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, Υ = 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. 11, 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 Excavation and Groundwater Control

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 Unit No. 2 Foundation Grouting

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: