Sengupta, Abhijit

From: Williams, Charles R. [Charles.Williams@pgnmail.com]
Sent: Thursday, December 31, 2009 4:37 PM
To: Lake, Louis; Thomas, George; Carrion, Robert; 'nausdj@ornl.gov'; Pugh, C-Glenn; 'daniel.fiorello@exeloncorp.com'
Subject: FM 1.5 Draft for Review
Attachments: FM 1.5.pptx; FM 1.5 Exhibit 1 Glenn-Craig email.pdf; FM 1.5 Exhibit 2 Containment Exterior Surface Survey Data.pdf; FM 1.5 Exhibit 3 RB Foundation Settlement discussion per FSAR.pdf

1 page 1 page 4 pages 2 page

Mr Lake and others,

Attached for your review is draft of FM 1.5 with Exhibits. Contact either me or Craig Miller with questions.

Thank you,
Charles Williams
919-516-7417

The message is ready to be sent with the following file or link attachments:

FM 1.5.pptx
FM 1.5 Exhibit 1 Glenn-Craig email.pdf
FM 1.5 Exhibit 2 Containment Exterior Surface Survey Data.pdf FM 1.5 Exhibit 3 RB Foundation Settlement discussion per FSAR.pdf

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P/126

The possibility of active solutioning occurring during the life of the structure has been considered as reported in Section 2.5.3.4. It is noted that these studies conclude that the present groundwater environment is not conducive to active solutioning.

2.5.7 FOUNDATION ANALYSIS

2.5.7.1 Loading Conditions

Class I structures are constructed to bear at various elevations ranging from 56.33 feet, in the nuclear service seawater pump pit area of the auxiliary building, to 91 feet in the turbine generator building area, to 112.5 feet for the diesel driven emergency feedwater pump building. The reactor building comprises the most heavily loaded plant unit, being supported by a 12½ foot thick, 147 foot diameter foundation mat, bearing at elevation 80.5 feet.

The average unit loading of the reactor building under operating conditions is reported to be about 7.8 ksf. Contact pressures were computed for the following static loading cases:

a. Dead load + prestress
b. Dead load + prestress + 1.5 loss-of-coolant accident pressure (1.50P)

The computer program used to obtain the results, modeled the mat as a thin circular plate and the soil was a Wickler type material (vertical springs - no interaction between springs).

For these cases the maximum contact pressures were 10.3 and 23.4 ksf, respectively.

The average unit pressures imposed by other plant units generally range between 2.5 and 7 ksf. The nuclear service seawater pump pit area which has been carried down to a base elevation of 56.33 feet imposes a gross unit loading of 8.3 ksf although the net imposed pressures are significantly less due to the considerable excavation unload.

2.5.7.2 Foundation Analysis

The bearing capacity of the foundation materials was analyzed to evaluate the deep crushing potential of the least competent foundation member within the Inglis Member - the Differentially Cemented Limerock. The analysis consisted of a "worst case" approach, considering that the entire foundation system above the dolarenite will respond as a weakly-cemented sand, containing discontinuities in the form of very loose zones of infill and/or cavities, of limited horizontal extent. The analysis investigated the required shear strength, with depth, to produce an adequate safety factor against local shear failure under operating loads imposed by the reactor building foundation system.

Comparison of the imposed loading with the conservatively estimated shearing strength of the foundation materials indicated that an adequate factor of safety against a bearing capacity failure would be achieved under the most unfavorable conditions which could be reasonably postulated. This conclusion, however, was predicated on the assumption that all significant voids occurring above elevation +30 feet would be filled so as to minimize local overstressing and possible future progressive failure.

Two basic criteria were used to establish the fact that all voids were adequately filled by consolidation grouting. They are:

a. Unit take of closure holes
b. Permeability tests

Based on grouting operations performed on Crystal River Unit 2, it was found that a tertiary unit take of 1.2 cubic feet/foot or less, averaged over the entire length of the hole, assured that all significant voids were adequately filled with grout. If the tertiary unit take exceeded 1.2 cubic feet/foot, quaternary holes were drilled in the offending area. The quaternary unit take was limited to 0.8 cubic feet/foot averaged over the entire length of the hole. Out of 1,833 consolidation holes, 846 were tertiary holes, 106 were quaternary holes and only one hole was a quinary hole. All unit take limitations were met.

Permeability tests are used as a post grouting testing procedure. The permeability of the foundation after grouting must be 7×10^{-3} cm/sec or lower. This figure was determined from extensive testing on the Crystal River Unit 2 foundation. Based on these tests coupled with direct observation of the foundation during excavation, it was determined that at a permeability of 7×10^{-3} cm/sec or less, the foundation was saturated with grout. Additional proof came from the fact that permeabilities were reduced from 10 cm/sec (ungrouted foundation) into the range of the primary permeability.

Out of 45 holes tested there was only one unaccountable failure. This test failure was believed to be attributable to internal leakage and failed by so little as to be considered negligible. There was no doubt, based upon the preceding, that the foundation was thoroughly grouted and all significant voids were filled.

The peak contact pressure of 23.4 ksf under the static loading condition of dead load + prestress + 1.5 x loss-of-coolant accident pressure gave a minimum factor of safety against bearing capacity failure of at least four. The factor of safety is controlled by the Differentially Cemented Limerock Member with a minimum shear strength of 18 ksf. The influence of seismic loading on shear strength and therefore on bearing capacity of the foundation material, characterized by the differentially cemented limerock and the dolomite, does not make it susceptible to a significant reduction considering the intensity and duration of the seismic loading imparted by the design earthquake. The influence of seismic loading on bearing capacity would not be critical considering that the ultimate bearing capacity of the foundation material is on the order of 100 ksf and that a factor of safety of 1.5 would yield a bearing value of at least 70 ksf.

A bearing capacity analysis for accident pressure conditions using strength parameters derived for static loading conditions indicates a reduced factor of safety against the bearing capacity failure. However, considering the transient nature of the accident loading, a bearing capacity failure would not be anticipated under accident pressure conditions.

A settlement analysis of the reactor building under static and wind loading was conducted using two multi-layered foundation models to investigate both total and differential settlements. Using very conservatively derived foundation parameters, differential settlements under the most unfavorable conditions which could be postulated indicated maximum angular distortions would be less than 3 to 4×10^{-4} radians. The corresponding upper limit total settlement, occurring at the center of the semi-rigid foundation mat, was found to be on the order of 7/8 inches.

Considering the load distribution characteristics of the superstructure, it was concluded that the estimated upper limit of total settlement would in all probability not be realized and that all but a very small fraction of settlement may be essentially elastic and would occur during construction. The total and differential settlements occurring after installation of equipment or instrumentation which would be sensitive to slight movement would therefore be expected to be a very small fraction of the estimated values.

To limit foundation settlements to within the order of magnitude defined by analytic studies, it was concluded that it would be necessary to excavate the irregular and occasionally low density surficial subsoils and decomposed rock. A foundation treatment consisting of excavation of unsuitable bearing materials and grouting of the solutioned rock system was derived.

Based on above information it appears RB settlement is not considered an issue. CR3 is also considered to be founded on rock

Containment Exterior Surface Survey Data

Surveys were performed of the containment dome and buttresses (field date 11/21/09). Attachment 1 includes a map of the dome and buttresses. The dome survey was performed to identify if there are significant changes on the surface of the dome by comparing the current survey data to the final dome survey performed in 1981. The dome was surveyed between 1977 and 1982 as technical specification surveillance required based on the dome delamination event. Procedure SP-180, Reactor Building Structural Integrity Dome Surveillance Program, was initiated to perform a survey of the dome to identify changes in dome elevation and an inspection of the dome surface identifying crack width and crack pattern. The final surveillance was performed in 1981 with an additional survey performed 3 months later due to exceeding acceptance criteria. The buttress survey was performed to determine the relative position of the buttress corners.

Dome Survey

The current survey of the dome was performed using SP-180 as a guide. The original benchmark and survey point pins were found on the surface of the dome. Elevations were taken at each of the benchmarks and survey points. Delta elevations were determined by subtracting the elevation of each survey point from the average of the three benchmark elevations. The change in elevation is shown below as well as the original acceptance criteria and the results from the last survey performed in 1982.

Survey Point Location No.	Change in ΔEL ft.(inches) 2009	Change in ΔEL ft. (inches) 1982	% Change 2009 to 1982	ΔEL Acceptance Limit ft.+/- (inches)
1	-0.056 (0.672)	-0.054 (0.648)	+0.04	0.030 (0.360)
2	-0.059 (0.708)	-0.042 (0.504)	+0.29	0.030 (0.360)
3	-0.064 (0.768)	-0.044 (0.528)	+0.31	0.030 (0.360)
4	-0.060 (0.720)	-0.050 (0.600)	+0.17	0.030 (0.360)
5	-0.037 (0.444)	-0.025 (0.300)	+0.32	0.025 (0.300)
6	-0.014 (0.168)	-0.019 (0.228)	-0.36	0.025 (0.300)
7	-0.024 (0.288)	-0.027 (0.324)	-0.13	0.025 (0.300)

In a letter dated 2/23/82 from Gilbert Associates, Inc. (GAI) to Florida Power Corporation, GAI concluded that the structural integrity of the dome was not adversely affected by the measured deflections outside of the Technical Specification acceptance limits. The deflections were considered to be indicative of a seasonal variation in thermal deflections of the structure, which are practically impossible to accurately predict. Similar to the 1982 survey, the 2009 survey was performed with dome apex surface temperature and internal ambient temperature within approximately 10°F. The baseline delta temperature was 50°F.

The % change from the 1982 survey is considered insignificant with respect to detecting a change in the structure similar to the delamination found between buttress 3 and 4. In addition, a review of the boroscope video of the seven core bores in the dome did not find any delamination.

Buttress Survey

Unlike the dome survey, the buttress survey does not have a historical procedure that contains baseline information or acceptance criteria. The buttress survey is used to determine the relative position of the outermost surface of the buttress at the corner of the buttress adjacent to the tendon bearing plate. Attachment 1 identifies the buttress corners that were within line of sight during the survey. Both corners of buttresses 1, 2, 5, and 6 were visible. One corner of buttress 3 was not visible; therefore, the surveyors chose to survey the face of the buttress and the containment wall at the buttress to wall interface. Only the buttress survey data at buttress 3 will be evaluated. Buttress 4 only had one corner visible.

The survey data consists of three coordinates, N/S (x), E/W (z), and elevation (y). The relative position of each buttress was determined by calculating the lateral offset and angle of verticality. The angle is determined using the x and z coordinates of the lowest and the highest reading to calculate a Δx and Δz. These dimensions are used with the difference in elevation between the lowest and highest reading to calculate the angle of verticality.

Buttress ID	Lateral Offset ft.(inches)	Survey Length (ft.)	Angle of Verticality
B1a	0.1122 (1.3461)	59.912	0.1071°
B1b	0.2776 (3.3312)	95.874	0.1659°
B1b*	0.2023 (2.4277)	89.118	0.1301°
B2a	0.1974 (2.369)	77.870	0.1452°
B2b	0.0435 (0.5220)	76.143	0.0327°
B3a	0.2175 (2.610)	53.106	0.2346°
B4	0.0375 (0.4500)	76.027	0.0283°
B5a	0.1760	80.705	0.1250°

Buttress ID	Lateral Offset ft.(inches)	Survey Length (ft.)	Angle of Verticality
	(2.1120)		
B5b	0.1263 (1.5156)	43.684	0.1656°
B6a	0.1126 (1.3512)	48.477	0.1331°
B6b	0.0883 (0.9996)	73.984	0.0684°

*The top two survey points were compared and found to have a lateral offset, from the highest to lowest survey point, of 0.9" in the East direction over a length of 6.8'. Surface variations exist that can cause a shift in lateral locations. The lateral offset for this location is reduced to less than 3" by eliminating the highest survey point.

The vertical alignment requirement for cast-in-place concrete for buildings for heights greater than 100 ft. is 1/2000 times the height but not more than 3 in. at outside corner of exposed corner columns and control joint grooves in concrete (Ref. American Concrete Institute (ACI), 117-90, Standard Specifications for Tolerances for Concrete Construction and Materials, Section 4.1, Vertical Alignment)

An examination of the outermost surface of the buttresses identified the following conditions:

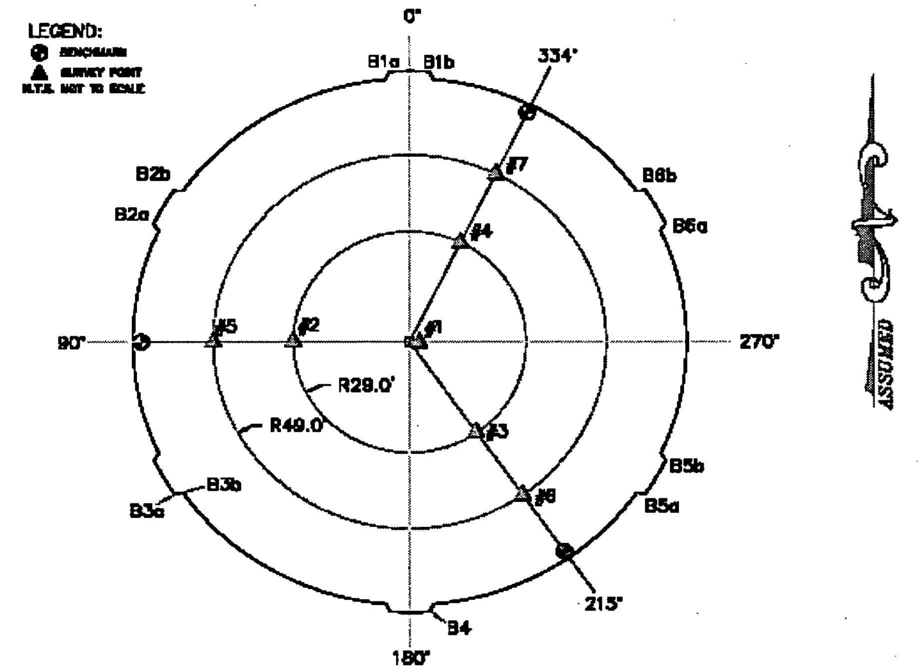
- Uneven surfaces between placements along form edges
- Cosmetic grout repairs along the face of the buttresses along the corner causes a radial change in to or out of the plane of the buttress
- Corners exhibit loss of cover concrete along the tendon bearing plate area, which causes a shift in lateral location of the corners

These conditions affect the accuracy of the survey data. As noted above, eliminating one survey point at Buttress ID B1b reduced the lateral offset by 0.9". The vertical alignment tolerance provided in ACI 117-90 is considered to be satisfied based on the localized surface variations affecting the accuracy of the survey data.

Prepared By: Martin E. Souther, PE Structural System Engineer

Reviewed By: Aaron Mallner, PE Engineer I and Bill Bayrd, PE Lead Engineer

Attachment 1



ENCLOSURE 1

ELEVATION BENCHMARK 90°	ELEVATION BENCHMANRK 215°	ELEVATION BENCHMARK 334°	#1	#2	#3	#4	#5	#6	#7
267.490	267.491	267.500	285.106	281.245	281.227	281.106	274.244	274.306	274.311

SURVEYORS NOTES:

1. This is not a boundary survey. The Purpose of this survey is to provide verticle and horizontal data on the outside of the reactor as directed by the client.
2. The elevations shown hereon are assumed as provided by the client.
3. For buttress data, see accompanying spreadsheet titled 08017A04.xls.

CERTIFIED TO: Progress Energy, Crystal River, Florida

Wallace L. Higgins, P.S.M. for GulfWest Surveying, Inc.
Florida Surveyor and Mapper registration no. 5632
Florida licensed business number 7314
Not valid without the signature and original raised seal of a Florida licensed surveyor and mapper.

SPECIFIC PURPOSE SURVEY FOR PROGRESS ENERGY

Citrus County, Florida

Sheet 1 of 1

GulfWest Surveying, Inc.
Professional Surveyors and Mappers
9469 W. Green Bay Lane
Crystal River, Florida 34428
Ph: 352.583.1252 Fax: 352.583.1253

From: Pugh, C-Glenn
Sent: Thursday, October 22, 2009 8:41 AM
To: Miller, Craig L
Subject: FW: RB Settlement

Craig,

To help close the loop on RB settlement I talked to several plant personnel involved in Maintenance, Ops, I&C Engineering, Civil Engineering, etc and no one can ever remember any instruments or programs for monitoring RB settlement. Our FSAR contains a statement that predicted settlement is essentially ignored and not a concern.

Consider this issue closed.

Glenn Pugh

1.5 Foundation Settling

Description: Foundation settling can cause added, asymmetrical stress in certain area in the containment.

Data to be collected and Analyzed:

1. Review of previous condition reports involving foundation cracks. No condition reports written since no foundation cracks were observed.
2. Review e-mail from Glen Pugh to Craig Miller, 22 October, 2009. (FM 1.5 Exhibit 1)
3. Buttress and Dome survey data. (FM 1.5 Exhibit 2)
4. Review FSAR Section for containment foundation. (FM 1.5 Exhibit 3, FSAR Chapter 2, Section 2.5.7)

Verified Refuting Evidence:

a. None of the personnel contacted by Mr. Pugh was able to remember an incident where equipment required adjustment because of settlement issues. (FM 1.5 Exhibit 1)
b. Buttress surveys do not have historical baseline data for comparison, but looking at the tilt or plumbness may indicate foundation settling. On buttresses that could be measured on both sides, a large difference in measured tilt is judged to be a result of construction tolerance rather than foundation settlement. Buttress #2 was measured as having a lateral offset of 4.17 inches on one side and 0.522 inch on the other. Foundation settlement would result in tilt at both locations. Buttress #4 was measured with very small tilt of 0.45 inch. No foundation settlement indicated. (FM 1.5 Exhibit 2)
c. Dome survey data compares elevation change data from 1982 and 2009. The measured change from 1982 to 2009 ranged from a +0.036 to – 0.240 inch. This movement is considered to be insignificant and not indicative of foundation settlement. (FM 1.5 Exhibit 2)
d. There is no record of containment cracking at or around the foundation consistent with the FSAR conclusions relative to the potential for foundation settlement. (FM 1.5 Exhibit 3)

Conclusion: There is no indication of containment settlement sufficient to cause added, asymmetrical stress in the containment.

Verified Supporting Evidence:

None

May identify additional perspective on this issue as RCA related efforts proceeds