Utley Limestone. Furthermore, the dikes are wholly confined to Upper Eocene sediments that are older than 33.7 mya, and it is not likely that such features would have been produced in units this old by historical seismicity and associated liquefaction. The applicant used stratigraphic constraints to suggest relative timing of dike formation (i.e., the applicant presented relative ages, rather than absolute age dates derived from radiometric dating methods). Use of stratigraphic data to determine relative age of a geologic feature is a standard method that is often applied when radiometric age dates are not available, and staff agrees that use of this method is appropriate in this case. In light of the information presented in the applicant's detailed response to Open Item 2.5-10, the staff agrees with the conclusions drawn by the applicant that the injection sand dikes are nontectonic in nature and pre-Quaternary in age. Therefore, Open Item 2.5-10 is resolved.

Based on a review of information presented by the applicant in SSAR Section 2.5.3.8.2 and the responses to RAI 2.5.3-1, RAI 2.5.3-2, and Open Item 2.5-10, the staff concurs with the applicant's conclusion that warped bedding, fractures, small-scale faults, clastic dikes, and injection sand dikes represent nontectonic deformation. The staff concludes that the applicant presented thorough descriptions of these features to enable assessment of nontectonic surface or near-surface deformation within the site area and at the ESP site in support of the ESP application as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on review of SSAR Section 2.5.3 and the applicant's responses to RAIs and Open Item 2.5-10 as set forth above, the staff concludes that the applicant properly characterized the potential for surface and near-surface tectonic and nontectonic deformation at the ESP site, including the possibility of Quaternary tectonic deformation along the Pen Branch fault. The staff also concludes that SSAR Section 2.5.3 provides accurate and thorough descriptions of the potential for surface and near-surface tectonic and nontectonic deformation at the ESP site, with emphasis on the Quaternary Period, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

2.5.3.4 Conclusions

As set forth in SER Sections 2.5.3.1, 2.5.3.2, and 2.5.3.3, the staff carefully reviewed the information on surface faulting submitted by the applicant in SSAR Section 2.5.3. On the basis of its detailed review, as fully described in the above SER sections, the staff concludes that the applicant provided a thorough and accurate characterization of surface and near-surface faulting and nontectonic deformation at the site as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff concurs that data and analyses presented by the applicant in the SSAR provide an adequate basis to conclude that there is no evidence to indicate that surface or near-surface faulting or nontectonic deformation presents a hazard for the site area.

Based on information from the applicant's thorough review of the literature on site area geology in regard to surface expression of faulting, and the applicant's literature review and geologic, seismic, and geophysical investigations of the site vicinity and site area, the staff further concludes that the applicant has properly characterized the potential for surface or near-surface faulting and nontectonic deformation at the ESP site.available, and staff agrees that use of this method is appropriate in this case. In light of the information presented in the applicant's detailed response to Open Item 2.5-10, the staff agrees with the conclusions drawn by the applicant that the injection sand dikes are nontectonic in nature and pre-Quaternary in age. Therefore, Open Item 2.5-10 is resolved.

2.5.4 Stability of Subsurface Materials and Foundations

Section 2.5.4 of this SER evaluates the stability of subsurface materials and foundations at the site of Vogtle Electric Generating Plant (VEGP) Units 3 and 4. Section 2.5.4.1 of this SER provides a summary of the relevant geologic and seismic information contained in Section 2.5.4 of the Site Safety Analysis Report (SSAR) of the VEGP Units 3 and 4 Early Site Permit (ESP) application and LWA request. SER Section 2.5.4.3 provides the staff's evaluation of SSAR Section 2.5.4, including with respect to the applicant's responses to any requests for additional information, the resolution of open items, and the results of confirmatory analyses performed by the staff. SER Section 2.5.4.4 summarizes the applicant's conclusions as well as the staff's conclusions, and confirms that the applicable regulations have been met by the applicant.

2.5.4.1 Summary of Application

With respect to the stability of subsurface materials and foundations, the SSAR addresses information items contained in the AP1000 Standard Plant Design, Design Control Document (DCD), Revision 15. The applicant developed geological, geophysical, geotechnical, and seismological information to be used as the basis for the evaluation of the stability of the subsurface materials and foundations at the proposed site. The applicant initially reviewed analyses and reports prepared for the existing VEGP Units 1 and 2 as well as the readily available geotechnical literature. The applicant then conducted field investigations and performed field and laboratory testing during the initial phase of the ESP site subsurface investigation. These subsequent investigations were conducted with the intent of obtaining additional site information to further the understanding of the VEGP site and to complement the existing geotechnical data from the previous investigations completed for VEGP Units 1 and 2.

The applicant augmented the ESP field and laboratory test data with field and laboratory data from an investigation it performed in support of a Limited Work Authorization (LWA) request, which the applicant submitted on August 16, 2007. In addition to performing this investigation to support the LWA request, the applicant conducted comprehensive site geotechnical field and laboratory investigations to enhance the existing ESP geotechnical data as well as to support the COL application that the applicant submitted to the NRC on March 31, 2008. This additional data allowed the applicant to further develop and understand the geotechnical data at the specific locations proposed for the VEGP Units 3 and 4 site structures and at the locations of the proposed borrow sources for the structural backfill materials. Because the staff determined that this additional information was necessary only to the staff's finding associated with the LWA request, the staff has summarized and evaluated these additional data and analyses separately from the ESP information in this section and Section 2.5.4.3, respectively. Finally, the applicant conducted a test pad program in support of the LWA request to establish site-specific design properties for the structural backfill and to verify that the proposed backfill materials would meet the AP1000 standard design siting criteria.

2.5.4.1.1 Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Sections 2.5.1.1 and 2.5.1.2 for detailed descriptions of the regional and site geology, including structural geology, physiography, geomorphology, geologic history, stratigraphy, structures, and hazards.

2.5.4.1.2 Properties of Subsurface Materials

SSAR Section 2.5.4.2 describes the static and dynamic engineering properties of the subsurface materials at the ESP site. In this section, the applicant described the subsurface materials, field investigations, laboratory tests, and the engineering properties it determined for the subsurface materials. The applicant also described the ESP and COL investigations and results for each stratum.

In support of the ESP application, the applicant submitted the following information:

Description of Subsurface Materials

SSAR Section 2.5.4.2.2 provides an overview of the subsurface profile and materials, including detailed descriptions of the underlying strata. The applicant categorized the soils underlying the ESP site into three groups based on their stability for geotechnical purposes. Group 1 soils include sands with silt and clay, Group 2 is the Blue Bluff clay marl layer, and Group 3 is made up of coarse-to-fine sand with interbedded thin seams of silt or clay. The applicant stated that the Group 1 soils would be completely removed and replaced with compacted backfill prior to construction of VEGP Units 3 and 4. In addition to grouping the soils, the applicant divided the VEGP site soils and bedrock into five strata:

1. Upper Sand Stratum (Group 1: Barnwell Group)
2. Marl Bearing Stratum (Group 2: Blue Bluff Marl or Lisbon Formation)
3. Lower Sand Stratum (Group 3)
4. Dunbarton Triassic Basin Bedrock
5. Paleozoic Crystalline Bedrock

The applicant developed the static design and engineering properties of the five strata from field and laboratory tests that it performed during the ESP and COL subsurface investigations, the results of which are summarized in Table 2.5.4-1 of this SSAR. A brief description of each stratum is provided below, including the soil and rock constituents and their ranges of thickness at the site. The applicant determined this information from 14 borings and 10 cone penetrometer tests (CPT) that it performed during the ESP subsurface investigation, and from 70 borings and 8 CPTs performed during the COL investigation. SSAR Figure 2.5.4-1a (Figure 2.5.4-1 of this SER) shows the locations of most of the ESP and COL borings. The applicant also provided cross-sectional profiles of subsurface conditions across the site and the nuclear island (SSAR Figures 2.5.4-3 through 2.5.4-5b; Figure 2.5.4-2 of this SER).

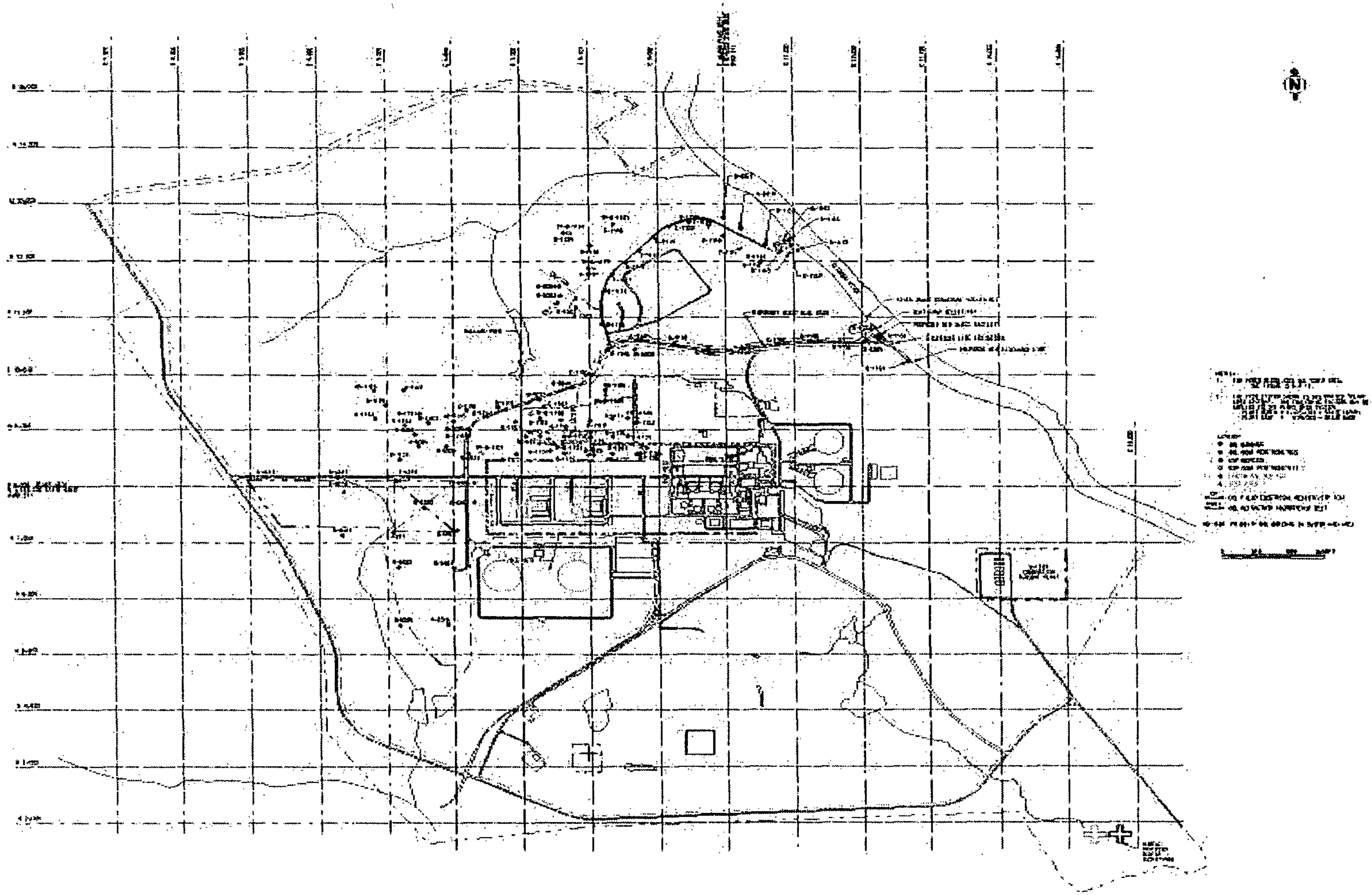


Figure 2.5.4-1 COL Site Boring Plan, Including Locations of ESP and Units 1 and 2 Borings (SSAR Figure 2.5.4-1a)

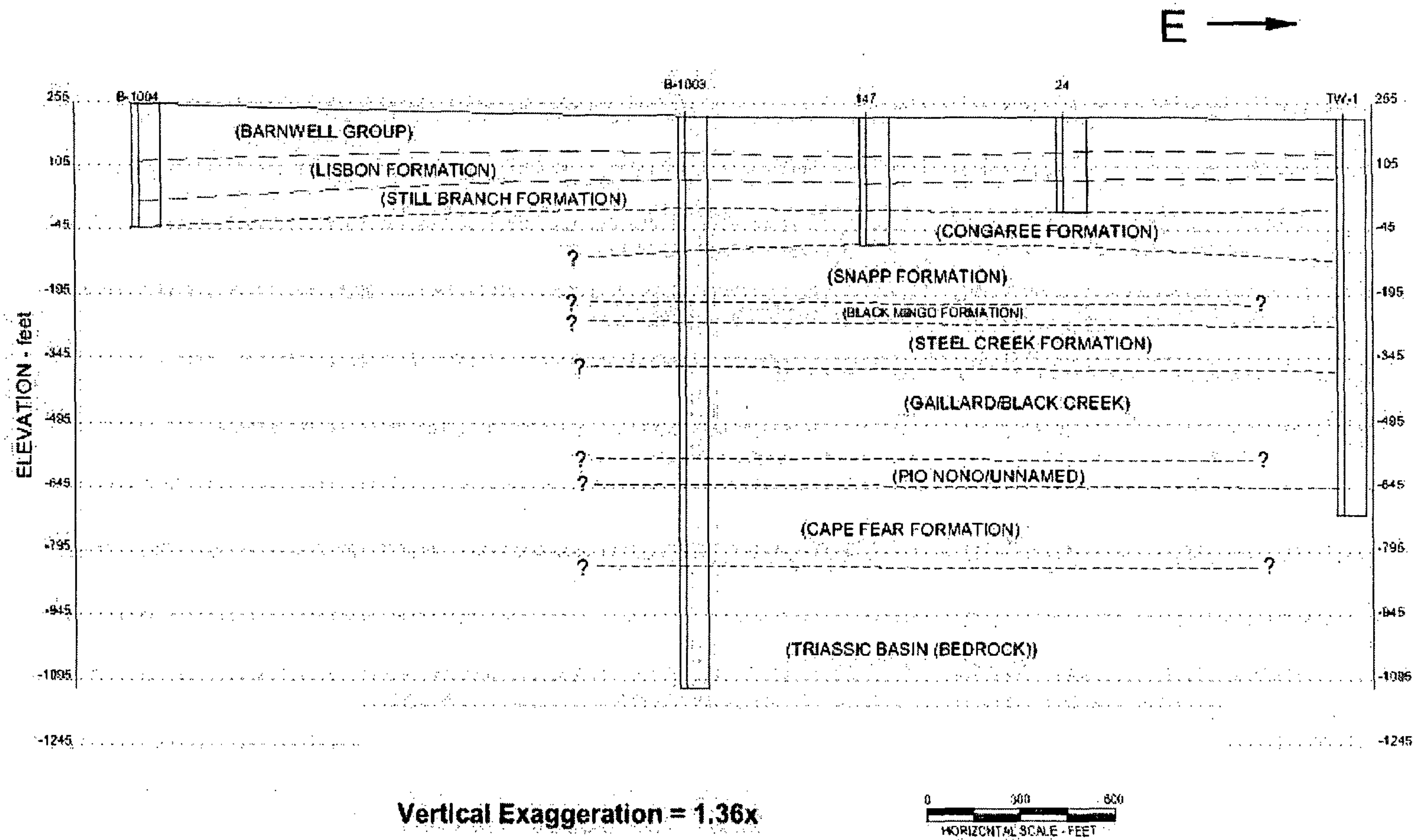


Figure 2.5.4-2 Subsurface Profile B-B' from SSAR Figure 2.5.4-1 (SSAR Figure 2.5.4-4)

1. Upper Sand Stratum (Barnwell Group). SSAR Subsection 2.5.4.2.2.1 describes the Upper Sand Stratum, or Barnwell Group, as consisting of predominantly sands, silty sands, and clayey sands with occasional clay seams, soft zones, and shell zones. The applicant encountered a shelly limestone layer, the Utley limestone, which contains significant solution channels, cracks, and other discontinuities, and observed severe fluid loss in the stratum while drilling. The applicant also determined that the stratum ranged in thickness from 24 to 48 meters (m) (78 to 157 feet (ft)), and attributed the large range to the westerly to northwesterly dip of the underlying Blue Bluff Marl. Based on its review of previous investigations for Units 1 and 2, the applicant determined that the Upper Sand Stratum is susceptible to liquefaction during seismic ground motion equivalent to the safe shutdown earthquake (SSE). The applicant found that the relative density of the stratum is highly variable, ranging from very loose to dense with clay lenses within the stratum ranging from soft to medium stiff. Therefore, the applicant concluded that the entirety of the Upper Sand Stratum, including the limestone layer, would need to be completely removed before it begins construction for VEGP Units 3 and 4.

   The applicant performed field Standard Penetration Tests (SPT) within the Upper Sand Stratum and obtained very high blow count values indicative of the previously observed shelly limestone and shell hash (mixture or pieces of shell) zones. Samples were recovered by the applicant at varying depths within the stratum and submitted for laboratory testing, including percent fines, moisture content, and Atterberg Limits (a measure of the relationship between percentage of fines and water content that affects the ability of a soil to remain plastic). The applicant indicated that the test results for percent fines ranged from 3 to 60 percent and 5 to 96 percent for the ESP and COL investigations, respectively, suggesting the stratum was made up of very fine grained sands, silts, and clay particles. From the results of the Atterberg Limits tests, the applicant determined a liquid limit of 43 to 97 for ESP investigations and an average of 72 for COL investigations. The applicant also determined a range of plasticity index from 21 to 67 for ESP investigations and an average index of 39 for COL investigations, indicating that the stratum's materials were inorganic and organic silts and clays of high plasticity. The natural moisture content of samples the applicant tested for Atterberg Limits ranged from 20 to 93 percent for the ESP investigations and again indicated the highly variable and fine grained nature of the sand, silt, and clay materials. The applicant calculated moist unit weights from 1,505 to 1,986 kilograms per cubic meter (kg/m^3; 94 to 124 pounds per cubic feet (pcf)) for fifteen samples, and specific gravities of 2.7 and 2.8 for two samples.

2. Blue Bluff Marl (Lisbon Formation). SSAR Subsection 2.5.4.2.2.2 describes the Blue Bluff Marl, which underlies the Upper Sand Stratum, in much greater detail because it is the load-bearing stratum at the proposed site of VEGP Units 3 and 4. The applicant stated that the Blue Bluff Marl consists of hard, slightly sandy, cemented, overconsolidated, calcareous clay with some shells and partially cemented, well-hardened layers varying between 19 to 29 m (63 to 95 ft) in thickness, with an average thickness of 23 m (76 ft) and a design ground water level at a depth of 16.7 m (55 ft). The top of the Blue Bluff Marl was mapped by the applicant between Elevation 37 and 42 m (122 and 140 ft) dipping downward towards the west side of the VEGP site. The applicant relied on 70 soil borings as part of its COL subsurface investigations to confirm its earlier ESP investigations of the Blue Bluff Marl. This reliance is especially important in the immediate area of the nuclear island, where 42 of the applicant's 70 borings penetrated into the Blue Bluff Marl layer. The applicant also considered the previous investigations completed for Units 1 and 2 to further determine the subsurface properties of the Blue Bluff Marl.

The applicant conducted a series of standard penetration tests (SPTs) within the marl layer at the VEGP site. The results of SPTs are reported as the total blows summed over the distance to give blows per meter (or per foot), a measure commonly referred to as the N-value. The average N-values from the SPTs conducted as part of the ESP and COL investigations were high, 272 and 233 blows per meter (bpm) (83 and 71 blows per foot (bpf)), respectively, which the applicant attributed to the hard to very hard consistency of the fossiliferous limestone, and cemented layers and nodules of the marl. As expected, the applicant noted that the SPT N-values increased with depth. Finally, although the applicant noted the presence of soft zones (N-values below 16 bpm (5 bpf)) in the marl at the adjacent Savannah River Site (SRS), none of the SPTs conducted on the marl underlying the VEGP site yielded N-values less than 30.48 bpm (10 bpf): therefore, the applicant concluded that soft zones were not present in the marl beneath the site of VEGP Units 3 and 4.

The applicant recovered samples from within the Blue Bluff Marl during the ESP and COL subsurface investigations and submitted these samples for laboratory testing of percent fines, moisture content, and Atterberg Limits. SSAR Tables 2.5.4-1 thru 2.5.4-4 provide a summary of these laboratory tests. The applicant also provided the average values from both the ESP and COL laboratory tests, which included: 48 and 74 percent fines; plastic limits of 29 and 34 percent; liquid limits of 51 and 67 percent; and a Plasticity Index of 22 and 33 percent, respectively. The natural moisture content of the samples the applicant tested ranged from 14 to 67 percent and 14 to 62 percent for the ESP and COL investigations, respectively, with an average of 35 percent for the ESP investigations and 33 percent for the COL investigations. The applicant also calculated moist unit weights from 1,521 to 2,130 kg/m^3 (95 to 133 pcf) for 69 COL samples, and specific gravities of 2.61 and 2.66 for eight COL samples.

As part of its ESP investigations, the applicant also performed 15 one-point unconsolidated undrained triaxial shear tests on marl stratum samples. From these tests the applicant found that the undrained shear strength of the marl ranged from 7 to 205 kilopascals (kPa) (150 to 4,300 pounds per square foot (psf)), far lower than the undrained shear strength measured by Southern for Units 1 and 2, which was between 12.5 and 23,900 kPa (260 to 500,000 psf). The applicant stated that the disagreement between the two results stems from "severe sample disturbance due to sampling technique (pitcher sampler) and preparation of testing specimen." During the COL investigation, the applicant performed several additional laboratory strength tests on relatively undisturbed marl stratum samples. Specifically, these tests included 27 unconfined compression, 11 UU triaxial, and 27 consolidated undrained (CU) triaxial tests. The applicant reported that the average undrained shear strength from the UU and CU tests was 564 kPa (11,800 psf), which supported the design value of 478 kPa (10,000 psf) obtained for Units 1 and 2.

The applicant monitored the average heave during excavation for Units 1 and 2 and observed an average heave of 3.75 cm (1.25 in.), which corresponded to an undrained Young's modulus value of 478,000 kPa (10,000,000 psf). Using the average value of shear strength results that failed at 2,394 kPa (50,000 psf), which was 766 kPa (16,000 psf), the applicant used the ratio of undrained shear strength to effective overburden pressure to calculate the preconsolidation pressure of 3,830 kPa (80,000 psf) and the overconsolidation ratio of 8. Due to this high preconsolidation pressure and the small foundation settlements measured by Southern during its VEGP Units 1 and 2 settlement monitoring program (less than 9.14 cm (3.6 in.), the applicant concluded that settlements due to new structures would be small. The applicant also measured the in-situ shear wave velocity which was used to calculate the dynamic shear modulus.

3. Lower Sand Stratum. SSAR Subsection 2.5.4.2.2.3 describes the Lower Sand Stratum, the top of which was mapped at a depth of about 50 m (165 ft) below the ground surface beneath the Blue Bluff Marl and underlain by the Dunbarton Triassic Basin rock. The applicant described the units of the stratum collectively as fine to coarse sands with interbedded silty clay and clayey silt, which, from top to bottom were identified as the Still Branch, Congaree, Snapp, Black Mingo, Steel Creek, Gaillard/Black Creek, Pio Nono/Unnamed, and Cape Fear formations. From the ESP subsurface investigations, the applicant determined that the Lower Sand Stratum was 275 m (900 ft) thick at the location of the one borehole (B-1003) that fully penetrated the stratum. Figure 2.5.4-4 of the SSAR illustrates the typical depths of the stratum as observed in B-1003.

   The applicant performed field SPTs during the ESP investigations and obtained an average N-value of 194 bpm (59 bpf). During the COL investigations, the applicant obtained SPT N-values for the Lower Sand in 42 penetrations as deep as 80 m (263 ft) into the unit, which averaged 230 bpm (70 bpf). The applicant observed that for the COL N-values, nearly all were above 98 bpm (30 bpf), indicative of very dense material. Furthermore, as was expected, both the ESP and COL investigation SPT N-values increased with depth. The applicant noted that the only evidence suggesting the presence of soft zones or loose material, a low N-value and lack of sample recovery, was an anomalous condition attributable to disturbed soil conditions at the bottom of the borehole caused by an imbalance between borehole and in-situ hydrostatic pressures.

   During the course of both the ESP and COL investigations, the applicant selected and submitted samples recovered from within the stratum for laboratory testing. The test results for percent fines and Atterberg Limits can be found in SSAR Table 2.5.4-1. The applicant reported that percent fines averaged 23.6 and 23 percent for the ESP and COL investigations, respectively. Atterberg Limit tests were performed as part of the ESP investigation and resulted in an average liquid limit percent of 47 percent, a plastic limit of 30 percent, a moisture content of 30 percent, and an average Plasticity Index of 17 percent. The applicant determined that samples with the higher percent fines and plasticity were from the silty clay and clayey silt layers. As part of the COL investigation, the applicant determined the moist unit weight of sixteen samples ranged from 1,810 to 2,178 kg/m^3 (113 to 136 pcf), with an average specific gravity of 2.67 for four samples.

4. Dunbarton Triassic Basin Rock. SSAR Subsection 2.5.4.2.2.4 describes the Dunbarton Triassic Basin rock as red sandstone, breccia, and mudstone, weathered through the upper 37 m (120 ft). The applicant drilled only one borehole deep enough to encounter the Dunbarton during the ESP investigation, B-1003. The applicant measured shear wave velocity in the upper 84 m (274 ft) of the rock profile and used the results to develop the shear wave velocity profile for site amplification. Finally, the applicant concluded that the rock was too deep to be of any interest to foundation design, and therefore performed no laboratory tests.

5. Paleozoic Crystalline Rock. SSAR Subsection 2.5.4.2.2.5 states that at a depth of 320 m (1,049 ft) below the surface, the applicant encountered the top portion of the weathered Dunbarton Triassic Basin rock. Beneath the adjacent SRS, the southeast dipping non-capable Pen Branch fault separates the Dunbarton Triassic Basin rock from Paleozoic crystalline rock to the northwest, a relationship the applicant suggested may occur at some depth below the VEGP site as well. According to the applicant, the results of a seismic

reflection survey at the VEGP site supported the continuation of the Pen Branch fault beneath the VEGP site, and therefore the presence of Paleozoic crystalline rock as well.

6. Subsurface Profiles. SSAR Figures 2.5.4-3, -4, and -5 present the typical subsurface profiles across the power block areas as determined from the ESP borings. The applicant presented the subsurface profiles across the power block area based on the COL borings in SSAR Figures 2.5.4-3a, -4a, and -5a.

Field Investigations

The applicant presented its field and subsurface investigation programs in SSAR Section 2.5.4.2.3. While the locations of borings completed for Units 1 and 2 were shown on site investigation maps and were referenced by the applicant, the applicant did not include boring logs from these previous investigations. The applicant utilized borings, geophysical surveys, CPTs, seismic CPTs, and test pits as part of the ESP and COL field investigations.

Laboratory Testing

SSAR Section 2.5.4.2.4 describes the laboratory testing of soil samples completed as part of the ESP and COL investigations. The applicant stated that laboratory testing was completed in accordance with the guidance presented in Regulatory Guide 1.138, was performed under an approved quality assurance program with work procedures developed specifically for the ESP and COL applications, and the soil samples were shipped from the onsite storage area to the testing laboratory under Chain-of-Custody procedures. The applicant focused the ESP laboratory test on verifying basic properties of the Upper Sand Stratum, the Blue Bluff Marl and the upper formations of the Lower Sand Stratum. The types and number of tests performed for the ESP investigations are listed in SSAR Table 2.5.4-3, while SSAR Table 2.5.4-4 presents the results. For the COL investigations, the applicant presented the types and number of tests in SSAR Table 2.5.4-3a and the results in Appendix 2.5C. The applicant also performed Resonant Column Torsional Shear (RCTS) testing on samples from the COL investigation and as a part of Phase 1 of the backfill test pad program at the Fugro facility in Houston, TX. The applicant presented the RCTS results for the COL investigation in Appendix 2.5C, Attachment G, while it presented the results for the test Pad program in Appendix 2.5D.

Engineering Properties

SSAR Section 2.5.4.2.5 describes the engineering properties for the soil and rock strata obtained during the ESP and COL subsurface investigations, and the chemical properties deduced as part of the COL investigation. The applicant used data from the COL borings in the immediate vicinity of the VEGP Units 3 and 4 nuclear island power block excavation areas as the basis for the determination of engineering properties. The engineering properties determined during the ESP investigations were derived from both the ESP subsurface and laboratory investigations and the data available from Units 1 and 2. The applicant determined the engineering properties of backfill from the COL and Test Pad program investigations. The applicant compared the properties from the ESP, COL and Test Pad Program to those developed during the previous field and laboratory testing programs conducted for Units 1 and 2 and concluded that the results were similar.

1. Rock Properties. SSAR Subsection 2.5.4.2.5.1 describes the engineering properties of rock at the VEGP Units 3 and 4 site. The applicant based Recovery and Rock Quality Designations (RQD) on results obtained from borehole B-1003, the deepest borehole drilled

during the ESP subsurface investigation, which extended 88 m (290 ft) into the bedrock. Although the applicant did not perform any laboratory strength testing of rock cores due to the extreme depth, suspension P-S velocity seismic testing in the borehole was performed to determine shear and compressional wave velocities.

2. Soil Properties. In SSAR Subsection 2.5.4.2.5.2, the applicant described the properties of the soil as determined from ESP and COL investigations, reviews of previous investigations for VEGP Units 1 and 2, and the Phase I test pad program results. To that end, the applicant performed sieve analyses, natural moisture content, and Atterberg Limits tests on Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples as part of the ESP and COL investigations, and made specific gravity measurements on Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples as part of the COL program. The applicant selected design values using the average of the test results for the respective soil strata.

Laboratory test data, SPT N-values, and shear wave velocity measurements from the ESP and COL investigations were used by the applicant to determine the undrained shear strength of the Blue Bluff Marl stratum. This data included UU and CU test results, in addition to laboratory strength testing data from the previous subsurface investigations and construction of VEGP Units 1 and 2. During the ESP investigation, the applicant correlated the average SPT N-value to an internal angle of friction of 34 and 41 degrees for the Upper and Lower Sand Stratum, respectively. Moist unit weights were determined by the applicant for select Blue Bluff Marl and Lower Sand Stratum samples from the ESP laboratory testing program, and Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples from the COL laboratory testing program. The applicant stated that the average unit weight for 15 ESP marl stratum and 3 Lower Sand Stratum samples was 1,922 and 1,970 kg/m^3 (120 and 123 pcf), respectively. During the COL laboratory testing program, the applicant measured the unit weight of 15 Upper Sand Stratum, 69 Blue Bluff Marl, and 16 Lower Sand Stratum samples, with average unit weights of 1,810, 1,842, and 1,970 kg/m^3 (113, 115, and 123 pcf). The applicant also included the in-situ moist unit weights from previous investigations for the Upper Sand Stratum (1,890 kg/m^3 (118 pcf)), the Blue Bluff Marl (1,906 kg/m^3 (119 pcf)), and the Lower Sand Stratum (1,874 kg/m^3 (117 pcf)).

The applicant compared the design SPT N-values for the ESP investigations with the range and average of the COL and Units 1 and 2 investigations. Based on the ESP results, the applicant concluded that the design SPT N-value for the Upper Sand Stratum (82 bpm (25 bpf)) was within the anticipated range and close to the average. Similarly, the applicant concluded that the design SPT N-value for the Blue Bluff Marl, taken as 328 bpm (100 bpf), also fell within the expected range and near the average N-value. However, when the design SPT N-value for the Lower Sand Stratum (203 bpm (62 bpf)) was compared to the results from the previous investigations, the applicant stated that the design value was less than the assumed range and average.

The applicant measured shear wave velocities by suspension P-S velocity tests and seismic CPTs during the ESP and COL subsurface investigations. Although suspension P-S velocity tests were performed in five ESP boreholes, the applicant acknowledged that only three of the tests extended into the Blue Bluff Marl and Lower Sand Strata, and it therefore the applicant performed tests in six additional COL boreholes. The applicant performed three seismic CPTs for the ESP investigation and eight for the COL; however, due to penetration resistance, the seismic CPTs did not extend into the Blue Bluff Marl. The applicant also determined the shear wave velocities for all strata based on all available data, including measurements from depths of

up to 88 m (290 ft) made during the previous VEGP units 1 and 2 investigations and data from seven deep borings performed at the SRS. The velocity ranges determined by the applicant were: 173 to 1,008 meters per second (m/s) (570 to 3310 feet per second (fps)) within the Upper Sand Stratum, 323 to 1298 m/s (1060 to 4260 fps) within the Blue Bluff Marl, 283 to 1423 m/s (930 to 4670 fps) within the Lower Sand Stratum, and 707 to 2849 m/s (2320 to 9350 fps) within the Dunbarton Triassic Basin. The applicant also calculated average shear wave velocities for the formations in the strata: 286 m/s (940 fps) in the Barnwell Formation and 348 m/s (1142 fps) in the Utley Limestone of the Upper Sand Stratum, 624 m/s (2050 fps) in the Blue Bluff Marl, and 533, 567, and 570 m/s (1750, 1863, and 1871 fps) in the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum, respectively.
SSAR Table 2.5.4-6 lists the shear wave velocities for all formations. Using both suspension P-S velocities and seismic CPT results, the applicant developed a complete shear wave velocity profile from the surface to a depth of 408 m (1340 ft).

The applicant derived high strain elastic modulus values for the Upper and Lower Sands, compiled in SSAR Table 2.5.4-1, using the relationship with the SPT N-value given in Davie and Lewis (1988). The applicant derived the high strain elastic modulus for the Blue Bluff Marl stratum using the relationship with undrained shear strength given in Davie and Lewis (1988). The applicant calculated shear modulus values using the relationship between elastic modulus, shear modulus, and Poisson's ratio. The applicant derived the low strain shear modulus values for the strata using the average shear wave velocity. The elastic modulus values were obtained by the applicant from the shear modulus values using the relationship described by Bowles (1982) between elastic modulus, shear modulus, and Poisson's ratio.

3. Chemical Properties. The applicant did not include chemical tests as part of the ESP laboratory testing program, because there were no aggressive chemical subsurface conditions identified during the license renewal aging management analysis of the buried concrete at VEGP Units 1 and 2.

In support of the LWA request, the applicant submitted the following information:

Field Investigations

The applicant's field investigations included the construction of a 6 m (20 ft) thick test pad to test the proposed borrow materials, which aided in the evaluation of the compacted backfill.

Engineering Properties

The applicant also determined the engineering properties of the proposed borrow materials and derived the engineering properties of the structural backfill from the data obtained from the COL investigation and Phase 1 of the test pad program.

Chemical Properties. SSAR Subsection 2.5.4.2.5.3 describes the chemical property testing of the proposed backfill material conducted as part of the COL investigation. The applicant performed laboratory testing for pH, chloride, and sulfate on samples from the Upper Sand Stratum in the power block area, test pits excavated in the switchyard borrow area, and soil samples from Borrow Area 4. Based on the average pH test results of 6.8, 5.2, and 5.4 for samples from the Upper Sand, switchyard, and Borrow Area 4, respectively, and corresponding average chloride test results of 188, 76, and 138 parts per million (ppm), the applicant concluded the soil was mildly corrosive. Citing average sulfate test results of 21, 9.8, and 16.3 ppm, the applicant indicated that the soil/concrete interaction would provide a mild exposure for sulfate attack.

2.5.4.1.3 Exploration

SSAR Section 2.5.4.3 summarizes the results of the subsurface investigation programs conducted by the applicant at the VEGP site, including the previous VEGP Units 1 and 2 program, and the Units 3 and 4 ESP and COL subsurface investigation programs.

Previous Subsurface Investigation Program

SSAR Subsection 2.5.4.3.1 summarizes field investigations completed in the early 1970s for VEGP Units 1 and 2. The applicant stated that although borings, geophysical surveys and groundwater studies were included in these field investigations, additional investigations were needed during the excavation of the power block areas to further understand and verify the subsurface conditions. The applicant stated that of the 474 borings completed for Units 1 and 2, twenty fell within, or are in the immediate vicinity of, the proposed VEGP Units 3 and 4 power block site, and the locations of these borings were provided on SSAR Figure 2.5.4-1b. Some of the investigations the applicant considered during the review of the previous programs included: electric logging, natural gamma, density, neutron, caliper, and 3-D velocity logs (Birdwell) in selected boreholes, water pressure and Menard pressuremeter testing of the Blue Bluff Marl, and fossil, mineral or soluble carbonate testing on recovered samples. The applicant supplemented test borings with geophysical methods, completing a total of 8,650 m (28,400 ft) of shallow refraction lines, 1,525 m (5,000 ft) of deep refraction lines, and subsurface cross-hole velocities from the ground surface to a depth of 88 m (290 ft). The applicant referenced the results of these investigations to support the data obtained during the later ESP and COL subsurface investigations.

ESP Subsurface Investigation Program

SSAR Subsection 2.5.4.3.2 describes the ESP subsurface investigation program performed in late 2005 over a substantial portion of the site which would contain the VEGP Units 3 and 4 reactors, switchyard, and cooling towers. The applicant utilized exploration points, as shown on Figure 2.5.4-1 of this SER, to confirm the results of the previous investigation. In addition, the applicant stated that it developed an exploration program, in accordance with Regulatory Guide 1.132, including an audited and approved quality assurance program, and site-specific work procedures. Once the program was established, the applicant performed a variety of field investigations, including 13 exploratory borings, ten CPTs, three seismic CPTs, in-situ hydraulic conductivity tests, five geophysical down-hole suspension P-S velocity logging, a topographic survey of exploration points, and laboratory testing of borehole samples. The applicant also completed a seismic reflection and refraction survey at the VEGP site to collect additional data, which helped delineate the rock profile associated with the non-capable Pen Branch fault.

a) Borings and Samples/Cores. SSAR Subsection 2.5.4.3.2.1 describes the thirteen borings drilled for the ESP investigation with depths from 27 m (90 ft) to 93 m (304 ft). The applicant advanced the borings using mud-rotary drilling techniques, polymer and/or bentonite drilling fluids, and an SPT sampler with automatic hammers to collect samples at continuous intervals to 5 m (15 ft) and at 1.5 to 3 m (5 to 10 ft) intervals thereafter. SSAR Table 2.5.4-7 provides a summary of the ESP boring and CPT locations and depths, and identifies the geophysical testing performed in the boreholes. In addition, the applicant obtained undisturbed samples of the Blue Bluff Marl using rotary pitcher samplers. In accordance with ASTM D 2488, the applicant processed the recovered soil samples by first visually describing the samples and placing them in a labeled moisture-proof glass jar before transporting the samples, in boxes, to an onsite storage facility. Finally, the applicant provided a summary of all undisturbed samples collected from the Blue Bluff Marl during the ESP investigation and described the materials encountered during the ESP borings as similar to those found in the borings from the previous investigation at the VEGP site.

The applicant performed one continuous core boring, B-1003, that was cased through the soil column to prevent cave-ins and allowed for coring of the rock at depths below 320 m (1,049 ft). The applicant placed the recovered soil and rock core samples in wooden boxes lined with plastic sheeting, and the onsite geologist visually described the core. The applicant's geologist computed and recorded the percentage recovery (average core recovery was 77 percent) and the rock quality designation (RQD), before the filled core boxes were transported to the onsite sample storage facility where the core was photographed.

b) Cone Penetrometer Tests. SSAR Subsection 2.5.4.3.2.2 describes the CPTs conducted in accordance with ASTM D 5778 during the ESP site investigations. Using a Type 2 piezocone, the applicant advanced each CPT to refusal at depths ranging from 2 to 35 m (6 to 116 ft); offset CPTs were performed for borings with shallow refusal depths. The applicant noted that, with few exceptions, all of the CPT locations met refusal at or near the top of the Blue Bluff Marl. The applicant performed down-hole seismic testing at 1.5 m (5 ft) intervals in three CPTs to measure shear wave velocity in the Upper Sand Stratum and pore pressure dissipation tests at depths between 17 and 30 m (56 and 99 ft) in four CPTs. SSAR Appendix 2.5A contains the CPT logs, shear wave velocity results, and the pore pressure versus time plots developed from the dissipation tests.

c) In-situ Hydraulic Conductivity Testing. The applicant installed fifteen observation wells in the ESP project limits and developed each by pumping until the pH and conductivity stabilized and the pumped water was reasonably free of suspended sediment. SSAR Subsection 2.5.4.3.2.3 describes the slug tests performed in each well in accordance with ASTM D 4044. The applicant described the slug test method as the lowering of a solid cylinder into a well to increase the water level, recording the time it took the well water to return to the pre-static level, then rapidly removing the cylinder and again recording the time it took the water to recover to the pre-static level. To record the water levels and time intervals during testing, the applicant used electronic transducers and data loggers. SSAR Section 2.4.12 and Appendix 2.4A contain additional details.

d) Sample Re-evaluation. SSAR Subsection 2.5.4.3.2.4 describes the revisions the applicant made to the ESP data report based on additional laboratory data and upon re-evaluation of samples. Upon re-examination of the coarse grained fractions, previously described in the Blue Bluff Marl and Utley Limestone as gravel, the applicant found the samples consisted of angular, gravel-sized, carbonate particles that were attributed to mechanical breakage of cemented nodules, shells, cemented limestone, and fossiliferous limestone by the split barrel sampler. The applicant also redefined the top of the Utley Limestone in some of the ESP boreholes based on the identification criteria developed for the COL subsurface investigation program.

COL Subsurface Investigation Program

SSAR Subsection 2.5.4.3.3 details the COL subsurface investigation conducted over a large portion of the site, including the VEGP Units 3 and 4 power block areas, cooling towers, switchyard/borrow areas, haul road, intake structure, pump house, pipeline, and other construction-related areas, locating the exploration points in accordance with guidelines in RG 1.132. As part of its investigation, the applicant completed 174 exploratory borings across the site, 21 CPTs, eight seismic CPTs, geophysical down-hole suspension logging in six boreholes, electrical resistivity testing along ten arrays across the site, geophysical refraction microtremor (ReMi) testing across four arrays, horizontal and vertical surveys of all exploration points, and laboratory testing, including RCTS tests for selected borehole samples. The applicant stated that it performed the field investigations under an audited and approved quality assurance (QA) program using approved work procedures developed specifically for the COL site investigation. Prior to the start of the field investigations, the applicant established an onsite storage facility for soil samples which included an inventory control system. SSAR Table 2.5.4-7a provides a summary of the locations of COL borings, CPTs and test pits.

1. Borings and Samples/Cores. SSAR Subsection 2.5.4.3.3.1 describes the 174 borings drilled to depths of 6.5 to 128 m (21.5 to 420 ft). Using mud-rotary methods, polymer and/or bentonite drilling fluids, and an SPT sampler with automatic hammers, the applicant sampled the soil at 0.75 m (2.5 ft) intervals within the upper 4.5 m (15 ft) and at 1.5 to 3 m (5 or 10 ft) intervals thereafter. The applicant stated that the soils encountered in the COL borings were similar to those encountered during the ESP and Units 1 and 2 investigations at the VEGP site. The applicant used the same sample processing and storage procedures that were used for the ESP investigation. The applicant also obtained relatively undisturbed samples from the Upper Sand Stratum using the direct push method, and, due to the very hard/dense nature of the materials, used a Pitcher sampler (a double-tube core barrel sampler) for sampling the Blue Bluff Marl and Lower Sand Stratum.

2. Cone Penetrometer Tests. The applicant advanced 21 CPTs to refusal for the COL investigation. SSAR Subsection 2.5.4.3.3.2 states that refusal was generally encountered at or near the top of the Blue Bluff Marl stratum and ranged in depth from 20 to 30.5 m (65.4 to 100.4 ft). The applicant performed seismic testing in eight of the CPTs located in the power block and cooling tower areas of Units 3 and 4.

3. Test Pits. The applicant excavated eight test pits in the proposed borrow areas. SSAR Subsection 2.5.4.3.3.3 describes how a geologist visually examined the excavation walls, prepared a Geotechnical Test Pit log based on the visual examination in accordance with ASTM D 2488, and collected representative bulk samples of the material types in moisture retaining glass jars. The applicant also used a backhoe to backfill the test excavation with the excavated materials.

4. Resistivity. Using the Wenner four electrode test method, the applicant performed field resistivity testing along ten arrays in the proposed switchyard, cooling tower and circulating water line areas of the site. SSAR Figures 2.5.4-1a and -1b illustrate the locations of arrays and SSAR Subsection 2.5.4.3.3.4 states that the locations and array lengths were adjusted to accommodate obstructions. The applicant used electrode spacings from 1 to 91 m (3 to 300 ft) to determine the soil resistivity at increasing depths.

2.5.4.1.4 Geophysical Surveys

SSAR Section 2.5.4.4 includes four subsections summarizing the applicant's previous geophysical investigations for VEGP Units 1 and 2, the geophysical program used for the ESP investigation, the geophysical surveys performed as part of the COL investigation, and geophysical surveys from the Phase I test pad program conducted in support of the LWA request.

In support of the ESP application, the applicant submitted the following information:

Previous Geophysical Survey Programs

SSAR Subsection 2.5.4.4.1 describes the geophysical seismic refraction and cross-hole surveys used to evaluate the subsurface materials during the investigations for VEGP Units 1 and 2. The applicant used the seismic refraction survey to determine the depths to seismic discontinuities based on compressional wave velocity measurements, and obtained shallow and deep refraction profiles throughout the site for a combined total depth of 8,650 and 1,525 m (28,400 and 5,000 ft), respectively. The applicant conducted a cross-hole survey in the power block area to determine the in-situ velocity data for both compressional and shear waves to a depth of 88 m (290 ft), or approximately 25 m (82 ft) below sea level, in six boreholes. The applicant also determined cross-hole velocities by lowering three-component geophones into four of the boreholes to equal levels and generating energy at the same level in a fifth hole.

The applicant also examined compressional and shear wave velocity data from the previous investigations, and used the velocities to determine the Young's Modulus and Shear Modulus for the 88 m (290 ft) closest to the surface. The applicant stated that the seismic (compressional) wave velocities ranged from 426 to 2,026 m/s (1,400 to 6,650 fps) with a shear wave velocity of 182 to 502 m/s (600 to 1,650 fps) for the Upper Sand Stratum (depth from 0 to 27 m (90 ft), while the Blue Bluff Marl stratum, and the underlying Lower Sand Stratum, had a compressional wave velocity of 2,072 m/s (6,800 fps), with shear wave velocities from 487 to 548 m/s (1,600 to 1,800 fps) from 27 to 88 m (90 to 290 ft). The applicant calculated a range of Young's and Shear Moduli for the Upper Sand and the Blue Bluff Marl, including the Lower Sand Stratum.

ESP Geophysical Surveys

SSAR Subsection 2.5.4.4.2 describes the geophysical surveys performed by the applicant as part of the ESP investigations, including suspension P-S velocity tests and down-hole seismic CPTs, as well as a discussion and interpretation of results.

1. Suspension P-S Velocity Tests in Boreholes. The applicant conducted suspension P-S velocity tests in five ESP borings, two of which did not extend below the Upper Sand Stratum. The applicant referred to Ohya (1986) for the details of equipment used to create the seismic compressional and shear waves and to measure the seismic wave velocities. SSAR Subsection 2.5.4.4.2.1 describes the suspension P-S velocity logging system used by the applicant, which incorporated a 7 m (23 ft) probe containing a source near the bottom, and two geophone receivers spaced 1 m (3.3 ft) apart. The applicant lowered the probe into the borehole, where the source generated a pressure wave at depth that was converted to seismic waves (P-wave and S-wave) at the borehole wall. These waves were converted back to pressure waves in the fluid and received by the geophones, which sent the data to a

recorder at the surface. The applicant repeated the procedure every 0.5 to 1.0 m (1.65 to 3.3 ft) and used the results to determine the average velocity of a 1 m (3.3 ft) high column of soil around the borehole.

The applicant defined the shear wave and compressional wave velocities for each stratum to the maximum depth of 407 m (1,338 ft). The average shear wave velocities determined by the applicant were 331 m/s (1,089 fps) for the Upper Sand stratum, 717 m/s (2,354 fps) for the Blue Bluff Marl, and 695 m/s (2,282 fps) for the Lower Sand Stratum, while average compressional wave velocities were 784 m/s (2,572 fps), 2,070 m/s (6,793 fps), and 2,014 m/s (6,610 fps), respectively. The applicant also presented typical values for shear wave velocities for each geologic formation contained within the Lower Sand Stratum; 518 m/s (1,700 fps) in the Still Branch, 594 m/s (1,950 fps) in the Congaree, 624 m/s (2,050 fps) in the Snapp, 716 m/s (2,350 fps) in the Black Mingo, 807 m/s (2,650 fps) in the Steel Creek, 868 m/s (2,850 fps) in the Gaillard/Black Creek, 874 m/s (2,870 fps) in the Pio Nono, and 826 m/s (2,710 fps) in the Cape Fear. The shear wave and compressional wave velocity range was also measured for a portion of the Dunbarton Triassic Basin rock, which the applicant determined was between 707 to 2,849 m/s (2,320 to 9,350 fps) and 2,225 to 5,596 m/s (7,300 to 18,360 fps), respectively. The applicant concluded that shear wave velocities increased linearly with depth at a very high rate, a rate that lessened once shear wave velocities achieved values of about 1,615 m/s (5,300 fps). The applicant noted that sound rock with an average shear wave velocity of 2804 m/s (9,200 fps) was not encountered at the site, but was extrapolated from the measured results. The applicant used both shear and compressional wave velocities to calculate Poisson's ratios for the Upper Sand, Blue Bluff Marl, Lower Sand and Dunbarton Triassic Basin rock strata.

2. <u>Down-Hole Seismic Tests with Cone Penetrometer</u>. SSAR Subsection 2.5.4.4.2.2 describes the three CPTs performed at 1.5 m (5 ft) intervals as part of the ESP investigation. The applicant obtained measurements at depths within the Upper Sand Stratum since all CPTs reached refusal at the top of the Blue Bluff Marl. To complete this test, the applicant located a seismic source on the surface that generated shear waves, and it mounted two geophones horizontally near the bottom of the cone string to record incoming seismic data. The applicant measured shear wave velocities that were lower than those determined by the suspension P-S velocity technique: these lower velocities may reflect site variability.

3. <u>Discussion and Interpretation of Results</u>. The applicant recommended design values for each stratum based on shear and compressional wave velocity measurements. SSAR Subsection 2.5.4.4.2.3 states that seismic CPTs and suspension velocity logging were used to develop the values for the Upper Sand Stratum, but, due to the CPT refusal at the top of the Blue Bluff Marl, only suspension velocity logging results were used to determine the values for the Blue Bluff Marl and Lower Sand Stratum. The applicant did not make any shear or compressional wave velocity measurements for compacted fill during the ESP subsurface investigation, but it recommended values for the compacted fill based on data from VEGP Units 1 and 2, values which would be confirmed during the COL investigations and Phase 1 of the test pad program.

<u>COL Geophysical Surveys</u>

SSAR Subsection 2.5.4.4.3 describes the suspension P-S velocity tests, down-hole seismic CPT tests, and ReMi tests performed during the COL site investigation.

1. Suspension P-S Velocity Tests in Boreholes. The applicant conducted six suspension P-S velocity tests using the equipment described by Ohya (1986) to measure the seismic wave velocities. The method used by the applicant was the same as was used during the ESP investigations summarized in the previous section of this SER. The applicant defined the shear wave velocity to a maximum depth of 128 m (420 ft). Shear wave velocities were determined by the applicant for the Blue Bluff Marl (386 to 909 m/s (1,267 to 2,984 fps)) and the Lower Sand Stratum (227 to 781 m/s (745 to 2,563 fps). The applicant also provided the average velocities for the geologic formations contained within the Lower Sand Stratum; 494 m/s (1,621 fps) for the Still Branch, 567 m/s (1,863 fps) for the Congaree, and 570 m/s (1,871 fps) for the Snapp. As with the ESP investigation, the applicant also determined a range of Poisson's ratios and Figure 2.5.4-3 of this SER illustrates the shear wave velocity profile through borehole B-1003.

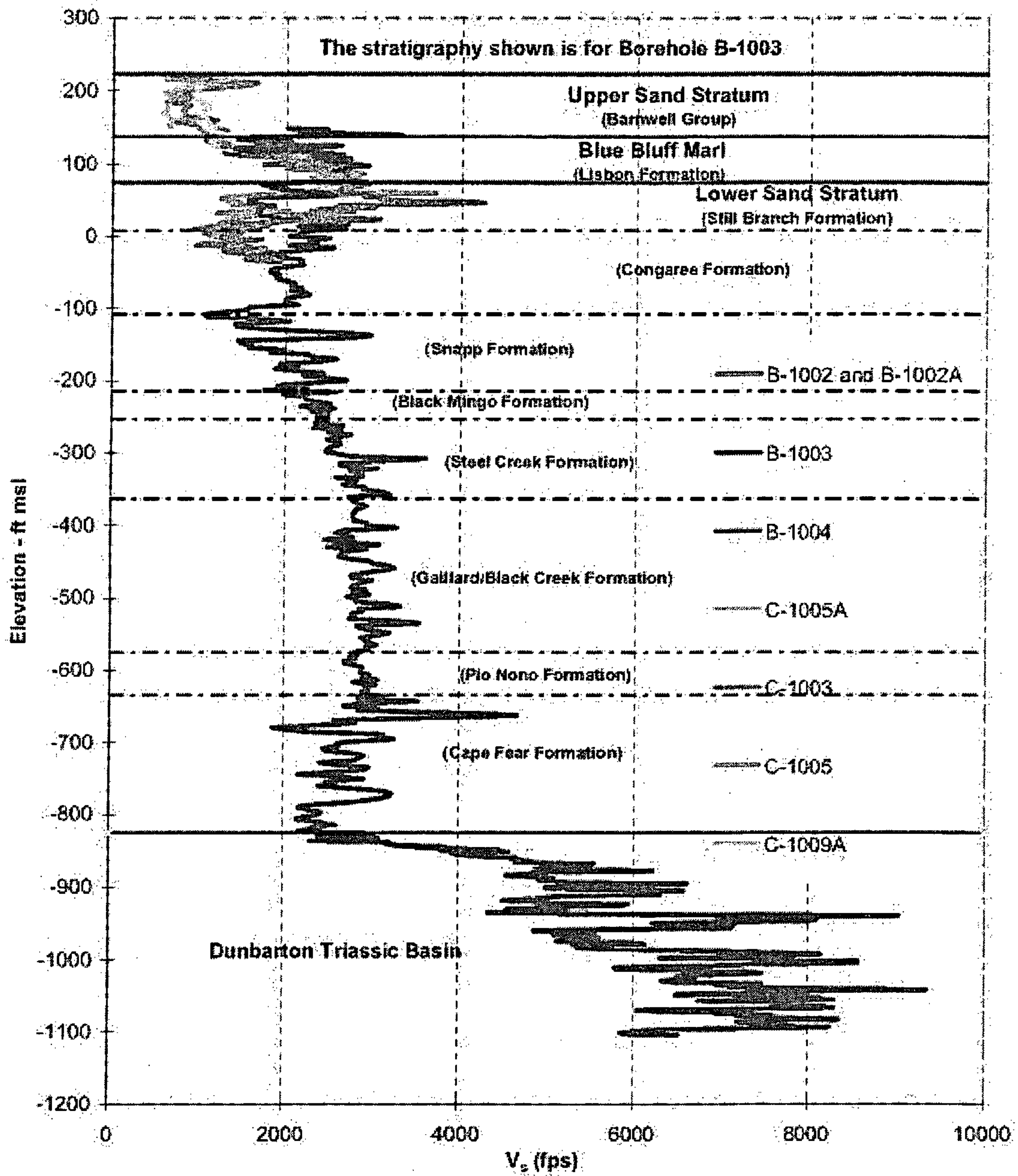


Figure 2.5.4-3 Shear Wave Velocity Measurements (SSAR Figure 2.5.4-6)

2. Down-Hole Seismic Tests with Cone Penetrometer. SSAR Subsection 2.5.4.4.3.2 describes the eight CPTs performed at 0.2 m (0.6 ft) intervals as part of the COL investigation. The method used by the applicant was the same as was used during the ESP investigations which the applicant presented in SSAR Subsection 2.5.4.4.2. Although penetrations depths ranged from 20 to 30.5 m (65.4 to 100.4 ft), CPT soundings could not penetrate the dense/hard materials encountered in the Utley Limestone and Blue Bluff Marl, and therefore the applicant was only able to obtain measurements in the Upper Sand Stratum. The applicant reported shear wave velocity measurements of 132 to 1,158 m/s (435 to 3,802 fps), and it plotted the summary of the average COL shear wave velocity profiles in the Upper Sand Stratum in SSAR Figure 2.5.4-6a.

3. Refraction Microtremor Testing. The applicant conducted ReMi testing across two arrays in the power block areas of the existing VEGP Units 1 and 2 and two arrays in the footprint of the proposed Units 3 and 4. SSAR Subsection 2.5.4.4.3.3 states that although ReMi testing was originally intended to establish the shear wave velocity characteristics of the existing backfill at Units 1 and 2, the applicant noticed frequency interference from the equipment of the operating plant on the ReMi. Although the applicant attempted to overcome the interference and consulted with Dr. K. Stokoe, the applicant concluded that the results did not truly represent the shear wave velocity profile, and therefore these results were not considered in the COL geophysical survey conclusions.

In support of the LWA request, the applicant submitted the following information:

Geophysical Surveys in Compacted Fill

The applicant conducted a test pad program that included the construction of a 6 m (20 ft) deep compacted test fill pad using the proposed backfill materials. SSAR Subsection 2.5.4.4.4 describes the geophysical surveys conducted at three different levels within the test pad to evaluate the shear wave profile in the compacted backfill. The applicant stated that it determined the shear wave velocity using the Spectral Analysis of Surface Waves (SASW) method at various stages of construction and upon completion of the test pad; the cross-hole method was used to measure shear wave velocity through the compacted test fill. Upon completion of the test pad, the applicant installed and measured compressional and shear wave velocities between three cased boreholes extending through the test pad into native materials. The applicant incorporated the results, along with RCTS test results, into the analysis to develop the shear wave profile through the entire depth (about 27 m (90 ft)) of proposed backfill.

2.5.4.1.5 Excavation and Backfill

SSAR Section 2.5.4.5 summarizes the excavation and backfill for VEGP Units 3 and 4, including the extent of safety-related excavations, fills, and slopes; excavation methods and stability; an overview of backfill design; a discussion of backfill sources; quality control and ITAAC; control of groundwater during excavation; and retaining wall construction.

In support of the LWA application, the applicant submitted the following information:

Extent of Excavations, Fills, and Slopes

SSAR Subsection 2.5.4.5.1 describes the substantial excavations necessary for construction of VEGP Units 3 and 4. The applicant presented subsurface profiles providing the grade elevation range across the site, one of which is presented as Figure 2.5.4-2 in this SER. Since the existing ground elevation was at Elevation (El.) 67 m (220 ft) above mean sea level (msl), while the base of the nuclear island foundations for the proposed new units would be at about El. 55 m (180 ft) msl, the applicant determined that the entirety of the Upper Sand Stratum would be excavated for the Units 3 and 4 power blocks. Based on the borings completed during the ESP and COL subsurface investigations, the applicant concluded that the total depth of excavation to the top of the Blue Bluff Marl will range from 24 to 27 m (80 to 90 ft) below the existing grade, with deeper localized excavations using conventional excavating equipment to remove potentially weathered zones in the upper portion of the Blue Bluff Marl.

The applicant stated that once the excavation was complete, Seismic Category 1 backfill would be placed from the top of the Blue Bluff Marl to the bottom of the nuclear island foundation. Although Seismic Category 2 backfill would be used above the nuclear island foundation level, the applicant stated that all of the backfill placed above the foundation would be engineered to the same criteria as Seismic Category 1 backfill. The applicant also described plans to construct a retaining wall along the perimeter of the nuclear island to facilitate construction and backfilling operations with Seismic Category 2 backfill behind it to final grade or foundation elevation of non-nuclear island structures. The applicant described this backfill as granular material selected from portions of the excavated Upper Sand Stratum and other acceptable onsite borrow sources.

Excavation Methods and Stability

SSAR Subsection 2.5.4.5.2 describes the applicant's plans to excavate and stabilize the large volume of Upper Sand Stratum that needs to be removed. The applicant described plans to use conventional equipment to remove any weathered material encountered at the top of the Blue Bluff Marl, and would slope any necessary excavations to facilitate placement of compacted structural fill. The applicant described the overall excavation as an open-cut excavation, with slopes no steeper than 2-horizontal to 1-vertical (2h:1v), and adhering to OSHA regulations (OSHA 2000). The applicant stated that all slopes would be sealed and protected from the highly erosive sandy soils. The applicant determined that where vertical cuts were required due to space constraints, sheet pile or soldier and lagging walls would be adequate support. The applicant determined there were no permanent slopes that need to be considered for stability in the nuclear island area. Finally, the applicant concluded that dewatering operations would be needed once the excavation progressed to depths beneath the groundwater table, approximately El. 45 to 47 m (150 to 155 ft), based on groundwater monitoring results from SSAR Section 2.4.12.

Control of Groundwater During Excavation

SSAR Subsection 2.5.4.5.6 refers to SSAR Subsection 2.5.4.6.2 for a discussion of construction dewatering. However, the applicant stated that because the Upper Sand Stratum soils were highly erosive, the tops of all excavations would be sloped back to prevent runoff, and sumps and ditches constructed for dewatering purposes would be lined, although the applicant did not describe the liner material.

Backfill Design

The applicant established the design of the Seismic Category 1 and Seismic Category 2 backfill for VEGP Units 3 and 4 through analysis and testing of the proposed borrow materials during the COL investigation, Phase I of the test pad program, and the previous site investigations for VEGP Units 1 and 2. SSAR Subsection 2.5.4.5.3 describes the selection and compaction requirements for the backfill. The applicant stated that it selected materials for Seismic Category 1 and Seismic Category 2 backfill that were sands and silty sands that met the gradation requirements specified in SSAR Table 2.5.4-14. According to the applicant, material not within the requirements was evaluated on a case-by-case basis to assess the overall impact of the material on backfill design, although the applicant considered borrow material that did not meet the limits on percentage of particle sizes smaller than the No. 200 (0.075mm) sieve to be unacceptable for use. The applicant stated that all Seismic Category 1 and 2 backfill materials would be compacted to a minimum of 95 percent of the maximum dry density as determined by the ASTM D 1557 standard test method.

The applicant utilized a two-phase test pad program to establish site-specific design properties for the structural backfill materials, verify the materials would satisfy the AP1000 standard plant design siting criteria for a shear wave velocity of at least 304.8 m/s (1,000 fps), and finalize the placement procedures and equipment. For Phase I, the applicant constructed a 6 m by 18 m by 6 m (20 ft by 60 ft by 20 ft) test pad below grade in the switchyard borrow area using methods similar to those used to construct the VEGP units 1 and 2 structural backfill. The applicant stated that it utilized field and laboratory tests, including density, SASW, SPTs, moisture density relationships, grain size distribution, percentage of fine material and plasticity, shear, and shear modulus and damping relationships, to determine the backfill properties. SER Table 2.5.4-1 presents the calculated shear wave velocity profile based on field measurements of velocity in the test pad and in laboratory samples. After interpreting this data, the applicant concluded that the siting criterion for a shear wave velocity of at least 304.8 m/s (1,000 fps) at the nuclear island foundation had been achieved using the proposed backfill materials within the thickness of the test pad.

Table 2.5.4-1 Estimated (ESP) Shear Wave Velocity and Dynamic Shear Modulus Values and Calculated (COL) Shear Wave Velocity Values for Compacted Backfill

Estimated (ESP)			Calculated (COL)	
Depth m (ft)	Vs (fps)	Gmax (ksf)	Depth m (ft)	Vs (fps)
0 to 1.8 (0 to 6)	573	1,255	0 (0)	550
1.8 to 3 (6 to 10)	732	2,049	1.5 (5)	724
3 to 4.2 (10 to 14)	811	2,510	3 (10)	832
4.2 to 5.5 (14 to 18)	871	2,898	6 (20)	975
5.5 to 7 (18 to 23)	927	3,280	9.1 (30)	1,064
7 to 8.8 (23 to 29)	983	3,694	12.2 (40)	1,130
8.8 to 11 (29 to 36)	1,040	4,130	15.2 (50)	1,183
11 to 13.1 (36 to 43)	1,092	4,553	18.2 (60)	1,228
13.1 to 15.2 (43 to 50)	1,137	4,940	21.3 (70)	1,267
15.2 to 17 (50 to 56)	1,175	5,274	24.4 (80)	1,302
17 to 19.2 (56 to 63)	1,209	5,588	25.9 (85)	1,318
19.2 to 21.6 (63 to 71)	1,232	5,796	26.3 (86.5)	1,327
21.6 to 24 (71 to 79)	1,253	6,001	26.8 (88)	1,327
24 to 26.2 (79 to 86)	1,273	6,186	-	-

The applicant stated that Phase II of the test pad program would be used to finalize the placement procedures and equipment, including the material placement procedures and equipment types, construction methods, compaction requirements and methods, and the testing protocol, that would be used during the emplacement of backfill. The applicant described plans to use onsite borrow material excavated from the switchyard and nuclear island areas and its eventual intent to incorporate the backfill placement and compaction methodologies into its earthwork specifications and implementing procedures prior to beginning approved excavation and backfill operations. The applicant completed the Phase II test pad program in July 2008 and incorporated the results into the revised SSAR. The applicant evaluated the results of the various types and combinations of equipment and methodologies used during the program and stated that it determined the optimum placement and compaction strategy for the material types proposed for structural backfill. The applicant stated that it planned to develop its soils specification and structural backfill implementing procedures prior to the start of approved construction activities. However, the applicant did provide the staff with the draft procedures used for the test pad program, which the applicant stated it would use as the basis for its actual specification and procedures. The applicant also stated that the final specifications and corresponding implementing procedures would be developed in accordance with the applicant's approved quality assurance/quality control program prior to its commencement of any actual construction activities approved under the LWA.

Backfill Sources

SSAR Subsection 2.5.4.5.4 describes the backfill material sources that the applicant identified at the Vogtle site through borings and laboratory testing programs and analyses. The applicant identified onsite borrow material sources, including the acceptable portion of Upper Sand Stratum material excavated from the power block and switchyard area north of the power block, and from an alternative location (Borrow Area 4) that was identified and investigated during construction of VEGP Units 1 and 2. The applicant stated that flowable backfill may be used in small restricted areas where adequate compaction may not be achieved; this flowable backfill would be designed to have similar strength characteristics as the proposed compacted backfill materials. The applicant stated that approximately 2,750,000 cubic meters (m^3; 3,600,000 cubic yards(yds^3)) of material were necessary to complete backfilling of the planned 3,000,000 m^3 (3,900,000 yds^3) excavation. Based on the COL investigation and laboratory testing, the applicant estimated that 30-50 percent of the material excavated from the power block area would be suitable backfill material; however, as the suitable and unsuitable materials were generally inter-layered, the applicant conservatively estimated the recovery of about 900,000-1,500,000 m^3 (1,200,000-2,000,000 yds^3) of usable material.

The applicant determined that 1,200,000 m^3 (1,600,000 yds^3) of backfill needed for the power block areas was available from an old borrow stockpile area, developed during the construction of Units 1 and 2 and located to the north of the power blocks in the area of the switchyard for Units 3 and 4. SER Figure 2.5.4-4 (SSAR Figure 2.5.4-15) show the plan and section views, respectively, of this borrow area. The applicant explored the switchyard area with fifteen SPT borings and five test pits during the COL investigation and determined that the needed volume of suitable backfill material was available at the switchyard borrow source. The applicant classified the material as silty sands and poorly graded sands, with lesser amounts of clayey sands in some samples.

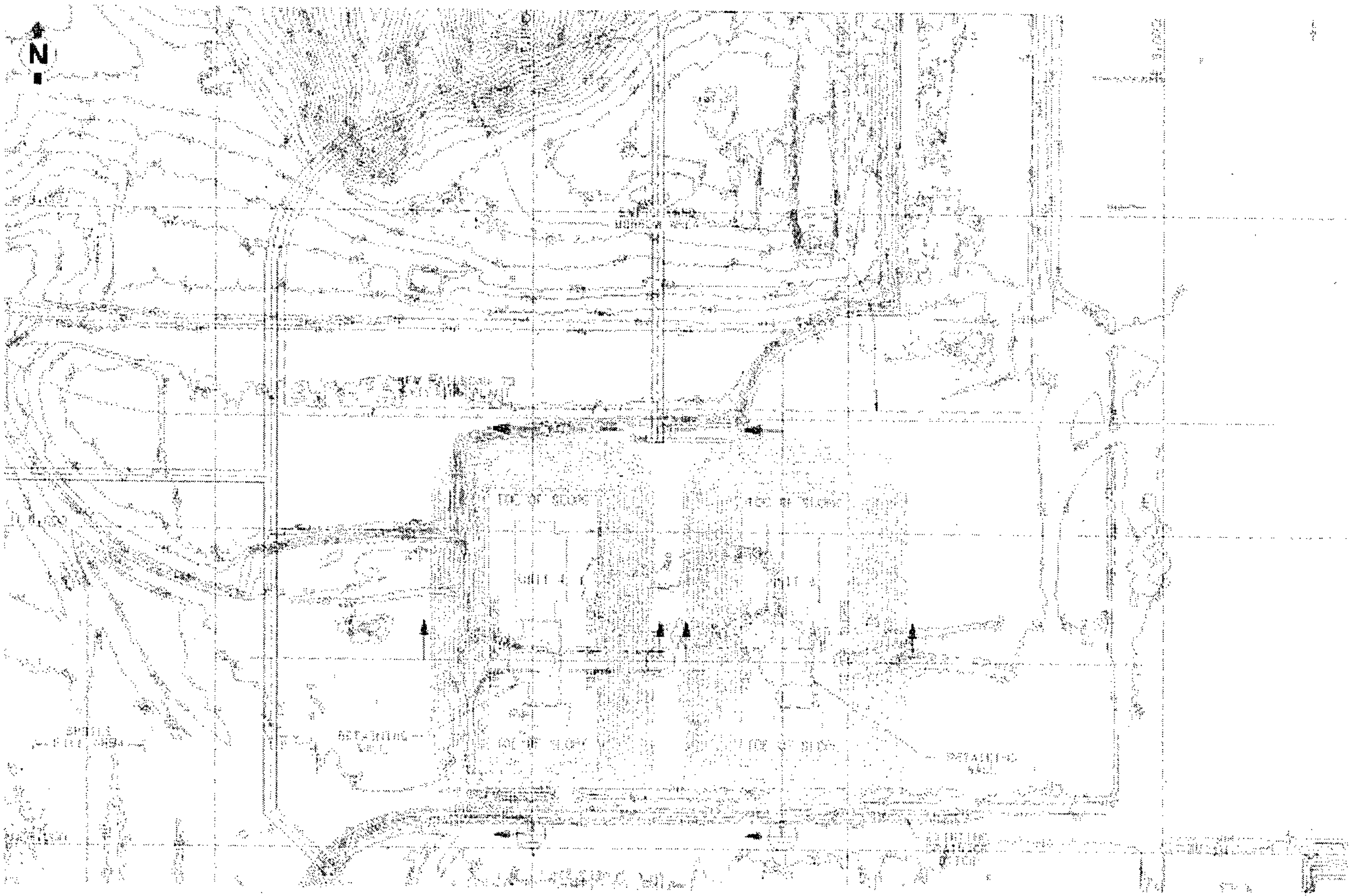


Figure 2.5.4-4 Power Block Excavation and Switchyard Borrow Areas (SSAR Figure 2.5.4-15)

In addition to the switchyard borrow source, the applicant also explored an alternative borrow source, Borrow Area 4, located about 1,220 m (4,000 ft) north of the power block area. Utilizing the results of four SPT borings and three test pits to add to the exploration data for Units 1 and 2, the applicant concluded that approximately 900,000 cubic meters (1,200,000 cubic yards) of suitable backfill material were available from the surface (approximate El. 246 ft) to a depth of 11 m (36 ft; approximate El. 210 ft) at Borrow Area 4.

Quality Control and ITAAC

SSAR Subsection 2.5.4.5.5 describes the quality control and quality assurance program that would be established by the applicant to verify that the backfill was constructed to design requirements as well as the applicable Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC). The applicant detailed plans to use a soil testing contractor with an onsite laboratory and a separate earthwork contractor, each of which would be monitored independent of the other. From the soil testing contractor, the applicant expected that sufficient laboratory modified compaction and grain size distribution tests would be performed to ensure that variations of fill material were addressed.

The applicant stated that an additional quality control program would be applied to all aspects of the backfill testing program, from qualification of borrow material to confirmatory shear wave velocity testing of the as-placed backfill. Qualification of the borrow materials would include soil classification, grain size distribution, and laboratory moisture-density relationship (modified Proctor compaction) tests. These results were used by the applicant to determine the acceptability of borrow materials and the optimum moisture content for field soil compaction. The applicant stated that field density testing would be performed to verify the compaction requirements were met. For earthwork in limited areas, where fill was compacted with hand equipment, there would be one test for every 608 square meters per meter (2,000 square ft per ft) of material placed; for mass earthwork for both Seismic Category 1 and Seismic Category 2, a minimum of one test for every 382 cubic meters (500 cubic yards) of compacted fill, but no less than one test per every lift was performed, and at least two field density tests per lift were located within the footprint directly beneath the nuclear island.

The applicant also planned to review backfill test results, backfill-related non-conformance reports, and QA audits of backfill operations to determine if the as-built backfill met the requirement of 95 percent for minimum compaction for backfill under Seismic Category 1 structures. Only the field density tests performed on backfill directly beneath the nuclear island would be used in the evaluation that would be submitted by the applicant in a report to support ITAAC closure.

Shear wave velocity tests, as measured by the SASW method, would be performed by the applicant on the completed backfill to confirm that the shear wave velocity at the bottom of the nuclear island foundation was greater than or equal to 304.8 m/s (1,000 fps). The applicant also described plans to develop a report to document that the ITAAC requirement for shear wave velocity was met. Preliminary measurements of the shear wave velocity characteristics of the backfill made when placement of backfill reached the approximate elevation of the bottom of the nuclear island foundation, SASW measurements taken within the foundation footprint, representative measurements from locations outside the nuclear island footprint, and SASW measurements made at finish grade would all be used by the applicant to document that the backfill shear wave velocity profile at the elevation of the bottom of the foundation and below was greater than or equal to 304.8 m/s (1,000 fps). Finally, the applicant described plans to use a second method, such as cross-hole testing or seismic CPT, to measure shear wave velocity at one of the finish grade reference locations to validate the SASW results at the same reference. In the event that the velocity measurements do not provide adequate evidence to support closure of the ITAAC, the applicant stated that additional testing and evaluations would be completed before the final report to close the ITAAC is completed. A table of the backfill ITAAC was also provided in the SSAR (now SER Table 2.5.4-2):

Table 2.5.4-2 Backfill ITAAC

Design Requirement	Inspections and Tests	Acceptance Criteria
Backfill material under Seismic Category 1 structures is installed to meet a minimum of 95 percent modified Proctor compaction.	Required testing will be performed during placement of the backfill materials.	A report exists that documents that the backfill material under Seismic Category 1 structures meets the minimum 95 percent modified Proctor compaction.
Backfill shear wave velocity is greater than or equal to 1,000 fps at the depth of the NI foundation and below.	Field shear wave velocity measurements will be performed when backfill placement is at the elevation of the bottom of the Nuclear Island foundation and at finish grade.	A report exists and documents that the as-built backfill shear wave velocity at the NI foundation depth and below is greater than or equal to 1,000 fps.

Retaining Wall

SSAR Subsection 2.5.4.5.7 describes the applicant's plans to construct a mechanically stabilized earth (MSE) retaining wall within each power block excavation to facilitate construction of the nuclear island. The applicant stated that the MSE wall would permit backfilling of the excavations before construction of the nuclear island foundations and substructure walls as well as act as the exterior formwork for the foundation and substructure walls. The applicant also described plans to waterproof the surface of the pre-cast concrete MSE wall facing panels before placing the concrete for the nuclear island foundation and substructure walls.

2.5.4.1.6 Groundwater Conditions

SSAR Section 2.5.4.6 describes the groundwater conditions at the site, including groundwater measurements and elevations, and construction dewatering.

In support of the ESP application, the applicant submitted the following information:

Groundwater Measurements and Elevations

In SSAR Section 2.5.4.6.1, the applicant presented a summary of groundwater conditions at the site of VEGP Units 3 and 4; additional detailed discussions can be found in SSAR Section 2.4.12. The applicant stated that groundwater was present in unconfined conditions in the Upper Sand Stratum and in confined conditions in the Lower Sand Stratum at the VEGP site. The applicant concluded that the Blue Bluff Marl was an aquiclude, a unit which absorbs and holds but does not transmit water, separating the unconfined water table aquifer in the Upper Sand from the confined Tertiary aquifer in the Lower Sand, with groundwater generally occurring at depths between 19 and 21 m (65 and 70 ft) below the existing ground surface.

In mid-2005, prior to the start of the ESP subsurface investigation program, the applicant installed ten observation wells in the unconfined aquifer and five wells in the confined aquifer. The applicant also used the existing wells, thirteen in the unconfined aquifer and nine in the confined aquifer, to monitor groundwater levels at the site. The groundwater levels in the unconfined water table wells ranged from elevation (El.) 40 to 50 m (132 to 165 ft), and the levels in the confined aquifer ranged from El. 25 to 39 m (82 to 128 ft). The applicant performed hydraulic conductivity (slug) tests in the wells, using the same method that was described in SSAR 2.5.4.3.2.3. Based on the slug test results, the applicant concluded that the hydraulic conductivity (k) values for the unconfined water table aquifer in the Upper Sand Stratum ranged from 4.4 x 10^{-5} to 9.3 x 10^{-4} cm/second, while the values for the confined Tertiary aquifer in the Lower Sand Stratum ranged from 1.3 x 10^{-4} to 7.5 x 10^{-4} cm/sec.

Due to groundwater levels that would be higher than the depth of planned excavations at the site, the applicant described its plans to temporarily dewater the excavations that extended below the water table during construction of the new units, and further stated that the dewatering would be performed in a manner that minimized the effects of drawdown on the environment and the operating units. The applicant expected the drawdown effects would be limited to the VEGP site and would have only a negligible effect on the existing Units 1 and 2.

The design groundwater level for VEGP Units 3 and 4 was at El. 50 m (165 ft) msl based on the results of ten years of groundwater monitoring prior to and during the ESP subsurface investigation. The El. 50 m (165 ft) msl level also corresponded to the design groundwater level for the existing VEGP Units 1 and 2, and the applicant based the static stability of the proposed structures on this design groundwater level.

In support of the LWA request, the applicant provided the following information:

Construction Dewatering

Due to the relatively impermeable nature of the Upper Sand Stratum and underlying Blue Bluff Marl, the applicant concluded that sumps and pumps would be sufficient for construction dewatering, and dewatering would be accomplished using gravity-type systems for sump-pumping of ditches that would advance below the progressing excavation grade. SSAR Subsection 2.5.4.6.2 also describes the dewatering methods used during construction of Units 1 and 2, which included a series of ditches oriented in an east-west direction and connected by a north-south ditch that drained to a sump equipped with four high-volume pumps. The applicant stated that the dewatering plans for Units 3 and 4 would use similar methods.

2.5.4.1.7 Response of Soil and Rock to Dynamic Loading

SSAR Section 2.5.4.7 describes the applicant's estimates of the amplification and attenuation of the seismic acceleration at sound bedrock through the soil and rock column. The applicant stated that it compiled data from shear wave velocity profiles of soils and rock, variations of the shear modulus and damping values of soils with strain, and site-specific seismic acceleration-time history, all analyzed using an appropriate computer program.

In support of the ESP application, the applicant provided the following information:

Shear Wave Velocity Profile

SSAR Subsection 2.5.4.7.1 describes the shear wave velocity profiles developed for both soil and rock in the site area.

1. Soil Shear Wave Velocity Profile. During the ESP investigation, the applicant collected a variety of measurements to obtain estimates of shear wave velocity in the soil, estimates that were later confirmed during the COL investigation. The applicant used P-S velocity and CPT down-hole seismic testing to measure the shear wave velocity as part of the ESP subsurface investigations. The applicant developed the shear wave velocity profile used in the seismic amplification/attenuation analysis from the ESP investigation, shown on SSAR Figure 2.5.4-7, and the soil profile used consists of compacted backfill from 0 to 26 m (86 ft), Blue Bluff Marl from 26 to 45.5 m (86 to 149 ft), Lower Sand Stratum from 45.5 to 320 m (149 to 1,049 ft), and Dunbarton Triassic Basin and Paleozoic Crystalline Rock below 320 m (1,049 ft).

   The applicant stated that when compared, the profile of the combined data set (COL) in the middle and upper portions of the Blue Bluff Marl was in good agreement with the ESP profile, although, in the lower portions of the Blue Bluff Marl and the Lower Sand Stratum, the COL profile exhibited slightly lower shear wave velocity values than in the ESP profile. The applicant concluded that the COL shear wave velocity generally increased with depth and supported the findings of the ESP.

2. Rock Shear Wave Velocity Profile. SSAR Subsection 2.5.4.7.1.2 states that due to the thickness of sediments at the VEGP site, the applicant needs to know the shear wave velocity profile and material properties for the site down to the depth where the material shear wave velocity is approximately 2804 m/s (9,200 fps). Since the site is underlain by both the Triassic Basin and Paleozoic crystalline rocks, the applicant considered the effect of shear wave velocities and the material properties of both rocks and their geometries. The applicant concluded that shear wave velocities measured at the top of the Triassic Basin, including the weathered portion, did not reach 2,804 m/s (9,200 fps). The applicant then compared deep borehole shear wave velocity data available from the Savannah River Site (SRS) with data from borehole B-1003 to determine the character of the rock shear wave in the Triassic Basin. The applicant concluded that a weathered zone 61 m (200 ft) thick was present at the top of the Triassic Basin, characterized by the shear wave velocity rapidly increasing with depth to a point where there was a relatively high shear wave velocity, but still less than 2,804 m/s (9,200 fps). The applicant observed a gentler shear wave velocity gradient increasing with depth below the weathered zone. Finally, the applicant noted an arrangement of gentle gradients and shear wave velocities at the top of the unweathered Triassic basin that was interpreted as a continuation of the site-specific profile from borehole B-1003.

   After considering data suggesting that the non-capable Pen Branch fault separated the Triassic Basin from the Paleozoic crystalline rocks, as well as the structural geometry of the rock units and the fault, and the velocity profiles from SRS investigations, the applicant stated the shear wave velocity profile through the Triassic Basin probably would not reach 2,804 m/s (9,200 fps) before encountering the Paleozoic crystalline rock, where the shear wave velocity was interpreted as at least 2,804 m/s (9,200 fps). Accounting for the variability of the depth where the Paleozoic crystalline rock was encountered and the uncertainty of the shear wave velocity gradient, the applicant considered six rock shear wave velocity profiles to comprise the base case used in the seismic amplification and attenuation analysis. The applicant also considered the deep boring rock shear wave velocities from three SRS locations, velocities that suggested additional geometries for the shear wave velocity profiles of the Triassic Basin and the Paleozoic crystalline rock that could impact site response. A closer inspection of the shear wave velocity profile from three SRS locations suggested there was a low velocity zone at the bottom of the Triassic basin where the Pen Branch fault was encountered. The applicant determined through sensitivity analyses that the alternate shear wave velocity models suggested by these observations resulted in insignificant variations in the site response relative to the six profiles previously considered.

Variation of Shear Modulus and Damping with Shear Strain

SSAR Subsection 2.5.4.7.2 describes the variations of the shear modulus and damping with shear strain for both the ESP and COL analyses. Site-specific shear modulus and damping curves are presented as Figures 2.5.4-6 and 2.5.4-7 of this SER.

1. Shear Modulus (ESP Analysis). SSAR Subsection 2.5.4.7.2.1.1 describes the variation of shear modulus with shear strain as determined during the ESP analysis at the VEGP site. The applicant derived the shear modulus from the unit weight data and shear wave velocity of the soil, the determination of which was described in SSAR 2.5.4.7.1. Using the SHAKE2000 (Bechtel 2000) analysis, the applicant tabulated values for shear modulus, as well as the low strain values for the existing soils and rock and for compacted backfill as shown in Tables 2.5.4-1 and 2.5.4-3 of this SER, respectively. The applicant also used the EPRI curves for sands and clays (EPRI TR-102293 1993) to derive the dynamic shear modulus reduction in terms of depth for granular soils (Upper and Lower Sand Stratum) and in terms of the Plasticity Index (PI) for cohesive soils (Blue Bluff Marl) using a PI of 25 percent for the clay of the Lisbon Formation. Table 2.5.4-4 of this SER provides the results of the shear modulus reduction factors. The applicant also used the shear modulus reduction factors developed for the neighboring SRS, selected based on their stratigraphic relationship to the site of VEGP Units 3 and 4, for the ESP analysis. The applicant equally weighted the site amplification factors using the EPRI and SRS shear modulus degradation relationships as described in SSAR Subsection 2.5.2.5.1.2.1.

2. Shear Modulus (COL Analysis). SSAR Subsection 2.5.4.7.2.1.2 describes the development of site-specific dynamic shear modulus reduction curves using RCTS test results from the Blue Bluff Marl, Lower Sand Stratum, and the proposed borrow materials for the compacted backfill. The applicant tested undisturbed samples from both the Blue Bluff Marl and Lower Sand Stratum, plotted the shear modulus reduction data against shearing strain, and overlaid the data on the EPRI curves for clay or for depth for granular soils. The applicant stated that for the Blue Bluff Marl, the site-specific data followed the EPRI trend of the relationship with plasticity index, while the Lower Sand Stratum followed the EPRI trend for depth for granular soils.

3. Damping (ESP Analysis). SSAR Subsection 2.5.4.7.2.2.1 describes the derivation of the damping ratio from EPRI in terms of depth for granular soils, such as the Upper and Lower Sand Strata, and in terms of Plasticity Index for cohesive soils, such as the Blue Bluff Marl, as conducted as part of the ESP site analysis. The applicant used the EPRI curves for sands to derive the damping ratios for the granular soil strata (compacted backfill and Lower Sand Stratum), and the EPRI curves for clays to derive the damping ratios for the Lisbon Formation using a PI of 25 percent. SER Table 2.5.4-4 provides the calculated damping

ratios. The applicant also used certain damping ratio values developed for the SRS, selected based on their stratigraphic relationship to the VEGP site. The applicant stated that it weighted the mean site reduction and site amplification factors using EPRI and SRS shear modulus degradation relationships.

4. Damping (COL Analysis). SSAR Subsection 2.5.4.7.2.2.2 describes the development of the site-specific damping curves from the RCTS test results performed on samples from the Blue Bluff Marl, the Lower Sand Stratum, and the proposed borrow materials for compacted backfill. The applicant stated that it plotted the RCTS damping relationships for the Blue Bluff Marl samples, which were then overlain on the EPRI curves for clay, and it concluded that the site-specific data followed trends that were consistent with the EPRI damping relationships for PI. The applicant also derived site-specific curves for low and high PI materials based on the similarity of the EPRI PI curves. Utilizing similar plots and overlays for the Lower Sand Stratum and clayey samples, the applicant concluded that the site-specific data for both the sand and clay samples followed trends consistent with the EPRI relationships for depth for granular soils and were based on the EPRI curves for depth for granular soils.

Table 2.5.4-3 Design Dynamic Shear Modulus and Typical Shear Wave Velocity from ESP Investigations (Taken from SSAR Tables 2.5.4-2 and 2.5.4-6)

Geologic Formation	Depth m (ft)	Elevation m (ft)	Gmax (ksf)	Vs (fps)
Upper Sand Stratum (Barnwell Group)	0 to 4.8 (0 to 16)	68 to 63 (223 to 207)	7,000	1,400
	4.8 to 12.5 (16 to 41)	63 to 55.4 (207 to 182)	2,286	800
	12.5 to 17.7 (41 to 58)	55.4 to 50.2 (182 to 165)	2,580	850
	17.7 to 26.2 (58 to 86)	50.2 to 41.7 (165 to 137)	2,893	900
Blue Bluff Marl (Lisbon Formation)	26.2 to 28 (86 to 92)	41.7 to 40 (137 to 131)	6,978	1,400
	28 to 29.5 (92 to 97)	40 to 38.4 (131 to 126)	10,321	1,700
	29.5 to 31 (97 to 102)	38.4 to 36.8 (126 to 121)	15,750	2,100
	31 to 32 (102 to 105)	36.8 to 35.9 (121 to 118)	10,321	1,700
	32 to 33.8 (105 to 111)	35.9 to 34.1 (118 to 112)	17,286	2,200
	33.8 to 37.5 (111 to 123)	34.1 to 30.5 (112 to 100)	19,723	2,350
	37.5 to 45.4 (123 to 149)	30.5 to 22.5 (100 to 74)	25,080	2,650
Lower Sand Stratum	45.4 to 47.5 (149 to 156)	22.5 to 20.4 (74 to 67)	14,286	2,000
Still Branch	47.5 to 65.8 (156 to 216)	20.4 to 2.1 (67 to 7)	9,723	1,650
Congaree	65.8 to 101 (216 to 331)	2.1 to -32.9 (7 to -108)	13,580	1,950
Snapp	101 to 134 (331 to 438)	-32.9 to -65.5 (-108 to -215)	15,009	2,050
Black Mingo	134 to 145 (438 to 477)	-65.5 to -77.4 (-215 to -254)	19,723	2,350
Steel Creek	145 to 179 (477 to 587)	-77.4 to -111 (-254 to -364)	25,080	2,650
Gaillard/Black Creek	179 to 243 (587 to 798)	-111 to -175 (-364 to -575)	29,009	2,850
Pio Nino	243 to 262 (798 to 858)	-175 to -193 (-575 to -635)	29,418	2,870
Cape Fear	262 to 320 (858 to 1,049)	-193 to -251 (-635 to -826)	26,229	2,710
Dunbarton Triassic Basin	320 (1,049)	-251 (-826)		2,710
	333 (1,093)	-265 (-870)		5,300
	403 (1,323)	-335 (-1,100)		7,800

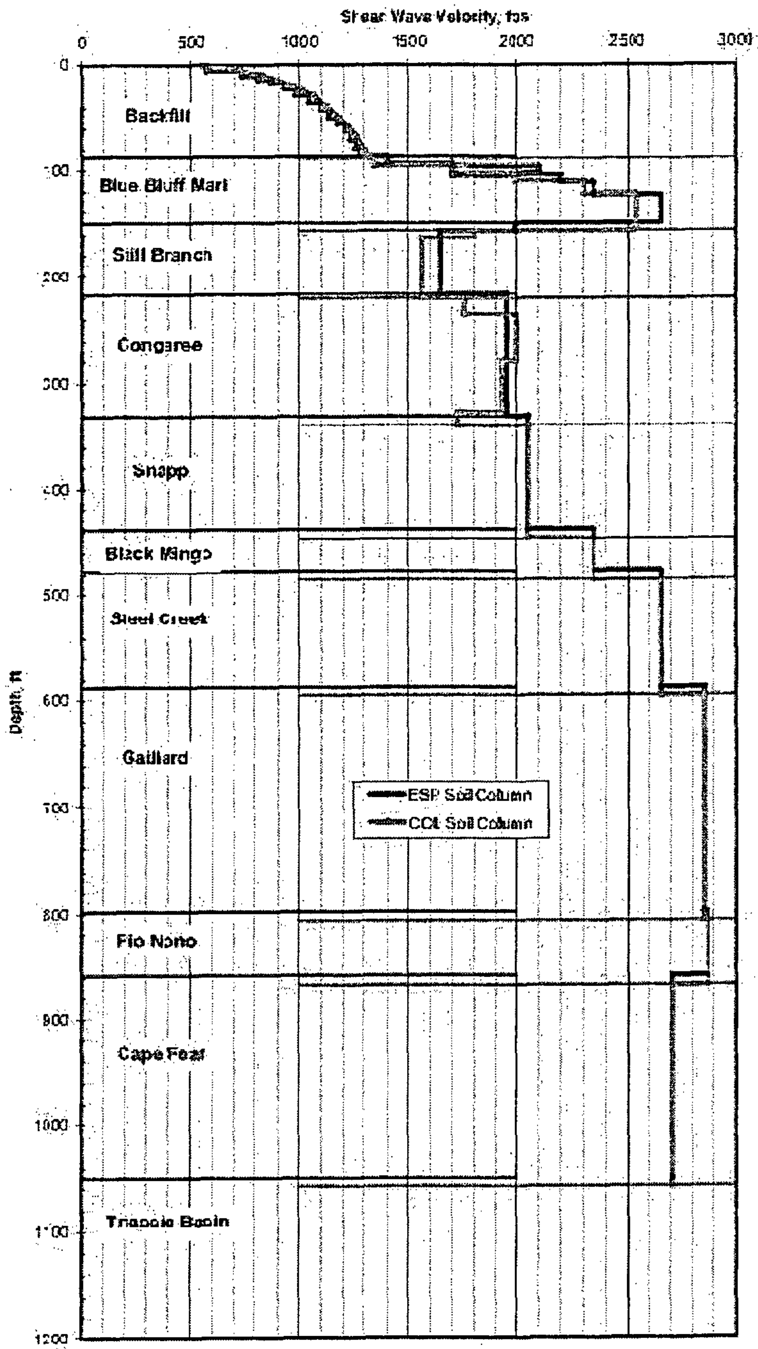


Figure 2.5.4-5 Shear Wave Velocity Profile – ESP and COL Soil Column (SSAR Figure 2.5.4-7a)

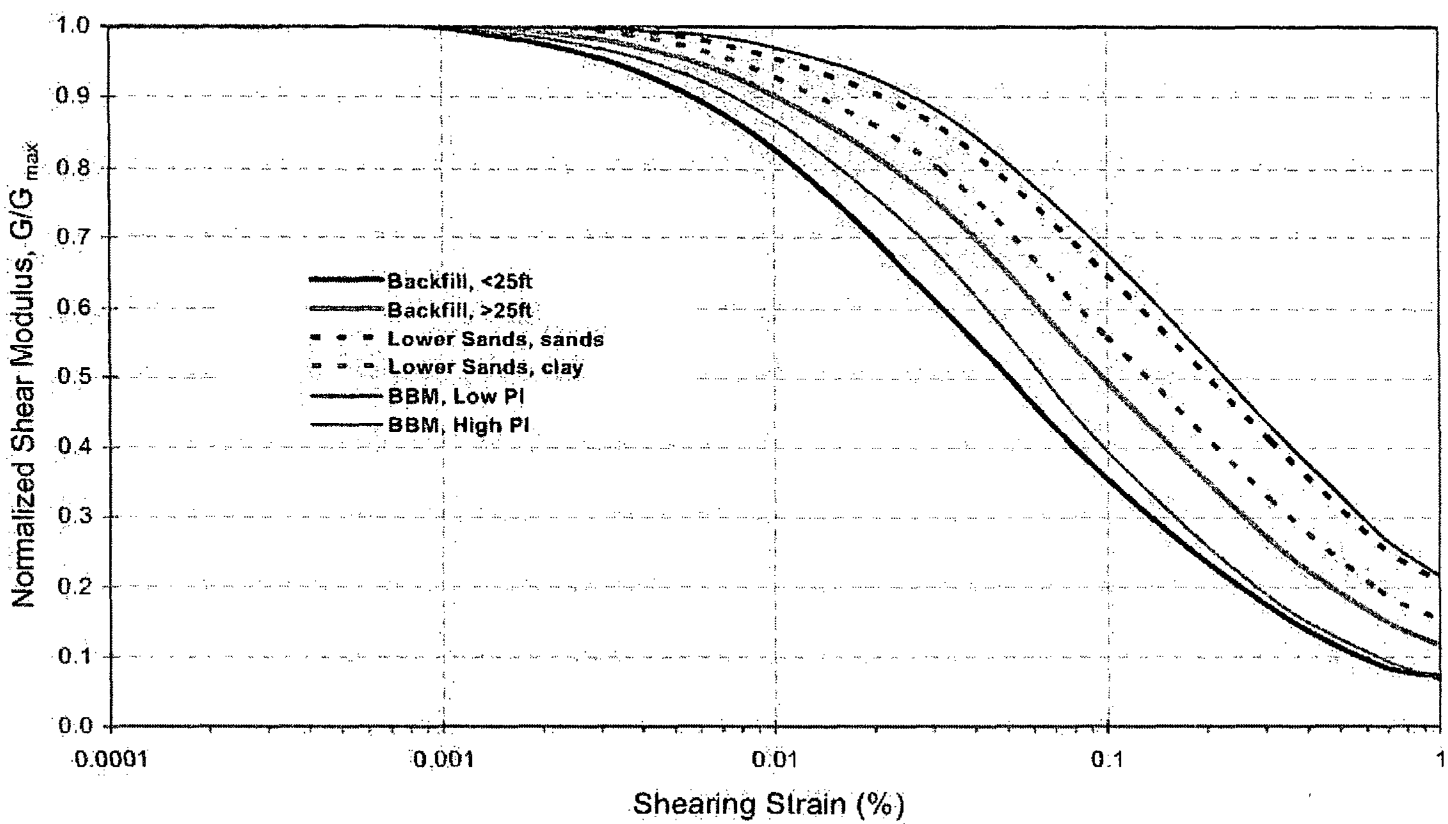


Figure 2.5.4-6 Site-Specific Shear Modulus Reduction Curves (SSAR Figure 2.5.4-9a)

Soil/Rock Amplification/Attenuation Analysis

SSAR Subsection 2.5.4.7.3 describes the use of the SHAKE2000 computer program to determine the site dynamic responses for the soil and rock profiles. The applicant stated that SHAKE2000 used an equivalent linear procedure to account for the non-linearity of the soil by employing an iterative procedure to obtain values for shear modulus and damping that were compatible with the equivalent uniform strain induced in each sublayer. At the beginning of the analysis, the applicant assigned a set of shear modulus and damping value properties to each sub-layer of the soil profile, properties which were used during the analysis to calculate the shear strain induced in each sub-layer. The applicant then modified the shear modulus and damping ratio for each sub-layer based on the shear modulus and the damping ratio versus strain relationships, repeating the analysis until strain-compatible modulus and damping values were achieved.

Comparison of ESP versus COL Soil Column

SSAR Subsection 2.5.4.7.5 compares the subsurface data collected and evaluated during two distinct phases referred to as the ESP and COL investigations, including Phase 1 of the test pad program. The applicant described the ESP investigation as limited in scope but broad in aerial coverage, whereas the COL investigation was extensive in scope but limited to the Units 3 and 4 power block areas. SER Figure 2.5.4-5 presents the stratification and shear wave velocity profiles of the ESP and COL soil columns. The applicant stated that the offset in the soil stratification between the soil columns reflected refinements due to the additional data collected during the COL investigation. The applicant concluded that a comparison of the ESP and COL shear wave velocity profiles indicated good agreement between the data sets and consistency of trends within the strata.

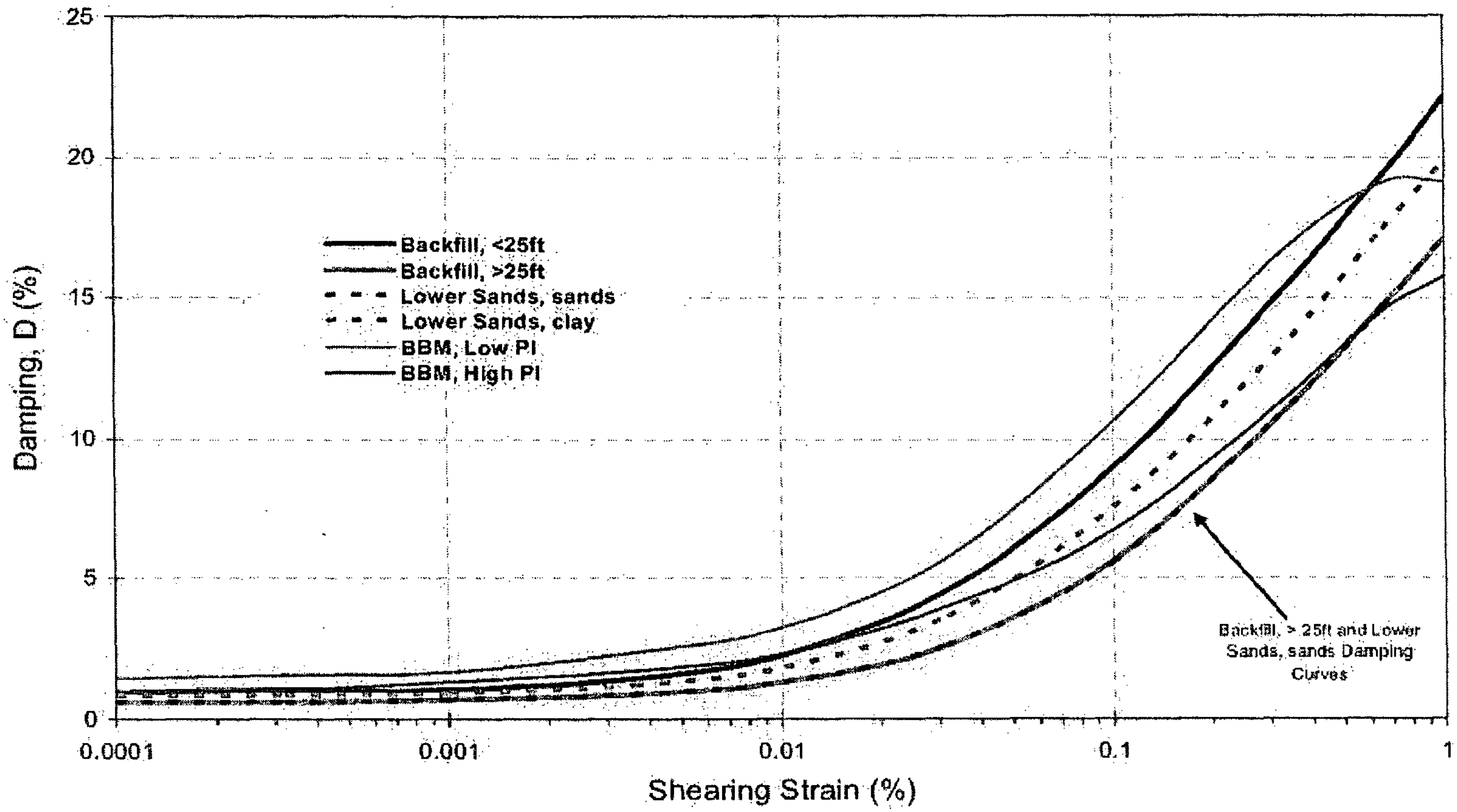


Figure 2.5.4-7 Site-Specific Damping Ratio Curves (SSAR Figure 2.5.4-11a)

In support of the LWA request, the applicant submitted the following information:

Shear Wave Velocity Profile

Soil Shear Wave Velocity. The applicant collected shear wave velocity data from the ESP and COL investigations, and it stated that the ESP data was derived from the backfill shear wave velocity data determined during the previous investigations conducted for VEGP Units 1 and 2, while the COL investigations considered the shear wave velocity data determined for the structural backfill to be used at the VEGP Units 3 and 4 site.

The applicant measured shear wave velocity in the field by the applicant during Phase 1 of the test pad program, as well as through RCTS and other methods from the COL investigations. The applicant used this data, along with laboratory test data, to evaluate the shear wave velocity of the backfill and develop the shear wave velocity profile for the backfill. During the COL investigation, the applicant calculated the shear wave velocity values from 0 to 27 m (88 ft) in the backfill, 27 to 47.5 m (88 to 156 ft) in the Blue Bluff Marl, 47.5 to 322 m (156 to 1,058 ft) in the Lower Sand Stratum, including the Still Branch, Congaree, and Snapp Formations, and in the Dunbarton Triassic Basin and Paleozoic crystalline rock below 322 m (1,058 ft). The applicant stated that it combined and averaged the data from the six COL profiles and two ESP data profiles to produce SSAR Figure 2.5.4-7a (reproduced as SER Figure 2.5.4-5), an average shear wave velocity profile for the data. The applicant stated that the figure illustrates the relationship and similarity between the ESP and COL data sets.

Table 2.5.4-4 Summary of Site-specific Modulus Reduction and Damping Ratio Values

Stratum	Backfill				Blue Bluff Marl				Lower Sands			
Sub strata	< 7.6 m (25 ft)		> 7.6 m (25 ft)		Low PI		High PI		Sands		Clay (Congaree/Snapp)	
Shear Strain (%)	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio
0.00010	1	0.97	1	0.62	1	1.44	1	1	1	0.62	1	0.86
0.00032	1	1.05	1	0.62	1	1.56	1	1.05	1	0.62	1	0.87
0.00100	0.998	1.05	1	0.7	1	1.67	1	1.32	1	0.7	1	0.93
0.00359	0.942	1.44	0.975	0.89	0.96	2.34	0.9965	1.71	0.997	0.89	0.99	1.21
0.01019	0.826	2.26	0.902	1.3	0.867	3.23	0.97	2.3	0.954	1.32	0.928	1.8
0.03170	0.603	4.55	0.748	2.6	0.673	5.75	0.88	3.97	0.828	2.6	0.8	3.62
0.10000	0.355	8.97	0.495	5.64	0.395	10.63	0.679	6.715	0.649	5.59	0.56	7.54
0.30690	0.172	14.94	0.269	10.65	0.187	16.39	0.433	11.115	0.411	10.65	0.327	13
0.65313	0.089	19.38	0.158	14.73	0.1	19.08	0.2785	14.545	0.263	14.68	0.198	17.42
1.00000	0.072	22.12	0.117	17.11	0.068	19.12	0.217	15.77	0.209	17.11	0.154	19.87

Variation of Shear Modulus and Damping with Shear Strain

1. Shear Modulus (COL Analysis). In addition to the information summarized from this section in support of the ESP application, the applicant also included the following information in support of the LWA request. SSAR Subsection 2.5.4.7.2.1.2 describes the variation of shear modulus with shear strain as determined during the COL analysis at the VEGP site. The applicant developed the site-specific dynamic shear modulus reduction curves from the results of RCTS tests on Blue Bluff Marl and Lower Sand strata samples, as well as on samples from the proposed borrow materials. As part of the COL analysis, the applicant also tested five bulk soil samples from test pits in the proposed borrow sources. The tests conducted by the applicant included percent fines (8 to 25 percent), moisture-density and index testing on the samples. The applicant stated that RCTS tests were performed on the bulk samples at two different levels of compaction (at 95 percent and 97 percent, or at 95 percent and 100 percent), using confining pressures based on representative depths throughout the proposed 27 m (90 ft) backfill soil column. The applicant concluded that the results disclosed little variation based on the level of compaction. The applicant then plotted the shear modulus reduction data against shearing strain, overlaid the data on the EPRI curves for depth for granular soils, and concluded that the site-specific data followed trends consistent with the EPRI relationships for depth for granular soils.

2. Damping (COL Analysis). In addition to the information summarized from this section in support of the ESP application, the applicant also included the following information in support of the LWA request. SSAR Subsection 2.5.4.7.2.2.2 describes the development of the site-specific damping curves from the RCTS test results performed on samples from the Blue Bluff Marl, the Lower Sand Stratum, and the proposed borrow materials for compacted backfill. The applicant stated that it developed site-specific damping curves for the borrow material for samples under low confining pressure (less than 7.5 m (25 ft) deep) and for samples under higher confining pressures (more than 7.5 m (25 ft) deep) based on the similarity of the EPRI curves for depth for granular soils.

Two-Dimensional Effects Site Response Analysis (Bathtub Model)

SSAR Subsection 2.5.4.7.4 states that the model for the site dynamic response analysis, as discussed in SSAR Section 2.5.2.5, depicting the backfill above the Blue Bluff Marl as a continuum, did not account for the extent of the excavation and backfill or any impacts of the Upper Sand Stratum on site response. Therefore, the applicant stated that it evaluated these impacts by considering the site response with both the Upper Sand Stratum in place and replaced by backfill. According to the applicant, the average shear wave profile of the stratum was developed and used to characterize shear wave velocity of the Upper Sand. The applicant provided a more detailed discussion of these analyses and results in SSAR Section 2.5.2.9.2.

MSE Backfill Shear Wave Velocity Profile

SSAR Subsection 2.5.4.7.6 provides further discussion on the Mechanically Stabilized Earth (MSE) retaining wall presented in SSAR Subsection 2.5.4.5.7. The wall, as shown on SER Figure 2.5.4-17, consists of wall facing panels and tensile elements embedded in the structural backfill placed behind the wall face. Immediately behind the wall face and away from the wall face for a distance of about 5 ft, the applicant plans to place backfill in thinner lifts and utilize smaller hand-operated compaction equipment to achieve the compaction criteria of at least 95 percent of the maximum dry density as determined by the ASTM D 1557 standard test method. Beyond a distance of about 5 ft away from the wall face, the applicant's mass earthwork operations will place and compact the structural backfill in thicker lifts utilizing larger self-propelled equipment. The applicant stated that due to the likely different compaction procedures and the presence of the MSE wall face, the shear wave velocity profile of the backfill within the 5 ft wall face zone may be reduced. The applicant investigated the effect of this possibility by using a reduced velocity profile for the full height of the wall, identified as the MSE best estimate, in a soil structure interaction analysis and presented the results in SSAR Appendix 2.5.E. The applicant concluded that the results show no differences in the seismic structural responses from the potentially reduced shear wave velocity behind the MSE wall.

2.5.4.1.8 Liquefaction Potential

SSAR Section 2.5.4.8 describes soil liquefaction as the process where loose, saturated, granular deposits lose a significant portion of their shear strength due to the buildup of pore pressure as a result of cyclic loading such as that caused by an earthquake. The applicant stated that multiple factors contributed to liquefaction potential, including geologic age, state of soil saturation, density, grain size distribution, plasticity, and intensity and duration of earthquakes. The applicant stated that, in general, when the following criteria are met, liquefaction can occur: 1) the design ground acceleration is high, 2) the soil is saturated (i.e., the soil is close to or below the water table), and 3) the site soils are sands or silty sands in a loose or medium dense condition.

In support of the ESP application, the applicant submitted the following information:

At the VEGP site, the applicant identified the Upper Sand Stratum, consisting of sands of varying fines content, as meeting all three criteria. According to the applicant, liquefaction was not a concern in either the Blue Bluff Marl or the Lower Sand Stratum, although the applicant addressed the liquefaction potential of the coarse-grained materials within the Blue Bluff Marl. Due to the potential susceptibility of the Upper Sand Stratum to liquefaction, the applicant completely removed the entire potion of the Upper Sand Stratum during construction of VEGP Units 1 and 2, and replaced it with engineered backfill. The applicant stated that it planned for a similar removal and replacement procedure during construction of VEGP Units 3 and 4.

Acceptable Factor of Safety Against Liquefaction

The applicant used Regulatory Guide 1.198 (RG1.198) as a guide for liquefaction analysis. RG 1.198 considers factors of safety (FS) less than or equal to 1.1 against liquefaction to be low, FS between 1.1 and 1.4 to be moderate, and FS equal to or greater than 1.4 to be high.

Previous Liquefaction Analyses

SSAR Subsection 2.5.4.8.2 describes the applicant's evaluation of the liquefaction potential of the Upper Sand Stratum performed during the VEGP Units 1 and 2 investigations. The applicant determined that the Upper Sand Stratum below the groundwater table was susceptible to liquefaction when it was subjected to the maximum SSE acceleration of 0.2g developed for Units 1 and 2. To account for this potential, the applicant removed the Upper Sand Stratum to an approximate El. of 39.5 to 41 m (130 to 135 ft) in the Units 1 and 2 power block area and replaced it with compacted structural backfill. The applicant evaluated, using cyclic strength data from test specimens, the liquefaction potential of the compacted structural backfill in the power block area and determined an FS against liquefaction of 1.9 to 2.0. The applicant concluded that this was an adequate factor of safety against liquefaction for the compacted backfill for VEGP Units 1 and 2.

Liquefaction Analyses Performed for the ESP Application

SSAR Subsection 2.5.4.8.3 describes the liquefaction analyses performed for the strata at the VEGP site as part of the ESP application, including the Upper Sand, Blue Bluff Marl, and compacted backfill.

1. Liquefaction Analyses of the Upper Sands. Based on the previous investigations and excavations for VEGP Units 1 and 2, as well as on the proximity of proposed Units 3 and 4, the applicant stated that it did not perform a liquefaction study as part of the ESP investigation because the unit would be completely removed and replaced with select compacted non-liquefiable structural backfill up to plant grade within the footprint of the power block.

2. Liquefaction Analyses of the Blue Bluff Marl. The applicant identified the Blue Bluff Marl as a cemented, overconsolidated, calcareous, fine-grained silt and clay material that exhibited a high factor of safety against liquefaction; however, the applicant stated that since it found some lenses of silty fine sand during the COL investigation, additional analyses were performed. The applicant stated that it evaluated the data from SPT, CPT, and shear wave velocity measurements, with the SPT measurement method being the most well developed and well recognized. The applicant calculated the cyclic stress ratio (CSR), a measure of the stress imparted to the soils by the ground motion; then the cyclic resistance ratio (CRR), a measure of the resistance of soils to the ground motion; and finally used the ratio of the CRR to the CSR to determine the FS.

    a. Liquefaction Potential Based on SPT Data. The applicant presented SPT N60-values versus elevation for the 70 COL investigation borings in the VEGP Units 3 and 4 power block area and stated that the results were indicative of non-liquefiable coarse-grained soil samples. The applicant stated that of eight soil samples it analyzed, three were potentially liquefiable, with calculated FSs against liquefaction of 1.43, 1.75, and 2.19, and in all cases, greater than 1.1. Therefore, the applicant concluded the FS against liquefaction in the Blue Bluff Marl was adequate based on the SPT data.

    b. Liquefaction Potential Based on Shear Wave Velocity Data. The applicant stated that it measured shear wave velocity (Vs) data in the Blue Bluff Marl by P-S logging in six power block area borings during the COL investigation to evaluate the potential for liquefaction. Following the recommendations in Youd et al, the applicant stated that it corrected the shear wave velocity values for overburden (Vs1), and calculated Vs1 values from 253 to 508 m/s (830 to 1666 fps). Based on the relationship between Vs1, cyclic resistance ratio (CRR), and liquefaction presented by Youd et al., the applicant concluded that the Blue Bluff Marl was non-liquefiable.

Liquefaction Conclusions

Based on its analysis of the potential for liquefaction, the applicant concluded that the only potentially liquefiable soil was the portion of the Upper Sand Stratum below the groundwater table. The applicant stated that for this reason, the Upper Sand Stratum was removed and replaced with compacted structural backfill during construction of Units 1 and 2 and that the same would be done during construction of Units 3 and 4. Through various analyses, the applicant concluded that the liquefaction potential of the compacted structural backfill material, consisting of materials and using methods similar to those for VEGP Units 1 and 2, was not a concern. Finally, the applicant determined that the FS against liquefaction of the Blue Bluff Marl (greater than 1.1) was adequate.

In support of the LWA request, the applicant provided the following information:

> Liquefaction Analyses of the Compacted Backfill. In SSAR Subsection 2.5.4.8.3.3, the applicant stated that the structural backfill would be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 1557, and that the backfill materials, construction, and field compaction methods would be consistent with those used during construction of Units 1 and 2. The applicant evaluated the properties of backfill from the proposed borrow sources during Phase 1 of the test pad program through field and laboratory testing of the materials, and by consistent comparison with results from Units 1 and 2, and concluded that for the design basis earthquake, liquefaction was not a concern for the compacted backfill at Units 3 and 4.

2.5.4.1.9 Earthquake Design Basis

SSAR Sections 2.5.2.6 and 2.5.2.7 discuss in detail the Safe Shutdown Earthquake (SSE). SSAR Section 2.5.2.8 discusses the Operating Basis Earthquake (OBE).

2.5.4.1.10 Static Stability

In support of the ESP application, the applicant submitted the following information:

SSAR Section 2.5.4.10 describes the two scenarios used for the bearing capacity and settlement analyses for VEGP Units 3 and 4. The first scenario, as identified by the applicant, was the Containment and Auxiliary Building foundations, which would be constructed at about El. 55 m (180 ft) msl, a level that corresponded to a depth of 12 m (40 ft) below the final grade of El. 67 m (220 ft) msl, and 15 to 18 m (50 to 60 ft) above the top of Blue Bluff Marl bearing stratum based on the ESP subsurface investigation. The second scenario was the construction of the other foundations in the power block area, which the applicant stated would be placed at depths of about 1.2 m (4 ft) below final grade. Based on the results of the ESP and COL investigations and Phase I of the test pad program, the applicant determined that the soils supporting the nuclear island did not exhibit extreme variations in subgrade stiffness and considered the site to be uniform.

Bearing Capacity

For calculation purposes, the applicant modeled the containment building mat as a circle with a diameter of about 43 m (142 ft) placed at a depth of 12 m (39.5 ft) below finished grade, while other structures would be founded at an approximate depth of 1.2 m (4 ft) below grade. The applicant assumed that all structures in the power block area would be founded on a 27 meter (90 feet) thick layer of structural backfill compacted to a minimum of 95 percent.

Settlement Analysis

The applicant noted that, based on previous site experiences, the total settlement for large mat foundations that support major power plant structures can exceed the limit of 5.08 cm (2 inches) suggested in geotechnical literature. The applicant stated that the settlements of VEGP Units 1 and 2 foundations were from 6.8 to 8.1 cm (2.7 to 3.2 in) for containment buildings, 2.8 to 4.8 cm (1.1 to 1.9 in) for the control building, 7.4 to 8.4 cm (2.9 to 3.3 in) for the auxiliary building, and 6.35 to 9.1 cm (2.5 to 3.6 in) for the cooling towers, all of which were significantly below the maximum design values. The applicant also provided the ratio of measured to predicted settlement for these structures, which ranged from less than 0.50 to about 0.75, which indicated that the subsurface soils were stiffer than anticipated.

The applicant also acknowledged that differential settlement between buildings could affect the pipe connections between those buildings, and therefore it measured differential settlements between the basemats of Units 1 and 2 and reported that they were generally within the limit of 1.9 cm (0.75 in) suggested in geotechnical literature and smaller than the design limit. The applicant noted that the settlements were essentially elastic in that they took place during construction of the units and reflected the elastic nature of the compacted backfill, the heavily overconsolidated Blue Bluff Marl, and the underlying Lower Sand Stratum. The results of laboratory consolidation tests that the applicant conducted on relatively undisturbed samples from the Blue Bluff Marl and Lower Sand Strata confirmed that the elastic behavior and very stiff and dense nature of the strata. Furthermore, the applicant confirmed the very dense nature and the expected performance under load of compacted backfill would be similar to VEGP Units 1 and 2 based on the results from the test pad program. The applicant concluded that settlement could be limited to one inch while differential settlement between footings could be limited to 1.27 cm (½ inch) for footings supporting smaller structures. As an additional strategy, the applicant planned to install piping as late in the construction schedule as practicable and install pipe supports only when construction of the structure to which the pipe connected was near completion.

> Displacement Monitoring. The applicant described plans to develop a detailed instrumentation plan to monitor heave in subsurface soils due to excavation, changes in pore pressures due to excavation and dewatering, and settlement due to construction of the structures. This plan will also include displacement monitoring at depth in order to estimate and confirm moduli of the subsurface soils. The applicant stated that instrumentation would be regularly monitored, including conventional survey, electronic instrumentation, and remote telemetry, where practical. Finally, the applicant stated its intention to place particular emphasis on differential movement and structure tilt. The applicant will develop the plan prior to construction activities.

In support of the LWA request, the applicant provided the following information:

The applicant stated that an earthwork specification for compacted backfill would be developed after Phase 2 of the test pad program was completed. The Phase II test pad program was completed by the applicant in July 2008 and the results used by the applicant to develop draft construction specifications and structural backfill placement procedures. The applicant stated that its final soils specification and backfill implementing procedures are to be finalized in accordance with its quality program, which would be approved as part of the LWA request, prior to the start of any construction activities authorized by the LWA. The applicant also stated that a coefficient of friction of 0.45 against the concrete foundation for the proposed sand and silty sand compacted backfill materials was expected to be achieved, and a site-specific evaluation was conducted and presented by the applicant in Appendix 2.5E of the revised SSAR. The staff's evaluation of the coefficient of friction against sliding is discussed in SER Section 3.8.

Bearing Capacity

Allowable static bearing capacity values were calculated with Terzaghi's bearing capacity equations using an internal angle of friction of 36 degrees for the compacted backfill as developed by the applicant from field and laboratory testing of the borrow materials during the COL investigation and Phase 1 of the test pad program. With an FS of 3.0, the applicant determined that the site conditions provided an allowable bearing pressure of 1,627 kPa (34 ksf) under static loading conditions for the nuclear island; that capacity is greater than the AP1000 DCD requirement of 411.77 kPa (8.6 ksf). The allowable bearing capacity values for foundations placed on compacted fills at depths of about 1.2 m (4 ft) below finished grade are shown on SSAR Figure 2.5.4-13.

The applicant also evaluated the allowable bearing capacity of the structural backfill under the nuclear island for dynamic loading conditions, again using Terzaghi's bearing capacity equation for local shear and Soubra's method with seismic bearing capacity factors using Terzaghi's bearing capacity equation for general shear with an internal friction angle of 36 degrees. To simulate the potential for higher edge pressures during dynamic loading, the applicant considered 3 foundation widths, corresponding to 10, 25 and 50 percent of the width, of the nuclear island basemat. Using a width of 25 ft and a FS of 2.25, the applicant concluded that site specific conditions provided an allowable bearing pressure greater than 2,011 kPa (42 ksf) under dynamic loading conditions for the nuclear island, which was greater than the required 1676 kPa (35 ksf) for dynamic bearing as provided in the DCD as well as the Vogtle site specific maximum dynamic demand for the ESP soil profile of 862 kPa (18 ksf).

The applicant also evaluated the bearing capacity of the structural backfill in terms of the ratio of the ultimate bearing capacity against the structure demand, and this capacity over demand (C/D) ratio provided an alternative measure of the margin of safety against bearing failure. The applicant evaluated the C/D ratios for the static and dynamic demand conditions as provided in the DCD as well as the maximum dynamic demand from the Vogtle site specific seismic evaluation. The applicant stated that the C/D ratios were higher than those typically utilized for standard practice, and that while the results did not take into account settlement of the structures, the significant margin suggested that the settlements would be minimal and within the requirements of the AP1000 DCD.

Settlement Analysis

The applicant performed a detailed settlement analysis for VEGP Units 3 and 4 using elastic properties similar to those used in the analysis for VEGP Units 1 and 2. In the analysis, the applicant incorporated excavation, dewatering, and a timeline of construction to estimate basemat displacement time histories. According to the applicant, the results of the analysis indicated that for the assumed loads, the predicted total settlements ranged from about 5.08 to 7.62 cm (2 to 3 in), with a tilt of approximately 0.63 cm (¼ in) in 15 m (50 ft), a differential settlement between structures of less than 2.54 cm (1 in), and the predicted heave due to foundation excavation ranged from about 2.54 to 6.35 cm (1 to 2 ½ in). The applicant noted that the results were similar to the movements measured for Units 1 and 2.

2.5.4.1.11 Design Criteria

SSAR Section 2.5.4.11 summarizes the design criteria provided in the AP1000 DCD, Revision 15, and covered in various sections of the SSAR. The applicant summarized the geotechnical criteria, except for the criteria that pertain to structural design (e.g., wall rotation, sliding, or overturning), which is discussed in Section 3.8 of this SER. As noted by the applicant in SSAR Section 2.5.4.8, the acceptable factor of safety (FS) against liquefaction of site soils was greater than or equal to 1.1. SSAR Section 2.5.4.10 specifies bearing capacity criteria, including the minimum FS of 3 when applied to bearing capacity equations and against breakout failure due to uplift on buried piping. For soils, an FS of 2.25 can be used when dynamic or transient loading conditions apply. SSAR Section 2.5.5.2 specifies that the minimum acceptable long-term static FS against slope stability failure is 1.5. SSAR Section 2.5.5.2 states that the minimum acceptable long-term seismic FS against slope stability failure is 1.1.

2.5.4.1.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 describes the techniques employed by the applicant to improve the subsurface conditions. For the ESP and COL investigations, the applicant did not consider any ground improvement techniques beyond the removal and replacement of the Upper Sand Stratum, while the test pad program defined the materials and methods for the backfill that would replace the Upper Sand Stratum. The applicant also described plans to improve surficial areas outside the power block excavation through densification with heavy vibratory rollers, and other ground improvement methods, such as the use of piles, as warranted.

2.5.4.2 Regulatory Basis

The applicable regulatory requirements for reviewing the applicant's discussion of stability of subsurface materials and foundations are:

1. 10 CFR 50.55a, "Codes and Standards," requires that structures, systems, and components be designed, fabricated, erected, constructed, tested and inspected to quality standards commensurate with the importance of the safety function to be performed.

2. 10 CFR Part 50, Appendix A, General Design Criterion 1 (GDC 1), "Quality Standards and Records," requires that structures, systems and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. It also requires that appropriate records of the design, fabrication, erection, and testing of structures, systems, and components important to safety be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit.

3. 10 CFR Part 50, Appendix A, General Design Criterion 2 (GDC 2), "Design Bases for Protection Against Natural Phenomena," as it relates to consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

4. 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants," establishes quality assurance requirements for the design, construction, and operation of those structures, systems, and components of nuclear power plants that

prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.

5. 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," as it applies to the design of nuclear power plant structures, systems, and components important to safety to withstand the effects of earthquakes.

6. 10 CFR Part 100, "Reactor Site Criteria," provides the criteria that guide the evaluation of the suitability of proposed sites for nuclear power and testing reactors.

7. 10 CFR 100.23, "Geologic and Seismic Criteria," provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

The related acceptance criteria are described in SRP Section 2.5.4:

1. Geologic Features: In meeting the requirements of 10 CFR Parts 50 and 100, the section defining geologic features is acceptable if the discussions, maps, and profiles of the site stratigraphy, lithology, structural geology, geologic history, and engineering geology are complete and are supported by site investigations sufficiently detailed to obtain an unambiguous representation of the geology.

2. Properties of Subsurface Materials: In meeting the requirements of 10 CFR Parts 50 and 100, the description of properties of underlying materials is considered acceptable if state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soils and rocks in the site area.

3. Foundation Interfaces: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of the relationship of foundations and underlying materials is acceptable if it includes (1) a plot plan or plans showing the locations of all site explorations, such as borings, trenches, seismic lines, piezometers, geologic profiles, and excavations with the locations of the safety-related facilities superimposed thereon; (2) profiles illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials; (3) logs of core borings and test pits; and (4) logs and maps of exploratory trenches in the application for a COL.

4. Geophysical Surveys: In meeting the requirements of 10 CFR 100.23, the presentation of the dynamic characteristics of soil or rock is acceptable if geophysical investigations have been performed at the site and the results obtained wherefrom are presented in detail.

5. Excavation and Backfill: In meeting the requirements of 10 CFR Part 50, the presentation of the data concerning excavation, backfill, and earthwork analyses is acceptable if: (1) the sources and quantities of backfill and borrow are identified and are shown to have been adequately investigated by borings, pits, and laboratory property and strength testing (dynamic and static) and these data are included, interpreted, and summarized; (2) the extent (horizontally and vertically) of all Category I excavations, fills, and slopes are clearly shown on plot plans and profiles; (3) compaction specifications and embankment and foundation designs are justified by field and laboratory tests and analyses to ensure stability and reliable performance; (4) the impact of compaction methods are incorporated into the structural design of the plant facilities; (5) quality control methods are discussed and the

quality assurance program described and referenced; (6) control of groundwater during excavation to preclude degradation of foundation materials and properties is described and referenced.

6. Ground Water Conditions: In meeting the requirements of 10 CFR Parts 50 and 100, the analysis of groundwater conditions is acceptable if the following are included in this subsection or cross-referenced to the appropriate subsections in SRP Section 2.4 of the SAR: (1) discussion of critical cases of groundwater conditions relative to the foundation settlement and stability of the safety-related facilities of the nuclear power plant; (2) plans for dewatering during construction and the impact of the dewatering on temporary and permanent structures; (3) analysis and interpretation of seepage and potential piping conditions during construction; (4) records of field and laboratory permeability tests as well as dewatering induced settlements; (5) history of groundwater fluctuations as determined by periodic monitoring of 16 local wells and piezometers.

7. Response of Soil and Rock to Dynamic Loading: In meeting the requirements of 10 CFR Parts 50 and 100, descriptions of the response of soil and rock to dynamic loading are acceptable if: (1) an investigation has been conducted and discussed to determine the effects of prior earthquakes on the soils and rocks in the vicinity of the site; (2) field seismic surveys (surface refraction and reflection and in-hole and cross-hole seismic explorations) have been accomplished and the data presented and interpreted to develop bounding P and S wave velocity profiles; (3) dynamic tests have been performed in the laboratory on undisturbed samples of the foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysterietic damping properties of the soils and the results included.

8. Liquefaction Potential: In meeting the requirements of 10 CFR Parts 50 and 100, if the foundation materials at the site adjacent to and under Category I structures and facilities are saturated soils and the water table is above bedrock, then an analysis of the liquefaction potential at the site is required.

10. Static Stability: In meeting the requirements of 10 CFR Parts 50 and 100, the discussions of static analyses are acceptable if the stability of all safety-related facilities has been analyzed from a static stability standpoint including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, and lateral loading conditions.

11. Design Criteria: In meeting the requirements of 10 CFR Part 50, the discussion of criteria and design methods is acceptable if the criteria used for the design, the design methods employed, and the factors of safety obtained in the design analyses are described and a list of references presented.

12. Techniques to Improve Subsurface Conditions: In meeting the requirements of 10 CFR Part 50, the discussion of techniques to improve subsurface conditions is acceptable if plans, summaries of specifications, and methods of quality control are described for all techniques to be used to improve foundation conditions (such as grouting, vibroflotation, dental work, rock bolting, or anchors).

In addition, the geologic characteristics should be consistent with appropriate sections from: Regulatory Guide 1.27, "Ultimate Heat Sink for Nuclear Power Plants," Regulatory Guide 1.28, "Quality Assurance Program Requirements (Design and Construction)," Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants," Regulatory Guide 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites," and Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)."

2.5.4.3 Technical Evaluation

This section discusses the staff's evaluation of the geotechnical investigations conducted by the applicant to evaluate the stability and determine the static and dynamic engineering properties of the subsurface materials and foundations at the site of VEGP Units 3 and 4, in particular with respect to the specific LWA activities requested. The applicant presented technical information in SSAR Section 2.5.4 resulting from field and laboratory investigations, data gathered during the ESP phase site investigations, and additional field and laboratory data from a COL level investigation in support of the LWA request. The applicant used the subsurface material properties from its field and laboratory testing to evaluate the site geotechnical conditions and to derive the design values for the ESP, LWA request, and COL application. The staff also identified, summarized and considered the applicant's responses to Requests for Additional Information (RAIs) and Open Items from the SER with Open Items.

2.5.4.3.1 Description of Site Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Section 2.5.1.1 for a description of the regional and site geology. Section 2.5.1.3 of this SER presents the staff's evaluation of the regional and site geology.

2.5.4.3.2 Properties of Subsurface Materials

The staff focused its review of SSAR Section 2.5.4.2 on the applicant's description of the subsurface materials, field investigations and laboratory testing, and the static and dynamic engineering properties of the subsurface materials at the VEGP site. The applicant stated that the soils encountered during the ESP investigation, and during subsequent investigations supporting the LWA request and COL application, constitute alluvial and Coastal Plain deposits and were divided into three groups for stability of subsurface materials and foundation purposes; Group 1, the Upper Sand Stratum or Barnwell Group, which would be removed and replaced with structural backfill; Group 2, the Blue Bluff Marl Bearing Stratum or Lisbon Formation, which is the load bearing layer at the site; and Group 3, the Lower Sand Stratum, consisting of several formations. The Dunbarton Triassic (206 to 24 million years ago [mya]) basin rock, and the Paleozoic (543 to 248 mya) crystalline rock underlie the soil layers at the site. The applicant determined the static and dynamic properties of the three principal soil groups and compacted structural backfill through field investigations and laboratory testing performed in accordance with RG 1.138. The applicant performed grain size distribution (gradation), Atterberg Limits, natural moisture content, unit weight, and triaxial shear laboratory tests. The applicant concluded that the engineering properties obtained from the subsurface investigations and laboratory testing program were similar to those obtained from the previous VEGP Units 1 and 2 investigations. SSAR Table 2.5.4-1 summarizes the geotechnical features of the strata and their corresponding engineering properties as determined during the aforementioned investigations.

The staff's evaluation of the information provided in support of the ESP application is as follows:

In RAI 2.5.4-1, the staff asked the applicant to clarify the discrepancy in different SSAR sections on the number of borings drilled during the ESP field investigation. The applicant explained in its response that in one section it referred to the total number of borings as 14, which included the two borings without any sampling. In other SSAR sections, the applicant did not include these 2 additional borings. With this clarification, the staff considers RAI 2.5.4-1 resolved.

Geotechnical Parameters of the Lower Sand Stratum and the Blue Bluff Marl

In RAI 2.5.4-3, the staff asked the applicant to provide justification for developing geotechnical parameters for the Blue Bluff Marl and Lower Sand Stratum (the main load-bearing layers) using only the data from four borings with no significant sampling in the Lower Sand Stratum. In its response, the applicant stated that three ESP borings completely penetrated the Blue Bluff Marl and another nine borings extended partially into the marl. Among the three, borings B-1002 and B-1004 penetrated through the marl into the Still Branch and Congaree Formations and boring B-1003 went as deep as 407.8 meters (1,338 ft) into the bedrock. The applicant obtained a total of 58 SPT N-values and corresponding samples, as well as 12 tube samples from the Blue Bluff Marl and the Lower Sand Stratum, and performed P-S velocity logging in the three borings that penetrated the marl. In addition to its ESP investigation, the applicant stated that it considered the soil engineering properties from the previous investigations of Units 1 and 2.

From its review of SSAR Section 2.5.4 and the applicant's response to this and other RAIs, the staff found in the SER with Open Items that the applicant relied more on the previous investigations for the existing Units 1 and 2 than on its ESP field investigations to obtain geotechnical parameters for the ESP site. The staff determined that, while the applicant could use data from the previous investigations as a reference to support the current site characterization, the applicant should not have relied on the previous data to demonstrate the suitability of the ESP site because those data were generated by following different regulatory requirements, regulatory guidelines, and industry standards, and by using different investigation technologies. In addition, soil property variation between the two sites made reliance on the previous data inappropriate. Therefore, the staff concluded that the applicant did not conduct sufficient field and laboratory tests to reliably determine the subsurface soil static and dynamic properties for the soils beneath the Blue Bluff Marl at the ESP site. This was identified in the SER with Open Items as Open Item 2.5-11.

In response to Open Item 2.5-11, the applicant stated that the ESP investigations were intended for limited study of the site conducted in accordance with RS-002, "Processing Applications for Early Site Permits," and following the example of other recently accepted ESP studies. However, the applicant indicated that additional investigations were ongoing at the site as part of the COL investigation, including 68 power block borings, 42 of which penetrated the Blue Bluff Marl, as well as geophysical and laboratory testing, all of which were included in later revisions of the SSAR. The staff reviewed the guidelines of RS-002, as well as the additional borings and analyses conducted as part of the COL investigation and described in Revision 4 of the SSAR. Based on the inclusion of additional borings, which followed the guidance presented in RG 1.1.32 and RG 1.138, and which penetrated the load-bearing Blue Bluff Marl, the staff concludes that the applicant conducted sufficient field and laboratory tests at the site of VEGP Units 3 and 4 to adequately determine the static and dynamic property values included in Revision 4 of the SSAR. Based on this conclusion, the staff considers Open Item 2.5-11 closed. Furthermore, the closure of Open Item 2.5-11 also resolves the portions of RAI 2.5.4-3 that relate to the properties of subsurface materials at the site of VEGP Units 3 and 4.

In RAI 2.5.4-3, the staff also asked the applicant to explain the low SPT blow count values (as low as 9 bpf) in the Lower Sand Stratum below the Blue Bluff Marl, because low SPT blow count values often indicate the presence of soft soil layers. For comparison, the average blow count for the same layer is about 60 bpf. The applicant explained that this low SPT N-value (9 bpf) in the Lower Sand Stratum could be due to the existence of disturbed materials at the bottom of the drill hole because other geophysical measurements at the same depth showed no physical or strength abnormalities. After reviewing the applicant's response, the staff agreed that the disturbed materials at the bottom of the drill hole may have caused this anomalously low SPT value in the Lower Sand Stratum. However, because the Lower Sand Stratum is one of the load-bearing layers and the applicant was also committed to performing more borings during the COL stage, the staff considered that obtaining additional data on the Blue Bluff Marl and Lower Sand Stratum during the COL stage to confirm the absence of soft materials in these load-bearing layers would be acceptable. Accordingly, in the SER with Open Items, the staff identified this as COL Action Item 2.5-1.

However, in the revised SSAR, the applicant incorporated significant information obtained during the COL site investigations. The applicant included the results of additional subsurface borings, test pits, and SPTs. The staff reviewed this information and determines that none of the additional data provided as part of the applicant's COL investigation results suggests the presence of a soft material within the load-bearing layers at the VEGP Units 3 and 4 site. The inclusion of this information in the revised SSAR addresses the needs of COL Action Item 2.5-1. Therefore, the staff concludes that COL Action Item 2.5-1 is no longer necessary.

The staff considered the existence of the very low SPT N-values measured from the ESP field tests, and in RAI 2.5.4-3(c), asked the applicant to explain whether there were any indications of soft zones in the Upper Sand Stratum, such as those encountered at the SRS. In its response to RAI 2.5.4.-3(c), the applicant stated that it encountered "soft zones" with SPT N–values of 5 bpf in the Upper Sands at ESP boreholes B-1001, B-1004, B-1005, and B-1006. The applicant also stated that if these kinds of soil are saturated with water they would liquefy during certain seismic events, which may result in surface settlement of several inches. The applicant then referred to its RAI 2.5.4-2(a) response, which provided further details about the extent of the soil replacement in the power block area that would occur during the COL stage.

After reviewing the applicant's response to RAI 2.5.4-3, the staff concluded in the SER with Open Items that, because the extent of the excavation and backfill will be limited in both the vertical and horizontal directions at the ESP site, it was not clear from the response that the purpose of the placement of backfill material is to eliminate the existence of such soft zones located outside the foundation area. Although these soft zones are outside of the immediate foundation area, these soft zones can still have potential adverse impacts on the foundation and the structures of the nuclear power plant. In its response, the applicant committed to take six more deep borings (250 ft to 400 ft deep) during the COL subsurface investigation. Although this information was not necessary at the ESP stage to determine whether 10 CFR Part 100 is satisfied, the issue of confirming the locations of the soft zones and evaluating the potential impact of the soft zones on the foundation and structures was identified as COL Action Item 2.5-2 in the SER with Open Items.

However, in the revised SSAR, the applicant included the additional boring logs and data obtained as part of its COL site investigations, which the staff reviewed. The summary of this additional information can be found in Section 2.5.4.2.2 and 2.5.4.2.3, where the applicant stated that an additional 174 borings were completed as part of the COL investigations. The applicant used these additional borings to confirm the locations of soft zones within the Upper

Sand Stratum at the Unit 3 site, to evaluate the potential impact these zones would have on the stability of the plant foundations and safety-related structures, and to verify the ESP characterization of the Upper and Lower Sands, as well as to further validate the ESP characterization of the Blue Bluff Marl. Using this information, the applicant confirmed that the Upper Sand Stratum is too variable and potentially unstable a stratum and further supported the applicant's decision to completely remove the material. Since this information further confirmed the locations of soft zones within the site area, and addressed the minimum number of borings as requested by COL Action Item 2.5-2, the staff finds that COL Action Item 2.5-2 is no longer necessary.

Effective Angle of Internal Friction

In RAI 2.5.4-9, the staff asked the applicant to clarify how the effective angle of internal friction was determined for the soils underlying the ESP site. The applicant responded that it estimated the effective angle of internal friction of 34 degrees using an empirical correlation associated with SPT N-values (Bowles 1982). From its review of the applicant's response, the staff considered that the internal friction angle calculated based on SPT N-values varies significantly, depending on the correlations used. For example, for N-values between 10 and 40, the corresponding soil internal friction angle values vary from 30 degrees to 36 degrees (Peck 1974) or from 35 degrees to 40 degrees (Bowles 1982). More importantly, the N-values measured for the ESP site are all below 20 (from 3 to 19), according to SER Table 2.5.4-3. Therefore, the use of a friction angle of 34 degrees based on an N-value of 25 for the Upper Sand Stratum appeared to be inappropriate. In the SER with Open Items, the staff concluded that the applicant did not provide reliable effective angles of internal friction for the subsurface soils because it did not have sufficient SPT N-values from the ESP investigation to support its calculation. The internal friction angle for the subsurface soils is one of the input parameters in calculating bearing capacity and determination of earth pressure coefficients. Therefore, in the SER with Open Items, the issue regarding the effective angles of internal friction for the subsurface soils was designated as Open Item 2.5-14.

The applicant responded to Open Item 2.5-14 by stating that the effective angle of internal friction of the subsurface soils was estimated based on empirical correlations associated with SPT N-values. Furthermore, the applicant summarized the measured SPT N-values, noting that a large number of values were recorded in the Upper Sand Stratum, which would be removed during construction. Some N-values measured below the Upper Sand did not achieve a full 12 inches of penetration, which the applicant attributed to either the high relative density of the material encountered or the intact nature of the in-situ material. The applicant updated the SSAR to incorporate the additional COL investigation data, such as N-values and shear strength testing, which was used to verify the effective angle of internal friction.

The staff reviewed the response to Open Item 2.5-14, focusing its review on the additional data provided in the revised SSAR. In the revised SSAR, the applicant provided the effective angle of internal friction for both the Upper and Lower Sand Strata (34 and 41 degrees, respectively). The applicant used an empirical correlation associated with the average SPT N-values (Bowles 1982) from the ESP investigation, based on N60 equals 25 bpf, which the staff agrees is an acceptable method by which to determine the effective angle of internal friction. Based on the inclusion of the effective angles of internal friction in the revised SSAR, which were determined using an acceptable method of correlation to the empirical averages of Bowles, the staff considers Open Item 2.5-14 closed. The closure of Open Item 2.5.4-14 also resolves RAI 2.5.4-9.

High Strain Elastic Modulus

In RAI 2.5.4-11, the staff asked the applicant to explain: (1) why it used the Davie and Lewis' (1988) relationship to estimate the high strain elastic modulus (E) for the Upper and Lower Sand Strata underlying the ESP site; (2) what the consensus is about using the Davie and Lewis relationship between SPT and E; and (3) the extent of the application of the Davie and Lewis relationship. In response to RAI 2.5.4-11, the applicant stated that Bechtel used the Davie and Lewis relationship extensively to estimate settlement when compared to observed settlements for a wide range of foundation sizes on granular materials from clean sands to silty sands to gravels, such as the medium-dense, silty sand of the Upper Sand Stratum and the very dense silty sand of the Lower Sand Stratum. Therefore, the applicant believed that the Davie and Lewis relationship is applicable to the Lower Sands. In addition, the applicant found that the Davie and Lewis relationship provided an E value that was closer to the median value of five different relationships for both sand strata than were the four other E and N (the SPT N-value) relationships detailed in SER Table 2.5.4-5, which is taken from the applicant's response to RAI 2.5.4-11. The applicant also implied that Davie and Lewis' relationship provided reasonable predictions of settlement when compared to measured settlements, and with a reasonable consensus.

Table 2.5.4-5 - Summary of Calculation of Elastic Modulus E

Reference	Relationship	E, ksf	
		N = 25 bpf	N = 62 bpf
Bowles (1987)	E = 10 (N + 15) ksf	400	770
D'Appolonia et al. (1970)	E = 432 + 21.2N ksf	962	1,746
Parry (1971)	E = 100N ksf	2,500	6,200
Schmertman (1970) and Schmertman et al. (1978)	E = 30N to 50N ksf	750 to 1,250	1,860 to 3,100
Yoshida and Yoshinaka (1972)	E = 42N ksf	1,050	2,604
Median		**1,006**	**2,232**
Davie and Lewis (1988)	E = 36N ksf	900	2,232
Note: The references shown above are cited in Davie and Lewis (1988) and are listed at the end of the response to this RAI.			

Based on its review of the applicant's response to RAI 2.5.4-11, the staff concurs with the applicant's conclusion about the applicability of the Davie and Lewis' relationship in estimating elastic modulus. However, the applicant needed to use appropriate SPT N-values to obtain a reasonable E value. Since the N-values obtained from the ESP investigation and the design undrained shear strength values determined by the applicant for the ESP soils are not reliable for very limited data, the staff determined in the SER with Open Items that the applicant did not have sufficient site-specific data to justify the determination of the design parameter E for the Upper and Lower Sand Strata. Therefore, in the SER with Open Items, the issue of using appropriate SPT N-values to determine a reasonable elastic modulus value for the Upper and Lower Sand Strata was designated as Open Item 2.5-16.

In response to Open Item 2.5-16, the applicant referenced the guidance of RS-002 regarding the determination of the engineering properties of the soil and rock strata underlying the site. The applicant stated that the elastic modulus was derived from representative data collected during the ESP site investigation and the measured SPT N-values from the Lower Sand Stratum. Finally, the applicant conducted additional SPTs and provided the data in the revised SSAR.

The staff focused its review of the response to Open Item 2.5-16 on the additional information provided by the applicant in both the response and the revised SSAR, and on the guidance of RS-002. The applicant provided the derived elastic modulus for each of the subsurface strata at the VEGP Units 3 and 4 site (SSAR Table 2.5.4-1). Based on the inclusion in the revised SSAR of additional SPTs, which indicated the hard to very hard and the dense to very dense natures of the Blue Bluff Marl bearing stratum and the Lower Sand Stratum, respectively, from which the elastic modulus was derived, the staff concludes that the applicant has provided sufficient information to close Open Item 2.5-16. The closure of Open Item 2.5-16 also resolves RAI 2.5.4-11.

Determination of Unit Weight Values

In RAI 2.5.4-12, the staff asked the applicant to explain how unit weight values were determined for different soils and why there was a discrepancy between the average values given in the SSAR text and those listed in SSAR Table 2.5.4-1. The applicant explained in its response that the unit weight values were determined based on the laboratory test during the ESP subsurface investigation. However, the applicant used the average values of unit weight based on VEGP Unit 1 and 2 laboratory test results because there were more test data available, despite results that differed from those obtained from ESP tests. The staff considered that the unit weight values for underlying soils are very basic soil property parameters used in many calculations/analyses. However, the applicant did not have sufficient data to calculate the unit weight values for the ESP subsurface soils and instead used the values from previous investigations. In the SER with Open Items, the staff concluded that it was not acceptable for the applicant to use these previously determined engineering parameters in this manner. Accordingly, this issue was designated as Open Item 2.5-17 in the SER with Open Items.

In response to Open Item 2.5-17, the applicant provided the tabulated unit weight for 15 samples from the Blue Bluff Marl and 3 samples from the Lower Sand Stratum. The number of measurements was limited to be consistent with the scope of the ESP site investigation program as designed by the applicant. Additional unit weight measurements were included by the applicant in the revised SSAR and are provided in Table 2.5.4-1 of this SER.

In its review of the response to Open Item 2.5-17, the staff focused on the additional unit weight measurements provided in the revised SSAR Table 2.5.4-1. The staff also considered the description of these additional unit weight measurements and concludes that a sufficient number of samples was measured and that the value ranges of the samples tested are consistent for the sand, silt, and clay materials that were tested. Therefore, the staff concludes that the information provided by the applicant in the revised SSAR with respect to the unit weight measurements for the Blue Bluff Marl and Upper and Lower Sand Strata at the site is acceptable and follows the guidelines presented in RG 1.138. Accordingly, the staff considers Open Item 2.5-17 closed. This closure also resolves RAI 2.5.4-12.

Chemical Tests

The staff noted that, in SSAR Section 2.5.4.2.5.3, the applicant stated that chemical tests were not included in the ESP laboratory testing program. The applicant also stated in the SSAR that chemical tests would be required for the backfill materials placed in proximity of planned concrete foundations and buried metal piping, and the applicant committed to conduct these chemical tests in the COL investigation phase. Accordingly, the need to provide chemical test results on the backfill was identified as COL Action Item 2.5-3 in the SER with Open Items.

However, in a later revision to the SSAR, the applicant included additional information on the excavation and backfill plans for the site of VEGP Units 3 and 4, including the chemical tests performed on the backfill materials, the results of which are included in SSAR Appendix 2.5C. These plans and tests were evaluated by the staff as part of the information provided in support of the LWA request. Because the application now contains this information in the SSAR, the staff concludes that COL Action Item 2.5-3 is no longer necessary.

Blue Bluff Marl Design Shear Strength

In RAI 2.5.4-7, the staff asked the applicant to explain why the undrained shear strength values (7.2 kPa (150 psf) to 205.9 kPa (4,300 psf)) from the UU tests performed on the Blue Bluff Marl samples were significantly lower than the SSAR specified design value, 478.9 kPa (10,000 psf), and to explain why these values differed substantially from the values (12.0 kPa (250 psf) to 23,946.4 kPa (500,000 psf)) obtained from previous investigations conducted for Units 1 and 2. The staff also asked the applicant to justify the use of a 478.9 kPa (10,000 psf) design value based on the SPT N-values measured during the ESP investigations.

In response to RAI 2.5.4-7, the applicant stated that the laboratory measurements of undrained shear strength for the Blue Bluff Marl (Lisbon Formation) yielded low values because the tests were performed using one confining pressure corresponding to the overburden pressure. The applicant also listed some qualitative factors to explain why these laboratory values were low. These factors included (1) being unable to push the CPTs below the Barnwell Group and into the Lisbon Formation (Blue Bluff Marl), (2) Shelby tubes being unable to penetrate into the Lisbon Formation without being damaged, which indicated that the soils were very hard, and (3) possible disturbance of samples obtained by pitcher barrel due to sampling, storage, and transportation processes. For these reasons, the applicant adopted an undrained shear strength design value for the Blue Bluff Marl from the FSAR for VEGP Units 1 and 2. The applicant further provided empirical correlations between the PI value, SPT N-value, shear wave velocity, and the undrained shear strength to justify the use of the SSAR design value of 478.9 kPa (10,000 psf).

From its review of the applicant's response to RAI 2.5.4-7, the staff found in the SER with Open Items that the qualitative and quantitative information provided by the applicant did not justify the use of the SSAR design strength value of 478.9 kPa (10,000 psf) for the Blue Bluff Marl, based on the following five considerations:

1. The design strength value obtained from the previous investigation for Units 1 and 2 was generated using different regulatory requirements, different industry standards, and different testing technologies. The applicant can use the data or engineering values from the previous investigation as a reference to support the current decision, but may not use the data as a direct input to calculate engineering parameters or previous engineering values directly for the ESP site.

2. As for the qualitative reasoning presented by the applicant, being unable to push the CPT and Shelby tubes through the Blue Bluff Marl does not justify the applicant's use of a design strength value much higher than the values obtained from the testing. According to Appendix 2.5 A to the SSAR, because soil samples collected from the Blue Bluff Marl contain gravels, it is possible that the CPT and Shelby tubes engaged gravels causing it to be difficult for them to push through the soil. Therefore, this factor does not support the adoption of a specific value of 478.93 kPa (10,000 psf) as the design shear strength for the Blue Bluff Marl.

3. If, as the applicant implied, the samples used in the ESP tests were disturbed because of the sampling, storage, and transportation processes, then there would be no reliable ESP laboratory test results to support the determination of the design value for the ESP site.

4. The applicant did not justify the applicability of the empirical correlations used in its response, such as the correlations between the undrained shear strength and PI, N-value, or shear wave velocity. Specifically, Mayne (2006) developed the correlation between shear wave velocity and shear strength from one group of clays, and the applicant used this correlation in its response to RAI 2.5.4-7, but this correlation may not be applicable to the Blue Bluff Marl at the ESP site. Furthermore, Mayne recently recommended another correlation developed by Laval University Group (2007) based on data from three groups of clays. This correlation resulted in a lower shear strength value than the one originally developed by Mayne (2006).

5. Even if an empirical correlation is applicable, the applicant did not use appropriate input parameters. Instead, the applicant used inappropriate input parameters, based on very limited data, and values that vary significantly. For example, the design PI value of 25 is an average value based on 18 data points ranging from 5 to 58, with 3 points above 50. The applicant obtained the N-value 80 from a total of 58 samples; among the samples there were only 23 actual measured N-values, ranging from 27 to 81. The applicant extrapolated the N-values linearly for 35 measurements in which the sampler did not penetrate 12 inches, and most of those data ended up having the cutoff value of 100. As mentioned previously, most of the 35 SPT measurements did not penetrate 12 inches because the samplers were in contact with gravels. Therefore, the average N-value does not meaningfully represent the general soil properties due to the lack of actual measurement and possible gravel engagement during the SPT tests.

Based on the above considerations, the staff concluded in the SER with Open Items that the applicant did not provide sufficient data to reliably derive the undrained shear strength value for the Blue Bluff Marl for the design. Accordingly, this was identified as Open Item 2.5-12 in the SER with Open Items.

In response to Open Item 2.5-12, the applicant stated that SPTs and split-spoon sampling were conducted in almost all the ESP borings in accordance with ASTM D 1586 to provide a measure of the relative density for cohesionless soils and consistency for cohesive soils. The applicant also described the split-spoon sampling process, in which the sampler is driven into massive in-situ materials, converting the material to coarse-grained soils through the crushing process. The applicant indicated that it is this crushing process that was responsible for the high recorded N-value of the materials sampled. Although the applicant followed the guidance of Appendix X2 of ASTM D 2488 to identify the materials sampled during the ESP investigations, it acknowledged that this method has led to some confusion regarding the presence of gravel-sized particles taken from the borings. The applicant clarified this confusion by stating that gravel-sized particles were the result of the crushing process, and were not reflective of actual gravel encountered in the subsurface. The applicant also described ongoing laboratory tests (grain size distribution, Atterberg Limits, and carbonate content) that confirmed the visual reclassifications of the samples. Finally, the applicant revised the SSAR to include additional field and laboratory test results, which were used to verify the undrained shear strength of the Blue Bluff Marl.

The staff reviewed the information provided in response to Open Item 2.5-12. In particular, the staff focused on the applicant's classification of the crushed material from the split-spoon sampler in accordance with Appendix X2 of ASTM D 2488. The staff evaluated the applicant's explanation that no gravel was encountered in the subsurface, but that gravel-sized particles were produced from the crushing of more massive materials, such as micritic limestone or fossiliferous shale beds, which would explain the isolated occurrence of shell fragments in the subsurface investigations. The staff considers this a more likely explanation for the occurrence of "gravel-sized" particles resulting from sampling of the Blue Bluff Marl as this can happen when attempting to sample very hard material. And although this sampling method can produce "gravel-sized" particles, these "fragments" are not actual gravel and should not have been identified as such by the applicant. The applicant acknowledged this error and, in subsequent review of the sample material, was able to correctly identify the materials as resulting from the crushing of very hard massive materials. The staff also considered the additional field and laboratory tests included by the applicant in the revised SSAR as summarized in this SER. Based on the application of the appropriate ASTM guidance for reclassification of the gravel-sized particles encountered at the site, and the additional field and laboratory test results provided in the revised SSAR, in particular the Atterberg Limits and carbonate content tests indicating the presence of limestones and fossiliferous shales, the staff considers Open Item 2.5-12 closed. The closure of Open Item 2.5-12 also resolves the remaining issue from RAI 2.5.4-7.

In RAI 2.5.4-8, the staff asked the applicant for the following:

1. a description of the previous laboratory testing methods and results which indicate that the Blue Bluff Marl is highly preconsolidated,

2. justification for the assumption of an undrained shear strength of 766.3 kPa (16,000 psf) while the undrained unconsolidated test results yielded values from 7.2 to 205.9 kPa (150 to 4,300 psf).

3. justification for the conclusion that "the pre-consolidation pressure of the Blue Bluff Marl was estimated to be 3,831.4 kPa (80,000 psf)," and

4. justification for the conclusion that "settlements due to loadings from new structures would be small due to this pre-consolidation pressure" for the Blue Bluff Marl.

In its response to RAI 2.5.4-8, the applicant provided the following information:

1. The original data and interpretation were based on laboratory tests performed for VEGP Units 1 and 2, which included 191 one-point UU triaxial tests and 38 consolidation tests. The applicant used vertical pressures that reached 3,065 kPa (64000 psf) to perform consolidation tests for all 38 samples. Most of the test results (void ratio versus vertical effective stress curves) showed very flat curves, which indicated that the preconsolidation pressure had not been achieved.

2. The undrained shear strength of 766 kPa (16,000 psf) was an average value based on VEGP Unit 1 and 2 test data calculated from 185 one-point UU triaxial tests that disclosed undrained shear strength values of less than 2,394.6 kPa (50,000 psf).

3. The applicant used the Skempton (1957) method to estimate the preconsolidation pressure of the Blue Bluff Marl by relating the preconsolidation pressure to the PI value

and the undrained shear strength. The applicant concluded that the Lisbon Formation was highly overconsolidated because the calculations showed that the overconsolidation ratios (OCRs) were in the range of 3.6 to 5, and most of the consolidation test results on 38 samples from the Lisbon Formation, reported in Bechtel (1974b), showed very flat curves, which indicated that the preconsolidation pressure exceeded 3,065 kPa (64,000 psf).

4. The applicant also concluded that the settlement due to loadings from new structures would be small based on observation of VEGP Units 1 and 2 and that the settlements would take place during the construction phase.

Based on its review of the applicant's response to RAI 2.5.4-8, the staff found in the SER with Open Items that it was inappropriate to use the average undrained shear strength value for VEGP Units 1 and 2 as an input value to calculate preconsolidation pressure and OCRs for the Blue Bluff Marl at the ESP site because the previous value was obtained based on different regulatory requirements, regulatory guidelines, industry standards, and testing technologies. In addition, the spatial variation of the soil properties also made reliance on the VEGP Units 1 and 2 values inappropriate. Moreover, the previous shear strength value differs significantly from the one obtained during the ESP testing. Therefore, the applicant did not have sufficient sampling and testing results to reliably derive the input undrained shear strength used in calculating the preconsolidation pressure and OCRs of the Blue Bluff Marl. Accordingly, this was designated as Open Item 2.5-13 in the SER with Open Items.

In response to Open Item 2.5-13, the applicant stated that the ESP site investigation was limited in scope due to the depth of knowledge available based on VEGP Units 1 and 2. The applicant also noted that although the ESP borings disclosed field measurement data consistent with the previous investigations, there was some confusion regarding the material descriptions as was discussed in response to Open Item 2.5-12. The applicant clarified this issue in its revision to the SSAR, which also included calculations of preconsolidation pressure and overconsolidation ratios for the Blue Bluff Marl using additional test data from the ESP investigation.

The staff focused its review of Open Item 2.5-13 on the additional information provided in the revised SSAR related to preconsolidation pressure and the OCRs for the load-bearing Blue Bluff Marl. The staff also considered the closure of Open Item 2.5-12 as referenced in the applicant's response to Open Item 2.5-13. Based on the applicant's revisions to the SSAR to include preconsolidation pressure of 3,831 kPa (80,000 psf) and an OCR of 8 for the Blue Bluff Marl based on additional site investigations that indicated that settlements due to loadings from new structures would be small due to the high preconsolidation pressure, the staff concludes that the applicant has sufficiently addressed the calculations identified in Open Item 2.5-13 and therefore the staff considers Open Item 2.5-13 closed. Furthermore, the closure of Open Item 2.5-13 resolves RAI 2.5.4-8.

In RAI 2.5.4-10, the staff asked the applicant to provide the relative density of the Blue Bluff Marl. The applicant stated in its response that the design value of the undrained shear strength for the soil was 478.9 kPa (10,000 psf) and its preconsolidation pressure could be as high as 3,831 kPa (80,000 psf); therefore, the applicant concluded that the Blue Bluff Marl was highly overconsolidated and behaved as hard clay or soft rock material, not as a granular material. The applicant further stated that relative density does not apply to the Blue Bluff Marl. From its review of the applicant's response, the staff concluded in the SER with Open Items that test data for the Blue Bluff Marl were very limited. As described in the SSAR, the limited laboratory test data showed that the percent fines content ranged from 24 to 77 percent, the moisture content ranged from 14 to 67 percent, and the PI ranged from non-plastic to 58 percent. Each of the above-mentioned parameters does not exclude the possibility of the marl being liquefied. In addition, the undrained unconsolidated tests yielded undrained shear strength values from 7.2 to 205.9 kPa (150 to 4,300 psf), which significantly differ from the design shear strength value of 478.9 kPa (10,000 psf), as indicated in the discussion of RAI 2.5.4-7. Therefore, the applicant's response did not support the conclusion that the Blue Bluff Marl would behave as a hard clay or soft rock material because the applicant did not use the ESP soil engineering values to calculate relative density for the Blue Bluff Marl. Accordingly, the need to demonstrate that the Blue Bluff Marl would behave as a hard clay or soft rock material, and thus not need to be addressed using relative density, was designated as Open Item 2.5-15 in the SER with Open Items.

The applicant's response to Open Item 2.5-15 referenced the response to Open Item 2.5-12 and the confusion in subsurface material description. The applicant also stated that while it is technically correct to identify some Blue Bluff Marl samples as sands and gravels, this description does not accurately indicate the in-situ structure of the marl. The applicant conducted laboratory testing to evaluate the carbonate content of the marl materials previously identified as sands and gravels, which the applicant concluded were indicative of a soft rock or hard clay material with lesser amounts of coarse sand and no determinable gravel present. The applicant further stated that the material that was previously identified as gravel was reclassified as limestone fragments. Again, the applicant included the results of additional data and site investigations in the revised SSAR.

The staff considered both the applicant's response to Open Item 2.5-15 as well as the closure of Open Item 2.5-12, which was referenced therein. Since additional laboratory data and site investigations were provided in the revised SSAR that clarified the composition of the Blue Bluff Marl, and the staff concluded in Open Item 2.5-12 that there was no determinable gravel in the subsurface material, the staff concludes that the applicant has provided a sufficient explanation, including supporting data and analyses, to prove that the marl will behave as a hard clay or soft rock material at the ESP site. Based on the resolution of Open Item 2.5-12 and the additional information regarding to composition of the Blue Bluff Marl in the revised SSAR, the staff considers Open Item 2.5-15 closed. Furthermore, with the closure of Open Item 2.5-15, the staff also considers RAI 2.5.4-10 resolved.

Following the submittal of the revised SSAR and the LWA request, the staff issued further requests for additional information to address the supplemental information. These supplemental RAIs are evaluated throughout the following sections and are identified with an "S."

The staff's evaluation of the information provided in support of the LWA request is as follows:

Field Investigations

Similar to its request in RAI 2.5.4-1, in RAI 2.5.4-1S the staff asked the applicant to 1) clarify how it had arrived at the number of ESP soil borings as 174 and to provide a detailed accounting of these additional borings, and 2) identify how many of the penetrations would be unusable for the site-specific analyses because they were taken through the Upper Sand Stratum material that would be excavated and replaced. In response to RAI 2.5.4-1S, the applicant provided a table that broke the number of borings down by series number, subject (i.e., location within the site or specific structure), and the exact number of borings at the subject location. The table indicates that the applicant completed 40 borings in the Unit 3 power block and cooling tower area, and 37 in the Unit 4 power block and cooling tower area. The remaining 97 borings were distributed across the rest of the site of VEGP Units 3 and 4. With this information, the staff was able to account for the number of total borings and their locations within the site, and the staff accordingly considers Item 1 of RAI 2.5.4-1S resolved. Also in this response, the applicant stated that 70 soil borings were located in the immediate vicinity of the combined power block footprint with exploration depths varying from 6.5 to 128 m (21.5 to 420 ft). The applicant further explained that with the exception of two offset borings, each of these borings was drilled through Upper Sand Stratum and advanced into the Blue Bluff Marl. The applicant further stated that 42 of these 70 borings penetrated the Blue Bluff Marl and advanced into the Lower Sand Stratum. With this information, the staff considers Item 2 of RAI 2.5.4-1S resolved because, as the applicant advanced 68 of the 70 borings through the Upper Sand Stratum and into the underlying layers, almost every boring produced usable site-specific data. However, the applicant's response that only 42 borings penetrated the Blue Bluff Marl led the staff to request additional information identified as RAI 2.5.4-20S.

In RAI 2.5.4-20S, the staff asked the applicant to provide additional information to demonstrate that the 42 borings that penetrated the Blue Bluff Marl were sufficient to satisfy the site foundation criteria contained in Regulatory Guides 1.132 and 1.138, including the boring depth acceptance criteria. The staff also asked for clarification of the statement made in response to RAI 2.5.4-2S that only six of 70 borings penetrated the Lower Sand Stratum.

The applicant responded that, in keeping with RG 1.132, the borings were located beneath and adjacent to structures to provide the maximum aerial coverage, which resulted in a boring at the center of the safety-related structures and uniformly spaced inside and relatively close to the perimeter of the other power block structures. In the response to RAI 2.5.4-20S, the applicant provided a Table 1, Summary of COL Power Block Borings, which summarized the number of borings for each structure in each unit. The guidance in RG 1.132 for the density of site borings is one boring per 929 square meters (10,000 square feet): however, the applicant determined the density of its borings to be one boring per 501 square m (5,400 square ft). Regarding the boring depth acceptance criteria in RG 1.132, Appendix D of the RG states that "dmax, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loading is less than 10 [percent] of the effective in-situ overburden stress." The applicant noted that the foundation that will have the largest dmax is the nuclear island base mat. Based on the AP1000 DCD Revision 15 design bearing pressure under the base mat of 412 kPa (8.6 ksf), the applicant determined that the nuclear island base mat dmax is on the order of 82 m (270 ft). The applicant noted that three borings were drilled at each unit to a depth of at least 76 m (250 ft), and one boring at Unit 3 was drilled to a depth of 128 m (420 ft) while the deepest boring at Unit 4 was to a depth of 122 m (400 ft). As for other power block structures, the applicant noted that the other structures located in the power block were founded nominally at the surface, and that the exploration depth of the borings for these structures was generally 45.7 m (150 ft).

After considering the clarifications and additional information presented by the applicant concerning the RG 1.132 guidelines for boring spacing, depth, and density, the staff has determined that the applicant's response is sufficient to address the location of borings beneath and adjacent to structures to provide the maximum aerial coverage, the density of required borings, and the minimum depth requirements for boreholes because 1) the applicant exceeded the RG 1.132 guidance for density of site borings, 2) the applicant advanced a boring within each nuclear island power block to a depth well in excess of the RG 1.132 guidance for dmax, and 3) the applicant met the intent of the RG 1.132 guidance for spacing by locating a boring at the center of the safety-related structures and by uniformly spacing other borings around the inside and relatively close to the perimeter of the other power block structures.

With respect to the guidelines of RG 1.138, the applicant explained that specific guidance about the number of tests that should be performed was not provided in RG 1.138. In response to the RAI, the applicant provided the staff with a table that summarized the COL power block borings for each structure in each unit. Regarding the applicant's response concerning the laboratory testing guidelines in RG 1.138, the staff agrees with the applicant's statement that the RG does not provide specific guidance about the numbers of laboratory tests that should be performed and that this is most likely because the numbers and types of tests depend on various site-specific factors such as the location of borings with respect to significant structures, the depth of sampling (e.g., it may be within a zone of excavation), the type of sample materials (cohesive, cohesionless, soil or rock), and the sample type (disturbed or undisturbed). The RG states that the focus of laboratory investigations should depend on the design requirements and nature of problems encountered or suspected at the site (i.e., some level of determination about the types and quantities of testing needs to be left to professional judgment by the onsite personnel). The staff determined by its review of the applicant's referenced tables, in particular SSAR Tables 2.5.4-3, 2.5.4-3a, and 2.5.4-4, "Types and Numbers of Laboratory Tests for the ESP and COL Investigations and Summary of Laboratory Tests Performed on Selected Soils Samples", that summarize the laboratory test results performed on ESP boring samples, that the applicant has conducted a laboratory testing program sufficient to adequately characterize the engineering properties of the subsurface materials. The staff reached this determination because the laboratory testing program conducted by the applicant included a variety of conventional index (tests that determine the properties of soils that indicate the type and condition of soils and provide a relationship to structural properties such as strength, compressibility, permeability, swelling potential, e.g., particle size distribution and consistency limits) and geotechnical engineering tests as well as dynamic soil test (RCTS) such that the applicant was able to sufficiently characterize the properties of the site soils for the purpose of evaluating the stability of the site for the applicant's planned construction. Finally, the applicant stated that the listed number of borings penetrating the Lower Sand Stratum was a typographical error. Therefore, based on the applicant's responses to RAIs 2.5.4-1S and 2.5.4-20S, the staff concludes that these RAIs were adequately addressed by the applicant and considers them resolved.

Shear Wave Velocity Profiles

In RAI 2.5.4-4S, the staff requested that the applicant provide an assessment of the in-situ velocity profile through the Upper Sand Stratum. The applicant described the additional laboratory strength testing and shear wave velocity measurements performed in the Upper Sand Stratum in the power block and surrounding areas as part of its COL investigations. Figure 2.5.4-3 of this SER shows the in-situ shear wave velocity profile through the Upper Sand Stratum to the Dunbarton Triassic Basin rock. The applicant provided the test results of the laboratory strength testing, which included 10 consolidated undrained triaxial shear tests from relatively undisturbed Upper Sand Stratum samples, Atterberg Limits and chemical tests, and a plot of shear wave velocity measurements in the stratum. In follow-up RAI 2.5.4-23S, the staff asked the applicant to provide justification as to why the two-dimensional (2D) wave velocity consideration was not considered in the SSI analysis.

The staff reviewed the response to RAI 2.5.4-4S as it related to geotechnical engineering, especially the additional strength and shear wave velocity measurements included in the revised SSAR, and concludes that the applicant provided sufficient information to close the geotechnical engineering aspects of RAI 2.5.4-4S because the additional laboratory test results, particularly the Atterberg Limits, confirmed the variable nature of the Upper Sand Stratum and its corresponding low shear strength. Furthermore, the applicant collected additional shear wave velocity data in the Upper Sand Stratum that displayed values over a large range but generally below the required minimum of 304.8 m/s (1,000 fps), which also confirmed the variable nature of the Upper Sand Stratum materials and further validated the applicant's decision to completely remove this stratum. Since the response to RAI 2.5.4-23S specifically addresses structural engineering aspects at the VEGP Units 3 and 4 site, the staff evaluates the response in Section 3.8 of this SER.

The site characteristic values of shear wave velocities were specified for depth intervals and are given in Appendix A to this SER and SER Tables 2.5.4-6 and 2.5.4-7. The applicant determined these characteristic values from the geophysical surveys completed at the VEGP site. Because the values were determined from the results of the applicant's geophysical surveys, which the staff reviewed and found to be acceptable in Section 2.5.4.3.4 of this SER, the staff concludes that these values are acceptable for use as the site characteristics.

Table 2.5.4-6 Shear Wave Velocity for ESP Site Amplification Analysis

Geologic Formation	Depth (feet)	V_s (fps)
Compacted Backfill	0 to 6	573
	6 to 10	732
	10 to 14	811
	14 to 18	871
	18 to 23	927
	23 to 29	983
	29 to 36	1,040
	36 to 43	1,092
	43 to 50	1,137
	50 to 56	1,175
	56 to 63	1,209
	63 to 71	1,232
	71 to 79	1,253
	79 to 86	1,273
Blue Bluff Marl (Lisbon Formation)	86 to 92	1,400
	92 to 97	1,700
	97 to 102	2,100
	102 to 105	1,700
	105 to 111	2,200
	111 to 123	2,350
	123 to 149	2,650
Lower Sand Stratum	149 to 156	2,000
(Still Branch)	156 to 216	1,650
(Congaree)	216 to 331	1,950
(Snapp)	331 to 438	2,050
(Black Mingo)	438 to 477	2,350
(Steel Creek)	477 to 587	2,650
(Gaillard/Black Creek)	587 to 798	2,850
(Pio Nono)	798 to 858	2,870
(Cape Fear)	858 to 1,049	2,710
Dunbarton Triassic Basin & Paleozoic Crystalline Rock	> 1,049	see Table 2.5.4-11, Part B

Table 2.5.4-6 Continued, Six Alternate Profiles

Part B: Rock Shear-Wave Velocities - Six Alternate Profiles

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,402.5	8,000	8,700
1,402.5 to 1,405	8,005	8,703
1,405 to 1,525	8,059	8,739
> 1,525	9,200	9,200

Rock Vs profile corresponding to the location midway between B-1002 and B-1003.

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,450	8,000	8,700
1,450 to 1,550	8,090	8,760
1,550 to 1,650	8,180	8,820
1,650 to 1,750	8,270	8,880
1,750 to 1,830	8,360	8,940
1,830 1,900	8,414	8,976
> 1,900	9,200	9,200

Rock Vs profile corresponding to the location of B-1003.

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,450	8,000	8,700
1,450 to 1,550	8,090	8,760
1,550 to 1,650	8,180	8,820
1,650 to 1,750	8,270	8,880
1,750 to 1,850	8,360	8,940
1,850 to 1,950	8,450	9,000
1,950 to 2,050	8,540	9,060
2,050 to 2,127.5	8,630	9,120
2,127.5 to 2,155	8,679.5	9,153
2,155 to 2,275	8,733.5	9,189
> 2,275	9,200	9,200

Table 2.5.4-7 Shear Wave Velocity for COL Site Amplification Analysis

Geologic Formation	Depth (feet) (ft)	V_s (fps) (fps)
Compacted Backfill	0	550
	5	724
	10	832
	20	975
	30	1064
	40	1130
	50	1183
	60	1228
	70	1267
	80	1302
	85	1318
	86.5	1327
	88	1327
Blue Bluff Marl (Lisbon Formation)	88 to 96	1,341
	96 to 102	1,747
	102 to 110	1,988
	110 to 122	2,300
	122 to 156	2,541
Lower Sand Stratum (Still Branch)	156 to 164	1,820
	164 to 220	1,560
(Congaree)	220 to 236	1,757
	236 to 280	2,000
	280 to 328	1,926
	328 to 340	1,727
(Snapp)	340 to 447	2,050
(Black Mingo)	447 to 486	2,350
(Steel Creek)	486 to 596	2,650
(Gaillard/Black Creek)	596 to 807	2,850
(Pio Nono)	807 to 867	2,870
(Cape Fear)	867 to 1,059	2,710

Geotechnical Properties of the Lower Sand Stratum

In RAI 2.5.4-5S, the staff noted that, during its review of the ESP application, some samples below the Blue Bluff Marl were identified as having extremely low blow counts, which called into question the adequacy of the soil material for settlement and bearing capacity. The staff also noted that, although the applicant indicated through informal discussions that these low blow counts were anomalies, the LWA request did not contain an adequate discussion of this anomalous conclusion. Therefore, the staff requested that the applicant provide the basis for the conclusion that the samples with low blow counts were anomalies.

In response, the applicant stated that 42 borings in the power block area penetrated the Blue Bluff Marl, 611 linear feet of drilling was conducted in the Lower Sand Stratum, and 111 SPT split barrel samples were collected from the Lower Sands. The applicant reported that the average corrected blow count reading in the Lower Sand Stratum was 250 bpm (75 bpf), indicative of a very high relative density. The applicant also stated that, with the exception of one value, all of the N60-values taken in the Lower Sand Stratum were greater than 30 bpf, again indicative of a dense to very dense material, although one N-value from a sample taken in the Still Branch Formation of the Lower Sand Stratum at an elevation of -12.6 to -13.1 m (-41.5 to -43 ft), and from which the split barrel sampler was unable to recover a sample, indicated very loose material. The applicant attempted to take an undisturbed sample (UD-11) from elevation -39.5 to -41.5, but no recovery was obtained in this sample. Since the applicant identified the material above this elevation as light gray sand (SP), the difficulty in sampling this material and the weight of hammer reading was an anomaly in sampling that was attributed to disturbed soil conditions at the bottom of the borehole. The applicant surmised that these conditions were likely the result of a hydrostatic pressure imbalance between the borehole and the in-situ hydrostatic pressure, with the resulting imbalance causing a quick condition to develop in the poorly graded sands at the attempted sampling depth. In such circumstances, the resulting disturbed poorly-graded sand will flow out of the sampler, which makes the material difficult to sample, as the applicant appears to have experienced in its lack of sample recovery at that depth. Overall, the applicant concluded that the SPT N-values behaved as expected by increasing with depth. Based on the applicant's response to RAI 2.5.4-5S and because the applicant encountered no other evidence of soft zones or loose material in the 611 linear feet of drilling conducted in the Lower Sand Stratum, the staff concurs with the applicant's explanation that it likely encountered an anomalous condition during sampling at this depth, as such a condition is not an unusual occurrence when attempting to sample very granular material. Therefore, the staff considers RAI 2.5.4-5S resolved. This explanation also addresses COL Action Item 2.5-2, concerning the location and extent of soft zones, which was resolved earlier in this section of the SER.

Geotechnical Properties of the Blue Bluff Marl

The staff identified multiple RAIs related to the properties of the Blue Bluff Marl (BBM). In RAI 2.5.4-2S, the staff requested that the applicant provide a description of the borings that penetrated into and through the BBM, of the number and types of samples recovered, as well as of the material underlying the BBM. In response to RAI 2.5.4-2S, the applicant stated that 70 borings were taken in the power block area; 42 of these borings penetrated the BBM, accounting for 863 linear m (2,831 linear ft) of drilling in this stratum. Additionally, seven hundred and forty-two SPT split barrel samples (disturbed samples) were obtained in the BBM, for which the applicant presented figures of the SPT N60 values and shear wave velocity measurements. From these SPT data, the applicant recorded an average N-value of 233 blows per meter (70 blows per foot) with a median value of 240 bpm (72 bpf); the average N60-value is 96. The applicant stated that nearly all of the SPT N60 values from the BBM were greater than 100 bpm (30 bpf). Additionally, the applicant stated that the number of borings penetrating the underlying Lower Sands (LS) was six of seventy, which accounted for 186 linear meters (611 linear ft) of drilling in this stratum. The number of borings that penetrated the Lower Sands was addressed in follow-up RAI 2.5.4-20S, which was previously discussed earlier in this section of the SER.

In follow-up RAI 2.5.4-21S, the staff requested that the applicant provide clarification of how the formulas provided in the response to RAI 2.5.4-2S were used to obtain corrected SPT blow counts. The applicant responded that the formula included in the response was provided as an explanation of how the measured N-values were interpreted in cases where full penetration of the 0.45 m (18-inch) sampler was not achieved due to the presence of very dense and very hard material, which occurred primarily in the BBM. The applicant clarified its conservative approach, which involved interpreting high measured N-values by recomputing the measured N-values using a simpler more intuitive approach; the applicant performed this recomputing where full penetration of the split barrel sample was not achieved due to very hard or very dense material. The applicant noted that the recomputation would not impact the majority of measured N-values where full penetration of the split barrel sampler was achieved, and where full penetration was not achieved because of the hardness or high relative density of the soil, the majority of computed N-values would be at the capped value of 333 bpm (100 bpf). The staff agrees with the applicant's recomputation of the N-values where the applicant was unable to achieve full penetration due to the very hard nature of the marl stratum, as this only affects a relatively small number of the total values measured, and the capped values are still indicative of a very dense or hard material that is the marl stratum. The recomputed and replotted data was included in the ESP Revision 4 for staff's review. Based on the review of the data presented in response to RAIs 2.5.4-2S and 2.5.4-21S, the staff found that the applicant provided sufficient data to enable the staff to determine that the applicant adequately sampled and tested the BBM Stratum and clarified the method used to correct SPT blow counts. Accordingly, the staff considers RAIs 2.5.4-2S and 2.5.4-21S resolved.

In RAI 2.5.4-3S, the staff asked the applicant to demonstrate how BBM samples were obtained and what degree of disturbance was involved. In response, the applicant stated that soil borings into the BBM were drilled using mud rotary methods and SPT tests; split barrel soil sampling was conducted in accordance with ASTM D 1586, generally at 1.5 m (5 ft) intervals. The applicant noted that many of the split barrel samples obtained from harder layers or lenses within the marl were fractured by the sampling process, and some of these samples had the appearance of angular sands or gravels. The applicant obtained relatively undisturbed (intact) soil samples using a three inch diameter thin-walled Shelby tube sampler in accordance with ASTM D 1587. The applicant stated that, in general, the samples taken in the Upper Sands were obtained through the direct push method, whereas samples taken in the BBM and Lower Sands were obtained using a Pitcher sampler, which is recommended for hard or dense soils and soft rocks, in accordance with ASTM D 6169, due to the very hard/dense nature of these materials. The applicant also stated that undisturbed samples and tubes were inspected, sealed, and transported to the climate-controlled on-site storage area following ASTM D 4220 guidelines, and samples were transported to various off-site testing laboratories according to the applicant-approved subcontractor procedures for sample transportation, including transporting RCTS samples by automobile to Houston, Texas.

In follow-up RAI 2.5.4-22S, the staff asked the applicant to provide a description of the approved transportation procedures used to move RCTS samples from the site to a test facility. In response, the applicant provided a copy of the applicant-approved subcontractor procedure (work instruction) for transporting undisturbed samples by automobile, which the staff determined provided adequate instructions for handling and securing the samples during transportation, consistent with standard industry and ASTM guidelines. Based on its review of the applicant's response, the staff finds that the applicant demonstrated its use of appropriate material sampling techniques using acceptable industry practices or standards. Therefore, the staff considers RAIs 2.5.4-3S and 2.5.4-22S resolved.

In RAI 2.5.4-6S, the staff asked the applicant to provide the basis for its determination of the design value for cohesion of the BBM of 478 kPa (10,000 psf) and to explain how this value is to be used. The staff indicated that it is important to understand the basis for this evaluation, whether any laboratory test data was available to support the proposed design value, and where in the facility evaluation the parameter would be used.

In response, the applicant reiterated that the design value of 478 kPa (10,000 psf) for cohesion of the BBM was based on evaluating empirical correlations and laboratory test data from the ESP geotechnical investigation that was previously presented in response to RAI 2.5.4-7. The applicant also stated that this design value for cohesion of the BBM was based on evaluating empirical correlations and laboratory test data from the ESP geotechnical investigation, including 15 UU tests. The applicant collected additional data during the COL investigation to verify the design value developed during the ESP investigation. The applicant conducted UU and CU triaxial tests at various confining pressures, with results suggesting that the shear strength of the BBM increased with confining pressure as expected. The applicant stated that the marl is located at an approximate depth of 27 to 50 m (90 to 165 ft) with a design ground water level at a depth of 16.7 m (55 ft), and a range of confining pressures, based on overburden conditions, of 320 to 646 kPa (6,500 and 9,700 psf). The applicant noted that within this range, UU test results yielded minimum shear strength of 81 kPa (1,700 psf) and a maximum of 560 kPa (11,700 psf) while the CU test resulted in a minimum value of 134 kPa (2,800 psf) and a maximum value of 1,541 kPa (32,200 psf) for shear strength at the range of confining pressure. The applicant also noted that previously determined confining pressures corresponded to the upper limit of 766 kPa (16,000 psf) used in conducting the UU and CU triaxial tests, and at the higher confining pressure, the average UU and CU test results are 411 and 713 kPa (8,600 and 14,900 psf), respectively. From a review of the field and laboratory test data, the applicant concluded that, regarding the design undrained strength value of 478 kPa (10,000 psf), UU and CU tests conducted at confining pressures of 766 kPa (16,000 psf), empirical correlation with N-values, and empirical correlation with shear wave velocity, all support the design value of 478 kPa (10,000 psf).

The applicant used undrained shear strength of the marl stratum to evaluate the bearing capacity of the nuclear island, incorporating the shear strength value into the calculation of allowable bearing pressure through superposition, as follows from the RAI response:

$$qo = c \cdot Nc \cdot \zeta c + q \cdot (Nq) \cdot \zeta q + 0.5 \cdot \gamma' \cdot B \cdot N\gamma \cdot \zeta\gamma \qquad (1)$$

where:

- qo = ultimate bearing pressure (ksf)
- c = soil cohesion (ksf)
- q = effective overburden pressure at bottom of foundation level (ksf)
- γ' = effective unit weight of soil (kcf)
- B = foundation width (ft) = 101 ft
- L = foundation length (ft) = 254 ft
- Nc, Nq, $N\gamma$ = bearing capacity factor

ζc, ζq, $\zeta \gamma$ = foundation shape factor

In this superposition analysis, the foundation is placed on a "strong" layer (compacted structural fill) that is underlain by a "weaker" layer (BBM). The capacity of the "strong" layer is evaluated alone to obtain qo'. The capacity of the "weaker" layer is evaluated alone to obtain qo". The governing capacity, q, is determined by evaluating the effect of the "weaker" layer on the bearing capacity by the following equation:

qo = qo".exp{0.67.[1+(B/L)].(H/B)} (2)
qa = qoFS, with Factor of Safety (FS) = 3

where: qo" = ultimate bearing pressure of the foundation sitting on the surface of the Blue Bluff Marl (ksf)

H = thickness of compacted structural fill between the bottom of the foundation and the top of the BBM (ft) (H=43.5ft)

qo = ultimate bearing pressure at the foundation level

qa = allowable bearing pressure at the foundation level

For the "strong" (backfill) layer where: Φ = 34o, γ'moist = 120 pcf, γ'sat = 130 pcf

Nc = 42.16	Nq 2 9.44	Nγ = 41.06
ζc = 1.28	ζq =1.27	$\zeta\gamma$ =0.84
q = 4.74 ksf	γ'= 0.076 kcf	

From equation (1)
qo'=0.0 x 42.16 x 1.28 + 4.74 x (29.44) x 1.27 + 0.5 x 0.076 x 101 x 41.06 x 0.84 ≈ 0 + 177.2 +132.4 = 309.6 ksf

For the "weak" (Blue Bluff Marl) layer where: c = 10 ksf

Nc= 5.14	Nq 1.0	Nγ= 0.0	ζc= 1.08
ζq = 1.0	$\zeta\gamma$= 0.84	q = 8.49 ksf	

From equation (1)
qo" = 10 x 5.14 x 1.08 + 8.49 x (1.0) x 1 = 55.5 + 8.5 = 64 ksf.

Through superposition using equation (2), the ultimate bearing pressure at the foundation level is:
qo = 64 x exp{0.67 x [1+(101/254)] x (43.5/101)) = 95.8 ksf

Thus, with a factor of safety of 3 and the su of the BBM = 10 ksf, the allowable bearing pressure at the foundation level is:
qa = 95.8/3 or 31.9 ksf

The applicant explained that it used the same method to evaluate the allowable bearing pressure for other pressures as well. Based on the AP1000 standard design, where foundation pressure is 411 kPa (8,600 psf), the applicant provided additional consideration of contact pressure of the foundation and contact pressure projected to the top of the BBM. The applicant explained its methodology as follows:

Foundation Load = area x foundation pressure = 254ft x 101ft x 8.6ksf = 220,625 kips

Foundation pressure influence at the top of BBM =
Foundation Load / projected area, so
220,625 / (297.5ft x 144.5ft) = 5.1 ksf

Where: projected area = {(L + 2(H x s)) x (W + 2(H x s))}
H = 43.5 ft
s = slope of zone of influence (lv:2h) = 0.5

In conclusion, the influence of the foundation load decreases with depth such that at the top of the BBM, the load has diminished by 41 percent (5.1/8.6). Based on the above, using su = 10 ksf for the BBM:

- With the NI founded on the fill, the FS against bearing failure is 958/5.1 = 18.8
- With the NI founded directly on the BBM, FS = 64/8.6 = 7.4

Using su = 6.5 ksf for the BBM:

- With the NI founded on the fill, the FS = 66.8/5.1 = 13.1
- With the NI founded directly on the BBM, FS = 44.6/8.6 = 5.2

Based on the applicant's response to RAI 2.5.4-6S, including the calculations the applicant presented in its response, the staff concludes that the applicant adequately explained the basis for the determination of the 14.8 kPa (10,000 psf) design value. This conclusion is based on data and assessments provided by the applicant, as verified by the staff's confirmatory calculations and review of the laboratory triaxial test data provided in SSAR Revision 4. Furthermore, based on the applicant's response and review of the calculations presented, the staff concludes that the applicant explained how the 478 kPa (10,000 psf) design value will be used in the calculation of a factor of safety against bearing failure. However, although the staff was able to resolve most issues related to RAI 2.5.4-6S, the staff noted some areas of additional concern. The staff noted that the applicant's response to the RAI addressed only static bearing capacity evaluations for failure conditions; settlement considerations, which normally control the allowable pressures under large rigid basemats, were not included in the calculations. The staff also noted that the response did not address dynamic effects, which are the overwhelming effects on the computed toe pressures, and the staff requested the additional information. In follow-up RAI 2.5.4-24S, the staff requested that the applicant provide information addressing settlement considerations for static bearing capacity evaluations, and dynamic effects on the computed toe pressures.

In response to RAI 2.5.4-24S, the applicant stated that additional static and dynamic bearing capacity evaluations were underway, including localized punching failure of backfill materials supporting the nuclear island. The applicant conducted these assessments as part of the Phase 1 test pad program and used conventional analyses assuming safety factors of 3 and 2, for static and dynamic bearing capacity, respectively. Finally, the applicant evaluated settlement characteristics of the site and included all results in the revised SSAR.

The staff reviewed the applicant's response to RAI 2.5.4-24S, in particular the additional information and evaluations provided in the revised SSAR. The applicant stated that the soils supporting the nuclear islands did not exhibit extreme variations in subgrade stiffness and that the proposed Vogtle site could be considered uniform. The applicant presented in Section 2.5.4.2.2.2 that subsurface data has disclosed that the Blue Bluff Marl has a nearly even top over the length of the excavation footprints with relatively uniform thickness and consistent properties. Over this will be placed approximately 27.4 m (90 ft) of structural backfill that will be placed and compacted in level uniform lifts or layers. Results of the Phase 1 and 2 test pad program disclosed that the materials proposed for structural backfill have consistent engineering properties including density, shear wave velocity and N-values.

The applicant stated in SSAR Revision 4 that it based its allowable static bearing capacity values on Terzaghi's bearing capacity equations using an internal angle of friction of 36 degrees for the compacted backfill as developed from their field and laboratory testing program during the Phase 1 test pad program and COL investigation. The applicant evaluated the influence of the Blue Bluff Marl on the allowable bearing pressure using procedures outlined by Vesic, procedures which are acceptable to the staff as they are in common use. With a factor of safety of 3.0, the applicant determined that the site conditions provide an allowable bearing pressure of 1,628 kPa (34 ksf) under static loading conditions for the nuclear island. The staff concurs with this determination because the applicant used equations from Terzaghi and procedures from Vesic that are commonly used and widely accepted industry method.

The applicant also evaluated the allowable bearing capacity of the nuclear island under dynamic loading conditions, and again the methods of analysis were based on Terzaghi's bearing capacity equation for general shear using seismic bearing capacity factors from Soubra and Terzaghi's bearing capacity equation for local shear. Using a factor of safety of 2.25, the applicant determined that the site conditions provided an allowable bearing pressure of 2,011 kPa (42 ksf) under dynamic loading conditions for the nuclear island. Both the static and dynamic bearing capacity values are well below the minimums specified in Revision 15 of the AP1000 DCD. The staff concurs with the determination because again the applicant used widely accepted equations and factors for such evaluations.

Finally, the applicant conducted laboratory consolidation tests on relatively undisturbed samples of the Blue Bluff Marl and the Lower Sand Stratum, and the results confirmed the elastic behavior and very stiff and dense nature of the two strata. Also, the applicant's test pad program assessed the properties of the proposed compacted backfill and the results confirmed the very dense nature of the materials and showed that the expected performance under load will be similar to VEGP Units 1 and 2. The applicant performed a detailed settlement analysis using similar elastic properties used for the VEGP Units 1 and 2 and incorporated excavation, dewatering, and construction duration to determine basemat displacement histories. The applicant stated that the results predicted total settlement ranges of from 5.08 to 7.62 cm (2 to 3 inches), with an approximate tilt of .635 cm in 15.24 m (¼ inch in 50 ft), and a differential settlement between structures of less than 2.54 cm (1 inch). The applicant noted that these results are similar to actual movements measured for VEGP Units 1 and 2.

The staff concludes that the applicant provided sufficient information in response to RAI 2.5.4-24S to address both the static and dynamic bearing capacities for the materials supporting the nuclear island as it presented results based on site-specific test results input into equations and factors commonly in use to determine bearing capacities and settlements. Therefore, the staff considers RAI 2.5.4-24S closed. Furthermore, the closure of RAI 2.5.4-24S also resolves RAI 2.5.4-6S.

The staff finds that the applicant conducted a subsurface investigation program consistent with the guidelines presented in RG 1.132 to adequately characterize the subsurface conditions and materials, and it performed laboratory testing consistent with the guidelines presented in RG 1.138 to adequately determine the engineering properties of the subsurface materials and used the results to perform analysis to predict how the site conditions will support the AP1000 design requirements as presented in Revision 15 of the AP1000 DCD. Based on the information and findings above, including the resolution of RAIs, and the closure of Open Items, the staff concludes that the discussion of the properties of Subsurface Materials is acceptable.

The applicant determined the static and dynamic properties of the three principal soil groups and compacted structural backfill through its field investigations and through laboratory testing performed in accordance with RG 1.138. The staff concludes that the applicant complied with the relevant guidance of RG 1.138.

In Revision 4 of the VEGP SSAR, the applicant included information on the chemical tests performed on the engineered backfill for the VEGP Units 3 and 4 site. These tests are summarized in Section 2.5.4.2.2 of this SER and Subsection 2.5.4.2.5.3 of the SSAR, and included pH, chloride, and sulfate tests. The applicant stated that, due to the high concentration of sulfate in the Upper Sand Stratum, switchyard and borrow area 4, the concrete placed at the site would face mild exposure to sulfate attack. However, since the most potentially corrosive unit, the Upper Sand Stratum, would be completely removed during site excavation, the staff does not consider the exposure to sulfate attack to be a significant issue at the VEGP Units 3 and 4 site. Since the applicant included the results of chemical tests as part of the revised SSAR, the staff concludes that COL Action Item 2.5-3, as identified in the SER with Open Items, is no longer needed.

The staff concludes that the applicant's description of the subsurface materials was acceptable in that 1) the applicant, following the guidance of RG 1.132 and RG 1.138, investigated and tested the subsurface materials to determine that the soils encountered were alluvial and coastal plain sediments and characterized the soils as sands with silt and clay, the clay marl bearing layer, and underlying coarse to fine sand with interbedded thin seams; and 2) the applicant obtained sufficient undisturbed samples to allow for the adequate characterization of each of these soil groups and determine the extent, thickness, hardness and density, consistency, strength, and static design properties. The applicant also provided sufficient information in the form of plots, plans, and boring logs; and laboratory test results and summaries that enabled the staff to determine that the applicant had adequately characterized the subsurface soils and rock materials and determined their engineering and design properties. Therefore, the staff concludes that the applicant's description of the subsurface materials and their properties at the site of VEGP Units 3 and 4, per the information obtained from the ESP, COL, and LWA investigations, is acceptable. This conclusion is based on the information and findings above, including the resolution of RAIs and Open Items, and the addition of information to the revised SSAR that rendered COL Action Items unnecessary.

2.5.4.3.3 Exploration

The staff's evaluation of the information provided in support of the ESP application is as follows:

Section 2.5.4.3 of NUREG-0800 directs the staff to compare the applicant's plot plans and profiles of Seismic Category I facilities with the subsurface profile and material properties. Based on the comparison, the staff can determine whether (1) the applicant performed sufficient exploration of the subsurface materials and (2) the applicant's foundation design assumptions contain an adequate margin of safety.

In RAI 2.5.4-20, the staff asked the applicant to justify why it did not provide the relationship of foundations to the underlying materials in the form of plot plans and profiles, the foundation stability with respect to ground water conditions, and a detailed dewatering plan. In its response, the applicant stated that it would provide this information as part of a COL application once more details become available regarding the foundation and site interaction. The staff concurs with the applicant that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied. Accordingly, in the SER with Open Items, this was identified as COL Action Item 2.5-4. However, later revisions of the SSAR by the applicant included details of the foundation and site interaction, such as plot plans and profiles showing the relationship of the foundations in relation to the underlying materials, in particular boring location plans, boring logs and subsurface profiles, site cross-sections, shear wave velocity measurements and profiles, shear modulus and damping curves, and power block excavation sections. The applicant also provided sufficiently detailed discussions of the ground water conditions, including liquefaction analyses, and provided details about its proposed dewatering system. Accordingly, the staff concludes that the inclusion of COL Action Item 2.5-4 is no longer necessary.

MACTEC Reports

In SSAR Section 2.5.4.3, the applicant heavily referenced a MACTEC report included as an appendix to the application. In RAI 2.5.4-17S, the staff asked the applicant to provide a description of the refraction microtremor (ReMi) testing method used for site geophysical testing as discussed in the MACTEC Report. The staff specifically requested information detailing the application of this method in determining S- and P-wave velocity profiles; the staff also asked the applicant to provide a justification to demonstrate the adequacy of using these data to determine site properties and the resulting impact on response analysis.

The applicant responded to RAI 2.5.4-17S by stating that ReMi testing was conducted in the power block areas for Units 1 and 2, and in the footprint area for Units 3 and 4. The applicant also stated that the original intent was to establish a shear wave velocity profile using this data; however, during collection, it became apparent that the vibration frequency of the existing plant equipment was interfering with the results. After attempts to overcome the interference were unsuccessful in the field, the applicant consulted with Dr. K.H. Stokoe to review the results, who expressed doubt that the results represented the true profile. Therefore, the applicant concluded that the ReMi testing results should not be considered in the COL geophysical survey. The staff reviewed the applicant's explanation of the ReMi testing at the site, including the summary provided in Revision 4 of the SSAR. The staff concurs with the applicant and Dr. Stokoe's assessment that the test results do not truly represent the shear wave velocity profile at the site. The staff concludes that the applicant has provided sufficient information to clarify RAI 2.5.4-17S, and therefore the staff considers the RAI resolved because the applicant did not use the suspect test results.

In RAI 2.5.4-18S, the staff again referred to the MACTEC report, which indicated that Dr. K.H. Stokoe would review the RCTS data generated for appropriate use in the site evaluations. The staff asked for a description of the details, depth, and completeness of Dr. Stokoe's review. The applicant responded by clarifying that RCTS testing is performed by Fugro Consultants at their Houston, Texas facility, Dr. Stokoe was involved in the initial set-up and review of that facility. The applicant also clarified Dr. Stokoe's review role in that Dr. Stokoe reviewed each RCTS draft report to assure quality of the results. Dr. Stokoe also reviewed the laboratory procedures and setup prior to the commencement of RCTS testing. Additionally, the applicant stated that the geotechnical engineering contractor that was used, MACTEC, independently audited the Fugro facility and conducted surveillances of RCTS testing in progress.

The staff reviewed the applicant's response to RAI 2.5.4-18S, particularly the assurances from the applicant that the review of RCTS data by Dr. Stokoe, the foremost expert on the RCTS test method, would ensure that the quality of data generated was appropriate for use in site evaluations. The staff considered Dr. Stokoe's involvement in the initial setup and review of the Fugro RCTS testing facility and concludes that, based on the experience and expertise of Dr. Stokoe, the depth and completeness of Dr. Stokoe's review should ensure that quality information has been generated because Dr. Stokoe is the foremost expert on the RCTS test method. Furthermore, the staff concludes that the independent audit by the applicant's contractor, the leading expert on the test method in question, would further ensure quality of data. Therefore, the staff concludes that sufficient information and details were provided by the applicant to close RAI 2.5.4-18S.

The staff's evaluation of information provided in support of the LWA request is as follows:

In Revision 4 of the SSAR, the applicant provided additional figures of the plot plans and subsurface material profiles. The staff reviewed these figures and determined that because the applicant conducted its program following the guidelines presented in RG 1.132, and because the foundation design assumptions contain an adequate margin of safety consistent with regulatory guidelines and accepted industry practices, such as those developed by the U.S. Army Corps of Engineers (USACE) and delineated in the USACE Manual, Engineering and Design – Slope Stability, EM 1110-2-1902, Office of the Chief of Engineers, the applicant performed sufficient exploration of the subsurface materials. This information removed the need for COL Action Item 2.5-4, which the staff previously identified in the SER with Open Items.

The staff concludes that, based on the information and findings above, including the resolution of RAIs and Open Items, and the addition of information to the revised SSAR that rendered COL Action Items unnecessary, the discussion of the exploration of the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable for approval of both the ESP application and LWA request.

2.5.4.3.4 Geophysical Surveys

The staff focused its review of SSAR Section 2.5.4.3 on the adequacy of the applicant's geophysical investigations to determine the soil and rock dynamic properties. The applicant conducted three down-hole seismic CPT tests and five suspension P-S velocity tests during the ESP site investigation. The applicant compared the soil and rock dynamic properties obtained from these tests with the results from previous geophysical surveys conducted for Units 1 and 2.

In RAI 2.5.4-3, the staff asked the applicant to explain how the base case shear wave velocity profile was developed based on only 12 borings, since most of the borings did not go deeper than 91.4 meters (300 ft). The staff asked additional questions as part of RAI 2.5.4-3, which was discussed and evaluated in Section 2.5.4.3.2 of this SER. In response to RAI 2.5.4-3, the applicant stated that the base case shear wave velocity profile was developed in association with the Lisbon Formation (Blue Bluff Marl), Still Branch Formation, and the upper portion of the Congaree Formation based on the results of the three suspension P-S velocity logging tests performed at the ESP site. One of the suspension P-S velocity logging tests extended into bedrock below the Lower Sand Stratum, and the applicant used those results to derive the base case shear wave velocity profile below the top of the Congaree Formation. The applicant explained that the randomization model captures the uncertainty in the base case shear wave velocity profile for the in-situ soils. The applicant used logarithmic standard deviation of shear wave velocity as a function of depth, which was set to values obtained from soil randomization performed at SRS. After reviewing the applicant's response, however, the staff found that shear wave velocities vary significantly among the three profiles (ESP, VEGP, Units 1 and 2 and SRS), with most terminating at a depth from 85.34 to 60.96 meters (280 to 300 ft)), and lower shear wave velocities measured from down-hole seismic tests than from the suspension P-S velocity measurements. Furthermore, the shear wave velocities from previous investigations were relatively lower than those obtained from the ESP investigations. Therefore, in the SER with Open Items, the staff concluded that the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile. This issue was identified in the SER with Open Items as Open Item 2.5-18.

In response to Open Item 2.5-18, the applicant stated that the shear wave velocity provided in the ESP was based on site-specific data from velocity measurements taken in the footprint of the ESP site. The applicant also described the development of the velocity profile, which used down-hole seismic CPT data and P-S velocity logging data for elevations above the BBM and P-S suspension logging measurements for elevations below and including the marl. The applicant gave consideration to profiles developed at nearby sites, such as Units 1 and 2 and SRS. However, although the profiles were consistent, they were not incorporated by the applicant into the ESP profiles. The applicant used additional data to re-evaluate the ESP profile following more detailed site investigations, and the applicant included these evaluations in the revised SSAR.

The staff focused its review on the additional information provided by the applicant in the revised SSAR, which included shear wave velocity profiles derived from the down-hole seismic CPT data, P-S velocity logging data, and P-S suspension logging measurements. The staff finds that the applicant's shear wave velocity testing through the ESP and COL subsurface investigations and during the 2 Phase test pad program demonstrated that the site and compacted structural backfill will support the DCD's required minimum shear wave velocity. Based on these revised profiles, illustrated in Figures 2.5.4-3 and 2.5.4-5 of this SER, the staff concludes that the applicant provided shear wave velocity profiles, derived from the results of ESP site investigations, that were sufficient to address the concerns of Open Item 2.5-18. Therefore, the staff considers Open Item 2.5-18 closed. Furthermore, the closure of Open Item 2.5-18 resolves the remaining portion of RAI 2.5.4-3 as it relates to geophysical investigations at the site of VEGP Units 3 and 4.

Based on the review of SSAR Section 2.5.4.4 and the applicant's response to RAI 2.5.4-3, described above, the staff concluded that although the applicant used various methods to determine compressional and shear wave velocities, including some of the latest technologies recommended in RG 1.132, the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile nor to address the velocity difference from different methods. However, in Revision 4 of the SSAR, the applicant provided additional information on the shear wave velocity measurements, including the use of multiple methods such as suspension P-S velocity tests, down-hole seismic tests with cone penetrometers, and, although unsuccessful, ReMi testing. Based on the review of SSAR 2.5.4.4 and the applicant's responses to the RAIs, the staff concludes that the applicant adequately determined the dynamic properties of soil and rock through its geophysical surveys at the site of VEGP Units 3 and 4 because the applicant conducted its exploration program following the guidelines in RG 1.132, which included fieldwork and laboratory testing performed under an approved quality program in accordance with approved industry standards and practices.

The staff concludes, based on the information and findings detailed above, including the resolution of RAIs and Open Items, that the discussion of the geophysical survey at the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable for approval of the ESP application and LWA request.

2.5.4.3.5 Excavation and Backfill

The staff reviewed SSAR Section 2.5.4.5, focusing on the applicant's description of anticipated foundation excavations for safety-related structures, backfills, and slopes; excavation methods and stability; backfill sources and quality control; and control of ground water during excavation. The applicant stated that the Upper Sand Stratum would be removed and replaced with Seismic Category I backfill from the top of the BBM to the bottom of the containment and auxiliary buildings at a depth of about 12.19 meters (40 ft) below the final grade. Backfilling would continue up around those structures to final grade. The excavation would be open-cut, with slopes no steeper than 2:1 (horizontal-to-vertical ratio). The applicant indicated that the guidelines used for VEGP Units 1 and 2 would be followed during the development excavation and backfill plans at the COL phase.

The staff's evaluation of the information provided in support of the ESP application is as follows:

Extent of and Plans for Excavation

Since there was no specific description of the excavation plans in the first revision of the SSAR, in RAI 2.5.4-2, the staff asked the applicant to clarify whether the excavation and backfill would only cover the footprint of the power block or would instead extend to a certain distance beyond the foundation footprint. In response to RAI 2.5.4-2, the applicant explained that safety-related footprints of the future Units 3 and 4 would have two respective backfilled excavations, and those excavations would extend beyond their respective power block footprints. The applicant established the minimum lateral extent of each excavation by determining the stress zone as defined by a 1:1 (horizontal-to-vertical) slope ratio, extending from the bottom of the turbine, containment, and auxiliary building foundations. The approximate bottom of the foundation elevations would be 65.8 meters (216 ft) above msl for the turbine building, 54.9 meters (180 ft) above msl for the containment, and 39.6 meters (130 ft) above msl to the top of the Lisbon Formation (Blue Bluff Marl) for the auxiliary buildings. The stress zone at the top of the Lisbon Formation would extend approximately 26.2 meters (86 ft) horizontally beyond the footprint of the power block structures. The applicant considered the turbine building foundation to be the governing factor of this horizontal extension (highest foundation); therefore, the 26.2-m (86-ft) extension was conservatively set for all four sides of the excavation. The applicant planned to backfill the entire excavation, including the power block footprint, stress zone, and areas beyond the stress zone, using compacted structural fill.

Due to the concern of a possible backfill impact on the seismic response evaluation of the site and structures, in RAI 2.5.4-2, the staff also asked the applicant whether it would implement the seismic hazard calculations to the free-ground surface, including the Barnwell Group in the base case site soil column, if the site excavations were not to extend significant distances to the side of the plant. In addition, the staff asked the applicant to explain the basis for its column analysis that presumed uniform backfill in all horizontal directions, while the actual excavation and backfill would extend only to the immediate vicinity of the plant. In its response, the applicant stated that the site excavations would extend to significant horizontal distances from the structures. With the base of the excavation extending approximately 26.2 meters (86 ft) outside of the building footprint, and with the excavation side slope ratio at 2:1(horizontal to vertical), the structural backfill would extend more than 54.9 meters (180 ft) beyond the containment and auxiliary buildings at their foundation level and would extend more than 76.2 meters (250 ft) beyond the edge of the turbine building at its foundation level.

Since there was no specific description regarding the backfill compaction control, in RAI 2.5.4-2, the staff also asked the applicant to explain how compaction control would be implemented if the backfill was to contain as much as 25 percent fines content. In its response, the applicant stated that sand and silty sand with no more than 25 percent fines was obtained from onsite sources for use as backfill, as structural backfill for Units 1 and 2, and that it would use the same structural backfill criterion for Units 3 and 4. The applicant would also implement compaction controls for placement of the backfill through an independent soil testing firm. This testing firm would maintain an onsite soils testing laboratory to control the quality of the backfill material and the degree of compaction, and to monitor the compaction through field density tests performed at a minimum frequency of one test per 928 square meters (10,000 square ft) per lift of placed compacted backfill. In addition, the applicant committed to develop more detailed testing compaction control criteria during the COL phase. The applicant met this commitment through the testing performed during its Phase 1 and 2 test pad backfill program. At the time the SER with Open Items was issued, no site excavation or backfill had been performed; therefore, the staff considered this design-related information immaterial to determining whether 10 CFR Part 100 is satisfied at the ESP stage. Subsequently, the applicant performed additional subsurface investigations and laboratory testing to gather additional ESP and later COL data, which the applicant used to develop the later revisions of the SSAR and also defined the LWA portion of the activities to be the removal of the Upper Sand Stratum and excavation to the top of the Blue Bluff Marl bearing layer, placement of structural backfill to the bottom of the nuclear island foundation, installation of the concrete working surface mudmat and waterproofing membrane, installation of the MSE walls and accompanying waterproofing membrane around the perimeter of the nuclear islands, and backfilling around the outside perimeter of the MSE walls up to final plant grade.

After reviewing the responses from the applicant to RAI 2.5.4-2, the staff, in the SER with Open Items, concluded that, although the applicant provided more information on the extent of excavation, backfill material, and its compaction control, the applicant needed to consider some related issues during the COL stage including: (1) the stress zone described in the applicant's response to RAI 2.5.4-2 was based on normal static stress evaluations, but the applicant needed to consider both static and dynamic load induced stresses; and (2) since the applicant indicated that excavations would extend from about 26.2 meters (86 ft) outside of the building footprint with 2:1 (horizontal-to-vertical) side slope ratios and then extend away from the power block, the applicant needed to include the backfill material placed in and around the power block structures in the structural model when evaluating SSI, as indicated in the currently revised Section 3.7 of NUREG-0800. Thus, in the SER with Open Items, the applicant's commitment to provide detailed excavation and backfill plans during the COL stage was identified as COL Action Item 2.5-5.

Revision 4 of the SSAR contains detailed information on the excavation and backfill plans for the VEGP Units 3 and 4 site. The summary of these plans can be found in Section 2.5.4.1.5 of this SER. The applicant included discussions of the extent of excavations, methods and stability of excavations, backfill design and sources, quality control and ITAAC, groundwater control, and retaining wall plans. This information specifically fulfilled the level of detail specified by COL Action Item 2.5-5. Therefore, COL Action Item 2.5-5 is no longer necessary.

Geotechnical Parameters of Backfill Materials

Because the applicant did not describe the determination of shear wave velocity for the backfill, in RAI 2.5.4-4, the staff asked the applicant to explain how it would determine shear wave velocity values at depths of 15.2 meters (50 ft) and deeper for the backfill materials and whether it considered the effects of confinement. In its response, the applicant reiterated the statement of SSAR Section 2.5.2.5.1.2.1.1:

> Shear-wave velocity was not measured for the compacted backfill during the ESP subsurface investigation (APPENDIX 2.5A). Interpolated values based on measurements made on backfill for existing Units 1 and 2 (Bechtel 1984) are used instead.

The applicant also clarified that the measurements made of backfill soil for existing Units 1 and 2 were laboratory measurements using resonant column tests. The applicant developed shear wave velocity profiles for the backfill using equations presented in the response.

After reviewing the response to RAI 2.5.4-4, the staff found in the SER with Open Items that the applicant attempted to apply the estimated shear wave velocity from the backfill for the existing units to the backfill for the ESP site. But the equation used in the estimation dated back to the 1960s and there was significant variability, or uncertainty, for the parameter K2 in the equation. The calculation also did not account for confinement effects. Since the ability to show that the backfill meets the minimum shear wave velocity requirement with minimum in-situ variability is a major concern in the COL phase, and the procedures presented in the SSAR did not provide such information, the staff determined in the SER with Open Items that additional information to address the backfill shear wave velocity should be submitted in the COL application. Accordingly, this was identified as COL Action Item 2.5-6 in the SER with Open Items.

SSAR Revision 4 includes information on the applicant's test pad program, which was used to produce the site-specific data necessary to develop a shear wave velocity profile for the engineered backfill at the site. The applicant included the results of the test pad program in the revised SSAR, and the engineering properties, including shear wave velocity are found in Table 2.5.4-1 of this SER. The staff agrees that this information specifically addresses the needs of COL Action Item 2.5-6 because the information is specifically related to the actual materials the applicant planned to use for structural backfill and the shear wave velocity profile was developed for these proposed site-specific materials.

In summary, based on a review of SSAR Section 2.5.4.5 and the applicant's responses to RAI 2.5.4-2 and RAI 2.5.4-4 described above, the staff determined that the applicant did not initially provide detailed information on excavation and backfill plans due to the limited knowledge of the exact location of reactors and fill materials. Regulatory Position C.6 of RG 1.132 recognizes that there may be limitations on the extent of geologic mapping that may be performed prior to a site being approved under the 10 CFR Part 52 licensing procedures. To address this need for construction mapping, in the SER with Open Items, the staff proposed the inclusion of a permit condition requiring that the ESP holder or an applicant referencing the ESP perform geologic mapping of future excavations for safety-related structures, evaluate any unforeseen geologic features that are encountered, and notify the NRC no later than 30 days before any excavations for safety-related structures are open for NRC's examination and evaluation. Accordingly, this was identified as Permit Condition 2. However, geologic mapping of excavations was included within the scope of the LWA request, as was the evaluation of any unforeseen geologic features that may be encountered. Since this information is included within the scope of the LWA request, the staff concludes that Permit Condition 2 is no longer necessary.

The staff's evaluation of the information provided in support of the LWA request is as follows:

Subsequent revisions to the SSAR included additional information for the staff to review regarding the excavation and backfill plans proposed in the LWA request for VEGP Units 3 and 4. During the review of the revised SSAR, the staff identified several areas requiring additional information.

Geotechnical Parameters of Backfill Materials

In RAI 2.5.4-7S, the staff requested that the applicant provide a discussion of the required shear wave velocity condition that needs to be met to ensure the backfill soil will satisfy the analysis criteria used for the SSI calculations of the AP1000 standard design. The staff asked that this discussion refer to both the minimum shear wave velocity and the acceptable variability of the measure velocity over the nuclear island footprint.

The applicant responded by stating that a description of the borrow sources could be found in its response to RAI 2.5.4-10S. The applicant also described the general backfill design program for Units 3 and 4 as being modeled after the program that was used for the existing units, and which included a limiting fines content of no more than 25 percent passing the No. 200 sieve (0.075 mm); the Proctor test was utilized as the compaction standard. Furthermore, the applicant provided a detailed description of the two-phase backfill test pad program, which was used to develop the site-specific backfill design to satisfy the standard plant design siting criteria in Revision 15 of the AP1000 DCD and to develop placement and compaction methodologies for the construction program. The applicant stated plans to use the results of these two phases to finalize the details of the backfill construction program, including material properties criteria, construction methods, compaction methods and requirements, and testing protocol, before describing the phases of the program in greater detail:

> Phase 1 will entail a test pad, constructed below grade, approximately [6 m] 20 feet thick using on site borrow from the switchyard area borrow source. The backfill will be placed in [20.32 cm] 8 inch loose lifts and compacted to 95 percent of the maximum dry density as determined by ASTM D 1557. The placement of the backfill will be comprehensively monitored and tested. During backfill placement, field testing will include compaction and shear wave velocity testing utilizing surface wave methods (SASW). Parallel testing will be performed in the laboratory for density, grain size, moisture, and plasticity. On completion of test pad construction, SPT borings will be drilled through the test pad and sampled continuously in the backfill and at [1.5 m] 5-foot intervals to a depth of [6 m] 20 feet in the in-situ soil. Shear wave velocity will be measured in the test pad using cross-hole techniques in accordance with ASTM D4428. Shear wave velocity measurements will also be taken at the finished surface of the test pad using surface wave methods. Results of the test pad field and laboratory measurements will be used to develop expected shear wave velocity characteristics of the backfill.

The applicant concluded by stating that the description of the shear wave velocity data developed during Phase 1 would be evaluated against the assumed shear wave and soil degradation characteristics of the backfill used in SSAR Revision 2, and if significant differences were found, the SSAR would be revised. The applicant noted only minor differences and revised the shear wave profiles in the SSAR accordingly. The applicant later included the RCTS test results in Revision 4 of the SSAR. Finally, the applicant stated that the results of Phase 2 of the test pad program would be used to develop procedures, in accordance with the applicant's quality control program, to ensure that the backfill was placed as specified by design requirements, to minimize variability of backfill, and to achieve acceptable results as required by the AP1000 standard plant design.

During the review of the applicant's response to RAI 2.5.4-7S, the staff considered the information provided and, in follow-up RAI 2.5.4-25S, asked the applicant to explain how the limitation of 25 percent fines was selected, how different the fines content could be to still be acceptable, and how the acceptable ranges of fines were defined for the Phase I Test Pad program and the production of backfill. RAI 2.5.4-28S, which is discussed later in this section, also relates to the two-phase test pad program for backfill. In response to RAI 2.5.4-25S, the applicant stated that, based on studies, tests and analyses of the structural backfill used for Units 1 and 2, the maximum percent fines to minimize potential settlement of the backfill was 25 percent. The applicant also developed the grain size distribution envelope that met the prescribed criteria outlined in the SSAR for the proposed materials parameters, such as percent fines, and included the results of the settlement calculations using the geotechnical properties of the backfill in Revision 4 of the SSAR.

The staff also reviewed the explanation of the percent fines for the backfill and concludes that the use of 25 percent fines will minimize settlement of the backfill at the site of VEGP Units 3 and 4, because the proposed backfill materials are very similar to those used for Units 1 and 2, in which the materials performed acceptably, and 25 percent fines is a widely-accepted industry value for sands and silty sands. Therefore, the staff considers RAI 2.5.4-25S resolved. The staff considered the detailed description provided in response to RAI 2.5.4-7S, including the details and implementation of the two-phase backfill test pad program and inclusion of the subsequent test results in Revision 4 of the SSAR. The applicant was able to demonstrate through the Phase 1 and 2 test pad programs that, by keeping the fines content to less than 25 percent, placing and compacting the proposed materials to at least 95 percent of the modified ASTM D 1557 standard, and performing laboratory testing to verify moisture content, that the grain size distribution of the sands and silty sands did not fall outside of the proposed grain size envelope; therefore, structural backfill materials would meet the requirement for minimum shear wave velocity. The applicant verified this information through in situ testing of the placed and compacted backfill materials, and shear wave velocity testing utilizing the SASW method at various times during the construction of the test pad and again upon completion of the test pad. These test results indicated that, by employing uniform and consistent soil placement and compaction methods, as demonstrated by the applicant during the Phase 2 portion of the test pad program, the final compacted materials will meet the requirement for shear wave velocity. Based on this additional information, in conjunction with the resolution of RAI 2.5.4-25S, the staff considers RAI 2.5.4-7S resolved.

Similar to the issue the staff addressed in RAI 2.5.4-7S, in RAI 2.5.4-14S, the staff requested that the applicant provide a discussion of how velocity testing of the compacted backfill would be performed and what assurances would be provided to ensure, in the completed condition, that the resultant velocities will meet target velocity requirements. In response to this RAI, the applicant referred to the velocity testing of compacted backfill that would be performed as part of the two-phase backfill test pad program described in the response to RAI 2.5.4-7S. The applicant also stated that "assuring the in-placed backfill meets the backfill design and construction requirements will provide the assurance that the shear wave velocity profile of the in-place backfill falls within an acceptable range consistent with the appropriate requirements stated in the Westinghouse DCD and the Vogtle site-specific analyses including the development of the GMRS and FIRS and the soil-structure interaction analyses."

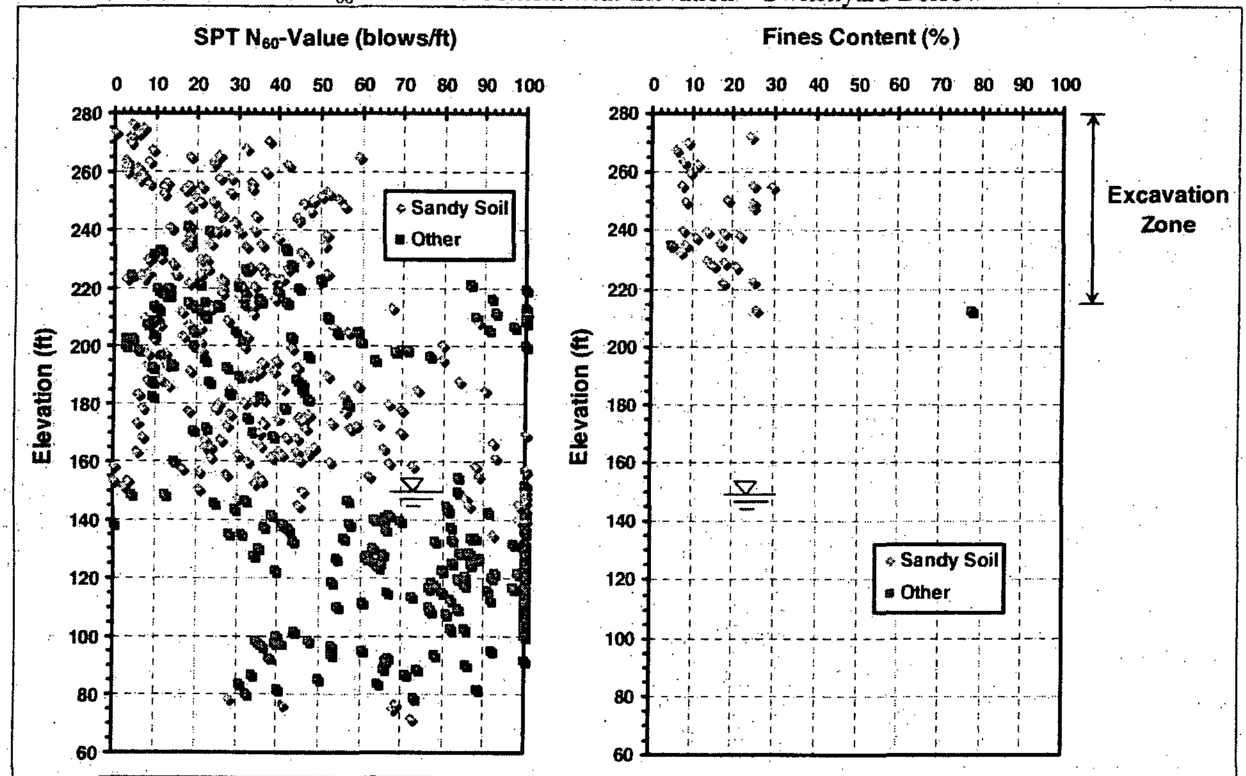
The staff reviewed the applicant's response and the backfill test program described in response to RAI 2.5.4-7S. The staff observed significant portions of both Phase 1 and 2 of the test pad program and actual in situ SASW shear wave velocity testing conducted on the compacted backfill and reviewed laboratory test results, as documented in the trip reports from the staff's December 2007 and July 2008 visits to the VEGP site (ML080110651 and ML082280539). Based on the staff's observation of the applicant's structural backfill placement and compaction methodologies, the applicant's SASW shear wave velocity testing and results, and the applicant's proposed soil specifications arrived at through laboratory testing, the staff concludes that the applicant has provided assurance that, during construction activities, if the applicant meets its soils specification and follows its backfill placement and compaction procedures as determined during the two-phase test pad program, the applicable soil density and shear wave velocity requirements will be met as specified in the proposed backfill ITAAC presented in SER Section 2.5.4.1.5. Based on the resolution of RAI 2.5.4-7S and the acceptable shear wave velocity results presented in the revised SSAR and reviewed by the staff, as well as the assurances that the soil density and shear wave velocity requirements will be met and confirmed through ITAAC, the staff concludes that the applicant supplied sufficient information to resolve RAI 2.5.4-14S. The staff's further evaluation of the proposed backfill ITAAC is provided below in this section of the SER.

<u>Volume and Sources of Backfill Materials</u>

In SSAR Section 2.5.4.5.3, the applicant stated that the volume of material to be excavated at the site was approximately 2.98 million (M) cubic meters (3.9M cubic yards), which will require 2.90 M cubic meters (3.8 M cubic yards) of structural backfill. The applicant further stated that only 30 percent of the excavated material will be available for reuse as structural backfill. In RAI 2.5.4-10S, the staff asked the applicant to perform additional investigations and testing at both horizontal and vertical intervals sufficient to determine the material variability of the remaining 70 percent of borrowed soil that will be used for backfill.

In response to this request, the applicant reiterated its previous conclusion that sufficient borrow material was identified at the site and that no additional investigations or testing was necessary. The applicant summarized the COL level investigation at the switchyard borrow area, including the results of 15 SPT borings that were drilled through these materials and five excavated test pits. Grain size, chemical tests, and compaction tests were part of the laboratory investigation described by the applicant for the borrow materials, an investigation which identified 1.9 million cubic meters (2.5 million cubic yards) of suitable borrow material. Again, the applicant referred to the backfill test pad program described in its response to RAI 2.5.4-7S for additional information on tests to be conducted on the borrow materials. Finally, the applicant described plans for investigations at an alternative borrow source, Borrow Area 4, which included four SPT borings and three test pits, and included preliminary comparison plats of N60 and Fines Content between the Switchyard Borrow area and Borrow Area 4 (SER Figures 2.5.4-8 and -9). However, in reviewing this response, the staff noted that survey results and/or figures were not provided to justify that sufficient material exists at the various borrow sources.

Plot of N_{60} and Fines Content with Elevation – Switchyard Borrow

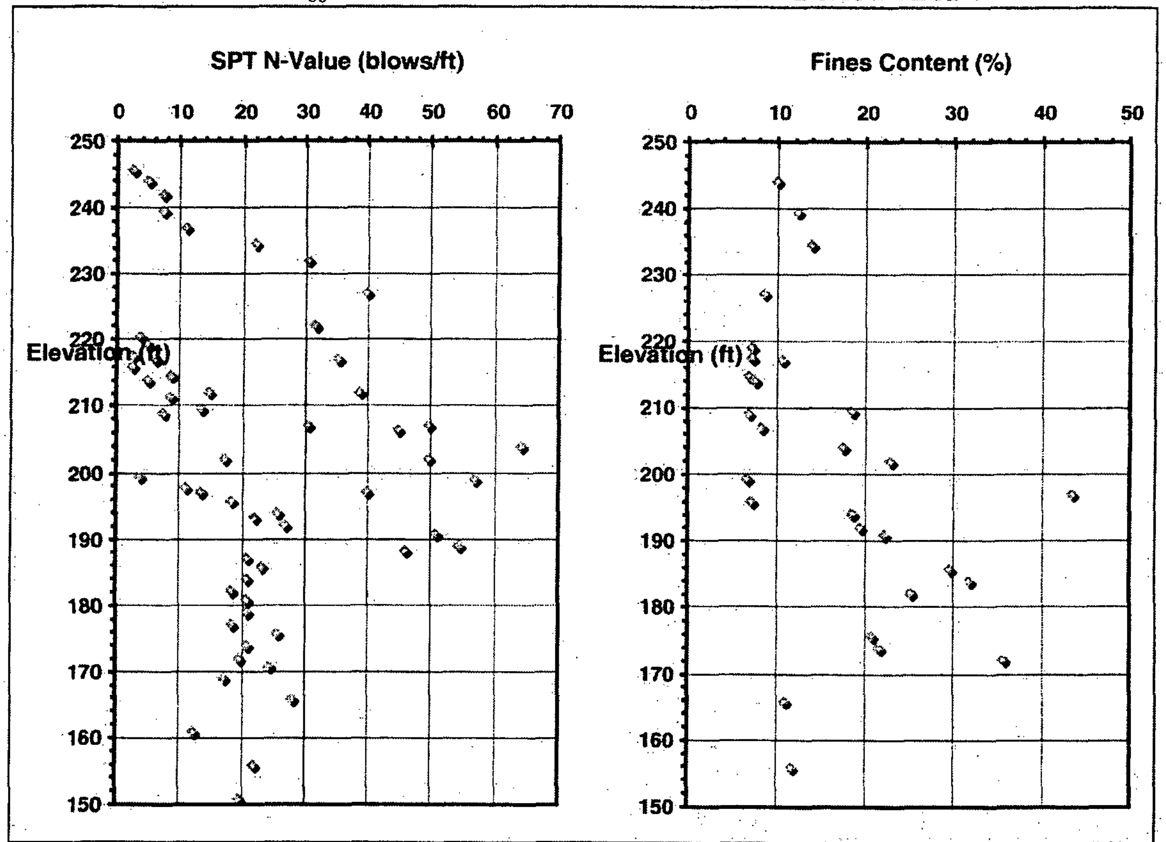


Presentation of test results for soils at the switchyard area: (a) SPT N60-value; (b) Fines content (MACTEC, 2007).

Note: It is assumed the measured SPT N = 100 blows/ft for refusal, and SPT N = 50 + 50/Δ/10 blows/ft for recorded blow counts of 50/(Δ ft). The energy-corrected SPT N_{60} does not exceed 100 blows/ft.

Figure 2.5.4-8 Plot of N_{60} and Fines Content with Elevation for Switchyard Borrow.

Plot of N_{60} and Fines Content with Elevation – Borrow Area 4



Presentation of test results for soils at Borrow Area 4: (a) SPT N60-value; (b) Fines content (MACTEC, 2007).

Note: It is assumed the measured SPT N = 100 blows/ft for refusal, and SPT N = 50 + 50/Δ/10 blows/ft for recorded blow counts of 50/(Δ ft). The energy-corrected SPT N_{60} does not exceed 100 blows/ft.

Figure 2.5.4-9 Plot of N_{60} and Fines Content with Elevation for Borrow Area 4

Accordingly, follow-up RAI 2.5.4-27S requested that the applicant provide clarification and justification of the quantity of suitable material in the switchyard area stockpiles, as well as describe how the percentage of reusable material excavated at the site was determined to be 30 percent. In response, the applicant stated that of the 2.75 million cubic meters (3.6 million cubic yards) of backfill required, two-thirds will come from the switchyard area and one-third will come from the power block excavations. The applicant also identified 1.5 million cubic meters (2.0 million cubic yards) of additional borrow material available at Borrow Area 4 and from the power block excavation. Details of the two major sources of backfill, the switchyard and power block areas, were provided by the applicant as follows:

> Switchyard Area: A detailed geotechnical investigation of the switchyard area was performed to confirm the suitability of the material in this area for use as backfill. As discussed in SSAR Section 2.5.4.5.4, the subsurface conditions in this area were explored with 15 SPT borings and five test pits during the COL investigation. Laboratory testing was conducted on representative samples to determine their engineering characteristics and to assess their suitability for use as backfill. These data, along with the backfill criteria as discussed in SSAR

Section 2.5.4.5.3, were used to estimate the horizontal and vertical extent of suitable borrow material in the switchyard area.

The field and laboratory test data from the switchyard borrow area borings were compiled onto logs of the borings. This information was used to develop subsurface profiles through the switchyard area and the volume of suitable borrow material was calculated using CADD. The surfaces of the suitable materials were projected onto a 3-D plot of the borrow area and the volume of suitable material was esstimated to be approximately 2,400,000 cubic yards. The material identified as suitable for use as backfill, identified as the Sands 1 Belt, extended down to the rough grade excavation surface.

The surfaces of the Sands 1 Belt of suitable borrow material are relatively horizontal (not undulating); therefore, segregation of the suitable material from unsuitable material is not expected to be an issue.

Power Block Area: Field and laboratory data were used to develop subsurface profiles in the power block excavation area. A total of 70 SPT borings in this area were considered, along with borings outside this footprint to add additional data and clarity to interpretation of the subsurface conditions.

Engineering judgment was used to correlate the layers of suitable borrow material identified in the borings for use in developing 3-D CADD surfaces. The materials identified by the applicant as the Sands 1, Sands 2, and Sands 3 layers constitute suitable borrow material, and the applicant calculated that the total quantity of borrow material in the excavation was approximately 2,000,000 cubic yards.

Prior to utilization of the subsurface data from the borings, approximately 30 percent of excavation materials were judged to be suitable material for backfill. However, analysis of the subsurface data indicated that over 50 percent of the material was suitable. For estimating purposes, the original conservative estimate of approximately 30 percent (1,200,000 cubic yards) has been maintained for use as backfill. The remaining 800,000 cubic yards of suitable borrow material will be segregated and stockpiled for potential future use.

The staff reviewed the information regarding the determination of borrow material availability at the VEGP site, including the site maps and figures provided to support the applicant's conclusion that sufficient borrow material exists in two areas at the site to be used as structural backfill, and the subsequent revised estimated quantities in SSAR Revision 5 that indicated that the applicant overestimated the quantity of available backfill from the borrow source located immediately north of the Units 3 and 4 power block areas. In SSAR Revision 5, the applicant indicated that rather than the original estimate of 2,400,000 cubic yards, approximately 1,600,000 cubic yards is available. However, the applicant also revised the estimated recovery of excavated material that could be designated for borrow material as an estimated 30-50 percent rather than its original conservative estimate of 30 percent of the material excavated from the power block areas as qualifying for reuse as Seismic Category I or II backfill. Based on its review, the staff concurs with the applicant that sufficient borrow material is available at the site based on the applicant's exploration through borings and laboratory testing to adequately determine the horizontal and vertical extent of acceptable materials, the applicant's use of computer-aided design and drafting (CADD) to calculate the volume of suitable materials, and the applicant's revised quantities of available borrow material which, although recalculated, indicated that sufficient quantities are still otherwise available. The estimated shortfall of 800,000 cubic yards in the switchyard area would be made up through enhanced recovery of the power block excavated materials and the materials at Borrow Area 4. As such, the staff concludes that RAI 2.5.4-27S is resolved. Given this resolution of RAI 2.5.4-27S, combined with the applicant's description of laboratory tests to determine the variability of borrow material at the site provided in response to RAI 2.5.4-10S, the staff agrees with the applicant's subsurface investigation and laboratory results and the method of calculating material quantities, and concludes that the applicant provided sufficient information to describe the variability and availability of borrow material at the site of VEGP Units 3 and 4. Therefore, the staff considers RAI 2.5.4-10S to be resolved. Additionally, the applicant provided sufficient information (as presented in SSAR Revision 5) regarding the available quantities of borrow material to demonstrate that the revised estimated quantities are equal to or greater than the estimated quantities of material required to backfill the VEGP Units 3 and 4 power block excavations.

In SSAR Subsection 2.5.4.5.3, the staff reviewed the information provided regarding the control of the uniformity of the backfill. Included in this review, the staff considered any plans regarding grain size tests, maximum dry density and optimum water content. In RAI 2.5.4-11S, the staff asked the applicant to ensure that the backfill underneath and to the sides of the nuclear island satisfies the AP1000 SSI criteria by providing a description of the program needed to assure the correlation of grain size distribution of the borrow material, and the corresponding maximum dry density and associated shear wave velocity is defined.

The applicant responded to RAI 2.5.4-11S by referring to the two-phase test pad backfill program that was described in the response to RAI 2.5.4-7S, which evaluated the range of acceptable backfill material properties at the site, including the maximum dry density and optimum water content for backfill, properties related to the grain size distribution, density, and shear wave velocity. According to the applicant, the test program would also specify the material property and field and laboratory testing criteria to ensure that the material would conform to the AP1000 standard plant criteria included in Revision 15 to the DCD.

The staff reviewed the backfill test program described in response to RAI 2.5.4-7S and referenced in response to RAI 2.5.4-11S, focusing its review on the correlation of grain size distribution, maximum dry density and shear wave velocity. The staff concluded, through review of the applicant's laboratory test results and results of the two-phased test pad program, that the applicant thoroughly characterized the material properties of the proposed structural backfill materials according to the guidance presented in RG 1.138. With the field density and shear wave velocity testing conducted during the two-phase test pad program, the applicant demonstrated that the soil placement and compaction methodology developed during the test pad program will ensure that soil specifications, resulting from its laboratory and field testing, will result in a uniformly placed and compacted backfill program that will meet the standard plant criteria in AP1000, as considered by the staff in review of the applicant's response and activities to address the RAI condition. Accordingly, the staff concludes that the applicant's plans to use the results of the test pad program to determine the final material properties and soil specifications for the compacted backfill are sufficient to ensure the appropriate correlation between material properties and soil specifications from the laboratory and field testing at the site, as well as to ensure conformance with the standard plant criteria. Therefore, the staff considers RAI 2.5.4-11S resolved.

Flowable Fill

In SSAR Subsection 2.5.4.5.3, the applicant indicated that a flowable fill would be used in place of compacted backfill to a very limited extent. In RAI 2.5.4-12S, the staff asked the applicant to specify: 1) the target properties of this material; 2) the required uniformity of the target properties; 3) the relationship of the flowable fill to the remainder of the compacted backfill; and 4) the potential extent of the material's use. The applicant provided a four-part response that addressed each portion of the RAI individually.

With respect to the target properties of the flowable fill, the applicant provided both the expected unit weight (1,922 to 2,242 kg/m^3 (120 to 140 pcf)) and the shear wave velocity, which would be determined empirically following the equation $V_s = (G_0/p)^{0.5}$ where Vs = shear wave velocity, p is soil density determined from unit weight of soil, and G_0 is shear modulus. The applicant then addressed the required uniformity of the properties, which it indicated would be adjusted to meet the strength requirements of the particular application. In an effort to maintain uniformity, the applicant described plans to produce the flowable fill in a ready-mixed concrete batch plant and transport the fill material using standard concrete mixing trucks to minimize the potential for component separation. The applicant also plans that most uses of the flowable fill at the site would be well removed from safety-related structures of the proposed units, but regardless of its eventual use, all flowable fill constituents, mix design, and placement will be controlled by widely-used industry specifications and procedures, the uses and locations of which will be documented on drawings. Regarding the relationship between flowable fill and the compacted backfill, the applicant stated that the flowable fill would have a higher load-bearing capacity, higher unconfined compressive strength, and greater bearing strength than the compacted backfill. Finally, the applicant addressed the potential extent of flowable fill at the VEGP Units 3 and 4 site, noting that flowable fill would be used where placement, compaction, and testing of compacted backfill was difficult. As was stated in response to the uniformity of the flowable fill, the applicant stated that flowable fill will be used at locations where the placement of soil backfill would be difficult or impractical to place and that those applications would be around piping, sewer and utility trenches, pipe bedding and slope stabilization well removed from the safety-related structures of the AP1000 units. Some potential locations where flowable fill may be used, as identified by the applicant, included the backfilling of sewer and utility trenches, road base, pipe bedding, and slope stabilization.

The staff considered the target properties and uniformity of the flowable fill, as well as the relationship to compacted backfill and potential extent of flowable fill at the site, provided in response to RAI 2.5.4-12S. The staff concludes that the applicant adequately addressed all aspects of the RAI by explaining the inclusion of target properties, its plans to maintain uniformity of fill, flowable fill's relationship to other backfill materials, and the extent of its usage at the site as described above; the staff therefore considers RAI 2.5.4-12S resolved, because while any use of flowable fill will be determined later, it will be controlled by specifications, procedures and drawings in accordance with the applicant's approved quality program.

Compaction of Backfill

SSAR Subsection 2.5.4.5.3 describes the classification of the backfill soils, including the percent compaction for each of the two categories. The applicant stated that the Seismic Category 1 backfill would be compacted to an average of 97 percent compaction, with no more than 10 percent of field compaction below 95 percent of the maximum dry density, while the Seismic Category 2 backfill would be compacted to an average of 93 percent, also with no more than

10 percent of field compaction below 95 percent. In RAI 2.5.4-8S, the staff asked the applicant to: a) correlate between density and velocity to ensure site characteristics and backfill requirements are met; b) justify how the 93 percent compaction minimum under Seismic Category I structures would not adversely impact soil density to the point the shear wave velocity falls below the minimum requirement; and c) justify how the average dry density of Seismic Category 2 backfill will meet the 95 percent compaction requirement that no more than 10 percent would fail below 95 percent.

The applicant provided a three-part response to RAI 2.5.4-8S, each part addressing one aspect of the RAI. First, the applicant responded to the correlation between velocity and backfill design and construction requirements. The applicant stated that this correlation was based on the two-phase backfill and test pad program described in response to RAI 2.5.4-7S. The program resulted in detailed design and construction parameters, including backfill selection criteria, placement techniques, compaction methods and requirements, and testing protocol, which the applicant then used to assure the expected shear wave velocity profile would be achieved. In response to the second part of the RAI, regarding minimum compaction requirements of the backfill, the applicant revised the backfill compaction specification to a single compaction requirement for both Seismic Category 1 and 2 backfill. The applicant stated that the criteria were revised to be 95 percent of the maximum dry density per the modified Proctor compaction standard as described and determined in accordance with ASTM standard D 1557, which should provide uniformity in placement and strength of the backfill. Finally, the applicant justified the average dry density of Seismic Category 2 backfill by stating that the same compaction requirements of Seismic Category 1 backfill would be applied to Seismic Category 2 and the 93 percent compaction requirement would be deleted; density for all backfill will be as required and verified by the backfill ITAAC presented in Section 2.5.4.1.5 of this SER and evaluated in the following section of this SER.

The staff focused its review of this additional information on the correlation of density and velocity, and the revision of the Seismic Category 2 backfill criteria to mirror that of Seismic Category 1. The staff noted that the change in the compaction and density requirements of Seismic Category 2 backfill to match the engineering criteria of Seismic Category 1 results in location being the only difference between Seismic Category 1 and 2 backfill. That is, Seismic Category 1 backfill will be beneath the Seismic Category 1 (safety-related) structures, and Seismic Category 2 backfill, although engineered to the same criteria as Seismic Category 1, will be beneath the Seismic Category 2 (non-safety-related) structures. The staff concludes that the applicant's plan to utilize the backfill and test pad program described in response to RAI 2.5.4-7S to correlate shear wave velocity to density is an acceptable plan to address the required correlation because shear wave velocity and density are functions of each other, i.e., the denser a material is generally, the higher the shear wave velocity. Furthermore, the staff concludes that the revision of Seismic Category 2 requirements to reflect the compaction requirements of Seismic Category 1 backfill is sufficient to address the compaction concerns raised for Seismic Category 2 backfill because both materials will be placed and compacted to an industry accepted minimum density in accordance with the backfill ITAAC evaluated in the following section of this SER. Based on these conclusions, the staff considers RAI 2.5.4-8S resolved.

SSAR Subsection 2.5.4.5.3 states that the two categories of backfill will be compacted to the Proctor density requirements given based on tests performed at a density of one test per 929 square meters (10,000 square feet). In RAI 2.5.4-9S, the staff requested that the applicant provide the basis for using a testing density of one test per 929 square meters (10,000 square ft) of lift and to explain how this distribution will provide assurance of adequate uniformity of shear wave velocity as used in the SSI analyses of the AP1000 standard design. The applicant responded by describing an evaluation that, with respect to justifying the testing frequency for performing field density testing of engineered backfill, would use the recommendations of ASME NQA-1-2004. The applicant revised the ESP application to conform to the testing frequency recommended by the aforementioned ASME code. Once again, the applicant referenced the backfill testing program described in response to RAI 2.5.4-7S and stated that the use of the ASME code for quality assurance requirements would provide an acceptable and consistent industry testing frequency for the development of the final construction specifications. The staff considers the applicant's utilization of ASME NQA-1-2004 as the recommended testing frequency for mass earthwork at nuclear facilities to be a suitable testing frequency for the density tests to assure uniformity of shear wave velocity as applied to the SSI analyses of the AP1000 standard design. In follow up RAI 2.5.4-26S, the staff requested that the applicant provide further clarification of how the ASME standard referenced in the response to RAI 2.5.4-9S will be implemented, and to provide justification of the testing density and how the applicant will ensure adequate uniformity of shear wave velocity.

In response to this supplemental request, the applicant stated that both the 152 cubic meter (2000 cubic yard) criteria and lift criteria will be applied and that the backfill testing program will provide the necessary assurance that the backfill will achieve the required shear wave velocity at the nuclear island foundation. The applicant further stated that the testing density for mass earthwork was consistent with the guidance of NRC Inspection Manual, Inspection Procedure 88131, which references the test frequency (testing density) of ASME NQA-1 initially cited by the applicant. The applicant then described the testing frequency in greater detail, stating that "early during placement of the production backfill, the frequency of field density testing is expected to exceed the minimum frequency until sufficient data are developed to document that the required degree of compaction is consistently being achieved, based on field engineering judgment." The applicant also made comparisons to the frequency of testing for the MOX facility at the Savannah River Site and the National Enrichment Facility in New Mexico. The applicant concluded that a higher frequency of in-place testing was required depending on the size of the area; six nuclear tests per lift for areas between 1858 and 5574 m^2 (20,000 and 60,000 ft^2), four tests per lift for areas between 929 and 1858 m^2 (10,000 and 20,000 ft^2), and three tests per lift for smaller areas.

During the review of RAI 2.5.4-26S, the staff focused its review on the applicability of ASME NQA-1 to nuclear power plant sites. The staff agrees with the use of the criteria from the inspection manual as it specifies testing frequencies consistent with those used successfully at other nuclear facilities. Based on the applicant's reliance on the code in question in the NRC Inspection Manual, Inspection Procedure 88131, as well as the comparison to other facilities handling special nuclear material, and the applicant's proposed backfill ITAAC, evaluated in the following section of this SER, whereby it will prepare final reports documenting the minimum 95 percent compaction and shear wave velocity equal to or greater than 304.8 m/s (1,000 fps) requirements, the staff concludes that the applicant adequately justified the testing density used to resolve RAI 2.5.4-26S. With the resolution of RAI 2.5.4-26S, the staff also considers RAI 2.5.4-9S resolved.

Backfill ITAAC, Test Pad Program and MSE

While reviewing the excavation and backfill section for the VEGP Units 3 and 4 site, the staff also considered the applicant's discussions of its proposed ITAAC for backfill soil, which is provided in table 2.5.4-2 from Section 2.5.4.1.5 of this SER.

In RAI 2.5.4-15S, the staff asked the applicant to address the following four issues: 1) include the requirement of minimum shear wave velocity of 304 m/sec (1,000 ft/sec) in the Design Requirement; 2) provide a detailed description of the testing program for the placement of the backfill materials as part of the inspections and tests; 3) describe the report that is referenced in the Acceptance Criteria; and 4) include the minimum shear wave velocity of 304 m/sec (1,000 ft/sec) in the Acceptance Criteria.

In its response, the applicant addressed all four issues simultaneously by stating that SSAR Subsection 2.5.4.5.3.2 would be updated to provide additional discussion of the design of engineered backfill. In Revision 4, the applicant revised the SSAR to include a description of the test pad program and RCTS testing to provide assurances that the minimum shear wave velocity would be met. With respect to the backfill ITAAC, the application stated that the conformance to shear wave velocity would be demonstrated through the test pad program and not through the ITAAC process. In reviewing this information, the staff determined that it was not inherently clear whether the normal variability would be sufficiently evaluated without adequate shear wave velocity testing. Accordingly, in follow-up RAI 2.5.4-28S, the staff asked the applicant to justify the adequacy of the production backfill test program to estimate the average velocities of placed soils and their variability.

The applicant replied by referring the staff to the response given for RAI 2.5.4-19S and to a structural backfill evaluation report it submitted with the RAI responses. In RAI 2.5.4-19S, the staff asked the applicant to address two issues related to MSE wall backfill placement and footing construction. On the first issue, the staff asked the applicant to provide information on how the procedures modified from Phase I of the test pad program and revised compaction procedures from Phase II would be developed, to indicate whether a section of the MSE wall would be included in Phase II, and if so, to explain how compaction around the wall would be accomplished. The staff also requested confirmation from the applicant that the procedures developed at the end of Phase II would be used during the placement of production backfill. Finally, the staff asked for information on how the soil wave velocity testing would be accomplished during the placement of the production backfill in and around the final nuclear island configuration.

In response to the first issue, the applicant stated that Phase II of the test pad program would focus on the establishment of placement procedures and equipment to be combined with the Phase I results to develop backfill specifications and procedures, including frequency and type of quality control testing. Based on preliminary testing as part of Phase I of the test pad program, the applicant concluded that shear wave velocity testing during production fill placement would not be necessary since the results of the test pad program indicated that proper controls on backfill gradation and compaction would result in a homogenous fill with minimum shear wave velocity meeting the criteria of the AP1000 DCD. The staff reviewed this information, particularly the conclusion that shear wave velocity testing would not be needed during placement of fill because the applicant intends to use its specific backfill placement and compaction procedures developed during the test pad program, in conjunction with its laboratory testing program, to control the structural backfill gradation and compaction density to produce a homogeneous soil backfill foundation that will result in a minimum shear wave velocity at the foundation level of the NI that meets the AP1000 DCD criteria. Thus, because the applicant will verify and document the shear wave velocity as required by ITAAC, the staff concludes that the applicant provided sufficient information to resolve the first issue of RAI 2.5.4-19S.

On the second issue of RAI 2.5.4-19S, the staff requested that the applicant describe in detail the concrete footer that will be installed at the start of construction of the MSE wall. The staff noted that this description should include such parameters as concrete mix design, and reinforcing steel sizes so that the staff could determine the adequacy of the design. The applicant responded that the MSE wall is an internally stabilized system of panels that would act as forms for pouring the nuclear island structures. In order for the panels to be erected, the applicant explained that a thin leveling pad, or footer, is needed to provide a stable working surface from which the panels can be erected. The applicant stated the specifications of the footer, including the 28-day concrete strength, which would be 17 MPa (2,500 psi) or above, and the dead-load pressure of the wall (less than 275 kPa (40 psi)). The applicant further stated that reinforcing steel will not be needed since the pad will be confined by its neighboring elements and shrinkage will be negligible. Finally, the applicant provided the profile dimensions of the footer (15.24 cm wide by 30.48 cm deep (12 in by 6 in)), stated the length to be equal to that of the MSE wall, and specified that the concrete mix would be designed in accordance with the governing ACI code. The staff reviewed these specifications, including the use of the governing ACI code for the concrete mix and concludes that the applicant provided an acceptable level of detail for the staff to determine that the design of the MSE wall footer is adequate because 1) the purpose of the concrete footer is to provide a clean smooth working surface for construction of the MSE wall and as such has no bearing capacity requirements, 2) the applicant stated that the design of the MSE wall considers that the horizontal soil reinforcements at or most near to the wall leveling pad (footer) have full effective pullout length so that the footer takes no or insignificant tension force when lateral pressure is exerted on the MSE wall system, 3) the 28 day compressive strength for the cast in place concrete footer will be a minimum of 17 MPa (2,500 psi) or greater and the dead load pressure exerted by the wall system will be at or less than 275 kPa (40 psi), 4) and the concrete will be designed in accordance with ACI-318, which is the governing code used for all nuclear plant construction, and finally 5) the concrete footer will be allowed to cure to meet its design strength prior to the placement of MSE wall sections. Based on the above, the staff considers the second issue of RAI 2.5.4-19S to be resolved.

With the resolution of these two issues, which relate to geotechnical engineering aspects of the VEGP LWA request, the staff considers the geotechnical engineering aspects of RAI 2.5.4-19S to be resolved. Based on the resolution of these aspects of RAI 2.5.4-19S, which is referenced by RAI 2.5.4-28S, the staff also considers RAI 2.5.4-28S resolved based on the resolution of issue 1 for RAI 2.5.4-19S. Finally, since the resolution of RAI 2.5.4-15S was contingent upon the resolution of RAI 2.5.4-28S, the staff also considers RAI 2.5.4-15S to be resolved as well because the applicant included in the ITAAC for shear wave velocity all four of the items requested by the staff in RAI 2.5.4-15S.

SSAR Subsection 2.5.4.5.5 discusses the quality control program and ITAAC associated with the excavation and backfill at the VEGP Units 3 and 4 site. The applicant stated that a MSE will be used as a form against which the nuclear island structures would be poured; however, it was not obvious to the staff that the backfill immediately behind the MSE wall would be compacted to the same density criteria of the remainder of the fill. Accordingly, in RAI 2.5.4-13S, the staff asked the applicant to provide the procedures for compaction of the backfill immediately adjacent to the MSE wall.

The applicant responded to RAI 2.5.4-13S by stating that with the exception of within five feet of the panels, the backfill will be compacted using a large smooth drum vibratory roller. For the five feet immediately behind the panels of the MSE, the applicant planned to use small single or double-drum vibratory walk-behind rollers, walk behind vibratory plate compactors, and jumping jack compactors to achieve the requisite compaction. The applicant concluded that using these methods, the compacted fill would meet or exceed the established specifications. The staff reviewed this response, including the numerous tools which might be used to compact the fill adjacent to the MSE wall, and concludes that the applicant provided sufficient information in its response to resolve RAI 2.5.4-13S. The staff further based its conclusion on results from the Phase 2 of the test pad program, portions of which were observed by the staff and audited by Region II staff during the December 2007 and July 2008 visits to the VEGP site as documented in the staff-written trip reports (ML080110651 and ML082280539). During these trips, the staff observed the actual placement methodologies and subsequent field and laboratory test results for structural backfill materials placed adjacent to test portions of constructed MSE wall system. Therefore, the staff concludes that the applicant provided sufficient evidence to resolve RAI 2.5.4-13S.

The applicant described the extent of the excavations, planned backfills, and described its construction slopes, including providing adequate plans and profiles and boring logs supported by laboratory testing following the guidelines of RG 1.138. The applicant also described why and how the Upper Sand Stratum will be removed and replaced with engineered structural backfill, the specifications and locations of which the applicant adequately described in detail as discussed in this section. The applicant also established the design of its Seismic Category 1 and 2 structural backfill materials through analysis and testing, and provided sufficient test results in the form of laboratory test result summaries that adequately characterized the properties of the materials and provided sufficient information to allow the staff to determine material acceptability. The applicant conducted exploration and testing of potential borrow sources to identify backfill material sources, from which it was able to identify and verify that sufficient backfill material was available at the site. As discussed above, the applicant also proposed acceptable ITAAC for the structural backfill compaction density and shear wave velocity requirements and to provide documented evidence that testing is sufficient to verify that the AP1000 DCD requirements have been met. An associated ITAAC, concerning the applicant's approach to securing the waterproof membrane to the mudmat and placing the membrane against the vertical MSE wall, is evaluated in Section 3.8.5 of this SER. Finally, the applicant provided details for the MSE walls that will permit backfilling of the excavations up to plant grade.

Based on the information and findings above, including the resolution of RAIs and Open Items, the staff concludes that the discussion of the excavation and backfill plans at the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable, and that the proposed Backfill ITAAC are appropriate. The staff concludes that the geotechnical parameters of minimum soil backfill density of 95 percent as determined by ASTM D 1557, and minimum shear wave velocity of 1000 fps at the bottom of the NI foundation are acceptable criteria because 1) a minimum compaction of 95 percent is the accepted industry standard for nuclear construction, and 2) the minimum shear wave velocity of 1000 fps is as required by the AP1000 DCD. The staff agrees with the applicant's density testing frequency because it will use the ASME NQA-1 industry standard and because the ITAAC will require the applicant's shear wave velocity testing at the bottom of the nuclear island foundation as required by the AP1000 DCD.

2.5.4.3.6 Groundwater Conditions

In SSAR Section 2.5.4.6, the applicant provided some basic groundwater conditions based on the water well observations and a summary of the dewatering plan implemented for VEGP Units 1 and 2. The staff determined that this information is necessary to understand the ground water conditions and potential dewatering plan at the ESP site.

The staff's evaluation of the information provided in support of the ESP application is as follows:

The staff reviewed the groundwater conditions described by the applicant in SSAR Section 2.5.4.6.1. The staff's evaluation of this information can be found in Section 2.4.12 of this SER.

The staff's evaluation of the information provided in support of the LWA request is as follows:

In RAI 2.5.4-6, the staff asked the applicant to explain the dewatering procedures it will use for the construction of the new units. In its response to this RAI, the applicant stated that it would implement the same dewatering program as that developed for the VEGP Units 1 and 2 but with some deviations. The applicant considered the dewatering program deployed at Units 1 and 2 to be successful, and subsurface conditions at the ESP site and at Units 1 and 2 are similar.

After reviewing the applicant's response, the staff concluded that, since the applicant had not yet determined the reactors' location within the ESP site and did not have a site-specific dewatering program, the staff could not evaluate the groundwater conditions as they affect the loading and stability of foundation materials. The staff was also unable to assess the applicant's dewatering plans during construction as well as ground water control throughout the life of the plant. Because the plant specific dewatering program could not be planned until the reactor location is decided, the staff considered that this design-related information was not necessary to determine whether 10 CFR Part 100 is satisfied. Therefore, in the SER with Open Items, the staff identified the need for the submission of groundwater condition evaluations and a detailed dewatering plan during the COL stage as COL Action Item 2.5-7.

However, in the revised SSAR, the applicant described plans for temporary dewatering of the site during the excavation and construction of VEGP Units 3 and 4. These plans are summarized in Section 2.5.4.2.6 of this SER and include the sump-pumping of ditches to remove groundwater during construction at the site. The staff reviewed this information, especially the dewatering plans and groundwater characterization through observation wells, and concludes that due to this additional information, COL Action Item 2.5-7 is no longer necessary.

The staff considered the following information acceptable to meet the criteria of RG 1.132 and 10 CFR Part 100.23: 1) as the staff discusses in SER Section 2.4.12, groundwater conditions at the site were discussed in sufficient detail in SSAR Section 2.4.12, 2) the applicant installed fifteen observation wells at the site for the ESP subsurface investigation and also used an additional 22 existing wells for the groundwater monitoring program, 3) the applicant had a representative number of wells in both the unconfined water table aquifer in the Upper Sand Stratum and in the confined Tertiary aquifer in the Lower Sand Stratum, and concluded that the Blue Bluff Marl is an aquiclude that separates the unconfined WT aquifer and the confined Tertiary aquifer, 4) the applicant was able to determine the groundwater levels in the wells and determine the hydraulic conductivity (k) values, through "slug" testing, 5) the applicant determined that some temporary dewatering of excavations will be required during construction and that, due to the low permeability of the Upper Sand Stratum and Blue Bluff Marl, sumps and pumps would be sufficient for successful construction dewatering, and 6) the applicant determined that groundwater levels for VEGP Units 3 and 4 correspond to design levels for the existing Units 1 and 2. The staff also concludes that the applicant's use of a liner in the sumps and ditches is acceptable, even though the liner material was not specified, since the type of liner material is peripheral to the adequate performance of the liner except in special applications, such as hazmat, which are not involved in the proposed construction dewatering.

The staff considered the criteria of RG 1.132 and 10 CFR Part 100.23 in its review of SSAR Section 2.5.4.6 and, for the above reasons, concludes that the applicant's assessment of groundwater conditions at the site is acceptable.

2.5.4.3.7 Response of Soil and Rock to Dynamic Loading

The staff's review of the information provided in support of the ESP application is as follows::

The staff reviewed SSAR Section 2.5.4.7, focusing on how the applicant developed the base shear wave velocity profile and modeled soil modulus reduction and damping with respect to cyclic shear strain. The applicant derived shear modulus for the soil strata from the relationship relating the unit weight to shear wave velocity, as well as the dynamic shear modulus reduction and damping ratio curves derived from EPRI (EPRI TR-102293 1993). The applicant used the SHAKE2000 (Bechtel 2000) computer program to evaluate the site dynamic responses.

The applicant derived ESP soil shear modulus degradation and damping curves from the curves developed by EPRI (1993). In RAI 2.5.4-5, the staff asked the applicant to justify its application of the EPRI curves to fine-grained soils. In response, the applicant stated that EPRI (1993) developed degradation curves for soils from gravels to high plasticity clays, and thus it was appropriate to apply the curves to fine-grained soils. EPRI (1993) presented fine-grained soils in Figures 7.A-16 (shear modulus reduction curves) and 7.A-17 (damping ratio curves) in terms of soil plasticity and required the use of the plasticity index. The applicant referred the staff to its response to RAI 2.5.4-17 for more details on how it derived the degradation curves from the EPRI (1993) curves. The applicant further indicated that the soil degradation relationships for fine-grained soil (and coarse-grained soils) used in the SSAR would be verified by laboratory testing during the COL subsurface investigation. Figures 2.5.4-6 and -7 of this SER present the site-specific shear modulus and damping ratio curves, respectively.

After reviewing the applicant's response and references, the staff determined that although Section 7A.6 of the EPRI (1993) report recommends the modulus degradation and hysteretic damping strain-dependent curves for generic CEUS sites, these curves are intended for gravelly sands to low plasticity silty or sandy clays and should not be applied to either very gravelly or very clayey deposits. The curves presented in the report for silts and clays of high plasticity are significantly different from those for sandy soils. In its response to RAI 2.5.4-10, however, the applicant indicated that the BBM "is described as hard, slightly sandy, cemented calcareous clay, and with less than 50 [percent] fine material," which was different from the type of materials for which the curves were intended. Therefore, the staff concluded that the applicant did not adequately explain why it was appropriate to apply those relationships to the silt and clay soils at the ESP site. The report further stated that, while the generic curves are appropriate for preliminary site studies, one should use site-specific data for final evaluations. In conclusion, the staff agreed with the applicant that it needed to verify the soil modulus degradation and damping curves. However, the staff concluded that this verification should not wait until the COL stage. Without site-specific soil modulus degradation and damping curves, the determination of site-specific GMRS (SSE) is inadequate. In the SER with Open Items, the need to provide site-specific soil degradation and damping ratio curves for the site-specific soil amplification calculation discussed in SER Section 2.5.2 was identified as Open Item 2.5-19.

The applicant responded to Open Item 2.5-19 by stating that site-specific soil degradation and damping ratio curves were not developed as part of the ESP investigations at the VEGP Units 3 and 4 site. The applicant also referenced its responses to RAIs 2.5.4-5 and 2.5.4-17 with respect to the applicability of the generic EPRI curves to the materials at the VEGP site, stating that in addition to the EPRI curves, soil degradation and damping ratio curves from the adjacent SRS were also included in the analysis. Finally, the applicant stated that the data determined from the EPRI and SRS curves would be confirmed after RCTS testing was completed during the COL investigation. The staff considered this justification, including with respect to the assertion that the use of both generic and adjacent curves was sufficient, as well as the applicant's plans to confirm these conclusions during the COL phase of site investigations. Because the applicant confirmed the EPRI and SRS curves through RCTS testing performed as part of the COL investigations and included that information in the revised SSAR, the staff concludes that the applicant provided sufficient information to satisfy Open Item 2.5-19. Therefore, the staff considers Open Item 2.5-19 closed, which also resolves RAI 2.5.4-17 since the response provides a suitable description of how the soil degradation and damping ratio curves were developed.

The SSAR stated that the applicant used values of shear modulus and damping ratio to extend the EPRI curves beyond the 1 to 3.3 percent strain level. In RAI 2.5.4-13, the staff asked the applicant to justify how it extended the values beyond the 1 percent strain level and to provide a complete description and supporting data. In its response, the applicant stated that, even though it extended the EPRI curves beyond the 1 percent strain level, the maximum strains calculated during the site amplification analyses remained below 1 percent. But the applicant then stated that SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1, and 2.5.4.7.2.2 would be revised, along with associated tables and figures, to show the degradation curves only at a 1 percent or less cyclic shear strain. In light of the applicant's commitment to revise the shear modulus and damping ratio curves back to a 1 percent strain level without extrapolation, the staff concluded that this RAI could not be resolved until the revised SSAR sections were submitted for review. This was identified as Open Item 2.5-20 in the SER with Open Items.

In response to Open Item 2.5-20, the applicant updated the appropriate SSAR sections. The staff reviewed the revised figures and tables, and, based on the revisions to the SSAR and included tables and figures, which reflect the revised degradation curves at 1 percent cyclic shear strain, the staff concludes that the applicant provided sufficient data in the revised tables and figures of SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1, and 2.5.4.7.2.2 to close Open Item 2.5-20. The closure of Open Item 2.5-20 also resolves RAI 2.5.4-13 since it provides the necessary updating of figures and tables referencing the excess percent strain that was previously modeled.

In RAI 2.5.4-17, the staff asked the applicant to provide a complete description, including sample calculations, to show how it derived the shear modulus reduction and damping curves and how it incorporated uncertainties in the site characteristics into the curves' development. The applicant explained in its response that it used the shear wave velocity to calculate the low strain dynamic shear modulus (Gmax) only. The EPRI (1993) curves simply showed the ratio G/Gmax versus cyclic shear strain, regardless of the initial value of Gmax. The shear modulus reduction and damping ratio curves for cohesionless materials were based on confining pressure at depth, or simply depth, but were based on the plasticity index for cohesive material like BBM. The applicant then described how the shear modulus reduction and damping ratio curves were derived from the EPRI (1993) curves for each layer included in the base shear wave velocity profile. The applicant also stated that, "shear modulus reduction and damping curves will be obtained using undisturbed samples collected during the COL subsurface investigation."

In addressing how uncertainties were incorporated, the applicant stated that EPRI shear modulus reduction curves were extended from the strain level of 1 percent to 3 percent and uncertainties were incorporated in the site parameters during the randomization process. SER Figures 2.5.4-6 and 2.5.4-7 show shear modulus reduction and damping ratio curves, respectively, for each layer in the profile. The applicant randomized the shear modulus reduction and damping ratios at one strain level using log-normal distributions with median values given by the corresponding base-case curves and logarithmic standard deviations taken from the statistical summaries obtained by Costantino (1997) for natural soils. For the engineered backfill, the applicant reduced these standard deviations by one-third to account for a more homogeneous soil mass. The applicant also used a hyperbolic parametric form to generate the shear modulus reduction and damping ratios at other strains from the randomized values obtained above. The applicant stated that this approach produced realistic curves with logarithmic standard deviations that approximate the Costantino (1997) values over a wide range of strains. The applicant assumed that the normal random variables associated with the log-normal shear modulus reduction and damping ratios had a correlation coefficient of -0.75.

After reviewing the responses from the applicant, the staff reached the following conclusions:

1. Although the EPRI (1993) curves were up to the 1 percent strain level, the applicant did not provide information on the strain levels associated with the 10^{-4}, 10^{-5}, and 10-6 uniform hazard response spectra (UHRS) at the bedrock in the site response analyses and did not indicate whether the laboratory data developed during the SRS testing program carried to those levels of strain.

2. The adequacy of the equivalent-linear approximations for site response deteriorates as strain levels exceed about 0.5 percent effective shear strain. The applicant did not justify the applicability of the equivalent-linear method used in the SHAKE2000 model analysis if the strain levels were to exceed 1 percent.

3. In its response to RAI 2.5.4-13, the applicant indicated that it would revise the 3.3 percent strain level extrapolation back to 1 percent for the EPRI (1993) modulus reduction and damping curves; however, its response to this RAI indicated otherwise.

4. The applicant needed to demonstrate that it can confidently obtain undisturbed samples for deeper depths (e.g., in the Blue Bluff Marl and lower sands of the Congaree and Lower Snapp formations) for use in site response and SSI studies.

5. The applicant also needed to test disturbed samples of the compacted fill material to estimate appropriate modulus reduction and damping properties for the SSI analysis.

6. Other RAI responses indicated that the applicant used both SRS and EPRI (1993) models in the site response analyses and weighted them equally. Considering that site-specific data are almost always desired over generic models, the applicant needed to evaluate the strain level difference in the surface UHRS at different exceedance levels that result from application of these different models and to justify whether the equal-weighting approach is appropriate.

Based on its review of SSAR Section 2.5.4.7, the related references, and the applicant's responses to RAIs described above, the staff concluded that the applicant did not have sufficient site-specific laboratory data to support the determination of the site response to dynamic loading. Although the applicant committed to provide the site-specific modulus reduction and damping curves during the COL stage, the staff determined that this issue, raised with a different perspective in RAI 2.5.4-13, needed to be resolved in the ESP application to provide site-specific shear modulus reduction and damping curves for the site SSE determination. Therefore, as stated earlier, resolving this issue was designated as Open Item 2.5-19 in the SER with Open Items; and the evaluation and closure of that Open Item was discussed in more detail above.

The staff's review of the information provided in support of the LWA request is as follows:

In supplemental RAI 2.5.4-16S, the staff asked the applicant to provide further discussions on the comparison of the EPRI 1993 soil degradation models to the SRS models, identify which model is more appropriate for the VEGP site, and explain how significant the models are to both site response and soil structure interaction (SSI) analyses. In its response, the applicant referenced its response to RAI 2.5.4-17 described above. The applicant also stated that both the EPRI and SRS curves were used as inputs into the SHAKE analysis at the VEGP ESP site. Also in the response, the applicant provided additional figures demonstrating the relationship between the EPRI-derived curves and those derived from the SRS data, selecting the SRS curves based on their stratigraphic relationship to the ESP site. Finally, the applicant stated the results of RCTS testing were used to develop site-specific data as well as confirm the derived curves. The staff agrees with the applicant that the SRS curves are more appropriate for the VEGP Units 3 and 4 site since the SRS curves represent a stratigraphy similar to that of the VEGP site. Based on the supplied response, especially the figures provided to compare the EPRI-derived and SRS curves and the selection of the SRS curves based on the stratigraphic correlation to the VEGP site, the staff concludes that the applicant provided the information to resolve RAI 2.5.4-16S.

In supplemental RAI 2.5E-2, the staff asked the applicant to provide a description and discussion of the effect of backfill adjacent to the MSE walls on SSI analysis results, due to the fill placement and compaction techniques within the zone immediately behind the wall. In its response, the applicant stated that to investigate the effect of differential compaction within 5 ft of the wall face zone, it used a reduced velocity profile for the full height of the wall (referred to as MSE best estimate) in a soil structure interaction analysis and stated that the results showed that there was no difference in the seismic structural responses from the potentially reduced shear wave velocity profile behind the MSE wall. The applicant performed the sensitivity analysis using two-dimensional (2D) seismic soil structure interaction SASSI models and presented the results of the sensitivity study of backfill behind the MSE wall; the applicant included the figures showing the FRS comparisons between the Vogtle 2D model with the reduced shear wave velocity directly behind the MSE wall and the Vogtle ESP best estimate (BE) 2D SASSI model at Nodes shown in SSAR Table 5.1-1. The figures also showed the AP1000 SASSI 2D SSI FRS envelope. Based on the applicant's response, the staff concludes that the applicant provided sufficient information to resolve RAI 2.5E-2 because the FRS for the model that included the lower bound (LB) backfill shear wave velocity (V_s) directly behind the MSE wall was almost identical to the FRS of the same model including no reduction in V_s directly behind the wall; therefore, the potentially reduced shear wave velocity of the backfill directly behind the MSE wall would not affect the nuclear island building responses because, as stated above, the shear wave velocities are almost identical.

Based on its review of SSAR Section 2.5.4.7 and the resolution of RAIs and closure of Open Items described above, the staff concludes that the applicant adequately determined the response of the soil and rock underlying the site of VEGP Units 3 and 4 to dynamic loading and that this determination is acceptable for both the ESP application and the LWA request.

2.5.4.3.8 Liquefaction Potential

In its review of SSAR Section 2.5.4.8, the staff evaluated the applicant's description of liquefaction potential and plans for future liquefaction studies at the ESP site. The staff's review focused on the applicant's conclusion that, based on the previous investigations and excavation completed for the VEGP Units 1 and 2, liquefaction would occur only in the Upper Sand Stratum.

The staff's evaluation of the information provided in support of the ESP application is as follows:

In RAI 2.5.4-14, the staff asked the applicant to justify why liquefaction analyses were not performed on the Blue Bluff Marl (BBM), since the unit has a relatively high variable fines content (24–77 percent) and saturation level (14–67 percent), and a potentially high ground motion level at the site. In response, the applicant first discussed the liquefaction potential for the BBM (Lisbon Formation) based on the material and age. The applicant then examined the field strength and shear wave velocity results to determine whether the marl would liquefy based on these results.

The applicant stated that, although the BBM frequently contained less than 50 percent of fine material, it had the appearance and characteristics of a calcareous claystone or siltstone and was a hard, slightly sandy, cemented calcareous clay. The design undrained shear strength of the marl was set as 478 kPa (10,000 psf) with a preconsolidation pressure as high as 3,831 kPa (80,000 psf), indicative of a highly overconsolidated material. Although the marl would be below the groundwater table, its compressed structure would prevent it from having the free water characteristic of a saturated granular material. Based on these characteristics, the applicant concluded that the BBM is not a material with liquefaction potential, regardless of the ground motion level. The applicant further indicated that liquefaction resistance would increase markedly with geologic age. Based on Youd et al. (2001), Pleistocene (1.8 mya to 10,000 year) sediments were more resistant, while pre-Pleistocene (older than 1.8 mya) sediments were generally immune to liquefaction. The BBM's age is late middle Eocene (40 to 41 million years old), much older than Pleistocene.

The applicant also stated that, based on Youd et al. (2001), there were thresholds for the N-values, tip resistance, and shear wave velocity beyond which the material was considered nonliquefiable (e.g., a sand with 35 percent or more fines or a soil with a corrected N-value over about 21 is not liquefiable). According to the applicant, of the 58 N-values measured in the marl for the ESP investigation, 5 were below 50, ranging from 27 to 46. Thus, if the marl were a potentially liquefiable material, a liquefaction analysis would be run for these five samples. An initial analysis of these five samples showed factor-of-safety values in excess of the accepted 1.35 value in all cases. All of the CPTs that penetrated into the marl had refusal at or near the top of the stratum; therefore, the applicant concluded that the measured tip resistance showed the material to be nonliquefiable. The applicant also stated that the typical shear wave velocities in the marl ranged from 426 to 807 m/s (1,400 to 2,650 ft/s) but dropped to 301 to 512 m/s (990 to 1,680 ft/s) when corrected for overburden. According to the applicant, Youd et al. (2001) indicated that, for a sand with 35 percent or more fines, soils with a corrected shear wave velocity in excess of about 190.5 m/s (625 ft/s) were nonliquefiable.

The applicant stated that, based on material and age, the BBM does not have the potential to liquefy, and that the CPTs, as well as shear wave velocities, consistently indicated the marl is nonliquefiable material. In addition, the applicant indicated that over 90 percent of the

SPT N-values indicated the marl as nonliquefiable material and the remaining N-values showed adequate factors of safety.

After review of the applicant's response, however, the staff was concerned that (1) the general observation of liquefaction occurrence with respect to age and material type did not exclude the liquefaction potential of the BBM because of the limitation of the observations, such as the possible gravel engagement during the SPT and CPT tests; and (2) limited test data, including N-values, tip resistance, and shear wave velocity, could not reliably exclude the liquefaction potential for the BBM. The staff concluded that limited data prevented the applicant from making a conclusion on the liquefaction potential for the BBM; therefore, the staff determined that the applicant did not have sufficient ESP soil property data to confirm that the BBM is not liquefiable. Accordingly, the staff in the SER with Open Items designated this issue as Open Item 2.5-21.

In response to Open Item 2.5-21, the applicant stated that additional boring logs were used to re-characterize the confusion surrounding the presence of hard layers (i.e. gravel) in the BBM that may have yielded anomalously high SPT results. The applicant provided updated boring logs, along with additional laboratory tests, which it stated showed that the BBM was a hard clay or soft rock material and therefore not prone to liquefaction. The applicant incorporated additional boring logs and field and laboratory test data into later revisions of the SSAR. The staff reviewed these additional boring logs and information and concludes that the soil property data support the applicant's conclusion that the BBM was not susceptible to liquefaction. The staff based its conclusion on the results of the liquefaction potential analyses performed for the application, including liquefaction potential based on SPT data, liquefaction potential based on shear wave velocity data, and liquefaction analyses of the compacted backfill. The applicant also determined that the Blue Bluff Marl is primarily cohesive but has some lenses of coarse grained materials, but these materials have an adequate factor of safety, greater than 1.1, against liquefaction. RG 1.198 states that factors of safety against liquefaction of 1.1 to 1.4 are considered to be moderate. Accordingly, the staff considers that the applicant has demonstrated an adequate factor of safety against liquefaction for the Blue Bluff Marl for Open Item 2.5-21 to be closed. The closure of Open Item 2.5-21 also resolves RAI 2.5.4-14, since the applicant provided the additional information required to confirm the liquefaction potential of the BBM.

The staff identified the site characteristic value for liquefaction potential and determined it should be defined as negligible. Because portions of the soil at the VEGP site are susceptible to liquefaction, the applicant stated that these soils would be either removed and replaced, or physically improved, such that the liquefaction potential is reduced to negligible and the factor of safety against liquefaction is increased to at least 1.1. The staff therefore proposes to include the following condition in any ESP that might be issued in connection with this application: The ESP holder shall either remove and replace, or shall improve, the soils directly above the Blue Bluff Marl for soil under or adjacent to Seismic Category 1 structures, to eliminate any liquefaction potential. This is **Permit Condition 1**.

The staff's evaluation of the information provided in support of the LWA request is as follows:

The staff reviewed the information provided by the applicant regarding the liquefaction potential of the backfill materials proposed for use at the site. Based on the properties of the backfill material described in SSAR Section 2.5.4.5.3, and the results of field and laboratory testing, the applicant concluded that, for the design basis earthquake, liquefaction was not a concern within the compacted backfill. Considering the dry density of 95 percent, and the relatively high blow count and shear wave velocity of the compacted backfill, the staff concurs with the applicant's conclusion that liquefaction potential of the compacted backfill was not a concern at the VEGP Units 3 and 4 site. Therefore, the staff concludes that the assessment of the liquefaction potential of the compacted backfill at the site is adequate to satisfy the criteria of 10 CFR Parts 50 and 100 with respect to the liquefaction potential of the materials underlying the Seismic Category 1 structures at the site.

Based on its review of SSAR Section 2.5.4.8 and the resolution of RAIs and closure of Open Items, the staff concludes that the applicant's assessment of the liquefaction potential of the soil and rock underlying the site of Units 3 and 4 is acceptable for both the ESP and LWA applications, subject to Permit Condition 1.

2.5.4.3.9 Earthquake Design Basis

SSAR Sections 2.5.2.6 and 2.5.2.7 present the applicant's derivation of the safe shutdown earthquake (SSE), and Section 2.5.2.8 presents the operating basis earthquake (OBE). Sections 2.5.2.3.6 and 2.5.2.3.8 of this SER provide the staff's evaluation of the applicant's determination of the SSE and OBE. Shear wave velocity profiles, soil modulus reduction, and damping curves described in Section 2.5.4 are critical inputs to the site seismic response and therefore to the SSE and OBE. However, the staff's analysis of these inputs is fully discussed in SER Section 2.5.2.

2.5.4.3.10 Static Stability

In its review of SSAR Section 2.5.4.10, the staff focused on the applicant's evaluation of bearing capacity and settlement of the bearing strata at the ESP site. The applicant used the following assumptions in calculating soil-bearing capacity and structure settlement: (1) placing all safety-related structures on the structural backfill above the Blue Bluff Marl after removal of the Upper Sand Stratum; (2) placing the base of the containment and auxiliary building foundations about 12.19 meters (40 ft) below final grade, or 15.3 to 18.3 meters (50 to 60 ft) above the top of the Blue Bluff Marl Stratum; and (3) placing other foundations in the power block area at depths of about 1.2 meters (4 ft) below final grade. The applicant modeled the containment building mat as a circle with a diameter of about 43.3 meters (142 ft) placed at a depth of 12.0 meters (39.5 ft) below finish grade in the calculations. The applicant determined that the allowable bearing pressure was 1470.3 kPa (30,700 psf) under static loading conditions and 2203 kPa (46,000 psf) under dynamic loading conditions. The settlement under an average bearing pressure of 239.5 kPa (50,000 psf) was 41 mm (1.6 in.).

In RAI 2.5.4-15, the staff asked the following of the applicant:

1. Justify the adoption of the Peck et al. (1974) settlement and differential settlement values as guidelines which suggest total settlement of no more than 50 mm (2 in.), and differential settlement of no more than 19 mm (0.75 in.). For footings that support smaller plant components, the total settlement should be no more than 25 mm (1 in.), and the differential settlement no more than 13 mm (.5 in,).

2. Explain the main causes for exceeding these settlement values at the foundation levels of Units 1 and 2 and whether it would take any measures to prevent settlements and differential settlements for the new units.

3. Justify the use of an average bearing pressure of 239.5 kPa (50000 psf) for the settlement analyses of compacted fills.

In response to this RAI, the applicant stated the following:

1. The geotechnical community has widely accepted and used the Peck et al. (1974) total settlement guidelines of 25 mm (1 in.) for column footings and 50 mm (2 in.) for mats. When limiting foundation settlements to these values, differential settlements are usually very small. The applicant further stated that, even if these settlement values were exceeded, it would not necessarily have adverse effects on structures, especially for large mat foundations which can efficiently distribute structural loads to the soil. The applicant used the VEGP Units 1 and 2 as an example where the calculated settlements of the containment buildings ranged from 102 to 109 mm (4 to 4.3 in.)

2. It (the applicant) will not use the settlement guidelines from Peck et al. (1974) for Units 3 and 4. The approach used for Units 3 and 4 consisted of estimating settlements for power block structures and using them as design values. The "VEGP Report on Settlement" prepared by Bechtel in 1986 provides comparisons of measured versus calculated settlements and concludes that the measured values did not exceed calculated or design values. The applicant would reanalyze and employ corrective measures in the event that monitored settlements exceed the design values. The applicant committed to follow the same approach for Units 3 and 4 and to revise SSAR Sections 2.5.4.10.2 and 2.5.4.11 accordingly in the next revision to the ESP application.

3. It (the applicant) used a bearing pressure value of 239.5 kPa (50,000 psf) in foundation settlement analysis for illustrative purposes because no design value was available during the ESP. The applicant will revise the calculation using design values during the COL application.

After reviewing the responses, the staff concluded the following:

1. A primary concern of potential total and differential settlements is how these settlements compare with what the design of the reactor takes into consideration. It is important to compare the estimated settlements, which are appropriate for evaluation of the acceptability of the site at the ESP stage, with those incorporated into the plant design to evaluate the degree of conservatism because there will be severe impact to the safety of the SSCs once unexpected differential settlements occur.

2. The contact pressures associated with the planned reactor model are of interest and need to be considered at the ESP stage to estimate potential settlement. Since the data for a given reactor facility are available, the applicant incorporated the data into the site evaluation. Based on the above considerations and in lieu of the fact that large settlements were observed at VEGP Units 1 and 2, the staff concludes that the applicant did not demonstrate quantitatively whether the observed large settlement that occurred at the existing VEGP units will occur at the VEGP site and have no impact on the new units. This was identified as COL Action Item 2.5-8 in the SER with Open Items.

In the revised SSAR, the applicant provided additional information on the settlement analysis for the ESP site. These analyses are summarized in Section 2.5.4.2.10 of this SER, and include details on the differential settlement and the application of the elastic properties of VEGP Units 1 and 2 to determine the settlement of Units 3 and 4. The staff reviewed the additional information supplied in Revision 4 and determines that because the applicant provided the information on settlement analysis using differential settlement and the elastic properties of the existing units, the response negates the need to include COL Action Item 2.5-8 in the final safety evaluation report.

In RAI 2.5.4-16, the staff asked the applicant to justify not analyzing the stability of all planned safety-related facilities in terms of bearing capacity, rebound, settlement, and differential settlements with the consideration of dead loads of fills and the reactor facility, as well as the lateral loadings. In its response, the applicant explained that this kind of information is not available at the ESP stage. Based on the applicant's response, the staff concluded that, since the applicant committed to provide more details regarding the bearing capacity, the staff agreed with the applicant that this information will not be available until the COL stage, and considered that this design-related information was not necessary to determine whether 10 CFR Part 100 is satisfied. Accordingly, this issue was designated as COL Action Item 2.5-9 in the SER with Open Items.

Revision 4 of the SSAR incorporates additional site investigation results from the COL stage, including bearing capacity calculations summarized in Section 2.5.4.2.10 of this SER. The staff reviewed this additional information from the COL site investigations, including the influence of the load-bearing layer (Blue Bluff Marl) on the allowable bearing pressure. The staff determined that because the applicant provided additional factors of safety and allowable bearing capacity details that the applicant determined as part of its COL investigation, the applicant provided adequate information to address concerns identified in COL Action Item 2.5-9. Therefore, the staff concludes that COL Action Item 2.5-9 does not need to be included in the FSER.

In RAI 2.5.4-18, the staff asked the applicant to provide detailed information on its determination of the allowable bearing capacity value. In its response, the applicant provided a detailed description of bearing capacity evaluations based on the Vesic (1975) formula. In addition, the applicant later clarified that the calculated value was net allowable bearing capacity, not the gross bearing capacity; therefore, the formula used in the actual calculation was slightly different from that presented in the reference. From its review of the applicant's response, the staff considered that the Vesic (1975) formula is based on primary assumptions of gross shear failure of soils under the foundation. Although this allowable bearing capacity formulation is applicable for general foundation analysis, the staff considers it inappropriate to use in nuclear power plant foundation design. The control factors of allowable contact pressure for a large and heavy structure typically are not general shear failure but are (1) settlements; (2) allowable pressures used in design of the wall/basemat intersection; and (3) toe pressures developed during potential overturning and sliding of the facility. Based on the above considerations, the staff concluded that the allowable bearing capacity value provided by the applicant is not appropriate when considering the expected governing issues controlling the site evaluation. This was identified as Open Item 2.5-22 in the SER with Open Items.

In response to Open Item 2.5-22, the applicant stated that the bearing and settlement analysis would be completed in late 2007 and would be incorporated in a later revision of the SSAR. When the applicant submitted Revision 4 of the SSAR, the staff reviewed the bearing capacity of the containment and auxiliary buildings, which the applicant stated was 2011 kPa (42 ksf) under dynamic loading conditions with a factor of safety of 2.25 and 1628 kPa (2.25 and 34 ksf) under static loading conditions with a factor of safety of 3.0. These bearing capacity values were identified by the staff as the site characteristic values. The staff also considered the settlement analysis performed by the applicant for the large mat foundations that will support the major power plant structures. The applicant concluded that the settlement at the site would be

5.08 to 7.6 cm (2 to 3 in), with a tilt of approximately 0.63 cm (¼ in) in 15 m (50 ft), a differential settlement between structures of less than 2.54 cm (1 in), and the predicted heave due to foundation excavation ranging from about 2.54 to 6.35 cm (1 to 2 ½ in).

As a result of a staff audit of seismic calculations, the applicant revised SSAR Subsection 2.5.4.10.1 for Revision 5. The applicant evaluated the allowable bearing capacity of the structural backfill under the nuclear island for dynamic loading conditions using both Terzaghi's bearing capacity equation for local shear and Soubra's method with seismic bearing capacity factors, which incorporates Terzaghi's bearing capacity equation for general shear with an internal friction angle of 36° (SNC 2008d). To simulate the potential for higher edge pressures during dynamic loading, the applicant considered three foundation widths corresponding to 10, 25, and 50 percent of the width of the nuclear island basemat. The applicant stated that the results from these two methods compared well with Terzaghi's approach for local shear, providing more conservative values, and it reported the computed average ultimate capacities for the three widths as 4261, 4788, and 5698 kPa (89, 100, and 119 ksf). The applicant reported that using a width of 7.62 m (25 ft) and a factor of safety of 2.25 for site-specific conditions provided an allowable bearing pressure greater than 2011 kPa (42 ksf) under dynamic loading conditions for the nuclear island. The applicant also noted that the value was greater than the DCD requirement of 1676 kPa (35 ksf) for dynamic bearing as well as the Vogtle site-specific maximum dynamic demand of 862kPa (18 ksf) for the ESP soil profile.

The applicant also evaluated the bearing capacity of the structural backfill in terms of the ratio of the ultimate bearing capacity against structure demand, and stated that this capacity over demand (C/D) ratio provided an alternative measure of the margin of safety against bearing failure (SNC 2008d). The applicant evaluated these C/D ratios for the static and dynamic demand conditions as well as the maximum dynamic demand from the Vogtle site-specific seismic evaluation. The applicant stated that the C/D ratios, 11.9 for DCD static, 2.9 for DCD dynamic, and 5.6 for the site-specific dynamic, were higher than those typically utilized for standard practice. While the results did not account for settlement of the structures, the applicant concluded the significant margin suggested that settlements would be minimal and within the DCD requirements.

Considering: 1) the updated bearing capacities determined for both static and dynamic conditions, which incorporated capacity-over-demand ratios as an alternative measure to the factor of safety against bearing failure; 2) the settlement analysis results, which showed minimal settlement; and 3) the displacement monitoring plans for the VEGP site, the staff concludes that the information provided by the applicant in the revised SSAR addressed the concerns identified in Open Item 2.5-22 and the staff considers the Open Item closed. The closure of Open Item 2.5-22 also resolves RAIs 2.5.4-15, 2.5.4-16 and 2.5.4-18. Based on its review of SSAR Section 2.5.4.10, including Revision 5, and the applicant's responses to the RAIs, as described above, the staff further concludes that the applicant provided an adequate assessment of the static stability of the ESP site through the incorporation of data and results for both ESP and COL site investigations, including through additional calculations performed by the applicant as a result of the staff's seismic calculation audit included in Revision 5. The site characteristics approved by the staff for minimum bearing capacity (static and dynamic) are included in Appendix A. Furthermore, the staff concludes that the applicant provided sufficient information with respect to the static and dynamic stability of the site to satisfy the applicable criteria of 10 CFR Parts 50 and 100.

2.5.4.3.11 Design Criteria

In SSAR Section 2.5.4.11, the applicant provided general geotechnical criteria, such as acceptable factors of safety against liquefaction, allowable bearing capacities, acceptable total and differential settlements, and an acceptable factor of safety against slope stability failure.

The staff's evaluation of the information provided in support of the ESP application is as follows:

The staff reviewed the information provided by the applicant regarding the applicable AP1000 geotechnical design criteria to determine if the applicant conducted an exploration and testing program sufficient to determine whether the site would support the design parameters. The staff focused on 1) the applicant's efforts to determine the ability of the Blue Bluff Marl bearing layer to support the plant structures and whether the overall site geology met site parameters, 2) the applicant's studies to determine static and dynamic bearing capacity and whether the site properties and properties of the engineered backfill met or exceeded site perimeters and required factors of safety, 3) whether the applicant's studies and backfill designs supported DCD shear wave velocity minimum requirements, and 4) whether the applicant sufficiently analyzed site liquefaction potential. As discussed in the previous sections, the staff concludes that the applicant conducted an exploration and testing program consistent with the guidance presented in RG 1.132, RG 1.138, and RG 1.198 to adequately characterize the site and verify that the site would support the AP1000 design criteria discussed and applied in Section 2.5.4 of this SER.

The staff focused its review on the design criteria, including the factors of safety against specific events, such as liquefaction and loading conditions. The application did not provide structural design criteria, such as wall rotation, sliding, or overturning. The staff also considered the applicant's incorporation of standard design criteria into the most recent revision (Rev. 4) of the SSAR. Based on the applicant's inclusion of site-specific design criteria, including the factors of safety against events such as liquefaction or loading, the staff considers the applicant's design criteria used in the ESP application to be acceptable, as the applicant has met the applicable standards of 10 CFR Part 50.

The staff's evaluation of the information provided in support of the LWA request is as follows:

The staff reviewed the information provided by the applicant regarding the design criteria required to support the LWA request to excavate, prepare the site, and backfill the proposed plant site to the bottom of the foundation within the nuclear islands and up to plant grade outside the MSE walls. To meet the requirement for the LWA, the applicant needed to characterize the site down to a depth sufficient to support the AP1000 site parameters for bearing capacity, shear wave velocity and liquefaction, and it also needed to develop the site-specific criteria for engineered structural backfill, MSE retaining walls, concrete mudmats, and MSE and concrete mudmat waterproofing materials sufficient to meet the intent of the DCD design for coefficient of friction. As discussed in the preceding sections, the staff concludes that the applicant presented sufficient information for a LWA request because the staff determined that the applicant 1) adequately characterized the site following the guidelines presented in RG 1.132, 2) performed field and laboratory testing following the guidelines presented in RG 1.132 and RG 1.138 to verify that the site and engineered structural backfill support the DCD minimum required shear wave velocity, 3) presented sufficient design details for the concrete mudmat and MSE wall, including constructing a test section for staff observation, and 4) worked with the DCD design organization to determine the proper waterproofing system and minimum required coefficient of friction for the system.

In RAI 2.5.4-19, the staff asked the applicant to justify the omission of additional design criteria and factors of safety (FS). In response, the applicant revised the SSAR to reference the applicable design criteria in the AP1000 DCD, Revision 15. The applicant also stated that the FS against liquefaction should be greater than 1.1; FS of 3 should be applied to bearing capacity equations, but this FS can be reduced to 2.25 when dynamic or transient load conditions apply; and the long-term static and seismic FS against slope stability failure was 1.5 and 1.1, respectively. Because the applicant incorporated the applicable design criteria from Revision 15 of the AP1000 DCD and the revised SSAR to include relevant factors of safety, the staff considers RAI 2.5.4-19 resolved. Furthermore, based on the closure of RAI 2.5.4-19, the staff concludes that the design criteria presented for an ESP at the VEGP Units 3 and 4 site is acceptable to satisfy the requirements of 10 CFR Part 50 because the revised SSAR contained a description and safety assessment of the site and the site evaluation factors identified in Part 100, including the information relative to the materials of construction, general arrangement and approximate dimensions of the facility sufficient to provide reasonable assurance that the final design will satisfy the design bases with adequate margin of safety.

Based on the applicant's inclusion of the design-specific criteria, including the factors of safety against events such as liquefaction or loading, the staff considers the applicant's design criteria to be acceptable for the LWA request, as the applicant has met the applicable standards of 10 CFR Part 50.

2.5.4.3.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 states that no ground improvement techniques were considered beyond the removal and replacement of the Upper Sand Stratum with engineered structural backfill; however, other ground improvement techniques will be considered as necessary. The staff therefore focused its review on the subsurface improvement plans, the most significant of which is the planned removal of the entirety of the Upper Sand Stratum. The staff reviewed the plans for removal of the Upper Sand Stratum, as described in Section 2.5.4.1.5, and for the reasons evaluated in Section 2.5.4.3.5 of this SER, as well as the applicant's consideration of other improvement techniques, as necessary, the staff concludes that the plans for subsurface improvement therefore satisfy the criteria of 10 CFR Part 100. The inclusion of the detailed plans for removal of the Upper Sand Stratum, as well as the applicant's consideration of additional ground improvement techniques make fulfills COL Action Item 2.5-11. Therefore, COL Action Item 2.5-11 is no longer necessary.

2.5.4.4 Conclusions

Based on its review of SSAR Section 2.5.4, related references, and the applicant's responses to the associated RAIs and Open Items described above, the staff concludes:

The applicant conducted a limited ESP investigation to determine the engineering properties of subsurface soils at the ESP site. The applicant supplemented the few field and laboratory tests conducted as part of the ESP investigation to determine static and dynamic and other engineering properties of the underlying soils with information from the subsequent COL investigation. The additional quantity and quality of the test results were sufficient for the applicant to reliably determine the engineering properties of the subsurface materials.

Therefore, the staff concludes that the applicant has adequately determined the engineering properties of the subsurface materials.

The applicant provided a site-specific shear wave velocity profile in a situation that assumed the shear wave velocity measured from the down-hole tests was lower than the shear wave velocity obtained from the suspension P-S velocity measurements; the shear wave velocities from previous investigations associated with VEGP Units 1 and 2 were also lower. Additionally, the applicant provided the results of soil dynamic testing on the samples from the ESP site to provide soil modulus reduction and damping curves to feed into the site response study and the site-specific shear wave velocity profile. The applicant also supplemented the SSAR with additional inputs to the development of the shear wave velocity profile and the shear modulus reduction curves. Therefore, the staff concludes that the applicant provided sufficient information to characterize the shear wave velocity profiles, and the shear modulus reduction and damping ratio curves, which are critical input to the site-specific ground motion response spectrum discussed in SER Section 2.5.2, as well as to the soil structure interactions discussed in SER Section 3.8.

The applicant provided an assessment of the liquefaction potential of the BBM, which was the load-bearing unit at the ESP site. Based on the results of extensive SPT and CPTs by the applicant, the staff concurs with the applicant that the BBM is not prone to liquefaction. The applicant also described the excavation and backfill plans, in extensive detail, to support both the ESP application and its LWA request. These plans included the use of a test pad program to better constrain the final engineering properties of the Seismic Category I backfill to be used. The staff concludes that the level of detail provided for the excavation and backfill plans, including quality control and ITAAC, is sufficient to address the requirements of 10 CFR Part 50.

The proposed Units 3 and 4 would be located above the load-bearing strata similar to that underlying the existing units, and the existing units already observed an unusually large settlement (both total and differential). The applicant provided a detailed settlement analysis to ensure that the SSCs for the AP1000 are safe. The staff finds that the applicant adequately demonstrated the stability of the subsurface materials in response to static and dynamic loading conditions at the ESP site. The applicant provided the bearing capacity for the containment and auxiliary buildings at the site, which were given as 2,010 kPa (42 ksf) under dynamic loading conditions with a factor of safety of 107 and 1,627 kPa (2.25 and 34 ksf) under static loading conditions with a factor of safety of 3.0. Based on these bearing capacities and the high factor of safety, the staff concludes that the bearing capacity of the site is acceptable to meet the requirements of 10 CFR Parts 50 and 100 with respect to the static stability of the site. The staff also reviewed the information and data from the applicant's field and laboratory investigations as well as the evaluations of the geotechnical engineering properties of the soils and rock underlying the ESP site. Additionally, the staff made several trips to the site to observe applicant activities and the geotechnical conditions of the site to determine whether the applicant followed the guidance contained in RG 1.132 and other relevant guidance in its ESP and LWA site-specific investigations.

Based on the above findings, the staff concludes that, in support of both the ESP application and LWA request, the applicant conducted sufficient site investigations and performed adequate field and laboratory tests and associated analyses, to provide sufficient information describing soil conditions underlying the ESP site, such as the possible existence of "soft zones" in the foundation-bearing layer. The applicant also demonstrated reliable engineering properties of the soils through the combination of its ESP and COL site investigations. This information was addressed and evaluated by the staff as part of its review of the LWA request. Therefore, the staff concludes that for the information required by the scope of the ESP, the applicant has provided sufficient information to characterize the subsurface materials at the ESP site of VEGP Units 3 and 4. Based on its review of the engineering properties of materials at the ESP site, the assessment of bearing capacity, liquefaction potential, and settlement, as well as the development of a shear wave velocity profile through the site, the staff finds that the applicant has met the requirements of 10 CFR 100.23 in that the applicant adequately demonstrated the overall static and dynamic stability of the site, identified the soil and rock engineering properties through field and laboratory testing, and characterized the soil subsurface profile.

In SSAR Section 2.5.4, the applicant identified the subsurface material properties as ESP site characteristic values. The first site characteristic specifies that there is no liquefaction below the Blue Bluff Marl layer (approximately 88 ft below the ground surface). The applicant demonstrated, in SSAR Section 2.5.4.8, that any liquefaction at the ESP site would be limited to the soils directly above the Blue Bluff Marl. The requirement to remove and replace or otherwise improve the liquefiable soils at the site to eliminate the liquefaction potential is Permit Condition 1. The second site characteristic value specifies a minimum bearing capacity of 1628 kPa (34 ksf) under static loading conditions and 2011 kPa (42 ksf) under dynamic loading conditions. These values are based on the VEGP site soil properties and the results of the applicant's ESP and COL investigations. Finally, the third design parameter specifies minimum S-wave velocities for the depth intervals given in SSAR Tables 2.5.4-11 and 2.5.4-11a. These S-wave velocity values are based on the applicant field geophysical surveys. The staff has reviewed the applicant's suggested site characteristics related to SSAR Section 2.5.4 for the inclusion in an ESP, should one be issued. For the reasons set forth above, the staff agrees with the applicant's proposed site characteristic and the values for those characteristics.

Based on the staff's review of the applicant's information regarding the LWA request, the staff concludes that the applicant conducted sufficient subsurface investigations and performed adequate field and laboratory testing and analyses to support that request. As discussed previously in this section of the SER, much of the information needed for the LWA request was also required for the staff's evaluation of the ESP application. The applicant had to first adequately characterize the proposed site to determine whether the site could support the applicable AP1000 design criteria for the LWA activities. As the staff has stated above, the applicant adequately characterized the site and verified that the site criteria for bearing capacity, liquefaction, and shear wave velocity could be met. The applicant also developed the criteria for the engineered structural backfill materials and verified that these criteria, in conjunction with the geologic site conditions, would further support the DCD design criteria for bearing capacity, liquefaction, and shear wave velocity. As the staff stated above, the applicant did so, following the guidance presented in the applicable Regulatory Guides.

Once the applicant determined that the site and proposed backfill materials would meet the AP1000 design criteria, the applicant determined whether sufficient material was available on-site to backfill the proposed excavation. The applicant also proposed a design for the MSE wall system. As part of the LWA request, the applicant showed the extent and depth of the excavation; disposition of the excavated materials as backfill or spoil; extent of temporary construction slopes and construction dewatering details; preparation of the marl bearing layer for placement of backfill and backfilling to the bottom of the foundation; placement of the MSE walls and nuclear island concrete mudmat working surfaces and waterproofing system; backfilling around the perimeter of the nuclear islands outside of the MSE walls to final plant grade; demonstration of mass and confined backfill placement techniques; and, finally, its demonstration of backfill density, shear wave velocity and, as evaluated in SER Section 3.8.5, waterproofing system friction coefficient, with proposed ITAAC to verify and document that the

AP1000 design criteria will be met. Therefore, for the reasons stated above, the staff concludes that the applicant has adequately demonstrated that it has met the applicable LWA requirements associated with the stability of subsurface materials and foundations for the requested LWA activities at the VEGP site.

2.5.5 Stability of Slopes

SSAR Section 2.5.5 describes the applicant's review of existing slopes at the ESP site and the applicant's plan for permanent cut and fill slopes during construction excavation. The applicant also discussed its plans for future slope stability analysis to take place during the design phase. The applicant did not perform slope stability analysis for the ESP site because there is no existing slope and the applicant cannot determine the future slope at the ESP phase.

2.5.5.1 Technical Information in the Application

The applicant stated that, since there were no existing slopes or embankments near the proposed location of VEGP Units 3 and 4, it did not perform a dynamic slope stability analysis. The applicant further stated that the site grading for construction of new units would result in nonsafety-related permanent cut and fill slopes. Permanent cut slopes would have a height of 15.2 meters (50 ft) or less and would be located several hundred meters away from planned or existing safety-related structures. Permanent fill slopes would have a height of 6.1 meters (20 ft) or less and would also be several hundred meters away from planned or existing safety-related structures. During the construction phase, the applicant will remove the soils above the Blue Bluff Marl and replace them with compacted structural fill. The applicant stated that the construction excavation cut slopes would be temporary (i.e., only during the construction period) and that they will be far away from the safety-related structures of the existing VEGP Units 1 and 2. The applicant committed to perform nonsafety-related permanent slope stability analysis for dynamic and static conditions, as well as excavation cut slope analysis for static conditions during the design stage, to ensure that these slopes will not pose a hazard to the public.

2.5.5.2 Regulatory Basis

SSAR Section 2.5.5 states that the applicant did not perform a slope stability analysis for the ESP site application. However, the applicant stated in SSAR Section 1.8 that it followed the guidance of NUREG-0800, Section 2.5.5, when it described the slope-related issues in SSAR Section 2.5.5. In its review of SSAR Section 2.5.5, the staff considered the regulatory requirements in 10 CFR 100.23(c) and 10 CFR 100.23(d). According to 10 CFR 100.23(c), applicants must investigate the engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site. Pursuant to 10 CFR 100.23(d)(4), applicants must evaluate siting factors such as natural and artificial slope stability.

2.5.5.3 Technical Evaluation

The staff focused its review of SSAR Section 2.5.5 on whether there are any existing or planned new slopes that would adversely affect the safety-related structures of the proposed new units due to any possible loading conditions and/or natural events. After reviewing the information provided by the applicant, the staff concludes that, because there are no existing significant slopes near the proposed ESP site, a detailed slope stability analysis is not necessary at the ESP stage. The staff considers the creation of permanent slopes during construction to be a design-related issue, which must be addressed at the COL stage. However, after reviewing the site construction plan layout and discussions with the applicant, the staff confirmed that the only permanent slopes are not safety-related. Therefore COL action item 2.5-12 is no longer needed.

2.5.5.4 Conclusions

Since there are no safety-related permanent slopes, the applicant did not perform any slope stability analysis. The excavation will create nonsafety-related permanent cut and fill slopes during the new units' construction stage, however, since these slopes are not permanent, they are not part of the staff's review.

2.5.6 Embankments and Dams

SSAR Section 2.5.6 presents a general description of existing and potential new embankments and dams at the ESP site.

2.5.6.1 Technical Information in the Application

SSAR Section 2.5.6 indicates that there is no earth, rock or earth, and rock fill embankments required for plant flood protection or for impounding the cooling water required for the operation of the plant. The applicant indicated that there are three existing nonsafety-related impoundments at the site—Mallard Pond, Debris Basin Dam 1, and Debris Basin Dam 2. The Mallard Pond is located to the north of the proposed switchyard, Debris Basin Dam 1 is located to the southeast of the proposed cooling towers, and Debris Basin Dam 2 is located to the southwest of the proposed cooling towers. The applicant stated that it would not use the impoundments for plant flood protection or for impounding cooling water for the operation of the plant. The pool level in Mallard Pond is below the elevation of 38.1 meters (125 ft) above msl. In the event of a dam breach at Mallard Pond, the water would drain to the north and away from the proposed new units. The pool levels in Debris Dams 1 and 2 are also below the elevation of 45.7 meters (150 ft) above msl, and, in the event of a dam breach, the water would drain to the south, away from the proposed new units. Therefore, the applicant concluded that there would be no need for embankments or dams for flood protection or for impounding the cooling water at the site.

2.5.6.2 Regulatory Basis

The applicant did not state which regulations SSAR Section 2.5.6 addressed; these topics are covered in NUREG 0800, Sections 2.4.4 and 2.5.5. However, in SSAR Section 1.8, Table 1-2, the applicant stated that it used RG 1.70 for guidance on format and content. Section 2.5.6 of RG 1.70 describes the necessary information and analysis related to the investigation, engineering design, proposed construction, and performance of all embankments used for plant flood protection or for impounding cooling water.

2.5.6.3 Technical Evaluation

In its review of SSAR Section 2.5.6, the staff evaluated the possible impact of a breach of existing embankments and dams on the proposed new units at the ESP site and evaluated the need for construction of any embankments or dams for flood protection. Based on the information provided by the applicant, the staff notes that the proposed finished grade elevation for the new units is approximately 67 meters (220 ft) above msl, and the existing pool levels for the three impoundments are 38.1 meters (125 ft) above msl for Mallard Pond, and 45.7 meters (150 ft) above msl for both Debris Basin Dams 1 and 2. These elevations are all below the proposed finished grade elevation. In addition, as the applicant discussed in Sections 2.4.3 and 2.4.4 of the SSAR, both probable maximum flood elevation (45.8 m (150.13 ft) msl) and the dam break level (54.3 m (178.10 ft) msl) are much lower than the proposed finished grade elevation Therefore, the staff concurs with the applicant's conclusion that no embankments and dams are required.

2.5.6.4 Conclusions

The applicant provided adequate information and analysis in SSAR Section 2.5.6, with reference to Sections 2.4.3 and 2.4.4 of the SSAR, regarding the embankments and dams at the ESP site. The applicant demonstrated that no embankments or dams are needed for flood protection at the ESP site under possible flood and dam breach conditions because of the proposed finished grade elevation.