3.0 SITE SAFETY ASSESSMENT

3.5.1.6 Aircraft Hazards

3.5.1.6.1 Introduction

For its ESP application, the applicant provided information evaluating the potential hazards associated with aircraft. The NRC staff reviews these evaluations to ensure that the risks associated with potential aircraft hazards are sufficiently low.

3.5.1.6.2 Regulatory Basis

The acceptance criteria for aircraft hazards 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 site location and area description.

- 10 CFR 52.17, insofar as it requires the applicant to provide the location and description of any nearby military or transportation facilities and routes.

- 10 CFR Part 100, as it relates to the following:

- 10 CFR 100.20(b), which requires that the nature and proximity of man-related hazards (e.g., airports, transportation routes, and military facilities) must 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 states that the potential hazards associated with nearby transportation routes, industrial, and military facilities must be evaluated and site parameters established such that potential hazards from such routes and facilities will pose no undue risk to the type of facility proposed to be located at the site.

RS-002, Section 3.5.1.6, specifies that these requirements are met if the probability of aircraft accidents having the having the potential for radiological consequences greater than the 10 CFR Part 100 exposure guidelines is less than about 10^{-7} per year. The probability is considered to be less than about 10^{-7} per year by inspection if the distance from the site meets all of the following criteria:

1. the site-to-airport distance (D) is between 5 and 10 statute miles and the projected annual number of operations is less than 500 D^2, or the site-to-airport distance (D) is greater than 10 statute miles, and the projected annual number of operations is less than 1000 D^2,

2. the site is at least 5 statute miles from the edge of military training routes, including low-level training routes, except for those associated with usage greater than 1000 flights per year, or where activities (such as practice bombing) may create an unusual stress situation, and

3. the site is at least 2 statute miles beyond the nearest edge of a Federal Airway, holding pattern, or approach pattern

If the above proximity criteria are not met, or if sufficiently hazardous military activities are identified, then a detailed review of aircraft hazards should be performed. Section 3.5.1.6 of RS-002 provides guidance on the performance of such reviews.

3.5.1.6.3 Technical Evaluation

Following the procedures described in RS-002, Section 3.5.1.6, the NRC staff reviewed Section 3.5.1.6 of the SSAR included in the VEGP application. In this section, the applicant provided information that addressed and analyzed aircraft hazards. The applicant's response to the NRC staff's RAI 3.5.1.6-1 further supplements this information with regard to the calculation of effective area being used in the aircraft hazards analysis.

In Section 2.2.2.6 of the SSAR, the applicant presented information concerning the airports, airways, and military training routes in the site vicinity that need to be evaluated for potential hazards with respect to nuclear units that might be constructed on the proposed ESP site.

The applicant stated that all airports in the VEGP site vicinity are greater than 10 miles from the site. The closest and largest commercial airport is the Augusta Regional Airport at Bush Field (Bush Field), which is located about 17 miles north-northwest of the VEGP site. According to the applicant, on the basis of FAA projections up to 2025, the number of airport operations (including landings and take-offs) is estimated to be about 43,000.

The applicant stated that this total number of projected aircraft operations is substantially less than the threshold number of operations set forth in RS-002, Section 3.5.1.6., which indicates that the probability for the aircraft accident is considered acceptable if the projected annual number of operations is less than 1000 D^2, where D is the site-to-airport distance in miles. The applicant also stated that other airports in the vicinity are much smaller than Bush Field. The applicant noted that the aircraft hazard threshold for these airports is greater than the 100,000 annual number of operations because of their distance from the site. This threshold annual number of operations is significantly higher than the estimated annual operations for each of these airports. Therefore, the applicant found that the hazard probability of these airports was acceptable and did not require a detailed evaluation of the potential hazards with respect to aircraft operations at these airports.

The applicant stated that there is a small unimproved grass airstrip located immediately north of the VEGP site (north of Hancock Landing Road and west of the Savannah River). This privately owned and operated airstrip has a 1650-foot turf runway oriented 80 degrees east- 260 degrees west. The airstrip is for personal use and the associated traffic consists of small single-engine aircraft. In addition, a small helicopter landing pad is located on the VEGP site. This facility exists for corporate use and for use in case of emergency. The traffic associated with either of these facilities may be characterized as sporadic. The applicant stated that because of the small amount and the nature of the traffic, these facilities do not present a safety hazard to the VEGP site.

The applicant stated that the closest military training route is VR97-1059, the nearest edge of which is located more than 6 miles from the VEGP site. Military aircraft using route VR97-1059 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). The applicant stated that the total number of military aircraft using route VR97-1059 is approximately 833 per year. According to RS-002, the aircraft accident probability for a military training route is considered to be less than 10^{-7} per year if the distance from the site is at least 5 miles from the edge of the military training route, including low-level training routes, except for routes that have a usage greater than 1000 flights per year or where activities may create an unusual stress situation. The applicant stated that since the VEGP site is located more than 5 miles from the edge of VR97-1509, and the total military flights (833 per year) using the same route is less than 1000 per year, no aircraft accident analysis is required for flights using VR97-1509. The probability number of 10^{-7} was cited from RG 1.70, Revision 3, issued November 1978, in reference to design basis external events.

The applicant stated in Section 2.2.2.6.2 of the SSAR 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 the VEGP site is within the 2 statute-mile limit specified in Section 3.5.1.6 of RS-002, the applicant performed a more detailed review of aircraft hazards associated with air traffic along the V185 Airway; and this analysis was presented in Section 3.5.1.6 of the SSAR. The applicant stated that the FAA does not maintain records of air traffic in Airway V185. Therefore, since the traffic data for Airway 185 is not available, the applicant calculated the maximum number of airway flights per year required to exceed the acceptance guideline crash probability of 10^{-7} per year as stated in RS-002 and NUREG-0800. The applicant estimated that the total number of flights traveling along Airway V185 would need to be greater than approximately 51,100 per year in order to exceed a crash probability of 10^{-7} per year. Since this value is higher than the projected yearly total of flights through 2025 at Bush Field, the applicant did not consider Airway V185 to pose a significant hazard to the VEGP site.

The NRC staff independently verified the applicant-identified airports. The NRC staff contacted the FAA, and obtained the Bush Field flight operations data for the 2000 through 2006. These data reveal that the average number of flight operations at Bush Field is about 42,363, which is comparable to the applicant's stated number. Therefore, the NRC staff agrees with the applicant's conclusion that all public and private airports in the vicinity of the VEGP do not have sufficient annual flight operations to warrant a detailed risk analysis for potential nuclear units at the ESP site.

The NRC staff verified the applicant's cited reference of 14 CFR Part 71, "Designation of Class A, B, C, D, and E Airspace Areas; Air Traffic Routes, and Reporting Points." The applicant used the information cited in this regulation in recommending the width of the airway as 4 nautical miles on either side of the centerline, for a total width of 8 nautical miles. The NRC staff also verified the applicant's effective area calculation based on applicant's reference of the 1996 U.S. DOE guidance. The FAA provided the NRC staff with the number of flights that traversed V185 airways (FAA, 2007). As a result of the large amount of data to be analyzed, as well as the limitations of computing time and data handling, the FAA estimated the flight count data by extracting the flight count along V185 airways for every Thursday (typically as this day of the week is observed to have large number of flights) from January 2003 through December 2006. Based on these FAA data, the NRC staff calculated the average number of flights along V185 airways to be about 3000 per year. Also based on this value and the guidance provided in RS-002, Section 3.5.1.6, the NRC staff independently estimated the annual probability of an aircraft traversing along V185, crashing into the plant to be about 6 x10-^{9}.

The NRC staff evaluated the applicant's analysis of military aircraft for route VR97-1059. Based on 3 years of military training route data for Route VR97-1059, Shaw Air Force Base determined the average number of military training flights to be 761 compared to the applicant's referenced data of 833. Because the actual flights are lower than the threshold value of 1000 flights per year, the NRC staff finds the probability to be less than 10^{-7} per year. Regarding the identification of any activities within VR97-1059 that could create an unusual stress situation, Shaw Air Force Base informed the NRC staff that practice bombings are not authorized within Route VR97-1059. However, Shaw Air Force Base indicated that military aircraft will fly to Poinsett Range, to practice bombing and strafing. Inert bombs are used at Poinsett Range, instead of live bombs. Poinsett Range is approximately 10 miles south of Shaw Air Force Base. The NRC staff calculated the distance from the VEGP site to Poinsett Range to be approximately 78 miles. The guidance contained in RG 1.70 specifies that an aircraft hazard analysis should be done for practice bombing ranges within 20 miles from the site. Because the distance from the VEGP site to Poinsett Range is greater than the 20-mile distance specified in RG 1.70, the NRC staff finds the practice bombing at Poinsett Range does not create any unusual stress situations.

The NRC staff has reviewed the applicant's assumptions and calculations and finds them to be reasonable, consistent, and acceptable. On the basis of its independent estimation of the probability of a potential aircraft crash, the NRC staff agrees with the applicant's conclusion that Airway V185 does not present a safety concern for the VEGP site.

3.5.1.6.4 Conclusions

The NRC staff has reviewed the applicant's aircraft hazard analysis using the procedures delineated in RS-002, Section 3.5.1.6. As set forth above, the NRC staff has independently verified the applicant's assessment of aircraft hazards at the site and has concluded that the estimated probability of an accident having the potential for radiological consequences in excess of the exposure criteria found in 10 CFR Part 100 is less than about 10^{-7} per year.

Based on these considerations, the NRC staff concludes that aircraft hazards do not present an undue risk to the safe operation of nuclear units at the proposed ESP site. Therefore, the NRC staff concludes that, with respect to aircraft hazards, the proposed site is acceptable for planned nuclear units, and that the site meets the relevant requirements of 10 CFR Part 52 and 10 CFR Part 100.

3.7 Seismic Design

The AP1000 seismic Category I and II structures, systems, and components (SSCs) are designed to withstand the effects of seismic loads as defined in terms of the certified seismic design response spectra (CSDRS).

Seismic Category I SSCs are designed to withstand the effects of seismic motions defined in terms of the CSDRS and to maintain their specified design functions. Seismic Category II and nonseismic structures are designed or physically arranged (or both) so that seismic motions defined in terms of the CSDRS cannot cause unacceptable structural interaction with or failure of seismic Category I SSCs.

3.7.1 Seismic Design Parameters

3.7.1.1 Introduction

In its application, SSAR Part 2, Section 3.8, the applicant submitted details for performing work within the scope of the limited work authorization (LWA) request in accordance with 10 CFR 52.17(c) and 10 CFR 50.10(d). The scope of the applicant's LWA request involves soil foundation work and the placement of a concrete mudmat, a waterproofing membrane, concrete forms, a mechanically stabilized earth (MSE) retaining wall, and drains. The applicant, in SNC letter AR-08-1337, dated September 10, 2008 (ADAMS Accession No. ML082590048), states that the scope of the LWA request excludes the placement of steel reinforcement, embedments, and concrete for the structural foundation (basemat).

The scope of the staff's review of the applicant's LWA request is limited to SRP Sections 3.7.1,"Seismic Design Parameters," 3.7.2,"Seismic System Analysis," and 3.8.5, "Foundations." These sections address the applicant's LWA request to install a mudmat with an embedded waterproofing membrane. The mudmat, as indicated in SSAR Section 3.8, is to be placed over competent soil and constructed in two halves, with a waterproofing membrane placed between the two halves.

Accordingly, the staff evaluated the applicant's (1) seismic analysis and design, including (a) the design ground motion, (b) the foundation input response spectra, and (c) the supporting media for seismic design, and (2) applicable seismic system analyses, including (a) the foundation stability of the nuclear island (NI) against sliding and overturning, (b) the maximum dynamic bearing pressures developed beneath the foundation basemat, and (c) the horizontal seismic shear stresses developed between the basemat and the top of the mudmat, between the two halves of the mudmat through the waterproofing membrane, and between the bottom of the mudmat and the foundation soils.

The staff will perform the remaining review of the applicant's seismic design (i.e., for portions outside the scope of the LWA request) during its review of the Vogtle subsequent combined operating license (SCOL) application currently on Docket 52-025 and 52-026. The staff's review of Vogtle SCOL FSAR Section 3.7 will reflect the findings of the LWA review as appropriate.

3.7.1.2 Regulatory Basis

The staff relied on the following applicable regulatory requirements in reviewing the applicant's discussion of seismic design parameters:

- 10 CFR Part 100, Subpart B, which is applicable to power reactor site applications on or after January 10, 1997, refers to 10 CFR 100.23 for seismic criteria. This section describes the criteria and nature of investigations required to obtain the geologic and seismic data necessary to determine the suitability of the proposed site and the plant design bases. In addition, 10 CFR 100.23 refers to Appendix S to 10 CFR Part 50 for the definition of the minimum SSE ground motion for use in design.

- 10 CFR Part 50, Appendix S, is applicable to applications for a design certification or combined license pursuant to 10 CFR Part 52. Appendix S requires that, for SSE ground motions, SSCs will remain functional and within applicable stress, strain, and deformation limits. The required safety functions of SSCs must be assured during and after the vibratory ground motion through design, testing, or qualification methods. The evaluation must take into account SSI effects and the expected duration of the vibratory motion. Appendix S also requires that the horizontal component of the SSE ground motion in the free field at the foundation level of the structures must be an appropriate response spectrum with a PGA of at least. 0.10g.

- 10 CFR 52.79(b) applies to a COL referencing an ESP and requires information sufficient to demonstrate that the design of the facility falls within the site characteristics and design parameters specified in the ESP.

- 10 CFR 52.79(d)(1) applies to a COL referencing a design certification and requires that COL applications include information sufficient to demonstrate that the characteristics of the site fall within the site parameters specified in the design certification.

In addition, the seismic design parameters should be consistent with appropriate sections from:

- Regulatory Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, December 1973

- Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 3, March 2007

- Regulatory Guide 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," March 2007

Section 3.7.1 of NUREG-0800 provides specific guidance concerning the evaluation of seismic design parameters.

3.7.1.3 Technical Evaluation

3.7.1.3.1 Technical Information Presented by the Applicant

To support the technical basis for the LWA, the applicant incorporated by reference (IBR) the AP1000 DCD, Revision 15, Tier 1 Section 3.3, and Tier 2 Sections 2.5.2.3, 3.7.1, 3.7.2, and 3.8.5.

In addition, the applicant performed site-specific seismic analysis and provided the results in SSAR Appendix 2.5E, "AP1000 Vogtle Site Specific Seismic Evaluation Report, Revision 3."

3.7.1.3.1.1 AP1000 Standard Design

AP1000 DCD, Revision 15, Tier 2, FSAR Section 3.7.1, describes the CSDRS for the AP1000 design. These response spectra, as indicated in DCD FSAR Figures 3.7.1-1 and 3.7.1-2, are based on Regulatory Guide (RG) 1.60 amplified in the high-frequency region from 9 Hz to 25 Hz. The AP1000 CSDRS have peak ground accelerations (PGAs) of 0.30g in both the vertical and horizontal directions, which are applied at the foundation level in the free field for rock sites and at the finished grade for other generic soil conditions.

3.7.1.3.1.2 Vogtle Design Ground Motion Response Spectra

In SSAR Section 2.5.2, the applicant described its approach for developing the GMRS using the performance-based method described in RG 1.208. The Vogtle site-specific GMRS are defined at the free ground surface in the free field, which is defined as the finished grade level (Plant elevation 220 feet).

In SSAR Appendix 2.5E, Figures 3-4 and 3-5, the applicant compared the Vogtle GMRS to the AP1000, Revision 15, CSDRS. The free-field GMRS PGAs at the finished grade level are approximately 0.26g in the horizontal direction and 0.23g in the vertical direction and are bounded by the AP1000 CSDRS free-field PGA of 0.30.

SSAR Appendix 2.5E, Figures 3-4 and 3-5, indicate that the Vogtle site-specific GMRS exceed the AP1000 CSDRS in the approximate frequency ranges of 0.4–0.7 Hz and 7–60 Hz for the horizontal direction and 0.5–0.6 Hz and 12–50 Hz for the vertical direction. As a result of these exceedances, the applicant performed a site-specific soil-structure interaction (SSI) analysis to demonstrate either the suitability of the AP1000 standard design (Revision 15) or to justify the adequacy of the mudmat, the waterproofing membrane, and the NI structure stability. The applicant described these analyses in SSAR Appendix 2.5. FSER Section 3.7.2 describes the staff's review of these SSI analyses.

In SSAR Appendix 2.5E, Section 3.0, the applicant described its approach for developing the site-specific foundation input response spectra (FIRS) for Vogtle Units 3 and 4. The Vogtle FIRS are free-field outcrop spectra (determined using the entire soil column from bedrock to the free surface) at the foundation basemat elevation (i.e., 40 feet below the finished grade level). In SSAR Appendix 2.5E, Section 3.0, the applicant defined the Vogtle FIRS at 5-percent equipment damping, which is consistent with requirements in DCD FSAR, Tier 2, Section 2.5.2.3.

For the purpose of performing site-specific SSI calculations, the applicant chose to use the 5-percent damped spectrum as input to the three deterministic SSI soil profiles determined from the site-specific probabilistic site response analysis at the median or best estimate, 14th percentile or lower bound, and 84th percentile or upper bound levels. For each profile, the applicant computed the surface and corresponding in-column spectra for use in the three deterministic SSI calculations. The applicant determined the corresponding in-column spectra for each case at an elevation in the profile which is the same as the FIRS outcrop and which represents the free-field particle motions at that depth. For each of the three SSI cases, the applicant generated three enveloping time histories (two horizontal and one vertical) to envelop the in-column spectra at 40 feet below the finished grade level. The applicant then used these in-column time histories as input to the SSI cases at 40 feet below the finished grade.

3.7.1.3.1.3 Percentage of Critical Damping Values

For seismic analysis of Category I structures, the applicant used values of critical damping consistent with RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants." The applicant assumed the critical damping value for reinforced concrete to be 7 percent and the maximum critical damping value for free-field soil layers to be less than 15 percent.

3.7.1.3.1.4 Supporting Media for Seismic Category I Structures

In SSAR Appendix 2.5E, Section 2.0, the applicant described the Vogtle site characteristics. The subsurface materials for the Vogtle site consist of approximately 90 feet of loose to dense sands, 70–80 feet of very hard, slightly sandy clay (i.e., Blue Bluff Marl), 900 feet of dense sands, and Triassic sandstone at 1049 feet. The applicant will excavate the upper 90 feet of soil and replace it with approximately 50 feet of compacted granular fill materials from the top of Blue Bluff Marl to the free ground surface. The fill material will be taken from borrow materials available locally. SSAR Appendix 2.5E, Section 2.0, states that the location of the water table is expected to be at least 15 feet below the bottom of the basemat of the NI. This is consistent with the applicant's proprosed site characteristic value of 165 feet MSL for the highest ground water elevation at the site, and the proposed elevation of the bottom of the AP1000 nuclear island basemat (180 feet MSL). The SSI analysis relied on the ground water table to be 15 feet below the basemat elevation.

3.7.1.3.2 NRC Staff's Technical Evaluation

The scope of the staff's review of SSAR Appendix 2.5E is limited to those sections that support the applicant's LWA request to install a mudmat with an embedded waterproofing membrane. To this end, the staff evaluated the applicant's technical basis for developing appropriate seismic design parameters for (1) comparing with the AP1000, Revision 15, CSDRS, (2) satisfying regulatory requirements, and (3) using appropriate input motions to the site-specific SSI analyses. The staff's evaluation of the applicant's seismic design parameters was performed in accordance with SRP Section 3.7.1.

The applicant used the site-specific seismic design parameters (e.g., GMRS, FIRS, and associated randomized soil profiles) to support SSI analyses which evaluated the effects of NI dynamic response. In addition, the applicant performed SSI analyses to demonstrate that a basemat sliding coefficient of friction of 0.45 between the mudmat and the supporting soils, would prevent the NI structure from sliding against the supporting soils under the SSE seismic loads.

3.7.1.3.2.1 Design Ground Motion Response Spectra

As stated previously, SSAR Appendix 2.5E, Figures 3-4 and 3-5, indicate that the Vogtle site-specific GMRS exceed the AP1000, Revision 15, CSDRS in the approximate frequency ranges of (0.4–0.7 Hz) and (7–60 Hz) for the horizontal direction and (0.5–0.6 Hz) and (12-50 Hz) for the vertical direction. As a result of these exceedances, the applicant performed a site-specific SSI analysis either to demonstrate suitability of the AP1000, Revision 15, standard design or to justify the adequacy of the mudmat, the waterproofing membrane, and the NI structure stability. The applicant described these analyses in SSAR Appendix 2.5E, "AP1000 Vogtle Site Specific Seismic Evaluation Report, Revision 3." FSER Section 3.7.2 describes the staff's review of these SSI analyses.

The staff reviewed the applicant's method for developing the site-specific FIRS and reviewed the applicant's methods for developing spectrum compatible time histories, randomized soil profiles, artificial shear wave velocity profiles, and degradation curves for Vogtle.

The staff's review found that (1) the process used to generate randomized shear wave velocity profiles and (2) the procedures used to generate the mean uniform hazard spectra at the free-ground surface at the top of the backfill satisfied the standard guidance described in RG 1.208 and in SRP Section 3.7. However, the staff found that the procedures used to generate the corresponding "outcrop" motions at the 40-foot depth (bottom of the NI foundation) were not in accordance with SRP Section 3.7. The motions included the effects of the downcoming waves in the calculation and were inconsistent with the need to generate the outcrop motions at a free-ground surface. To address the inconsistency in generating outcrop motions, the applicant compared the surface motions used as input to the SSI calculations for the best estimate, lower bound, and upper bound site-specific profiles using the FIRS outcrop motion at the 40-foot horizon (SSAR Appendix 2.5E, Section 3.0, Figures 3-6 through 3-8). The staff reviewed these comparisons and found that the applicant's approach to generating the SSI input motions from the FIRS motion resulted in conservative horizontal and vertical motion. Therefore, in the case of Vogtle Units 3 and 4, the staff accepts the applicant's approach to generating the FIRS outcrop motion and corresponding time-histories for use in site-specific SSI analyses.

The staff reviewed the applicant's compliance with the 10 CFR Part 50, Appendix S, requirement that the horizontal component of the SSE ground motion in the free field at foundation level be an appropriate response spectrum with a PGA of at least 0.10g. SSAR Appendix 2.5E (Figures 3-4 and 3-5) demonstrates that the Vogtle FIRS PGAs at the bottom of the NI foundation are approximately 0.26g in the horizontal direction and 0.23g in the vertical direction and thus are greater than the 0.10g regulatory requirement. On the basis of this comparison, the staff finds that the applicant has satisfied the Appendix S requirement.

3.7.1.3.2.2 Percentage of Critical Damping Values

As part of the detailed review of the applicant's SSI analysis, the staff reviewed the applicant's critical damping values used in the SSI analysis of seismic Category I structures (i.e., 7 percent for reinforced concrete and less than 15 percent for soil) and found them to be consistent with RG 1.61.

3.7.1.3.2.3 Supporting Media for Seismic Category I Structures

The staff reviewed the applicant's description of supporting media for the NI, including foundation embedment depth, depth of soil over bedrock, soil layering characteristics, highest groundwater elevation, dimensions of the foundation, and soil properties in SSAR, Section 2.5.4, and Appendix 2.5E. The staff finds the 40 foot embedment and dimensions of the foundation to be consistent with the AP1000, Revision 15, NI. Additionally, the staff finds that the SSI modeling assumptions relating to depth of soil over bedrock, soil properties, soil layering characteristics and groundwater elevation are acceptable based on conformance to the criteria discussed in SRP Section 3.7.1.

The staff reviewed the ESP-calculated best-estimate soil shear wave velocity profile, described in SSAR Appendix 2.5E, Section 4.0, and found that the shear wave velocity through the backfill soil was approximately 500 fps at the ground surface, greater than 1,000 fps at the bottom of the basemat, and about 2,800 fps at approximately 700 feet below grade. The staff finds that the 1,000 fps shear wave velocity at the bottom of the basemat meets the SRP Section 3.7.1.3 criterion for minimum shear wave velocity of the supporting foundation material.

3.7.1.4 Conclusion

On the basis of its review of the applicant's submittal, the staff concludes that the applicant has adequately developed seismic design parameters (e.g., GMRS, FIRS, and associated randomized soil profiles) for use in comparing to the AP1000, Revision 15, CSDRS, satisfying regulatory requirements, and performing a site-specific two-dimensional SSI analysis to evaluate foundation stability and basemat bearing pressures.

The staff's conclusions are based on the following five findings:

(1) The free-field GMRS PGAs at the finished grade level are approximately 0.26g in the horizontal direction and 0.23g in the vertical direction and are bounded by the DCD, Revision 15, SSE free-field PGA of 0.30.

(2) Although the applicant's method for developing the FIRS (at 40 feet below grade as outcrop motion) was not consistent with SRP Section 3.7, the method resulted in conservative seismic demand.

(3) The FIRS in the free field satisfied the minimum PGA value of 0.10g and is suitably broad banded.

(4) The critical damping values used in SSI analysis were consistent withdamping values used in RG 1.61.

(5) The 1,000 fps shear wave velocity at the bottom of the basemat meets the SRP Section 3.7.1.3 criterion for minimum shear wave velocity of the supporting foundation material.

Therefore, the staff finds that with respect to the LWA request, the applicant has met the applicable requirements of 10 CFR 52.79(b), 10 CFR 52.79(d)(1), and 10 CFR Part 50, Appendix S in that the applicant adequately demonstrated (1) that the relevant portions of the design of the facility falls within the site characteristics and design parameters specified in the ESP and AP1000 certified design (Revision 15) and (2) that the horizontal component of the SSE ground motion in the free-field at the foundation elevation is an appropriate response spectrum with a PGA of at least 0.10g.

3.7.2 Seismic System Analysis

3.7.2.1 Introduction

SSAR Section 3.0, Figures 3-4 and 3-5, indicate that the Vogtle GMRS exceed the AP1000 CSDRS in the approximate frequency ranges of (0.4–0.7 Hz) and (7–60 Hz) for the horizontal direction and (0.5–0.6 Hz) and (12–50 Hz) for the vertical direction. As a result of these exceedances, the applicant performed site-specific analyses to demonstrate the suitability of the AP1000, Revision 15, certified design.

The staff reviewed the applicant's site-specific two-dimensional SSI analyses to ascertain the appropriateness of the model(s) for estimating the maximum horizontal and vertical inertial loads on the NI resulting from SSE loading. These inertial loads are used to compute factors of safety for sliding and overturning and to compute maximum foundation bearing pressures anticipated to develop from the seismic motions. The staff focused its review on computer model descriptions, analysis assumptions, shear wave velocity profiles, and sensitivity studies on backfill properties. The staff did not evaluate in-structure response of the NI because it was not needed for the LWA request.

3.7.2.2 Regulatory Basis

The staff relied on the following applicable regulatory requirements in its review of the applicant's discussion of seismic systems analysis:

- 10 CFR Part 50, General Design Criterion (GDC) 2, requires that the design basis reflect appropriate consideration of the most severe earthquakes 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 historical data have been accumulated.

- 10 CFR Part 100, Subpart B of 10 CFR Part 100, which is applicable to power reactor site applications on or after January 10, 1997, refers to 10 CFR 100.23 for seismic criteria. This section describes the criteria and nature of investigations required to obtain the geologic and seismic data necessary to determine the suitability of the proposed site and the plant design bases. This section also indicates that Appendix S to 10 CFR Part 50 contains applications to engineering design.

- 10 CFR Part 50, Appendix S, is applicable to applications for a design certification or combined license pursuant to 10 CFR Part 52. Appendix S requires that, for SSE ground motions, SSCs will remain functional and within applicable stress, strain, and deformation limits. The required safety functions of SSCs must be assured during and after the vibratory ground motion through design, testing, or qualification methods. The evaluation must take into account SSI effects and the expected duration of the vibratory motion. Appendix S also requires that the horizontal component of the SSE ground motion in the free field at the foundation level of the structures must be an appropriate response spectrum with a PGA of at least 0.10g.

In addition, the seismic systems analysis should be consistent with appropriate sections from:

- Regulatory Guide 1.60, “Design Response Spectra for Seismic Design of Nuclear Power Plants,” Revision 1, December 1973

- Regulatory Guide 1.92, “Combining Modal Responses and Spatial Components in Seismic Response Analysis,” Revision 2, July 2006

Section 3.7.2 of NUREG-0800 provides specific guidance concerning the evaluation of seismic analysis methods.

3.7.2.3 Technical Evaluation

3.7.2.3.1 Technical Information Provided by the Applicant

3.7.2.3.1.1 Seismic Model Description

Structural Model

SSAR Appendix 2.5E, Section 4.0, describes the applicant’s seismic analysis performed using the dynamic analysis computer code, “System for Analysis of Soil-Structure Interaction” (SASSI). The NI seismic analysis models used two-dimensional SASSI stick models to represent the AP1000 auxiliary shield building (ASB), the steel containment vessel (SCV), and the containment internal structure (CIS). These models made conservative assumptions with respect to the structural configuration of the auxiliary shield building. The applicant modeled the reinforced concrete of the ASB and CIS with linear elastic constitutive models. The applicant used guidance provided in FEMA 356 to reduce the concrete modulus of elasticity by 20 percent to account for reduced stiffness under moderate seismic loading conditions.

To account for the SSI effects on the NI response, the NI SSI model includes adjacent buildings such as the annex, radwaste, and turbine buildings, which are idealized by either lumped masses (radwaste and turbine buildings) or as a two-dimensional stick model (annex building).

Soil Model

The soil adjacent to the NI foundation is modeled by 8 uniform layers, with a horizontal spacing of approximately 5 feet. Soil elements are used to connect the foundation elements to the adjacent soil layers. The soil below the NI foundation is modeled with 81 elements to a depth of 1050 feet.

Ground Motion Input

The control motion input for the SSI analyses is the FIRS outcrop motion located at the NI foundation elevation. The applicant developed the FIRS outcrop motion from the three ESP soil profiles shown in SSAR Section 4.0, Figure 4.1-2. The results of the probabilistic site response analyses define these lower bound, best-estimate, and upper bound soil profiles. The enveloping of results from the three deterministic SSI analyses is intended to account for uncertainties in soils at the site as well as variability of wave-field effects assumed for the SSI analysis.

The applicant also developed a separate FIRS outcrop motion using the three sensitivity (SEN) soil profiles shown in SSAR Section 5.0, Figure 5.0-1. The SEN soil profiles assumed a slightly greater soil shear wave velocity in the NI backfill to account for variations in backfill compaction. The applicant performed these additional two-dimensional analyses to determine sensitivity in calculated response to properties of the backfill materials.

3.7.2.3.1.2 Soil-Structure Interaction Analyses

SSAR Appendix 2.5E, Section 5.0, describes the applicant's SSI analyses. The applicant used these analyses to determine the maximum horizontal and vertical inertial loads on the NI resulting from SSE loading. The SSAR defines the SSI model which used the ESP soil profiles as the ESP model. Similarly, the SSAR defined the SSI model which used the SEN soil profiles as the SEN model.

Sensitivity Studies

The applicant performed three sensitivity studies to address uncertainties in backfill soil modeling and its effects on NI response calculations. These sensitivity studies compared the effects of (1) variable compaction of the entire backfill soil adjacent to the NI, (2) modeling simplifications for the backfill excavation geometry, and (3) variable compaction of backfill soil within a 5-foot zone adjacent to the NI while the backfill outside of this zone remain unvaried.

SSAR Appendix 2.5E, Section 5.0, describes the applicant's approach to and results from sensitivity studies on variable compaction of the backfill soil. These calculations used the SEN SSI model and assumed slightly higher shear wave velocities for the NI backfill. The applicant compared the results of the SEN and ESP SSI models at the six key locations of the NI structure, provided in SSAR Appendix 2.5E, Section 5.0, Table 5.1-1, and found that the soil backfill, with a slightly higher shear wave velocity, will not significantly affect the NI dynamic response.

SSAR Appendix 2.5E, Appendix A, describes the applicant's approach to performing sensitivity studies on backfill excavation geometry. In the ESP model, the soil adjacent to the NI is essentially modeled in one dimension (i.e., horizontal layers of infinite extent) and does not consider construction issues such as lateral extent and sloping excavation. The applicant developed two separate two-dimensional SASSI models, one with the backfill excavation explicitly modeled and one without. The applicant compared the analysis results from these models at the six critical locations of the NI structure and found that while there were minor differences at some locations, the overall effect on NI response is small.

SSAR Appendix 2.5E, Section 5.3, describes the applicant's approach to, and results from, sensitivity studies on variable compaction of backfill soil within a 5-foot zone adjacent to the NI. This study estimated the effects of potential reduced compaction immediately adjacent to the NI due to construction effects associated with the MSE wall planned for use as a temporary excavation support system. The applicant analyzed the reduced compaction by varying the soil shear wave velocity of the backfill in the 5-foot zone behind the MSE wall. The applicant compared the analysis results from these models at the six critical locations of the NI structure and found that while there were minor differences at some locations, the overall effect on the NI response is small.

3.7.2.3.2 <u>NRC Staff's Technical Evaluation</u>

This section of the SER provides the staff's evaluation of the seismic system analysis. The staff's review of this section is limited to the analysis required to approve the applicant's LWA request. As such, the staff reviewed the applicant's methods for performing two-dimensional SSI analysis and determining seismic forces to evaluate foundation stability and dynamic bearing pressures on soils.

3.7.2.3.2.1 <u>Two-Dimensional Versus Three-Dimensional Seismic Analysis</u>

SSAR Appendix 2.5E, Section 4.0 describes the applicant's approach to performing site-specific analyses. The applicant performed two-dimensional seismic analyses to evaluate seismic stability (i.e., sliding and overturning) and to compute maximum bearing pressures beneath the NI. To evaluate the sliding stability evaluation and estimate bearing pressure demand/capacity evaluations only, the staff accepted the use of these two-dimensional seismic analyses. Since applicant's NI seismic stability evaluations are considered approximate and since the calculated factors of safety from the two-dimensional analyses were found to be large for an embedded facility, the staff considered the use of the simplified two-dimensional analyses to be appropriate. The staff did not consider increased refinement in the analyses to be necessary for these calculations.

However, for development of in-structure response spectra (ISRS) and calculation of maximum seismic element force needed for structural and equipment design, the Staff considers that more refined SSI models are required. The basis for the staff's position is that the two-dimensional SSI calculations do not properly account for (1) effective radiation damping, (2) frequency-dependent impedance functions, (3) out-of-plane effects (e.g., torsion, coupled modes), or (4) vertical responses associated with irregular structural configurations. All of these can have significant impact on computed design response. It is the judgment of the staff that the two-dimensional SSI calculations may underestimate seismic demand. The staff will review in-structure response as part of the COL review of Vogtle FSAR Section 3.7.

3.7.2.3.2.2 Nuclear Island Backfill Soil Sensitivity Calculations

The staff reviewed the applicant's two-dimensional SSI model and found that the modeling approach was acceptable and performed in accordance with guidance in SRP Section 3.7.2. However, the staff found that the applicant essentially modeled the backfill soil adjacent to the NI in one dimension (i.e., infinite horizontal layers), and the model does not consider construction issues such as lateral extent and sloping excavation. To understand whether or not the lateral extent of the backfill excavation has a significant effect on SSI calculations, the staff issued RAI 2.5.2-25 requesting the applicant to compare the soil model response using both a two-dimensional SASSI and one-dimensional SHAKE, and a two-dimensional SASSI structural model response with backfill soil modeled as both infinite uniform layers and uniform layers with lateral boundaries.

In response to RAI 2.5.2-25, the applicant compared the motion response of a one-dimensional SHAKE analysis to a two-dimensional SASSI analysis. SSAR Appendix 2.5E, Figures 2.5.2-55a and 55b, show response comparisons for motions at the ground surface, at the base rock, and at the 40-foot horizon.

The staff reviewed the comparison of the one-dimensional SHAKE and two-dimensional SASSI model vibratory motion responses and found the differences between them to be small.

In response to RAI 2.5.2-25, the applicant also developed two 2-dimensional SASSI models which included the NI and adjacent buildings (SSAR Appendix 2.5E, Appendix A). The first model, "2D-AP-d5," did not account for the lateral extent of the NI. The applicant developed the second model, "Bathtub Model-d5," to represent the east-west cross-section of the Vogtle excavation (SSAR Appendix 2.5E, Figure A-1). The applicant used the same time histories for both analyses and then compared the responses at the six critical locations of the NI structure. Figures A-2 through A-13 illustrates response comparisons for several NI locations.

The staff reviewed the six key NI locations referenced in SSAR Appendix 2.5E, Section 5.0, Table 5.1-1 and agrees with the applicant that these six locations correspond to areas of the most significant safety-related equipment or locations of maximum NI displacement resulting from an SSE event. Therefore, these locations are acceptable to the staff for comparing seismic responses.

1. NI at Reactor Vessel Support Elevation
2. Auxiliary Shield Building at Control Room Floor
3. ASB Auxiliary Building Roof Area
4. ASB Shield Building Roof Area
5. Steel Containment Vessel near Polar Crane
6. Containment Internal Structure at Operating Deck

The staff reviewed the response comparisons of the two-dimensional SASSI models, 2D-AP-d5 and Bathtub Model-d5, and finds that the difference in in-structure response is relatively small. The results indicate that the influence of the backfill excavation geometry on NI dynamic response is insignificant. Therefore, the staff finds the applicant's approach acceptable.

3.7.2.3.2.3 Mechanically Stabilized Earth Backfill Soil Sensitivity

SSAR Section 2.5.4.5.7 describes the details of MSE wall design. Although most of the compaction of the backfill soil behind the MSE will be accomplished with heavy, self-propelled equipment, the applicant stated that the compaction of the zone adjacent to the wall (approximately 5 feet) will be done using smaller sized compactors with thinner lifts. Because of the potential for lower compaction in this zone, the staff was concerned that the resulting drop in shear wave velocity would affect the NI response. To clarify this issue, the staff issued RAI Appendix 2.5E-2 requesting that the applicant describe the effects of variable compactions behind the MSE wall on SSI calculations.

In response to RAI Appendix 2.5E-2, the applicant performed sensitivity studies on the effects of MSE wall backfill using two 2-dimensional SASSI models which are described in Section 5.3 of SSAR Appendix 2.5E. The first SASSI model assumed a backfill shear wave velocity of 515-909 fps with no difference in shear wave velocity in the 5-foot zone adjacent to the MSE wall. The second SSI model assumed the same backfill shear wave velocity as the first model, but the 5-foot zone had a reduced shear wave velocity of 421–755 fps. Section 5.3, Figures 5.3-1 through 5.3-19, of SSAR Appendix 2.5E show the results of these calculations. The analysis results indicate that the effect of reduced backfill compaction (resulting in lower shear wave velocities) on the seismic response at the six critical locations of the NI structure is small.

Based on a review of these results, the staff concurs with the applicant's conclusion that the potentially reduced shear wave velocity of the backfill directly behind the MSE wall does not significantly affect the NI building response for the Vogtle site. On the basis of these findings and the applicant's responses, the staff considers RAI Appendix 2.5E-2 resolved.

3.7.2.3.2.4 Soil-Structure Interaction Models

The staff reviewed the applicant's approach for developing two-dimensional models, described in SSAR Appendix 2.5E, for the purpose of site-specific SSI analysis. On the basis of its review of the backfill soil and MSE wall sensitivity calculations as discussed above, the staff finds the applicant's approach to modeling the Vogtle site soil conditions and AP1000 NI (Revision 15) acceptable for the purpose of calculating seismic demands for assessing foundation stability and dynamic bearing pressures.

The applicant provided the magnitudes of peak results of the seismic shear forces from the SSI model, which are summarized in FSER Table 3.7.2-1, in response to RAI 3.8.5-4. The applicant, in SSAR Appendix 2.5E, Section 6.1, describes the use of these values in performing stability evaluations.

The staff calculated the maximum seismic shear forces and obtained values consistent with the applicant's values in FSER Table 3.7.2-1. Therefore, the staff finds the applicant's maximum seismic shear forces to be acceptable for use in calculating foundation stability and bearing pressures.

Table 3.7.2-1 Vogtle Maximum Seismic Shear Forces

Reaction	Vogtle Lower Bound	Vogtle Best Estimate	Vogtle Upper Bound
Seismic Shear NS	78.3 E3 kips	82.5 E3 kips	89.0 E3 kips
Seismic Shear EW	88.9 E3 kips	89.8 E3 kips	95.8 E3 kips

3.7.2.4 Conclusion

On the basis of the review of the applicant's submittal, the staff concludes that the applicant has adequately performed site-specific seismic analysis for the purpose of determining the maximum horizontal and vertical inertial loads on the NI for use in stability and bearing capacity evaluations. The calculated loads referenced above (FSER Table 3.7.2-1) are acceptable.

The staff's above conclusions are based on the following three findings:

(1) The applicant demonstrated that the effect of excavation geometry (i.e., lateral extent of soil backfill) on the NI SSI response is minimal.

(2) The effect of higher shear wave velocity for backfill soil on the NI SSI response is minimal.

(3) The effect of reduced compaction in the 5-foot zone behind the MSE wall on the NI SSI is minimal.

The staff finds that with respect to the LWA request, the applicant has met the applicable requirements of 10 CFR Part 50, Appendix A (GDC 2), and 10 CFR Part 50, Appendix S, in that the applicant's evaluation accounted for the SSI effects and the expected duration of the vibratory ground motion.

3.8.5 Foundations

3.8.5.1 Introduction

As part of its review of the applicant's LWA request, the staff reviewed the use of the mudmat, which is located beneath the nuclear island (NI) basemat, and the waterproofing membrane, which is located between the two halves of the mudmat. Both are within the scope of the applicant's LWA request. The staff also reviewed the sliding stability of the NI structure during the SSE to ensure that the horizontal seismic shear force can be transferred safely through the mudmat and the waterproofing membrane without sliding to the supporting soils and that the NI structure will not slide relative to its supporting soils. The staff also reviewed the overturning stability of the NI structure during the SSE event and found reasonable assurance that the NI structure will not break into the ground (the supporting soils) during the SSE event.

3.8.5.2 Regulatory Basis

The NRC staff used the following applicable regulatory requirements in its review of the applicant's discussion of foundations:

- 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 1, as they relate to safety-related structures being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed

- GDC 2, as it relates to the design of the safety-related structures that are capable of withstanding the most severe natural phenomena, such as wind, tornadoes, floods, earthquakes, and the appropriate combination of all loads, and still perform their safety functions

- Appendix B to 10 CFR Part 50, as it relates to the quality assurance criteria for nuclear power plants

- 10 CFR 52.80(a), which requires that a COL application contain the proposed inspections, tests, and analyses, including those applicable to emergency planning, that the licensee 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 operate in conformity with the COL, the provisions of the Atomic Energy Act, and NRC regulations.

Section 3.8.5 of NUREG-0800 provides specific guidance concerning the evaluation of foundation design.

3.8.5.3 Technical Evaluation

3.8.5.3.1 Technical Information Presented by the Applicant

In SSAR Section 3.8.5, the applicant described the scope of the LWA request which includes construction of MSE retaining walls, a mudmat, and the waterproofing membrane between the two halves of the mudmat.

SSAR Section 3.8.5.1 describes the process for constructing MSE retaining walls to serve as formworks for the outer walls of the NI structure. The MSE wall will be founded on a concrete strip footing that is independent of the NI structure. The wall will be approximately 40 feet high and will be backfilled with engineered fill. Because the MSE wall only serves as a formwork and is not categorized as a seismic Category I structure, it does not require a review under SRP Section 3.8. SSAR 2.5.4 provides details of the MSE wall.

SSAR Section 3.8.5 describes the installation of a concrete mudmat, which will be placed within the confines of the MSE wall. The mudmat will consist of two 6-inch layers of concrete placed on engineered fill, as described in SSAR Section 2.5.4. An elastomeric spray-on waterproofing membrane will be sandwiched between these layers to provide protection from external flooding. The waterproofing membrane will be sprayed or brushed onto the entire mudmat surface as well as the MSE wall inner face. Before the installation, the applicant will develop a qualification program to evaluate the chemical and physical properties of the waterproofing membrane material. In addition, the applicant has proposed a site-specific ITAAC, shown in Table 3.8.5-1, to confirm that the waterproofing membrane can provide a coefficient of friction of 0.7 to prevent sliding of the upper portion of the mudmat from the lower portion of the mudmat during an SSE.

Table 3.8.5-1 Waterproof Membrane Inspections, Tests, Analyses, and Acceptance Criteria (SSAR Table 3.8.5.1-1)

Waterproof Membrane Inspections, Tests, Analyses, and Acceptance Criteria		
Design Commitment	**Inspections, Tests, Analyses**	**Acceptance Criteria**
1) The friction coefficient to resist sliding is 0.7 or higher	Testing will be performed to confirm that the mudmat-waterproofing-mudmat interface beneath the Nuclear Island basemat has a minimum coefficient of friction to resist sliding of 0.7.	A report exists and documents that the as-built waterproof system (mudmat-waterproofing-mudmat interface) has a minimum coefficient of friction of 0.7 as demonstrated through material qualification testing.

3.8.5.3.2 NRC Staff's Technical Evaluation

The staff's review of this section is limited to the analysis and design required to approve the applicant's LWA request.

3.8.5.3.2.1 Waterproofing Membrane

In SSAR 3.8, the applicant proposed to use an elastomeric waterproof membrane for providing external flood protection for the NI foundation. The applicant stated that it will specify the final thickness of the membrane based on the physical properties of the selected material, but it is expected to be approximately 0.080–0.120 inches thick. SSAR Section 2.5.4.5.3 provides further details of the waterproofing membrane.

During its initial review of SSAR Section 3.8, the staff found insufficient information with respect to the waterproof membrane material. The staff issued RAI 3.8.5-3 which requested the applicant to do the following:

a) Provide chemical and structural (mechanical) properties of the waterproof membrane.

b) Describe whether the waterproof membrane has been used in structures in which a minimum 0.7 coefficient of friction between the waterproofing membrane and concrete was achieved.

c) If no data indicate that a minimum 0.7 coefficient of friction between the waterproofing membrane and concrete exists, provide the basis for the adequacy of the design assumption that the upper portion of the mudmat will not move relative to the lower portion of the mudmat during earthquakes.

d) Describe the qualification and test programs and explain how they can be used to demonstrate that the waterproofing membrane meets the waterproofing and friction requirements stated in Section 3.8.5.1.1.

In response to RAI 3.8.5-3(a), the applicant provided the intended waterproof membrane product datasheet (Sterling Lloyd, Intergitank® Structural Waterproofing Membrane). This data sheet states that the membrane material is a liquid-applied, fully reactive elastomeric membrane which cures rapidly and is available in both spray and hand grades. Typical applications include tunnels, storage tanks and silos, canals and culverts, and low-level radiation tanking.
Table 3.8.5-2 identifies several relevant properties of the membrane material.

Table 3.8.5-2 Relevant Properties of Waterproofing Membrane Material

Property	Value
Typical Tensile Strength	1711 psi (11.8 MPa)
Typical Elongation at Break	>130%
Typical Tear Strength	400 lb/in (70N/mm)
Heat Aging at 70 °C for 1 Year (equivalent to 32 years aging at 20 °C)	No significant change in tensile strength or elongation at break
Resistance to Water Pressure (19.7 ft (6m) head of water)	No leak

In response to RAI 3.8.5-3(b), the applicant provided a test report (Sterling Lloyd TR 621). This test report describes a simplified test for evaluating the coefficient of friction between the membrane material and concrete blocks of various surface textures (smooth, slightly rough). This test report also demonstrated that the static coefficient of friction ranged from 0.40 (slightly textured membrane/slightly rough concrete surface) to 0.81 (slightly textured membrane surface/smooth concrete surface).

In response to RAI 3.8.5-3(c), the applicant stated that the assumption that the upper and lower portions of the mudmat will not move relative to one another is based on the results from Sterling Lloyd TR 621, which showed that a coefficient of friction of 0.7 (or greater) is achievable.

In response to RAI 3.8.5-3(d), the applicant stated that it will conduct qualification and test programs to demonstrate that the waterproof membrane meets the waterproofing and friction requirements stated in SSAR Section 3.8.5.1.1. To this end, the applicant proposed in ESP SSAR Section 3.8.5.1, Table 3.8.5.1-1, "Waterproof Membrane Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC)," to perform testing to confirm that the mudmat-waterproofing-mudmat interface has a minimum coefficient of friction of 0.7 prior to installation of the waterproof membrane.

The staff finds that the report provided by the applicant (i.e., Sterling Lloyd TR621) is based on friction testing using materials (i.e., cement block and elastomeric spray-on waterproofing) whose relevant properties are substantially similar to those proposed by the applicant for the design of the waterproof membrane described in SSAR Section 3.8.

Based on the above, the staff finds that there is sufficient technical information to conclude: (1) that the proposed waterproof membrane material can achieve a friction coefficient of 0.7; and (2) that the ITAAC to document the as-built waterproof system (mudmat-waterproofing-mudmat interface) has a minimum coefficient of friction of 0.7, as demonstrated through material qualification testing, is reasonable and verifiable.

Therefore, the staff considers RAI 3.8.5-3 resolved.

On the basis of the above discussion and the inclusion of the waterproof membrane ITAAC, the staff concludes that the applicant's proposed use of the waterproofing membrane for VEGP Units 3 and 4 is acceptable.

3.8.5.3.2.2 Stability Analyses

In SSAR Appendix 2.5E, Chapter 6, the applicant described the methods for evaluating the site-specific sliding stability and overturning stability of the AP1000 NI structure.

3.8.5.3.2.3 Sliding Stability

The applicant assumed a coefficient of friction of 0.45 between the basemat and the supporting soil. To further the staff's understanding of this assumption, the staff issued RAI 3.8.5-4 which requested that the applicant address the following:

a) For stability analysis during earthquakes, state whether the bottom of the mudmat is allowed to move relative to the supporting soils or not. If relative movement is predicted, state the maximum value of the horizontal movement during the SSE and the basis for accepting that amount of movement. If relative movement is not predicted, state the maximum magnitude of the horizontal force generated in the nuclear island structure during the SSE, and the magnitude of frictional force provided at the interface between the mudmat and the supporting soils.

b) If the magnitude of frictional force provided at the interface between the mudmat and the supporting soils is less than the maximum magnitude of the horizontal force generated in the nuclear island structure during the SSE, state the magnitude of forces due to the passive earth pressure on one side and the active earth pressure on the opposite side of the embedded nuclear island walls generated through the rotation of the nuclear island structure, and describe how these horizontal forces are in equilibrium so that the bottom of the mudmat will not move relative to its supporting soils. At that equilibrium stage, state: (1) the rotational angle of the nuclear island structure and the horizontal displacement at the top surface of the soils adjacent to the nuclear island structure during the SSE; and (2) whether or not buoyancy force due to ground water and vertical seismic forces were subtracted from the total weight of the nuclear island.

c) Describe how the shear loads (or stresses) in different regions of the upper portion of the mudmat are transferred through the waterproof membrane to the lower portion of the mudmat. State the maximum shear load (or stress) in the mudmat and the shear capacity of the waterproof membrane and the mudmat, and describe how these values were derived or obtained.

In response to RAI 3.8.5-4(a), the applicant stated that its stability analysis assumed that there is no relative movement at the bottom of the mudmat relative to the supporting soils. Furthermore, the applicant provided the maximum seismic shear force and the friction force between the mudmat and the supporting soil for the upper bound, best-estimate, and lower bound SSI cases in the north-south (NS) and east-west (EW) directions during the SSE (summarized in SER Table 3.8.5-3). Because the applicant provided the requested information, the staff considers RAI 3.8.5-4(a) resolved.

Table 3.8.5-3 Vogtle Maximum Seismic Shear and Friction Forces

Reaction	Vogtle Lower Bound	Vogtle Best Estimate	Vogtle Upper Bound
Seismic Shear NS	78.3 E3 kips	82.5 E3 kips	89.0 E3 kips
Seismic Shear EW	88.9 E3 kips	89.8 E3 kips	95.8 E3 kips
Friction Force	117.3 E3 kips	116.7 E3 kips	116.4 E3 kips

In response to RAI 3.8.5-4(b), the applicant stated that in all cases (see Table 3.8.5-3) the available friction forces exceed the maximum seismic shear forces.

The staff calculated the maximum seismic shear force and the friction force and obtained values close to the applicant's values listed in Table 3.8.5-3. Therefore, the staff considers RAI 3.8.5-4(b) resolved and agrees with the applicant's conclusion that the NI structure will not slide against its supporting soil during the SSE.

With respect to the shear strength of the mudmat, the applicant responded in RAI 3.8.5-4(c) that the shear strength of the mudmat is about 100 psi, based on American Concrete Institute (ACI) code calculations, because the minimum specified concrete compressive strength for the mudmat is 2500 psi. The staff agrees with the applicant's assessment of the shear strength for the mudmat. The applicant obtained an average shear stress of 25.1 psi by using the highest friction force of 117.3x103 kips, as listed in Table 3.8.5-3, divided by the footprint area of the NI structure as the required shear stress to be transferred through the mudmat and the waterproofing membrane. The staff considers the applicant's shear stress calculation of 25.1 psi to be conservative because it used the largest friction force of 117.3x103 kips instead of the largest seismic shear force of 95.8x103 kips. Therefore, the staff concludes that the mudmat, which possesses a shear strength of 100 psi, can safely transfer the required 25.1 psi shear stress through it.

With respect to the shear strength of the waterproofing membrane, the staff agrees with the applicant's statement that the waterproofing membrane, which possesses a tensile strength of 1700 psi, likely possesses a shear strength greater than 25.1 psi. Based on the soil conditions at the Vogtle site, the mudmat material strength, and the waterproofing membrane strength, the staff concludes that the NI structure can safely transfer horizontal seismic shear force through the mudmat and the waterproofing membrane, without sliding, to the supporting soils. Furthermore, the NI structure will not slide horizontally relative to its supporting soils during the SSE.

3.8.5.3.2.4 Overturning Stability

In SSAR Section 2.5.4.10.1, the applicant provided the site-specific dynamic bearing capacity of 42 ksf for soils at the Vogtle site. SSAR Appendix 2.5E, Chapter 7.0, summarizes the maximum dynamic bearing pressures on soils from the site-specific two-dimensional SSI analyses. The analyses results indicate that no structure will be overturned, and the maximum dynamic bearing pressures for the NI, radwaste, annex, and turbine buildings are 17.95 ksf, 1.68 ksf, 7.20 ksf, and 2.54 ksf, respectively, during the SSE. The minimum factor of safety with respect to a failure of the dynamic bearing capacity during the SSE is 2.34 (42 ksf divided by 17.95). The staff considers this minimum factor of safety to be adequate and concludes that the NI structure will not break into the ground (the supporting soils) during the SSE.

3.8.5.4 Conclusion

Based on its review of the applicant's submittal and responses to RAIs, the staff concludes that the applicant has demonstrated that the design of the MSE walls, the mudmat, and the waterproofing membrane, as stated in the LWA request, is adequate and can be constructed. The staff's conclusion is based on the following findings:

(1) The MSE walls are not seismic Category I structures, and they only serve as formworks for the outer walls of the NI foundation.

(2) Both the mudmat and the waterproofing membrane have sufficient shear strength to transfer the required shear stress, without sliding at interfaces, to the supporting soils.

(3) The waterproof membrane ITAAC is adequate for confirming that the mudmat-waterproofing-mudmat interface beneath the Nuclear Island basemat has a minimum coefficient of friction to resist sliding of 0.7.

(4) The soil condition at the Vogtle site is capable of preventing the NI structure, including the mudmat, from sliding horizontally relative to the ground (the supporting soils) and from breaking into the ground vertically during the SSE.

The staff finds that the applicant has met the applicable requirements of 10 CFR 52.80(a), 10 CFR Part 50, Appendix A (GDC 1 and 2), and 10 CFR Part 50, Appendix B in that the applicant adequately demonstrated: (1) that the COL application contains the proposed inspections, tests, and analyses that the licensee 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 operate in conformity with the COL, the provisions of the Atomic Energy Act, and NRC regulations; and (2) that the NI mudmat and waterproofing membrane are designed to resist an SSE event.