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 $7 \times 10^{-3}$ 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 $7 \times 10^{-3}$ 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 $4 \times 10^{-4}$ 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