APPENDIX 3.7A

SEISMIC DESIGN - PARAMETRIC STUDIES

A) Site Amplification Studies

The design response spectra are applied at elevation -47 ft- MSL (top of existing Pleistocene formation) based on extensive studies that little amplification occurs through the Pleistocene formation because the fundamental period of the soil column is long. These studies involved varying the physical properties of the materials comprising the soil column as well as assuming that the earthquake input occurred at various elevations.

The analyses utilized a lumped-mass model, where multilayered stratigraphy was considered within the soil column. There is an inherent error in this approach in that certain configurations of mass intervals may produce mathematical inconsistencies leading to totally incorrect results. This form of error is inherent in any numerical approximation involving finite intervals. Although the finite element analyses may reduce the occurrence of such inconsistencies, they do not eliminate them. In order to eliminate such inconsistencies, the analyses were modified to vary the element spacing. In this way, these computational inconsistencies were eliminated. Soil structure interaction was not considered in the amplification studies.

The analyses considered three significant levels in the site columns. The following soil parameters, unit weight and shear modulus were selected from laboratory test results (refer to Section 2.5) on a conservative basis.

Elevation* (ft. MSL)	Soil	Unit Weight (pcf)	Shear Modulus (ksf/psi)
+13 to -40	Alluvium	111	260 to 460/1805 to 3190
-40 to -317	Pleistocene-predominately clay	119	333 to 850/2310 to 5900
-317 to -500	Pleistocene		
	a. Clay	119	533 to 830/3700 to 5760
	b. Sand	125	1450 to 4350/ 10,090 to 30,200

The various shear modulus values utilized changed with the interaction and the strain computed at the different elevations within the column.

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* Prior to excavation.

The fundamental periods of the column and changes in the peak acceleration of the earthquake, as it is propagated through the different columns, is given below:

Assumed Elevation of Firm Base (level at which motion is applied) FT. MSL	Ratio of Peak Acceleration to Earthquake Peak at Base of Recent Information	Ratio of Peak Acceleration to Earthquake Peak at Ground Surface	Fundamental Period of Total Column (sec)
-40 (Base of Recent)	1.0	2.5	0.8
-317 (Top of 400 ft. Sand)	0.98	1.3	2.8
-500 (Base of 400 ft. Sand)			
a. Disregarding Sand Rigidity	0.64	1.1	4.3
b. Considering Sand Rigidity	0.56	0.9	3.7

These analyses demonstrate that the most conservative conditions for design will be to assume the safe shutdown earthquake acts at the base of the recent formations.

The predominant period for the SSE is 0.3 seconds. This period assumes that the earthquake occurs close to the site. It is comparable with the observed predominant period of California earthquakes of equivalent magnitude.

If the SSE was assumed to act at a distance of 50 miles from the site, its peak acceleration in rock would be far less than at the site. Based on California earthquakes, the peak acceleration in the rock would be 0.1 times its value at the site. The predominant period of the earthquake would be increased only slightly[1]. Thus, the earthquake would not be amplified by the column at the site. If the earthquake were 100 miles from the site, the rock peak acceleration at the site would be 0.05 times that of the earthquake; the predominant period at the site would be about 0.5 seconds, still far from the fundamental period of the site.

The analyses of site response spectra include all the contributions of all modes of vibration up to the 33rd harmonic. However, the fundamental through the fourth harmonic contributes 70 percent of the total accelerations. A distant earthquake (300 miles away) with a predominant period of one second at the site would be resonant in the third harmonic, which contributes only about 10 percent of the peak acceleration. Thus, the acceleration of this longer period distant earthquake could not be amplified appreciably.

→(DRN 00-1121)

The New Madrid earthquake, centered about 450 miles from the site might have had a predominant period of about two seconds at the site. This is close to the second harmonic of the site column. The second harmonic contributes about 16 percent of the peak.

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This, and higher harmonics, might have produced some amplification of the peak acceleration. This conclusion tends to be confirmed by the observed decay of peak acceleration with distance from the New Madrid epicenter. Utilizing the distance acceleration relationships for California earthquakes, the acceleration at the site produced by the New Madrid earthquake would have been much less than the MM intensity V interpolated from observations at New Orleans.

Based on these studies, a conservative approach is to consider the recommended acceleration and frequency spectra to occur at elevation -47 ft. MSL.

B) Selection of Soil-Interaction Model

The use of equivalent soil springs for the system mathematical model is based on equations of a rigid plate on a semi-infinite elastic half-space. The elasticity of the continuum is defined by the physical properties of the soil. Whether this is a simplified approach or whether this is a sufficiently conservative design basis are questions that can be related to the design alternative of finite element modeling. The finite element modeling of the soil is a discretization of a continuum with the elasticity of the element defined by the same soil properties that defined the continuum. The finite element method does not create new data. The physical properties of the Waterford 3 soil behave as a filter and a change in mathematical models does not effect a change in soil physics, i.e., eliminating the filter. In either analysis, response amplitudes in the low period range are attenuated. The use of a soil spring or finite element model would show this filtering action. The effect of the filter depends upon the soil modulus which is required for either type of model; therefore, an error in the modulus is equally reflected as an error in the first three modes of vibration in either of the models. This attenuated response in the low periods is not particularly significant for structures, but it is recognized as more significant for equipment. To evaluate the attenuations of the high frequency responses, the shear modulus of the soil was assumed as 1, 3, and 5 times a value of 6400 psi and three dynamic analyses were performed. Periods for modes higher than the third did not vary more than two percent for any case and much less than one percent for over 90 percent of the cases, as listed in Table 3.7A-1.

C) Shear Modulus During a SSE

Using the Seed-Idriss type analysis for the 400 foot soil column discussed in Section A of this Appendix, a time history of the shear strain can be obtained for each level in the column. By use of the shear strain vs shear modulus relation in Figure 2.5-78, this curve can be converted to one describing the variation of shear modulus with time at each level. This is illustrated in Figures 3.7A-1 and 2 for a level (level 19) about half way down the soil column and a level (level 13) at the top of the Pleistocene formation. The time at which the shear modulus reaches a peak value is not the same for all levels but slightly different from level to level as can be seen by comparing these figures. By utilizing all of this data for all levels in the soil, a statistical distribution of shear modulus can be obtained as shown in Figures 3.7A-3 and 4. As shown by these curves, the shear modulus for the SSE on this site averages about 8000 psi and varies essentially from 4000 psi to no higher than about 16,000 psi.

Methods presently used for developing an effective shear modulus for soil structure interaction utilize a single value obtained from averaging techniques and is generally determined on the basis of the modulus equivalent to that corresponding to about 70 percent of the peak shear strain.

A modulus for Waterford 3 corresponding to a time averaged shear strain, which is on the order of 30 percent of the peak shear strain, would be about 7600 psi for a SSE for this site.

All evidence supports the fact that the Waterford 3 site would react to the SSE defined for it in a manner corresponding to a high strain condition. It would be valid to represent the soil-structure interaction by means of the statistically averaged values of the shear modulus as obtained from the soil column analysis. This is valid for Waterford 3 since the "floating raft" principle used for the Waterford 3 foundation imposes little, if any, extra strain on the soil in the vicinity of the structures than exists in the free field.

Since this concept results in a long fundamental period of the structure, far beyond the fundamental periods of equipment and piping housed within, the range of the 5800 psi to 16,050 psi was used to analyze and design the structures and piping equipment housed within. This resulted in an extremely conservative design.

APPENDIX 3.7A: REFERENCES

1) Seed H B, Idriss, I M, Kieffner, F W, Characteristics of Rock Motions During Earthquakes, Report No. EERG 68-5, Earthquake Engineering Research Center, University of California, Berkeley, 1968.

TABLE 3.7A-1

EFFECT OF SHEAR MODULUS ON
STRUCTURE RESPONSE

	Period (Seconds)		
Mode	G=6400psi	G=19200psi	G=32000psi
1	0.927	0.545	0.429
2	0.404	0.287	0.274
3	0.220	0.182	0.157
4	0.134	0.133	0.131
5	0.109	0.108	0.106
6	0.090	0.090	0.090