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Sent:	Thursday, December 31, 2009 4:43 PM
To:	Lake, Louis; Thomas, George; 'nausdj@ornl.gov'; Carrion, Robert; Pugh, C-Glenn; 'daniel.fiorello@exeloncorp.com'
Subject:	Emailing: FM 1.6 Exhibit 3 FSAR Ch 2 Selected Pages.pdf, FM 1.6.pptx, FM 1.6 Exhibit 1 Gilbert Study Section 1.2.11.pdf, FM 1.6 Exhibit 2 Containment Exterior Surface Survey Data.pdf
Attachments:	FM 1.6 Exhibit 3 FSAR Ch 2 Selected Pages.pdf; FM 1.6.pptx; FM 1.6 Exhibit 1 Gilbert Study Section 1.2.11.pdf; FM 1.6 Exhibit 2 Containment Exterior Surface Survey Data.pdf

2 pages 9 pages 11 pages 4 pages

Mr Lake and others,

Attached for your review is the draft for FM 1.6 and its Exhibits. If you have any questions, please contact either Craig Miller or me.

Thank you,
Charles Williams
919-516-7417

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

FM 1.6 Exhibit 3 FSAR Ch 2 Selected Pages.pdf FM 1.6.pptx FM 1.6 Exhibit 1 Gilbert Study Section 1.2.11.pdf
FM 1.6 Exhibit 2 Containment Exterior Surface Survey Data.pdf

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P/125

Series TB and C borings in the turbine building area and the diesel generator area, were drilled between November 23, 1969 and December 18, 1969.

Phase III drilling, which included Boring Series CI, X, PH, ESP, and PGE, was conducted during construction using standard penetration and core sampling techniques.

The CI boring series was drilled to investigate a suspect foundation area detected north of the nuclear service seawater pump structure. This area was subsequently treated with chemical grout.

The X-series of holes was for the purpose of substantiating the adequacy of the foundation rock system beneath the decay heat pit and reactor building. The TB series of holes was for further exploration under the turbine building and the PH holes for the pump house foundation. The ESP & PGE holes were post-grouting holes to verify the condition of the foundation materials in two selected areas.

2.5.6 FOUNDATION CONDITIONS

On the basis of literature, field and laboratory studies, the subsoils and rocks within the nuclear power plant facility were found to be characterized by a sequence of surficial fill and irregularly stratified marine sediments underlain successively by the Inglis and Avon Park Limestones. The pertinent geotechnical characteristics of each of these generalized stratigraphic units are summarized as follows:

2.5.6.1 Surficial Fill and Terrace Deposits

The surficial fill and transported soil mantle were generally found to consist of silty and gravelly sands, silts, and clays all characterized by a variable and occasionally low density/consistency. These materials were also found to occasionally contain significant inclusions of organic materials (roots, humus, etc.) which would be subject to future decomposition. It was therefore concluded that the fill and soil mantle would exhibit an irregular and occasionally significant compressibility under load and would mobilize a variable and occasionally low resistance to shearing displacement.

2.5.6.2 Inglis Limestone

As discussed under Section 2.5.3.1.3, the Inglis Limestone has been subdivided into three distinct lithologic units generalized for purposes of analysis into:

a. Decomposed limerock: a surficial "weathering horizon" but occasionally occurring as interspersed zones within the rock mass.
b. Cap Rock: a relatively intact massive zone beneath the surficial zone.
c. Differentially Cemented Limerock which includes the middle and basal unit defined previously.

The supporting characteristics of the decomposed surficial zone of the Inglis Limestone were found to range from relatively poor to relatively good depending on the degree of alteration as correlated with density and resistance to penetration. These materials were also found to be subject to deterioration after a relatively short-term exposure.

The intact and relatively rigid Cap Rock Member, found (with local exception) to be continuous across the site, was rated as the most competent member of the Inglis Limestone. Thus the cap rock would be expected to be significantly stronger and less compressible than the other elements of the Inglis Formation.

The Differentially Cemented Limerock zone of the Inglis Limestone was characterized for analytic purposes as a fragmental, often friable and poorly cemented material, interspersed with strongly cemented, discontinuous strata and near-vertical, oriented discontinuities. The basal portion of the Differentially Cemented Limerock (and to a lesser degree the entire Limerock Member) was found to have a particularly heterogeneous lithology, to have been subject to intense solutioning and to contain subsequent secondary infilling.

Considering the extreme variation and engineering properties and the difficulty of obtaining data from the low yield zones, it was necessary to characterize the load-deformation response of the Differentially Cemented Limerock as a weakly cemented, discrete grained medium containing random discontinuities (solution cavities and altered rock zones). Because the Inglis Limestone Formation represents the most influential supporting member of the facility, extensive field and laboratory testing was conducted to investigate its response to foundation loading.

On the basis of in situ testing, wave propagation studies, and laboratory testing, it was concluded that the load-settlement response of the weaker elements of the Inglis Formation can be conservatively characterized by a Modulus of Deformation of 54 ksi as would be derived from the loading of a one foot diameter, rigid bearing plate seated on the foundation surface. It was concluded that the mobilizable shear strength of the Differentially Cemented Limerock will be dependent on confining pressure but, except for (infill and decomposed) materials associated with discontinuities within the formation, could be expected to have a (cohesive) shearing resistance on the order of 9 tons/ft^2, independent of confining pressure.

2.5.6.2.1 Avon Park Limestone

The Avon Park Doloranite Member generally encountered at depths usually in excess of 90 feet below the regional ground surface, was found to be the most uniformly competent foundation member. Although subject to solutioning, the doloranite was found to be rigid and relatively incompressible under the loads. Based on uniaxial compression test on representative core specimens, the average unconfined compressive strength was found to be on the order of 700 tsf.

2.5.6.3 Groundwater

The groundwater surface beneath the site fluctuates in response to tidal variation. The groundwater level beneath the area investigated was observed to usually range between elevation 88.0 feet and 90.0 feet, as measured daily in completed bore holes.

Periodic level observations indicate a response lag of approximately one-half to two hours between tidal crests, as measured in the intake channel and in the area studied. The amplitude of groundwater variation was found to be approximately 40% less than at sea level.

2.5.6.4 Solution Activity

Foundation conditions are strongly influenced by solutioning, particularly within the Inglis Formation. Solutioning producing cavities within the rock system has been most intense along a regional fracture system as described in Section 2.5.3.2. It has been concluded that the strength and compressibility of the foundation rock could be adversely affected by the influence of large discontinuities in the form of voids, compressible infill deposits or limerock highly altered by the proximity of solution activity.

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 x 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 x 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 x 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.

2.5.7.3 Foundation Treatment

To assure the continuity and integrity of the solutioned limestone within a specified depth extending downward from the bearing level of foundation elements, consolidation grouting using a cement base grout was accomplished subsequent to the removal of unsuitable surficial bearing materials.

2.5.7.3.1 Consolidation Grouting

The foundation grouting employed a peripheral grout curtain to aid in groundwater control and to provide confinement for subsequent interior consolidation grouting. Curtain grouting was conducted on a split-hole sequence with a maximum closure spacing of four feet and the interior consolidation grout pattern had a maximum final hole grid spacing of ten feet.

To be consistent with the foundation model, it was required that consolidation grouting extend down to at least elevation +30 feet. In accordance with specifications, (Ref 38), the grouting extended into dolomite and bottomed at an average elevation of +10 feet.

The foundation analysis concluded that post grout investigations should be conducted to document the effectiveness of consolidation grouting. When zones of questionable supporting abilities were encountered, supplemental chemical grouting was conducted. An appropriate silica-base grout and other approved chemical grouts with equal permeation and strength characteristics were used to stabilize materials which could not be penetrated with cement base grout.

2.5.7.3.2 Excavation and Groundwater Control

Subsequent to completion of curtain-wall grouting, unsuitable surficial materials were excavated down to the level of dense decomposed limerock, caprock, or dense differentially cemented limerock.

Subaqueous excavation utilized confined and unconfined excavation, the latter for conditions where the depth of excavation below water level was limited. A confined excavation (sheeted cofferdam) was used where conditions dictated. Bottom clean-out procedures included vacuum cleaning (air lifting) of any collected bottom sediments.

During the early phases of construction (excavation and placing of structural fill) and before consolidation grouting, dewatering was accomplished by means of 36 inch diameter pumped wells drilled to approximately elevation 25 feet. In addition, local surface sumping was used, where required, to facilitate placement of structural concrete fill in the dry.

At a later phase of construction, dewatering was accomplished in the decay heat pit area by gravity drawdown to filtered subdrains which discharge into a collection sump.

Dewatering was not necessary in the nuclear service seawater pump pit during excavation since a tremie concrete plug was placed within the sheet pile cofferdam and just below the bottom elevation of the structural mat. When the tremie plug was cured, the pit was dewatered to facilitate its construction in the dry.

2.5.7.3.3 Load-Bearing Fill

The foundation analysis concluded that excavated unsuitable materials could be replaced with load bearing fills suitable for support of foundation elements. Fill placed below groundwater level consisted of a crushed limestone aggregate (Zone l), suitably graded for underwater placement and for in-place grouting.

For above water placement structural fill concrete was used. Alternatively, a well graded, crushed limestone aggregate (Zone 2), which is capable of being compacted to a relatively high density, was available. Another material, friable crushed limestone (Zone 3), was used for placements outside of structure areas. The material quality requirement and compaction criteria of the three load bearing fill types were outlined by specifications (Ref 35).

2.5.8 FOUNDATION PREPARATION

2.5.8.1 Foundation Grouting

The foundation grouting program for Crystal River Unit 3 was begun in June of 1968 and was predicated on the grouting concept and procedures developed from Crystal River Unit 2 (Ref 39), the test grout area for Crystal River Unit 3 (Ref 40), and the recommendations of Woodward-Clyde & Associates (Ref 37).

Peripheral grout curtains were utilized around the main plant structure, the intake structure, and as supplements to existing curtain walls to the south and east of the main plant area.

In all cases, primary holes were located on approximately 32 foot centers and subsequently split-spaced down to the quaternary order. On occasion, quinary holes were drilled on either side of quaternary holes which did not close out properly. Grout curtains extended into the dolomite (average elevation +20 feet) underlying the site.

Grout mixes included 1:1 fly ash mix (cement: fly ash), limerock flour mixes (cement: fly ash: limerock flour), and neat cement. The 1:1 flyash mix was rarely used once the superior limerock flour mixes were developed. Eventually, the transition was made to pure neat cement grout and was used exclusively. The last two mixes precluded the need for a final waterproofing step.

The following unit grout takes depict the average of all curtain grouting performed:

Hole Order	Cubic Feet Per Foot
Primary	49.8
Secondary	9.8
Tertiary	4.1
Quaternary	1.2
Quinary	0.9

Consolidation grouting consisted of primary order holes on a 20 foot maximum grid spacing, with secondary and tertiary holes interspaced on a final maximum grid of ten feet. Grout mixes included various limerock flour mixes but these were subsequently replaced by neat cement grout. In accordance with the specification, all mixes conformed to the minimum strength requirement of 500 psi. The unit takes for the consolidation grouting are summarized as follows:

Hole Order	Cubic Feet Per Foot
Primary	10.6
Secondary	1.9
Tertiary	1.2
Quaternary	0.8

Field permeability tests were conducted throughout grouting operations and reflected consistent and satisfactory results.

2.5.8.2 Foundation Conditions

The results of the engineering geology investigation of the foundation rock system confirmed by construction observations revealed that the entire foundation system contains near vertically oriented fracture zones. Solutioning has occurred along bedding planes and particularly at interfracture areas, resulting in a network of essentially vertical solution channels, and a net gain of secondary porosity.

During construction, two major and two minor conjugate sets of fractures were traceable at the site and measurable in the excavations. One primary conjugate set consists of fractures parallel to the trend of the Ocala anticline (North 45° West) with cross fractures perpendicular to this regional trend. This fracture set is believed to have developed in response to tensile stresses resulting from the deformation associated with the Ocala Uplift, producing a regional joint system.

The second conjugate fracture set consists of a north-south trend with cross fractures of the set trending east-west. Two secondary conjugate fracture sets that were observed during excavation are oriented N60°W - N30°E and N30°W - N60°E. These are oblique cross fracture sets to the principal fracture systems and are considered to be the result of stress adjustment to the principal fracture systems.

The net effect of the fracturing in altering the rock mass is indicated in Figure 2-38. Contours of competent limerock surface generally bend around inter-fracture areas, indicating depressions or local variations in the soundness of the limerock from localized solutioning. The network of solution channels was found to be infilled with secondary deposits of fine quartz sand, silts, clays, and shell fragments. The secondary infill areas were observed to generally occur at inter-fracture areas, especially at the intersection of more than two fractures.

The contours of competent bearing material shown in Figure 2-46 were based upon subsurface information from the Phase I exploration program; a detailed Phase II boring program and subsequent field survey information based on actual final excavation.

The highly localized variations in the foundation rock system resulting from the fracturing, solutioning, and subsequent infilling, necessitated the tailoring of the excavation and backfilling techniques for each area of the facility to suit field conditions, and to the variation in unit loading across the facility.

Early excavation procedures revealed that the Cap Rock is not continuous across the site, as shown on the geologic sections in Figure 2-39 through Figure 2-44. A trough of eroded Cap Rock was encountered in the reactor building approximately 40 feet wide and extending down to elevation 68 feet. In the fuel handling and diesel generator areas Cap Rock was encountered; however, it was necessary to excavate unsuitable surficial material from localized pockets. In the southern portion of the auxiliary building Cap Rock was absent from the general area; however, the depth of the foundations in most of the area required that the foundations would be bearing on the Differentially Cemented Limerock of the Inglis Member, which is below Cap Rock level. It appears that the absence of Cap Rock is confined to two general areas, the 40 foot wide trough extending SSE through the western quadrants of the reactor building, and a second trough through the auxiliary building. The loss of the Cap Rock is believed to be the result of local fracturing, decomposition, and solutioning.

In keeping with the model concept established by Woodward-Clyde & Associates an analysis considered that the excavations and foundations would extend down to dense decomposed limerock when encountered above Cap Rock, to Cap Rock, or to dense differentially cemented limerock in the absence of Cap Rock. An acceptance criterion requiring a Standard Penetration Resistance of at least 30 blows/ foot for competent bearing material applicable to dense decomposed limerock or differentially cemented limerock was established by Woodward-Clyde & Associates to be consistent with the foundation analysis. The contours showing the elevation of competent bearing material in Figure 2-46 are based on this concept; the solid contours are based on actual excavation elevations and the dashed pre-excavation contours are based on drill hole data.

2.5.8.3 Reactor Building Foundation

Final design for the reactor building established that excavation be carried to elevation 80.5 feet beneath the foundation mat and to elevation 71 feet beneath the tendon gallery. During excavation the trough of eroded Cap Rock was encountered extending to below elevation 68 feet. At the completion of excavation, the base of the tendon gallery was at elevation 70 feet or lower and all Cap Rock had been penetrated beneath the gallery. The excavation bottomed on the differentially cemented limerock, which was highly resistant to further excavation.

Of the original natural foundation rock above elevation 70 feet, an estimated 30% to 35% was left in place at the conclusion of the excavation for the reactor building area. This is essentially a crescent-shaped pedestal consisting of a four to five foot thick layer of Cap Rock, underlain by differentially cemented limerock and sand lenses. This is adequate bearing material and was left in place as shown in Figure 2-49. The exposed foundation material in the eroded trough was dense differentially cemented limerock of adequate bearing capacity.

Most of the bottom of the excavation was covered with an uncompacted blanket of groutable coarse aggregate (Brooksville Limerock) which varies in thickness from 18 inches to three feet between elevations 67 and 70 feet. This material was required because of groundwater conditions and is within the foundation concept established for Class I structures (37). An impervious visquene membrane was placed on top of the coarse aggregate, and a load-bearing fill of 1,500 psi concrete was placed thereon to the bottom of the reactor mat at elevation 80.5 feet.

In order to grout the coarse aggregate, the first stage of the consolidation grouting in this area was drilled no deeper than the bottom of the coarse aggregate fill and pressure grouted. Subsequent grouting at deeper stages and high pressures subjected the aggregate fill to further penetration from all primary, secondary, and tertiary holes grouted in the area.

A leveling pad of structural concrete backfill, referred to as the mudsill, was placed over the native limerock and the coarse aggregate in several separate placements beneath the tendon gallery, varying in thickness from one to two feet. A visquene membrane was placed on top of the aggregate prior to concrete placement. Concrete blocks were mortared into position on the leveling pad as form walls for the gallery. Structural concrete was used to backfill behind the form walls outward to the walls of the excavation against competent bearing material, or to a distance equivalent to the height of the concrete fill. This fill was placed in several lifts to elevation 80.5 feet, on the bottom of the reactor mat. The placement sequence of concrete fill beneath the reactor building is shown in Figure 2-49.

2.5.8.4 Fuel Handling and Diesel Generator Building Foundations

In the fuel handling and diesel generator areas, structural concrete backfill was placed on dense to very dense material, as determined by detailed test borings and visual inspection. The procedure followed for the preparation of the foundation in these areas required the removal of all unsuitable surficial materials to approximately elevation 84 feet in the fuel handling area and 85.5 feet in the diesel generator area as established by detailed drilling. The above-water subgrade areas were then proof-rolled with a vibratory compactor capable of exerting 130 pounds of pressure per linear inch of roller. Dental work type excavation was performed to remove pockets of unsuitable infill materials, and the excavated pockets were backfilled with structural grout or concrete backfill to the surface contour. The area was hand cleaned to remove all loose surface materials, and groutable aggregate was placed in any localized depressions under water, which were also located in the plan for future pressure grouting. Structural concrete backfill was placed over the areas to the permanent base elevation of the foundation.

The use of a 30/70 cement/sand grout mixture and high slump structural fill concrete allowed the intimate filling of the surface irregularities and convolutions in the competent native limerock that were uncovered during the dental work operations. Both the grout mixture and the structural concrete have compressive strength greater than the native materials and therefore are considered as adequate backfill materials.

The locations of the dental work and the types of materials used for backfilling are shown on Figure 2-49.

2.5.8.5 Auxiliary Building Foundations

Foundation preparation in this area was similar to that in the fuel handling and diesel generator areas, requiring the removal of unsuitable surface materials and dental work where required. Additional excavation was performed where deemed necessary, after rough excavation and visual examination of the foundation material. Before final clean-up and placement of backfill materials the subgrade in accessible areas was proof-rolled with a 130 pound/inch vibratory roller. Any potentially compressible materials exposed by proof-rolling were removed and replaced by concrete backfill.

Where it was not feasible to use the vibratory compactor to prove the subgrade, the adequacy of the foundation materials was investigated by penetration resistance tests. One method utilized a 35 pound hammer to drive a one-inch diameter conical point. A requirement of 60 blows per foot was established for this test to conform to the 30 blow SPR criterion. The second method used a cone-penetrometer, which consisted of a 10 cm square cone mechanically advanced at a rate of approximately 2 cm/sec. The force applied and the depth penetrated were continually recorded throughout the test. The depth at which the cone resistance reached 750 Kg/cm^2 established the depth of suitable bearing material. The location at which the plug sampler and cone penetrometer tests were conducted are shown in Figure 2-47 and Figure 2-48.

After approval of the foundation material, 1,500 psi backfill concrete was placed immediately on the foundation. Where the approved material was underwater, Zone I (groutable aggregate) rock was placed to the top of water, covered with a visquene layer and the backfill concrete placed above the visquene. The maximum depth of groutable aggregate was approximately 4 feet.

Where springs were encountered, the water was controlled by channeling through troughs filled with Zone I rock or through Corrugated Metal Pipe (CMP) to sumps. The conduits were fitted with 2" pipes brought to grade for later

The shapes of the spectra were estimated by Weston using two methods; one, adjusting the average strong motion spectra for moderate distances computed by Housner, and another by application of a method developed by Estere and Rosenblueth. The resulting design response spectra represents the estimated spectra coinciding with established ground acceleration level of 0.05 g.

1.2.11 Foundation Analysis

As part of the PSAR studies, Woodward-Clyde & Associates was commissioned to investigate all relevant site, structural and geotechnical conditions; to formulate criteria for foundation design and construction; and to present conclusions relative to the anticipated response of the foundation system under imposed static and wind loads. The scope of the foundation analysis included review of available site information, preliminary structural data and the results of geologic studies conducted by Gilbert Associates, Inc. Supplemental field and laboratory studies, described herein, were conducted to establish information on the supporting characteristics of the foundation materials.

1.2.11.1 Foundation Conditions

Based on study of available geologic and subsurface data, the stratigraphic model shown as Figure 11 was adopted for the foundation analysis. The stratigraphy assumed consisted of a sequence of surficial fill and irregularly stratified marine sediments of Quaternary age underlain successively by the Inglis and Avon Park limestone units. A general description and conclusions concerning characteristics of each of the stratigraphic units are summarized as follows.

1.2.11.2 Fill and Quaternary Deposits

The capping deposits are relatively heterogeneous silts, sands and clays occasionally containing organic inclusions and in much of the area of study are mantled by an irregular thickness of decomposed limerock fill. This stratigraphic unit is rated as having an irregular and occasionally high compressibility.

1.2.11.3 Inglis Limestone

Beneath a weathered horizon of decomposed, friable limerock of variable density, a surficial member of the Inglis Limestone, termed "Cap Rock", is described as a thin discontinuous stratum of hard, strongly cemented, fossiliferous limerock – relatively rigid and massive. The remaining part of the formation, identified as "Differentially Cemented Limerock", is an intensely solutioned, differentially cemented, often friable and weakly cemented, fossiliferous limestone interspersed with hard discontinuous rock strata. Solutioning, particularly intense along regional fracture traces, has resulted in numerous cavities of limited horizontal extent.

The cavities are usually vertically oriented and generally contain a secondary infill of sand and silty sand.

Considering the extreme variation in engineering properties and the difficulty of obtaining data from low yield zones, the differentially cemented limerock was characterized as a weakly cemented sand containing random discontinuities in the form of solution cavities and associated highly altered limerock zones. The load-settlement response taken as representative of the weaker elements of the formation was assumed to be characterized by a modulus of deformation of 54 ksi as would be derived from the loading of a one foot diameter, rigid bearing plate. The in situ shear strength of the differentially cemented limerock although dependent on confining pressure and varying with the degree of cementation, was very conservatively assumed to have an in situ shear strength of 9 tsf, independent of confining pressure.

1.2.11.4 Avon Park Limestone

Beneath a thin discontinuous zone usually identified as a depositional discontinuity and termed the "Transition Zone", the Avon Park doloarenite member - although containing randomly distributed solution voids - was characterized as a rigid, relatively incompressible rock with a modulus of deformation of 140 ksi for the upper zone and 530 ksi for the underlying rock. Based on uniaxial compression tests on representative core specimens, the average unconfined strength of the formation was assumed to be 700 tsf. On the basis of the depth, extent and character of the "Transition Zone" materials (usually classified as a stiff to hard dolomitized silt), these materials were not considered to influence the foundation analysis.

1.2.11.5 Bearing Capacity Analysis

Analyses of bearing capacity were performed first assuming that foundations would be based on the differentially cemented limerock and that the limerock would react as a weakly cemented cohesive material. Thus, the bearing capacity expression is given by:

$$q_{ult.} = 6c\ (1 + 0.2^{D}/B) \qquad (01)$$

where, c is the in situ strength, D is the depth of the mat base below final grade, and B is the diameter of the mat. To assess the deep crushing potential of the limerock, an analysis of the imposed vertical stresses ($\Delta\sigma_Z$) was made to determine the average unit pressure imposed at various elevations below the mat. By letting $\Delta\sigma_Z = q_{ult.}$ and solving Eq. (01) for c, the influence of solution voids may also be qualitatively considered by assuming the average $\Delta\sigma_Z$ to be increased on a given horizontal plain in accordance with the following expression:

$$\Delta\sigma_Z{}^1 = \frac{\Delta\sigma_Z}{1-n} \qquad (2)$$

where n is the ratio of the total area of voids to the total stressed area under consideration, assuming an idealized regular distribution of solution voids.

For a conventional bearing capacity analysis assuming a mat width of 147 ft. and an average contact pressure of 7.8 ksf, the required shear strength for a safety factor of 3 is only in the order of 3.0 ksf.

Extending the analysis to consider the failure potential of any extensive weak zones within the foundation rock, the most critical condition is postulated at elevation +60 where the shear strength requirement for a safety factor of 3 is approximately 2.0 ksf. Should the void area ratio at elevation 60 be as much as 50 percent, the required shear strength for a safety factor of 3 would be doubled, indicating a shear strength requirement of 4.0 ksf. Comparison of these values with an average in situ shear strength of 9 tsf (assumed to characterize the differentially cemented limerock) indicates a wide margin of safety against a bearing capacity failure provided that massive unfilled solution voids are not present within a zone extending below the foundation down to about elevation +30.

Analysis of the 1.5 times accident pressure condition where imposed transient pressures at the center of the Reactor Building foundation mat are assumed to be about 35 ksf indicates that a bearing capacity failure will not occur although some localized overstressing and additional foundation settlement would be expected. A similar analysis to the foregoing was conducted assuming a c = o condition and solving for required frictional strength parameters. This analysis also demonstrated that the Reactor Building foundation mat was not subject to a bearing capacity failure under the most unfavorable condition which could be reasonably postulated.

1.2.11.6 Settlement Analysis

Settlement analysis was predicated on removal of the Quaternary deposits and of the immediately underlying loose to medium dense decomposed limerock horizon. Thus, it was assumed that foundation elements would bear directly either on the cap rock or the differentially cemented limerock units of the Inglis Limestone. It was also assumed that any load-bearing fill materials used beneath the foundation would consist of materials of a quality at least equivalent to the weakly cemented limerock materials.

A pseudo-elastic method of analysis was used by adapting a form of Equation (03) for a multi-layered foundation system as proposed by Vesic.[1]

$$\rho = \frac{\mu_1 \mu_2 (1-\gamma^2)\sigma_o d}{E} \qquad (03)$$

[1] Vesic, A.B. (1963) "The Validity of Layered Solid Theories", Proceedings, International Conference, Structural Design of Asphalt Pavements, University of Michigan.

where, μ_1, and μ_2 are embedment and shape factors, γ = Poissons Ratio, σ_o = average contact pressure, d = diameter of mat and E = Modulus of Deformation. The angular deformation of the Reactor Building mat under transient wind loading was also estimated in accordance with elastic theory using a pseudo-static method of analysis proposed by Weissman and White[1].

It was concluded that the foundation deformation contributed by the Inglis and Avon Park formations would occur as a small, essentially immediate deformation, the major settlement contribution being derived from the differentially cemented limerock member of the Inglis Limestone. Estimates of total operating load deformation of the Reactor Building foundation system considered the load superposition from the adjacent structures and from the exterior fills. Results of this analysis indicated the upper limit of total settlement of the mat to be in the order of 7/8 of an inch at the center of the semi-rigid mat foundation. It was noted that all but a very small fraction of this settlement would be expected to occur during construction - before installation of equipment or instrumentation which may be sensitive to slight differential movement.

Differential between the load center and edge of the mat was estimated to be in the order of 5/16 of an inch in 75 ft.

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(1) Weissman and White (1961) "Small Angular Deflexions of Rigid Foundations", Geotechnique, Vol. II, No. 3.

indicating a maximum angular distortion in the order of 3.5×10^{-4} radians for the most unfavorable supporting conditions. Considering an estimated additional angular distortion of 0.2×10^{-5}, due to wind forces, the total angular distortion from the center to the edge of the Reactor Building mat was not expected to exceed an order of magnitude of three to four times 10^{-4} radians under the most unfavorable wind and static loading conditions which could be postulated. It was also concluded that the settlement distortion of foundations supporting other components of the plant complex would be less than estimated for the Reactor Building foundation.

1.2.11.7 Foundation Treatment

On the basis of bearing capacity and settlement analyses, it was concluded that the continuity and integrity of the solutioned limestone within a zone directly beneath all foundation units extending at least down to elevation +30 in the Reactor Building area should be assured by cement grouting, primarily to fill all solution voids of significant extent and secondarily to provide some densification of loose discrete grained infill materials associated with solution voids. With respect to the optimum grout zone depth, consideration was given to extending consolidation grouting to the doloarenite in lieu of employing a quick-set additive or other procedures to minimize grout excape beyond the base of the consolidation zone. This latter alternative was adopted and consolidation grouting was accomplished using a procedure which employed a grout curtain to aid in groundwater control and to prevent lateral escape of grout during split-hole consolidation grouting.

1.2.11.8 <u>Excavation and Groundwater Control</u>

Considering the undesirable characteristics of the surficial materials, it was concluded that excavations should extend down to competent materials below the loose to medium dense decomposed limerock horizon. As it was expected that excavation of unsuitable materials would require excavations extending well below groundwater level, special groundwater control techniques were recommended to minimize detrimental ground loss by piping of foundation materials under excessive hydraulic gradients. It was therefore concluded that dewatering should be primarily accomplished by pumping from shallow sumps and other subdrainage systems filtered to preclude excessive removal of fines.

It was recognized that a piping potential would exist even with the most appropriate dewatering system and that piping may have localized detrimental influence on the stability of foundation materials. The occurrence of extensive infill deposits not detected by the subsurface exploration and which would be unsuitable for foundation support was also recognized.

It was therefore concluded that should check borings, cement grout-take analysis or permeability tests made after grouting indicate an area of comparatively high porosity or low density, chemical grouting would be required if the unsuitable materials were too extensive for removal and replacement and could be spanned by the foundation system.

An alternative subaqueous excavation technique was recommended utilizing a confined or unconfined excavation, the latter recommended for conditions where the depth of excavation below water level is limited. A confined excavation (sheeted cofferdam) was recommended where the depth of excavation below water level would exceed about 10 ft. over an extensive area. Bottom clean-out procedures were specified including vacuum cleaning (air lifting) of any collected bottom sediments.

1.2.11.9 Load-Bearing Materials

As the depth to suitable bearing materials was expected to vary considerably in some areas, it was anticipated that it would be desirable to utilize load-bearing fills beneath foundation elements. It was recommended that fill placed below groundwater level consist of a crushed limestone aggregate (Zone I), suitably graded for underwater placement and for in-place grouting. For above water placement, the use of well graded, crushed limestone aggregates (Zone II, Zone A and Zone B) was recommended. These materials are capable of being compacted to a high relative density and are graded (Zones A and B) to facilitate subdrainage. Alternatively, a lean concrete fill was recommended. A third material, friable crushed limestone (Zone III), was recommended for placement outside of structure areas.

The recommended material quality requirements and compaction criteria for the three load-bearing fill types are contained in Specification SP-5629, "Specifications for Excavation and Placement of Structural Fill". These criteria were developed from the results of compaction, uniaxial compression and triaxial compression tests on representative samples prepared in a manner to simulate anticipated field conditions. The strength and compressibility of both the grouted and compacted materials were found to be acceptable for foundation support.

1.2.12 <u>Unit No. 2 Foundation Grouting</u>

To prepare for grouting of the foundation of the proposed Unit No. 3 Nuclear facility, Unit No. 2 foundation was used to develop the techniques and materials necessary to provide adequate support for the structures. (See Volume III, Section 3.3.0 for detailed report.)

In order to establish an acceptable grouting process and to document the effectiveness of such a procedure, the following were performed at various stages in the grouting process:

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:

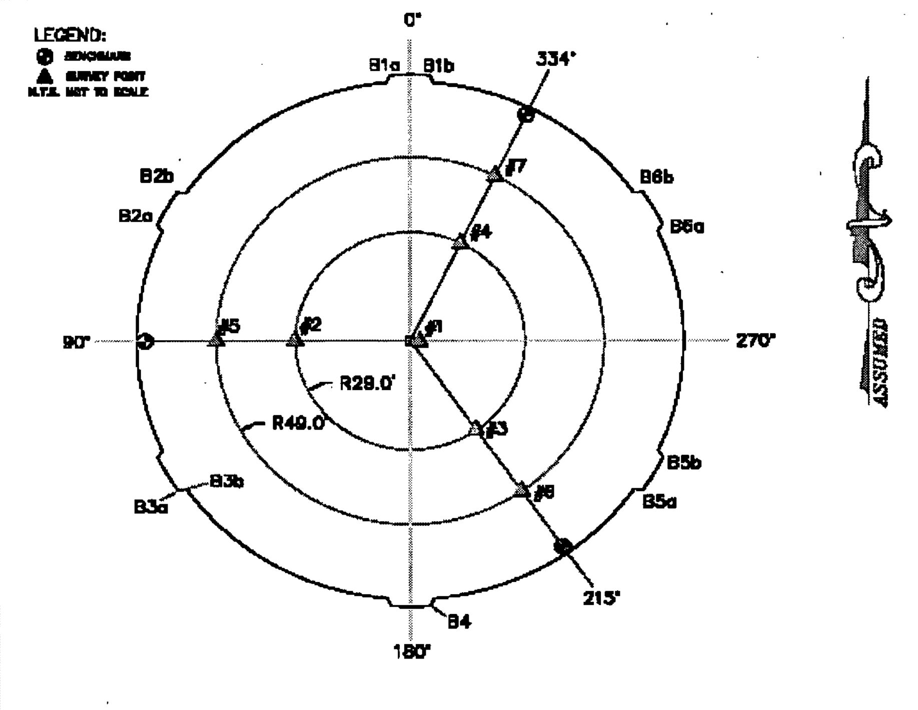
- Uneven surfaces between placements along form edges
- Cosmetic grout repairs along the face of the buttresses along the corner causes a radial change in to or out of the plane of the buttress
- Corners exhibit loss of cover concrete along the tendon bearing plate area, which causes a shift in lateral 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	274.308	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 09017A04.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

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

1.6 Inadequate Design against Ground Movement

Description: Ground movement, improper geo-strength selection, elastic deformation of the foundation base rock, dissolution of the ground limestone, etc. can distort the containment structure resulting in localized, added stress in certain areas of containment. The stress could result in crack initiation.

Data to be collected and Analyzed:

1. Geo-technical report – Gilbert Geo-technical Study, Section 1.2.11 (1971) (FM 1.6 Exhibit 1)
2. Buttress and dome survey data. (FM 1.6 Exhibit 2)
3. FSAR Chapter 2 Sections 2.5.6, 2.5.7, 2.5.8. (FM 1.6 Exhibit 3)

Verified Refuting Evidence:	Verified Supporting Evidence:
a.The 1971 Gilbert Study found Inglis and Avon rock limestone units below the area on which the containment was to be built. The Inglis was highly solutioned, the Avon rock less so. Where sound rock was encountered, the rock had a bearing capacity much greater than required to support the containment design. (FM 1.6 Exhibit 1) b.Settlement calculations predicted that settlement of 7/8 in occurring concurrently with loading during construction. To assure the continuity and integrity of the limestone, cement grouting under the containment was recommended. An additional study concluded cement alone was insufficient and chemical grouting was added. The Gilbert Study also confirms a successful grouting program under the containment was conducted. (FM 1.6 Exhibit 1) c.The FSAR and sited reports confirm the containment, including the grouting program, are compliant with the Geo-technical Study. (FM 1.6 Exhibit 3) See Attached Sheet	None

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

1.6 Inadequate Design against Ground Movement, cont.

Verified Refuting Evidence:

d. The Dome Survey (FM 1.5 Exhibit 2) measured changes in elevation from 1982 to 2009 from + 0.036 to – 0.240 inch. This movement is considered insignificant and not indicative of ground movement. The Buttress Survey has no historical baseline to compare. Evaluation compares tilt from the vertical as possible indication of ground movement. On buttresses that could be measured from both sides, a large difference in measured tilt is judged to be from construction tolerance rather than ground movement. Buttress #2 was measured as having a lateral offset of 4.17 inches on one side and 0.522 inch on the other. Ground movement would result in tilt measured at both locations. Buttress #4 was measured with a very small tilt of 0.45 inch. The data does not indicate ground movement.

Conclusion: Reviewed data confirms that the containment design is adequate against ground movement.

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

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