Vogtle PEmails

From: Davis, James T. [JTDAVIS@southernco.com]
Sent: Thursday, October 02, 2008 11:43 AM
To: Christian Araguas
Cc: Prunty, Robert; Damm, John; Pierce, Chuck R.; Prebula, John S.; Waites, Brandon Wiley
Subject: Proposed 2_5_4 Design Reference Changes (2).doc
Attachments: Proposed 2_5_4 Design Reference Changes (2).doc

Christian,

Proposed changes for discussion on today's phone call. Removed DCD/TR references and used Reg. Guides or industry standards.

<<Proposed 2_5_4 Design Reference Changes (2).doc>>

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From: Davis, James T.

Created By: JTDAVIS@southernco.com

Recipients:
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Proposed 2.5.4 Design Reference Changes

2.5.4.5.3 Backfill Design

(4th paragraph)

The Phase I test pad program is complete and is documented in Appendix 2.5D. The objective of this program was to establish site-specific design properties for the backfill, including density, compaction, gradation, and shear wave velocity, and to show that the backfill will satisfy the AP1000 standard plant design ~~siting criteria~~ ~~**(WEC 2007)**~~. The test pad was constructed below grade, was 20 ft deep, and was 20 ft x 60 ft in plan area. The test pad was constructed in the switchyard borrow area using methods similar to those used to construct the backfill for VEGP Units 1 and 2. The placement and compaction of the backfill were monitored and tested. Results of the test pad program demonstrated that the siting criterion for shear wave velocity of 1,000 fps at the NI foundation depth was achieved with the backfill material within the 20 ft thickness of the test pad.

2.5.4.10.1 Bearing Capacity

All structures in the power block footprint will be founded on the structural backfill compacted to a minimum of 95% (ASTM D 1557) as presented in Section 2.5.4.5. The structural backfill will be about 90 ft thick in the power block area. The containment and auxiliary buildings will be founded at a depth of about 40 ft below grade (about 50 ft of structural backfill beneath the foundations). For calculation purposes, the containment building mat was modeled as a circle with a diameter of about 142 ft placed at a depth of 39.5 ft below finish grade. Other structures will be founded at an approximate depth of 4 ft below grade. The allowable static bearing capacity values are based on Terzaghi's bearing capacity equations using an internal angle of friction of 36° for the compacted backfill as developed from field and laboratory testing of borrow materials during the Phase I test pad program (Appendix 2.5D) and the COL investigation (Appendix 2.5C). The influence of the Blue Bluff Marl on the allowable bearing pressure was evaluated using procedures outlined by Vesic (1975). With a factor of safety of 3.0 **(Bowles 1982)**, site conditions provide an allowable bearing pressure of 34 ksf under static loading conditions for the containment and auxiliary buildings. The allowable bearing capacity values for foundations placed on compacted fills at depths of about 4 ft below finished grade are provided in Figure 2.5.4-13.

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The allowable bearing capacity of the containment and auxiliary buildings under dynamic loading conditions was also evaluated. Analysis methods were based on Terzaghi's bearing capacity equation for general shear using seismic bearing capacity factors **(Soubra 1999)** and Terzaghi's bearing capacity equation for local shear. With a factor of safety of 2.25 **(IBC 2006)**, site conditions provide an allowable bearing pressure of 42 ksf under dynamic loading conditions for the containment and auxiliary buildings.

The results of settlement analyses are presented in Section 2.5.4.10.2.

2.5.4.11 Design Criteria

Applicable geotechnical-related design criteria are ~~provided in the AP1000 DCD **(WEC 2007)** and are~~ discussed in various sections of the SSAR~~. The criteria~~and are summarized below ~~are considered geotechnical-related criteria~~.

Section 2.5.4.8 specifies that the acceptable factor of safety against liquefaction of site soils should be ≥ 1.1 in accordance with Regulatory Guide 1.198.

Bearing capacity criteria are presented in Section 2.5.4.10. A minimum factor of safety of 3 is used when applying bearing capacity equations **(Bowles 1982)**. This factor of safety is also applied against breakout failure due to uplift forces on buried piping. For soils, this factor of safety can be reduced to 2.25 when dynamic or transient loading conditions apply **(IBC 2006)**.

Section 2.5.5.2 specifies that the minimum acceptable long-term static factor of safety against slope stability failure is 1.5~~. Section 2.5.5.3 specifies~~ and that the minimum acceptable long-term seismic factor of safety against slope stability failure is 1.1 **(Duncan 2005)**.

Appendix 2.5E describes the site-specific analyses that have been performed to show the acceptability of the AP1000 plant at the Vogtle site.

Section 2.5.4 References

(selected)

(Duncan 2005) Duncan, J.M., and S.G. Wright, *Soils Strength and Soil Stability*, John Wiley and Sons, Inc., New Jersey, 2005.

(IBC 2006) International Building Code, International Code Council, Inc., 2006.

~~**(WEC 2007)** "AP 1000 Design Control Document, Revision 16," Westinghouse Electric Company LLC., Pittsburgh, PA, May 2007.~~

2.5.5.2 New Slopes

(3rd paragraph)

The proposed permanent non-safety-related slopes will be analyzed for dynamic and static conditions during the design stage. The minimum acceptable factors of safety against stability failure of permanent slopes are 1.5 for long-term static conditions and 1.1 for long-term seismic conditions **(Duncan 2005)**. The construction excavation cut slopes will be analyzed for static conditions during the design stage. The minimum acceptable factor of safety against stability failure of excavation slopes is 1.3, based on what was used for Units 1 and 2. These analyses will be performed to ensure that these slopes will not pose a hazard to the public. Such analyses are not part of the ESP SSAR.

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