ATTACHMENT 4

REVIEW OF WATERFORD III BASEMAT ANALYSIS

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April 16, 1984

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INTRODUCTION

At the request of SGEB/NRR, the Structural Analysis Division of the Department of Nuclear Energy at BNL undertook a review and evaluation of the HEA Waterford III mat analysis documented in Harstead Engineering Associates (HEA) Reports, Nos. 8304-1 and 8304-2. Both reports are entitled, "Analysis of Cracks and Water Seepage in Foundation Mat". Report 8304-1 is dated September 19, 1983, while Report 8304-2 is dated October 12, 1983. Major topics addressed in the first report are:

(1) Engineering criteria used in the design, site preparation and construction of the Nuclear Power Island Structure basemat.

(2) Discussion of cracking and leakage in the basemat.

(3) Laboratory tests on basemat water and leakage samples.

(4) Stability calculations for the containment structure.

The second report concentrates on the finite element analysis and its results. Specifically, it describes:

(1) The geometric criteria and finite element idealization.

(2) The magnitude and distribution of the loads.

(3) The final computer results in terms of moments and shear versus the resistance capacity of the mat structure.

Supplemental information to these reports were obtained at meetings held in Bethesda, MD, on March 21 and 26, 1984, at the Waterford Plant site in Louisiana on March 27, 1984, and at Ebasco headquarters in New York City on April 4, 1984. At the close of the EBASCO meeting, a complete listing of the HEA computer run was made available to BNL.

The BNL efforts were concentrated on the review of the results presented in report no. 8302-2 and on the supplemental information contained in the computer run given to us by HEA. This computer run contains 9 load cases and their various combinations. The input/output printout alone consists of roughly two thousand pages of information. Selected portions were reviewed in detail, while the remaining sections were reviewed in lesser detail. Comments regarding the reviewed work are given in the sections that follow.

GENERAL COMMENTS

Basically, the HEA report concludes that large primary moments will produce tension on the bottom surface of the mat. For this condition, it is shown that the design is conservative. Furthermore, the shear capacity vs. the shear produced by load combinations are concluded to be adequate although a few elements were found to be close to the design capacity. Accordingly, the cracking of the top surface is attributed to "benign" causes such as shrinkage, differential soil settlement, and temperature changes.

Based on the discussions held with EBASCO and HEA, and on the review of data given to BNL, it is our judgement that the bottom reinforcement as well as the mat shear capacity is adequate. The statement that the cracking of the top surface is attributable to "benign" causes however has not been analytically demonstrated by HEA. In the BNL review of the reports and data, an attempt was made to ascertain the reasons for the existing crack patterns that appear around the outside of the reactor shield building as depicted in Figure D-1 Appendix D of the HEA Report 8304-2. Other effects influencing the structural behavior and safety were also investigated. Specifically, the structural analysis topics reviewed in more detail include:

(1) Dead loads and their effects.

(2) Buoyancy forces and their effects.

(3) Variable springs used for the foundation modulus.

(4) Vertical earthquake effects.

(5) The side soil pressures.

(6) The boundary constraint conditions used for the mat.

(7) Finite element mesh size and its effects.

(8) BNL check calculations.

STRUCTURAL ANALYSIS TOPICS REVIEWED

1. Dead Loads

As mentioned, EBASCO in their discussion and HEA in their reports have not shown analytically, the cause of the top surface cracks. In reviewing the HEA computer outputs, it was found that element moments and shears for individual loadings are explicitly given. Thus, for the case involving dead loads only, a number of elements in the cracked regions exhibit moments that can produce tension and thus create cracking on the top surface. This situation is shown in Table 1 which gives moment data for elements in some of the cracked regions. From the HEA report (page C-2-1-9) it seems that the top reinforcement, which is #11 @ 6" in each direction* is the minimum requirement for temperature steel according to the American Concrete Institute Building Code

*In a subsequent phone conversation, P.C. Liu of EBASCO stated that some additional reinforcement was added on the top surface in one direction. Even if this is the case the statement that follows is true for the unstrengthened direction and perhaps even for the strengthened direction.

TABLE 1

Area	ELEMENT	Mx D	Mx B	My D	My B	Mxy D	Mxy B	Normal Pressure Side Pressure Mx	Mx	Mxy
Area T2-R-12M-7FH	437	-242	173	-574	197	116	- 31	-294	-196	93
	212	644	595	207	91	106	- 25	-663	-392	79
	211	-605	205	-412	217	-296	48	-219	-416	- 76
	207	64	99	-136	136	- 81	15	-319	-193	50
	441	-105	168	172	-170	39	- 12	-347	-489	66
	436	-719	269	-1193	357	531	-130	-274	-258	117
	438	269	142	-159	158	- 60	26	-730	-347	27
	447	665	59	210	88	248	- 55	-653	-339	-127
	204	193	87	569	72	-143	28	-361	-420	24
	208	350	32	898	- 24	-241	75	-354	-771	- 49
	203	-676	260	-995	- 236	39	- 21	-574	-247	30
	426	-542	157	-705	310	332	- 65	-171	-486	61
Area R-P-2M-1A	259	62	148	-133	81	154	- 36			
	253	5	71	531	75	0	18			
	255	30	58	670	5	41	10			
	252	86	24	611	- 55	87	8			
	254	50	26	412	- 41	69	9			
	251	37	5	162	- 23	44	12			
	257	320	- 38	57	15	- 81	- 15			
	248	255	- 26	29	16	- 29	- 6			
	267	-236	80	87	118	- 64	28			
	269	-173	59	434	10	- 82	32			
Area R-P1-12A-9M	419	-314	137	-635	313	- 30	12			
	410	-371	71	-642	238	270	- 29			
	400	-315	108	-774	275	- 44	41			
	401	-180	42	-201	102	108	- 23			
	414	-304	118	-130	178	44	- 19			
	417	-200	93	440	41	- 17	- 15			
	404	- 64	17	428	- 32	98	- 18			

NOTE: D - Dead Load

B - Bouyancy

Specification (i.e., $A_s = .0018 \times 12 \times 144 = 3.11$ in^2/ft). The resisting moment capacity based on working stress design is about $M = A_s f_s jd = 3.12 \times 24 \times 131/12 = 817$ ft-kips/ft. The steel reinforcement strain for this moment is equal to

$$\varepsilon_s \ (=\varepsilon_c) = \frac{f_s}{E_s} = \frac{24}{29,000} = 0.00083 \text{ in/in}$$

while, the corresponding concrete stress is,

$$f_c = \varepsilon_c E_{s/n} = 0.00083 \left(\frac{29,000}{8}\right) = 3 \text{ ksi}$$

In checking the data in Table 1, it can be seen that element 208 has exceeded the working load capacity under the dead load condition and, thus the local area could have exhibited a crack when this load acted alone. Similarly, concrete cracking could occur under this load condition in elements 447, 212, 204, 253, 255, 269, 257, 417, and 404. Thus, the cracks on the upper surface outside of the shield wall could have been initiated after construction of the superstructure, before placement of the backfill. It should be noted that since no analysis is available for dead load without the superstructure, the reason for the basemat cracks inside of the shield wall cannot be explained by this reasoning.

2. Buoyancy Forces

The moment results from this analysis show that these forces when acting alone would mostly cause tensile stress on the upper surfaces. The moments causing these stresses are tabulated in Table 1 for groups of elements in the cracked regions. As can be seen, these moments are not as severe as those due to dead weight. By superpositon they could in some cases contribute to higher tensile stresses and thus result in further cracking in some of the upper surface areas.

3. Variable Springs Used for the Foundation Modulus

Moments and shears developed in the basemat were computed using the concept of the Winkler foundation; namely the soil is represented as a series of relatively uniform independent springs. The stiffness of the springs is ob tained from approximate analyses which are based on generalized analytical solutions available for rigid mats on the surface of elastic soils. The actual design of the mat was based on a series of iterative computer runs in which the soil stiffness was varied until the computed contact pressures under the mat were fairly uniform and equal to the overburden stress at the elevation of the foundation mat. This approach appears to be reasonable since the long term consolidation effects can be anticipated to cause effective redistribution of loads and cause the mat to behave in a flexible manner.

4. Vertical Earthquake Effects

Vertical earthquake effect was not discussed in the HEA reports. However, from the finite element analysis print out and conversation with HEA engineers, it was stated that this effect was included in the load combination cases by specifying an additional factor of 0.067, which was then applied to the dead and equipment load case. From the discussions and the review it is not clear to BNL whether an amplification factor due to vertical mat frequency was used or not. A rough check by the reviewers indicates that this factor could have some influence on the results.

5. Side Soil Pressure

According to the STARDYNE computer results obtained from HEA, the normal side soil pressures produce large moments that are opposite to those caused by the dead loads. As shown in Table 1 where moments of elements located in one of the cracked regions outside of the shield building are compared. The total moments in some cases (i.e. element 447 or 208) become quite small. In other regions there is in fact a reversal in the total bending moment which causes tension on the bottom surface and compression on the top. This compression would tend to close the cracks on the upper surface. Thus, it appears that this pressure is a very important load case for the mat design.

For the static or normal operating condition the lateral pressures are based on the at-rest stress condition and are uniform around the periphery of the structure. For the seismic problems the pressures are computed to approximately account for relative movements between the structure and the soil. On one side the structure will move away from soil (active side) and reduce the pressures while the opposite will occur on the other side (passive side). The actual computations made use of triaxial test data from site soils to arrive at the soil pressures rather than use the standard Rankine analyses. However, no dynamic effects on either the lateral soil or pore pressures was included. The sensitivity of the calculated responses to these effects are currently unknown.

6. Boundary Constraints

For equilibrium calculations no special consideration need be made for vertical case since the soil springs prevent unbounded structural motion. However, the same cannot be said for the horizontal case since soil springs are not used to represent the soil reactions. Rather the lateral soil forces are directly input to the model. To prevent unbounded rigid body motion, artificial lateral constraints must be imposed on the model From the output presented in the EBASCO and HEA reports, it is not possible to evaluate the impact of these assumptions. The stresses caused by the artificial boundaries should be calculated and compared with those presented.

7. Finite Element Mesh and its Effects

In general finite element models for plate structures require at least four elements between supports to obtain reasonable results on stress computations. The models used by both EBASCO and HEA violate this "rule of thumb" in the vicinity of the shield wall. The significance of this effect is demonstrated in Figure D-3 which presents a plot of moment taken through the center of the slab. The computed moments in adjacent elements 193, 194 and 455 are -3800, -2500 and +400K. The elements used in the HEA analysis are constant curvature elements so that the computed moments will be constant within each element. The steep moment gradient between the elements indicates that a finer mesh would be advisable. A similar effect was also noted when investigating the elements forming the junction between the lateral earth retaining walls and the base mat.

8. BNL Check Calculations

Due to the questions raised in the items above (4 through 7), it was decided to perform several equilibrium calculations to check the order of magnitude of the shear stress computed in the detailed finite element analyses presented by EBASCO/HEA.

Two types of average vertical shear stresses were computed in the base mat. The first type considers the average shear through a vertical section across the entire mat (one section in the E-W direction and the other in the N-S direction). These sections were chosen to include those elements which indicated high shear stresses in the HEA analysis and where the actual cracking pattern was noted. The highest average shear stress computed for any design load combination is 50 psi. The allowable shear stress for the case is 107 psi ($2\phi\sqrt{f'_c}$).

The second type of section considered is a circular punching shear section located a distance of d/2 outside the reactor shield wall. The peak value of shear stress due to both SSE overturning moments and normal operating loads (plus proper load factors) were always less than the allowable design shear ($4\phi f_c'$).

CONCLUSIONS AND RECOMMENDATIONS

(a) The Waterford plant is primarily a box-like concrete structure supported on a 12 foot thick continuous concrete mat which houses all Class 1 structures. The plant island is supported by relatively soft overconsolidated soils. To minimize long term settlement effects, the foundation mat was designed on the floating foundation principle. The average contact pressure developed by the weight of the structure is made approximately equal to the existing intergranular stresses developed by the weight of the soil overburden at the level of the bottom of the foundation mat. Thus, net changes in soil stresses due to construction and corresponding settlements can be anticipated to be relatively small.

(b) In reviewing the information, reports, and computer outputs supplied to BNL by EBASCO, HEA, and LPL, it is concluded that normal engineering practice and procedures used for the analysis of nuclear power plant structures were employed.

(c) Accepting the information pertaining to loadings, geometries of the structures, material properties and finite element mesh data, it is the judgement of the reviewers that:

(i) the bottom reinforcement as well as the shear capacity of the base mat are adequate for the loads considered.

(ii) the computed dead weight output data can be used to explain some of the mat cracks that appear on the top surface. The cracks that appear, could have occurred after the construction of the superstructure but before the placement of the backfill. Their growth would then be constrained by subsequent backfill soil pressure.

(d) Due to the existance of the cracks, it is recommended that a surveilance program be instituted to monitor cracks on a regular basis. Furthermore, an alert limit (in terms of amount of cracks, and or crack width, etc) should be specified. If this limit is exceeded, specific structural repairs should be mandated.

(e) It is also recommended that a program be set up to monitor the water leakage and its chemical content.

(f) BNL has reviewed the information provided by EBASCO, HEA, and LPL. The following questions concerning their analyses were developed:

(i) dynamic coupling in the vertical direction between the reactor building and the base mat.

(ii) dynamic effects of lateral soil/water loadings.

(iii) artificial boundary constraints in finite elements models.

(iv) fineness of base mat mesh.

Based upon our approximate calculations together with engineering judgement, we do not anticipate that the above questions will lead to major changes in calculated stress levels. Thus, it is our opinion that the safety margins in the design of the base mat are adequate. However, it is recommended some detailed confirmatory calculations be performed in the near future.