ENCLOSURE 3

NCS-TR-00008, Rev. 0
Criticality Risk from NPH in Outlying Buildings

nuclear operations group

Criticality Risk from NPH in Outlying Buildings

NCS-TR-00008, Rev. 0

December 16, 2013

Prepared by:	Larry L. Wetzel	12/16/13 Date
Peer Reviewed by:	Todd W. Stinson	16 DEC 2013 Date
NCS Manager Approval:	David W. Faidley	12/17/13 Date

Revision History

Revision	Date	Description
0		Original Issue

Table of Contents

Table of Tables

Table of Figures

1.0 Summary

The NRC conducted an inspection from 2/27/12 - 3/1/12 for Temporary Instruction 2600/015, "Evaluation of Licensee Strategies for the Prevention and/or Mitigation of Emergencies at Fuel Facilities". The stated purpose of the instruction was to "...independently evaluate the preventive and mitigative strategies and associated procedures to minimize the consequences of selected safety/licensing basis events and to review the adequacy of those emergency prevention and/or mitigation strategies for dealing with the consequences of selected beyond safety/licensing bases events." The inspectors performed a review of licensing basis documents and safety analyses to determine the facility design and licensing bases as they relate to natural phenomena hazards (NPH). Specifically, the inspectors evaluated the following hazards: earthquakes, high winds, flooding, and extended loss of power and water. The inspectors identified that B&W did not have adequate safety basis documentation to demonstrate compliance with the 10CFR70 requirements for NPH. The NRC opened an unresolved item, URI 70-27/2012-006-01, to further evaluate whether B&W is in compliance with the requirements of 10CFR70.62(c) and the performance requirements of 10CFR70.61 regarding accident sequences that are a result of natural phenomena events. This is being tracked under CA-201200689.

Out of this CA, COM-43657 was established

> Complete a Technical Work Record to justify why a detailed structural analysis is not necessary for outlying facilities (LTC Building B, Waste Treatment, and the Railyard Storage Building) to demonstrate compliance with 10CFR70 requirements for Natural Phenomena Hazards.

2.0 Background

NOG is geographically located at approximately 37.4 latitude and -79.2 longitude. According to the Virginia Tech Seismology Office, the two largest magnitude earthquakes recorded in Virginia occurred in Giles County on May 31, 1897 and in Louisa County on August 23, 2011. The 1897 earthquake was listed as VIII on the Modified Mercalli Intensity (MMI) Scale and was estimated as a 5.9 on the Richter Scale. The strength was estimated based on reports of cracked buildings and downed chimneys. For this earthquake, the Lynchburg area reported it was perceptible with bricks falling from chimneys and furniture and housewares being jostled. The 2011 earthquake was 5.8 on the Richter Scale with a MMI of VII in Louisa with an estimated MMI of IV in Lynchburg.

NOG-L contracted with Master Engineering and Designers to perform an engineering analysis on earthquake and wind loads. This was to establish a documented basis to show the main and A bays met the design basis in effect at the time of construction.

In parallel, NOG-L developed a basis that demonstrated the likelihood of a criticality is less than 1×10^{-6} per year which is incredible. Since it is incredible, no IROFS are credited.

The basis for this is the design basis earthquake has a frequency of 1×10^{-4} per year, the likelihood of fuel moving in to an optimum configuration is 1 in 10 and the likelihood of optimum moderation is 1 in 10 which results in a likelihood of 1×10^{-6} per year.

NUREG-1520 Section 3 permits external events such as earthquake with a frequency of occurrence which can be conservatively estimated at less than once in a million years to be considered incredible. Based on this, a criticality initiated by an earthquake is incredible on the NOG-L site.

Additionally, the consequences of a tornado strike on the facility based on NUREG/CR-4461, Rev. 2, "Tornado Climatology of the Contiguous United States" were documented in Ref. 2.

3.0 Master Engineers Structural and Wind Load Analysis

The assumptions and summary of the criteria for different time frames are listed below.

Prior to 1973

The first buildings at the Mt. Athos facility were designed and constructed in 1955. Prior to 1973 Virginia did not adopt a uniform building code and gave the local building officials the authority to define the applicable codes. Campbell County, where the facility is located, did not invoke a building code; however, according to the Engineer of Record (EOR) the older facilities were designed to Southern Building Code (SBC). The SBC during this time did not require seismic effects to be considered. The wind load was a uniform 20 psf.

The bays that were constructed during this time period are Bays 1M through 15M, 1T, 2T, BC, 1A. 5A, 6A and 7A. Also portions of bays 2A through 4A, 8A through 10A, 13A and 14A were constructed. The original gate house and some outlying buildings that are not considered during this assessment were also constructed during this time period.

Between 1973 and 1981

Virginia first formed the Virginia Uniform Statewide Building Code (VUSBC) in 1973 which adopted the BOCA 1970 as the construction standard. The BOCA 1970 required seismic effects to be considered. Seismic loads were determined based on a numerical constant that equated to a percentage of dead load depending on where the structure was in a zone map and the number of stories. For single story bays this would equate to a design acceleration of 0.033g. BOCA required a wind load of 15 psf. Virginia adopted the BOCA 1975 in 1976 and BOCA 1978 in 1978. These codes were not available for review at the time of the assessment; however, based on understanding of the progression of codes the design forces were similar to BOCA 1970.

The bays that were constructed during this time period are Bays 4A 2nd North Addition, Bay 10A, and Bay 1A/2A North Addition. The gate house addition was also constructed during this time period but is not considered in this assessment.

Between 1982 and 1990

VUSBC adopted the BOCA 1981 in 1982. This code incorporated a similar numerical coefficient as previous codes, except the percent of dead load was based on the zone, fundamental period of vibration and structure type. For single story bays this would equate to a design acceleration of 0.025g. The design wind pressure for the structures was based on wind speed maps and corresponding tables that equated wind speed and building height to pressure. The design wind load for a typical bay during this time period was 11 psf. In 1986 the VUSBC adopted BOCA 1984. This code's requirements for seismic and wind did not change.

The bays that were constructed during this time period are Bays 12A and 15A, a portion of Bay 16M, Bay 13 EAST Addition, and Bay 13/14 South Addition. The Rail Yard Storage Building was also constructed during this time period but is not considered in this assessment. In 1988 the VUSBC adopted BOCA 1987. There are no bays that were constructed under this code.

Between 1991 and 1997

VUSBC adopted the BOCA 1990 in 1992. This code incorporated the concept of an acceleration map instead of zones to determine the percent of dead load applied laterally on the structure. This code is also the first to introduce an importance factor for the structures and factors for the type of soil at the site. For typical bays the determined percentage would equate to a design acceleration of 0.042g. The design wind pressure also incorporated an importance factor in this code. The design wind load for a typical bay during this time period was 10.7 psf.

Bay 2A/3A North Addition was constructed under this code. In 1993 the VUSBC adopted BOCA 1993. There are no bays that were constructed under this code.

Between 1997 and 2003

VUSBC adopted the BOCA 1996 in 1997. This code was similar to the previous in that it considered an acceleration map along with a soil profile, type of building construction and fundamental period to determine a percentage of dead load to be applied in the lateral direction. For typical bays the determined percentage would equate to a design acceleration of 0.066g. The design wind pressure was determined similar to previous codes. The design wind load for a typical bay during this time period was 14.4 psf.

The bays that were constructed during this code are Bays 17M, the RTRT addition, and the overbuild on Bay 15A. Other buildings constructed but not considered in this assessment are the Project 2002 buildings and the ACF Buildings.

Between 2003 and 2009

VUSBC adopted the IBC 2000 in 2003. This code allowed for multiple methods of seismic analysis. The simplified equivalent static method used the same methodology as previous code editions to determine a percentage of dead load and permanent live load to apply laterally to the structure. The mapped accelerations for both a short and long period were determined and then converted into a design acceleration. The design acceleration was then reduced by a response factor which was based on the building construction type and increased by an importance factor. The resulting coefficient was a percentage that was applied to the dead load and permanent live load of the structure. For the typical bay type this percentage would equate to a design acceleration of 0.066g. The design wind pressure was based on mapped wind speeds, coefficients that took into account surrounding terrain and importance factors based on the use of the building. The resulting design wind load for a typical bay during this time period was 14.8 psf.

In 2005 the VUSBC adopted the IBC 2003 code. The seismic and wind requirements did not change from the IBC 2000 code. An addition to RTRT was constructed during this time period. Other buildings constructed under the provisions of these codes include Station One (pre-engineered building) and the Container Storage Facility.

In 2008 the VUSBC adopted the IBC 2006 code. The seismic requirements were determined the same way as for IBC 2003; however, the acceleration maps varied slightly. The design wind loads stayed the same. There were no buildings constructed under this code.

Current Applicable Design Code

Currently the VUSBC references the IBC 2009, which became effective in 2011. The seismic requirements are similar to those that were required in IBC 2006. When using the equivalent static load method, the resulting acceleration for a typical bay is 0.083g. The IBC 2006 refers to the ASCE Standard 7-05: Minimum Design Loads for Buildings and Other Structures (ASCE 7-05) for gravity and lateral design loads. The current steel design code is the American Institute of Steel Construction (AISC): Manual of Steel Construction, 13th Edition which allows the use of either the allowable stress design (ASD) or the load and resistance factor design (LRFD) methodology. Bay 3T was designed and is being constructed under this code.

The existing Main and "A" Bay buildings were analyzed using ASCE 31-03, "Seismic Evaluation of Existing Buildings".

The results of the evaluation indicated the following:

- Seismic loads have increased over the years.
- The oldest structures (buildings prior to 1973) did meet a seismic resistance of approximately 0.03g even though the code of record did not require seismic design.
- All structures after 1973 were designed for seismic loads in accordance with the design code of record.

Table 1: Summary of the Time Frame of Building Codes and External Loads

Year	Virginia Code	BOCA/IBC	Seismic S_A (%g)	Wind (psf)
Pre-1973	None	EOR used SBC	N/A	20
1973	VUSBS 1973	BOCA 1970	3.3	15
1982	VUSBS 1981	BOCA 1981	2.5	11
1991	VUSBS 1990	BOCA 1990	4.2	10.7
1997	VUSBS 1996	BOCA 1996	6.61	14.4
2003	VUSBS 2000	IBC 2000	6.6	14.8
2005	VUSBS 2003	IBC 2003	6.6	14.8
2011	VUSBS 2009	IBC 2009	8.3	14.8

Table 2: Summary of Time Frame of Building Construction

Time Frame	Buildings Constructed
Prior to 1973	Bays 1M through 15M, 1T, 2T, BC, 1A. 5A, 6A and 7A, parts of 2A through 4A, 8A through 10A, 13A and 14A
1973 to 1981	Bays 4A 2nd North Addition, Bay 10A, and Bay 1A/2A North Addition
1982 to 1990	Bays 12A and 15A, a portion of Bay 16M, Bay 13 EAST Addition, and Bay 13/14 South Addition, and Rail Yard Storage Building
1991 to 1997	2A/3A North Addition
1997 and 2003	Bays 17M, the RTRT addition, and the overbuild on Bay 15A, and ACF
2003 and 2009	An addition to RTRT, Station One (pre-engineered building) and the Container Storage Facility.

The conclusion of the analysis is that most of the bays would survive an earthquake of the strength specified in IBC 2009 and allow safe evacuation of the facility.

Table 3: Summary of ASCE 31-03 Analysis

Criteria	Bays
Met IBC 2009	Bays 1 - 4, 6, 8 - 17, BC - 6, 8 - 13, 15
Met 1973	Bays 5, 7, 7A, and 14A

Conclusions from the Master Engineers Report

The results of the evaluation indicated the following:

- Seismic loads have increased over the years.
- The oldest structures (buildings prior to 1973) did meet a seismic resistance of approximately 0.03g even though the code of record did not require seismic design.
- All structures after 1973 were designed for seismic loads in accordance with the design code of record.
- The facility is adequate for the current design wind load.
- The bracing failed in several bays from the increased seismic loads.

It was not stated in the report, but verbally communicated that it is reasonable that the structure is sufficient to allow most personnel to safely evacuate.

4.0 Buildings Not Addressed by Master Engineers

Railyard Storage Building

The Railyard Storage Building was constructed in 1990. It was designed to the code in effect at the time. It is a steel building with a sheet metal skin. As with the other similar style buildings it is expected to survive with the loads of the 2009 code.

A criticality is incredible based on the same basis described in Ref 1.

Waste Treatment

Consistent with the design of the Main and A Bays, the Waste Treatment facility would have been designed according to the building codes in place at the time. The facility does not handle more than a minimum critical mass of ^{235}U. As such, a criticality is not possible due to any natural phenomenon event.

Retention Tanks Building

Portions of the building are block construction and portions are steel structure with sheet metal. It is reasonable to assume it was designed consistent with the building codes in effect at the time of construction.

There are six tanks, each are approximately 5000 gallons. The solution is measured via an inline monitor prior to entry to a tank. This limits the concentration to 0.04 g $^{235}U/l$. The solutions are typically acidic which helps keep the uranium in solution. If tanks were to fail as a result of an earthquake, the liquid would fill the containment around the tanks and eventually overflow. Since the concentration is well below the minimum critical concentration of approximately 11.6 g ^{235}U/l, a criticality would not result.

LTC

LTC Building B is a block structure built prior to 1973. It would be expected to meet the 1973 building code. Being a block structure, it is possible damage from a large earthquake could occur, but the damage would not be significant.

The Building B labs, storage tubes and Hot Cells 2, 3, and 4 are limited to 350 g ^{235}U equivalent. As such, rearrangement of the fuel in a lab could not result in a criticality since there is not a minimum critical mass present. The vault in Building B does have multiple units in a rack system. Criticality would be incredible based on the same reasoning described in Ref. 1.

Hot Cell #1 is allowed up to three units. Three units is 1050 g ^{235}U equivalent. This is more than a minimum critical mass when water reflected, but less than a critical mass without water reflection. Criticality would be incredible based on the same reasoning described in Ref. 1.

5.0 Generic Discussion of Other NPH Events

Wind

The analysis by Master Engineers concluded:

> Wind loads will be determined in accordance with Chapter 6 of ASCE 7-05. The basic wind speed for the facility is 90 mph. The importance factor for wind on essential facilities is 1.15. The facility was demonstrated to be adequately designed for the wind load.

The analysis of wind speeds based in Ref. 2, provides the probability of a given wind speed from a tornado striking the facility.

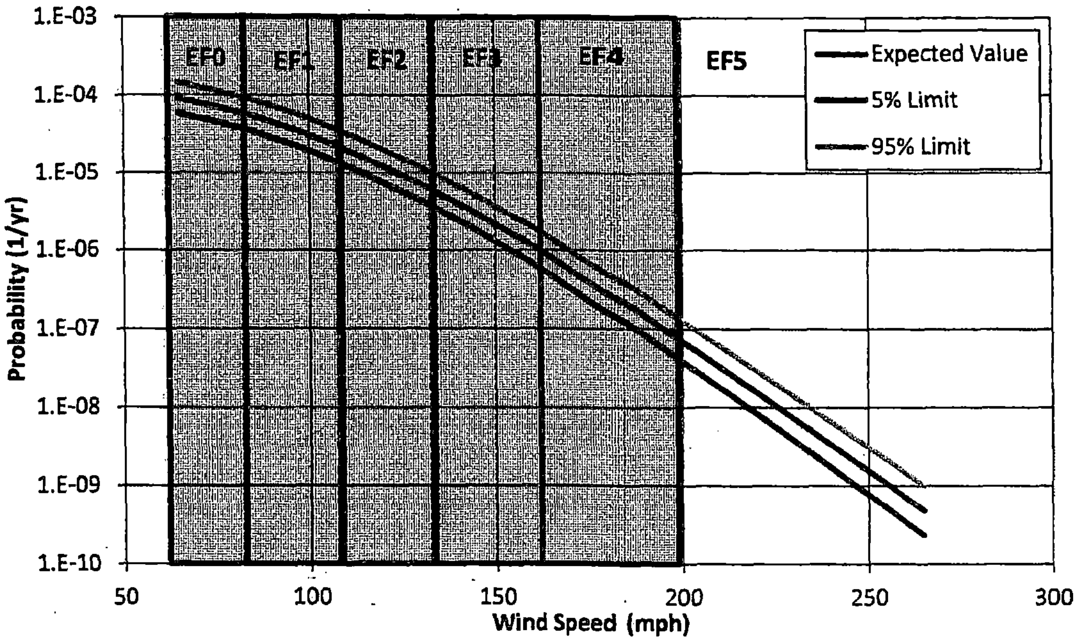

Figure 1: Total Probability for NOG-L based on the Eastern US

The mean probability of tornado with wind speeds of more than 90 miles per hour is approximately 5×10^{-5} per year.

The buildings on this site would survive a 90 mph wind. As noted, a tornado with wind speed of 90 mph has a probability of 5×10^{-5} per year. In order to cause significant damage to a building stronger winds are required, but the probability would be even lower. Substituting the 5×10^{-5} per year into the formula for overall likelihood in the Section 2, the likelihood of a criticality from a strong wind is 5×10^{-7} per year which again is incredible.

Flooding

The Facility is located on an elevated bend in a shallow gradient portion of the James River. The plant elevation is located substantially above the James River. The main site is approximately 500 feet MSL at its lower (plant north) fence line, with the principal facilities at 568 MSL or above. Most operations are not at risk of flooding. Past flooding events did not impact the NOG plant proper or the LTC. Critical facilities (active Uranium storage and processing portions of the site) are located more than 60 feet above the Standard Project Flood (SPF) level of 502 feet MSL and more than 40 feet above the expected Probable Maximum Flood (PMF) of 2 X SPF flow rate or 521 feet MSL for a Probable Maximum Flood ("highly unlikely" or < 10-5 by NUREG - 1520-Rev.1 page 3-D-1 definition) event without dam breaches and well above the </=523 feet MSL for a PMF event with dam breaches and wave action. In terms of Nuclear Power Plant Design Criteria -Standard Review Plan NUREG -0800, the facility is dry.

Two encapsulated material storage building first floors (Rail yard Storage and Container Storage) are at the Standard Project Flood (SPF) level of ~ 502 feet MSL, but higher than the maximum flood levels seen in historical floods over a 240 year period. Nuclear Criticality Analysis has demonstrated criticality control of these materials is not affected by total inundation, let alone partial flooding of these stored materials.

Lightning Strike

Electrical storms occur often during the spring and summer months resulting in minor damage to the site several times a year. The damage usually manifests itself in the form of failure of production or security equipment and the tripping of electrical breakers. Lightning protection is provided for most sensitive systems. However, no protective equipment exists that is 100% effective against lightning strikes. There are no indentified criticality scenarios that are a result of a lightning strike.

6.0 References.

Reference 1: NCS-TR-00002, Rev. 0, "Likelihood of a Criticality Accident Initiated by a Seismic Event at Mt. Athos," 10/16/2012

Reference 2: NCS-TR-00001, Rev. 0, "Probability of a Tornado Strike at NOG-L," 10/26/2012

Reference 3: "SEISMIC EVALUATION REPORT For Babcock & Wilcox Mt Athos Facility Lynchburg, Virginia," prepared by Master Engineers and Designers, Rev. 0 dated May 23, 2013.

7.0 Peer Review.

I have reviewed this technical work record. I have verified that:

- Design information and construction timelines included in this document is accurate.
- The methodology of the work record is logical and supports the drawn conclusions.
- Discussions on handling of fuel and the quantities of fuel processed in specific areas are accurate according to existing established NCS operating limits.