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.

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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 7×10^{-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.

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