SAFETY EVALUATION OF THE STRUCTURAL ADEQUACY OF THE WATERFORD 3 BASEMAT

STRUCTURAL AND GEOTECHNICAL ENGINEERING BRANCH

DIVISION OF ENGINEERING

Prepared by the Staff of the

Structural & Geotechnical Engineering Branch

Division of Engineering

Office of Nuclear Reactor Regulation

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EXECUTIVE SUMMARY

In response to a March 12, 1984 memo from the Executive Director for Operations, subject: "Completion of Outstanding Regulatory Actions on Comanche Peak and Waterford", the Structural and Geotechnical Engineering Branch was assigned the task of reevaluating the structural adequacy of the basemat and the related Category I structures at the Waterford Nuclear Power Plant. Concern was focused on the effect of cracks which had occurred in the concrete during construction, especially in view of some recent allegations pertaining to concrete construction at the site. The SGEB staff and its consultants from the Brookhaven National Laboratory (BNL) met with the applicant, Louisiana Power and Light and its architect-engineer consultant firm, EBASCO, a number of times. A visit at the site on March 27, 1984 provided the SGEB staff and consultants opportunity to see the cracks, question the builders, and examine records. Additional information was requested of the applicant.

Based upon the observations at the site and the review of information available to the SGEB staff, the SGEB staff and its consultants have completed a safety evaluation on the structural adequacy of the basemat and related Category I structures. A summary of the conclusions follows:

The geotechnical engineering staff has concluded that:

(1) The "compensated" foundation concept is sound and acceptable.

(2) The cracks in the foundation mat and superstructure were probably caused by differential settlements during construction.

(3) The differential settlements resulted from complicated soil conditions, high groundwater levels, compaction of shell filter strips and some concrete block construction procedures, and

(4) Movements of the foundation mat and the growth of the cracks will continue.

The structural engineering staff has concluded that:

(1) During the first three major concrete mat block pours, the applicant identified and documented construction difficulties.

(2) The mat is not currently in distress based on the observation of cracks.

(3) Verification of shear capacity of the mat under SSE needs to be done. As part of this verification program, selective nondestructive testing and evaluation are recommended to obtain information on cracks and potential voids in the mat and their effect on the mat.

(4) Significant corrosion of reinforcing bar due to the groundwater is believed to be unlikely at the site. Nevertheless, a surveillance program is recommended.

The structural and geotechnical engineering staff has jointly concluded that:

(1) A general surveillance (monitoring) program is recommended for all the cracks. For evident shear cracks, the length and size of a crack and its propagation against time should be marked and also recorded.

(2) The current monitoring program of foundation settlement should be expanded to enable more accurate measurements of differential settlements and crack growths.

(3) The applicant is required to either justify that its original analyses are still adequate in light of the NRC geotechnical engineering staff evaluation (enclosure 1), or perform additional analyses to account for the actual foundation soil conditions.

(4) The applicant must update its crack mapping records and submit its proposed surveillance programs for settlement, concrete cracks, and corrosion of reinforcing bars prior to issuance of the OL license.

(5) An independent report (enclosure 3) of our BNL consultants in general is supportive of the above conclusions. Our BNL consultants have indicated confidence in the functional performance of the mat, provided confirmatory measures including monitoring are accomplished.

II. Overall Evaluation

1. Introduction

The Structural and Geotechnical Engineering Branch (SGEB), Division of Engineering has been requested to provide structural design adequacy of the Waterford 3 "as-built" base mat. In the course of developing information needed for the assessment, the SGEB staff and its BNL consultants held meetings 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. Additional information was also obtained, via phone conversations, from the Region IV staff, the applicant and its consultants.

In brief, it was found that since its construction in late 1975, the concrete base mat of Waterford 3 has experienced cracks and accumulations of minimal amount of water through some of the cracks. These base mat cracks also caused several cracks in the reactor shield building and other structural walls supported by the mat. The cracks are generally believed to have been caused by differential settlements of the base mat and possibly due to some QA/QC deviations during concrete pours.

Technical evaluations of the analysis, design, construction and QA/QC aspects of the base mat were performed with their key findings described in the following sections.

Key recommendations to be incorporated as part of the Waterford 3 operating license are also listed in this evaluation.

2. Geotechnical Safety Evaluation of Waterford 3 Foundation

The geotechnical engineering staff concluded that:

(a) The plant foundation design, i.e., the "compensated" foundation concept, is sound and acceptable.

(b) The cracks in the foundation mat and other related structural elements were probably caused by differential settlement that occurred mainly during construction settlement.

(c) These differential settlements are believed to have resulted from complicated soil conditions, high groundwater levels, compaction of shell filter strips and the concrete block construction procedures.

(d) Movements of the foundation mat and the growth of the cracks will continue.

(e) Seasonal groundwater level fluctuation will cause some movement of the foundation mat.

(f) In order to examine and evaluate the future performance of the foundation, it is recommended that the current monitoring program be expanded to enable more accurate measurements of differential settlements and crack growths. All prominent cracks should be mapped and included in the program.

Enclosure 1 provides detail technical bases of the above conclusions.

3. Structural Safety Evaluation of the Base Mat

The structural engineering staff concluded that:

(a) The mat is not currently in distress based on the crack observation.

(b) Verification of shear capacity under SSE needs to be done. As part of this verification program, selective nondestructive testing and evaluations are recommended to obtain information on cracks and potential voids and their effect on the concrete mat.

(c) The licensee is required to either justify that its original analyses are still adequate in light of the NRC geotechnical engineering staff evaluation mentioned above, or perform additional analyses to account for the actual foundation soil

conditions.

(d) A general surveillance (monitoring) program is recommended for all the cracks. For evident shear cracks, the length and size of a crack and its propagation against time should be marked and recorded.

(e) Corrosion of reinforcing bars due to the ground water is believed to be unlikely at the site. Nevertheless, a surveillance program is recommended.

Enclosure 2 provides detail bases of the above conclusions.

4. Independent Evaluation of Base Mat Analysis by Brookhaven National Laboratory (BNL)

The Structural Analysis Division of the Department of Nuclear Energy at BNL was retained as staff consultant's to provide an independent evaluation of the structural adequacy of the base mat with emphasis in reviewing the analysis documents provided by the applicant and its consultants.

The BNL staff has concluded that:

(a) The net expected changes in soil stress due to construction and corresponding settlements of the mat should be relatively

small.

(b) Having reviewed the information reports and computer outputs supplied to BNL by EBASCO, HEA, and LPL[1], it was found that normal engineering practice and procedures used for nuclear power plant structures were employed.

(c) Accepting the information pertaining to loadings, geometries of the structures, material properties and finite element idealization as correct, it is the judgement of the BNL staff that:

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

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

[1] LP&L is the utility, Louisiana Power and Light, EBASCO is the engineering consulting firm to the LP&L and HEA is the Harstead Engineering Associates, Inc., for structural cracking evaluation.

(d) Due to the existence of the cracks, it is recommended that a surveillance 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. It is also recommended that a program be set up to monitor the water leakage and chemical content.

(e) The validity of the BNL conclusions depend mainly on the information supplied by EBASCO, HEA and LPL, either verbally, in reports or in computer outputs. While some checks for accuracy ad engineering approach were made pertaining to the supplied information some open questions still remain, especially those mentioned in the text under topics 4 thru 7 under the heading, "Structural Analysis Topics Reviewed" (Enclosure 3). It is suggested that the particular issues raised under these items be resolved.

These independent conclusions are supportive of those established by the SGEB staff. These conclusions, where they highlight issues to be resolved, will be resolved through the implementation of the staff's recommendations.

5. Evaluation on Corrosion Effects of Base Mat Rebar

The staff of the Chemical Engineering Branch has reviewed the licensee's proposed Limiting Conditions for Operation on the possible corrosion of basemat rebar due to groundwater penetration through cracks in the concrete basemat (Enclosure 4).

The following factors were considered in the evaluation:

(a) Analysis of groundwater at the site indicated a chloride concentration of approximately 35 ppm, which is significantly below the 710 ppm chloride corrosion threshold for rebar in the presence of oxygen (D. A. Hausmann, Materials Protection, pp. 23-25, October, 1969).

(b) The rate of seepage of groundwater through the 12-foot thick basemat is small, which restricts the access of dissolved oxygen, chlorides and carbon dioxide to the rebar-concrete interface.

(c) The slow movement of water through the basemat causes the water to become alkaline (pH=12.5) by contact with the calcium oxide and calcium hydroxide content of the concrete.

(d) The corrosion rate of steel by alkaline water is low.

On the basis of its evaluation, the staff concluded that there is reasonable assurance that the basemat rebar will not be significantly corroded by the penetraton of groundwater of the acidity and chloride content observed at the Waterford site.

6. Recommendations for Waterford 3 Licensing Action

The following requirements should be established prior to issuance of the OL license:

(a) The applicant should update his crack mapping records, including observable vertical or inclined cracks in Category I structures supported by the mat, 30 days prior to issuance of license.

(b) The applicant shall propose an expanded differential settlements and crack monitoring program and associated plant technical specifications within the next 30 days for staff review and acceptance.

(c) In order to expedite prompt resolution of the Waterford 3 basemat structural adequacy issue, it is recommended that the Division of Licensing forward and direct the Louisiana Power and Light Co. to implement the specific applicant's action items listed in Enclosure 5.

Enclosure 1
Geotechnical Safety Evaluation of
Waterford 3 Foundation

1. INTRODUCTION

The safety class structures at Waterford are supported on a continuous mat 270 feet wide, 380 feet long and 12 feet thick. The mat has been designed and constructed using the "compensated" or "floating" foundation concept in which the applied loads on the foundation soil, the Pleistocene clay, have been controlled so that the effective insitu stresses remain essentially the same as the stresses existing before construction. In this way, the overall settlements of the foundation soil are controlled.

In July 1977, a number of east-west oriented cracks were discovered at the top of the mat beneath the containment structure. Weeping water was reported to be low and not enough to form a sheen but enough to show the cracks and to moisten surrounding concrete. Epoxy grout was used to seal all the observed cracks in the mat inside the containment structure.

In May 1983, new cracks and accompanying weeping water were discovered in the base mat outside the containment structure. Some of those cracks were found that extended to vertical wall by an NRC investigation team in March, 1984.

This report summarizes the results of NRC's geotechnical engineering evaluation of the causes which may be responsible for the observed cracking. This report, also, addresses the present foundation conditions and anticipated future behavior of the mat.

2. SUBSURFACE CONDITIONS

Subsurface conditions at the site were investigated between 1970 and 1972. 64 soil test borings, 50 to 500 feet deep, were drilled. A general description of the subsurface conditions is presented in the attached Table 2.6 of the Waterford SER.

Extensive laboratory tests were performed on selected soil samples. Significant soil characteristics are presented in Table 2.6.

3. CONSTRUCTION SEQUENCE

The construction steps involved were:

a) Groundwater control:

Groundwater levels in the plant area were controlled during construction from 1972 to 1978 by pumping from 216 shallow wells and 34 deep wells around the perimeter of the plant area. The well tips were located at El. -40 feet for shallow wells and El. -95 feet for deep wells. From November 1972 to November 1974, dewatering was stopped and about 10 feet of standing water accumulated in the excavation. In January 1977, 12 additional wells were installed around the foundation mat area to provide additional groundwater control beneath the mat.

The groundwater level was raised in a controlled pattern in late 1977 by 12 recharge wells, located near the edge of the foundation mat with tips in the shell filter blanket. Additional groundwater recharing was achieved by watering the backfill. By the end of 1979, the groundwater was raised to normal level ranging from El. +3 to E. +12.

b) EXCAVATION

The excavation, about 60 feet below the original grade to El. -47, was done in four phases:

Phase I, grade to El. -5, April to July 1972

Phase II, El. -5 to El. -22, January to June 1975

Phase III, El. -22, E. -40, April to August 1975

Phase IV, El. -40 to El. -48, October 1975 to March 1976

Turbine building, grade to El. -40, January to March 1977

Phase IV excavation, cut into the upper Pleistocene clay from El. -40 to El. -48, was made in six strips, starting with a 120 ft-wide strip across the center of the common mat, and following the alternating strips north and south of the center strip.

c) BACKFILL AND CONCRETE PLACEMENT:

After each strip was excavated, the filter cloth, the shell filter layer and the concrete mat were constructed as soon as possible so as to reload the foundation soils and minimize heave. Marafi filter cloth was placed over the Pleistocene clay before the shell layer was placed. The shell filter layer, about a foot thick, was compacted by a vibratory roller for 10 passes.

The concrete mat was poured in 28 separate blocks from December 1975 to 1976. Each block had a thickness about 12 feet and an area which varied from 2000 to 5000 square feet. The construction of the super-structure was started in May 1977 with all concrete work completed in December 1980.

Backfill material of clean sand, was placed below El. +17 around the nuclear plant island structure from August 1976 to October 1978.

4. EVALUATION

The plant was designed to give a net reduction, by about 200 psf, of the applied effective soil loading at foundation level. Before construction began, the initial effective overburden pressure at foundation level was 3300 psf; after construction was completed the final effective static loading of the plant and backfill was 3100 psf. Therefore, the future settlement of the completed plant would be negligible.

During construction, the insitu vertical stresses were controlled by lowering the groundwater level simultaneously with the excavating of soils. The lowering of the groundwater level would give an increase in effective overburden pressure which compensated for the soil removed. Later as structural loads were applied, the groundwater level was raised to reduce the effective overburden pressure and compensate for the structural loading. By this

technique, the differential settlement of the foundation soil would be reduced and its effects on structures would be minimized.

The construction procedures are generally sound. However, the control of insitu vertical effective stresses and groundwater levels was quite difficult because of the subsurface soil conditions. Numerous construction difficulties, encountered during construction, may have contributed directly or indirectly to the observed cracking of the foundation mat. Those construction problems included:

a) Dewatering:

As discussed in 3(a) above, the tips of the dewatering wells were located at El. -40 ft., in the recent alluvium stratum, for shallow wells and at El. -95 ft, in the silty sand layer, for deep wells. The silty sand layer is an identified aquifer at the site. Because of the very low permeability of the upper Pleistocene clay, those wells did not completely lower the groundwater level in the foundations soils to below El. -49, as evidenced by some of the piezometric readings. Locally, those high groundwater conditions appear to have caused soil disturbance, mud spurt, standing water in some area of the excavation and difficulties in compaction of the shell blanket.

b) Variable foundation soil conditions:

The foundation mat was founded on the upper Pleistocene clay. These clays were considered to be fairly uniform and over-consolidated in the design and construction of the mat. However, within the boundary of the foundation mat, the permeability and the compressibility of the clay layer varied significantly from one location to another as evidenced by the results of the piezometric and heave monitoring during construction. The measured heave at various location was 2 to 4 times the anticipated maximum heave used in the mat design; this indicates that the differential settlements of the mat would be greater than anticipated.

c) Variable degrees of compaction in the six shell filter strips:

The compaction procedures were selected based on the results of a test fill program. However, due to the variability of the supporting soil and groundwater conditions, the degree of compaction in these shell filter strips varied widely, from 80 to 98 percent. Filter strip number 1, 97.5 feet long and 270 feet wide, was compacted to an average of 95 percent. Filter strip number 2, 58.5 feet long and located immediately north of strip number 1, was compacted to an average of 80 percent. Shell filter was placed in standing water in the west half of strip number 2. A mud spurt area of abut 120 sq. ft. occurred in strip number 2 during compaction. Filter strip number 4, 48.5 feet long, was compacted to 98 percent.

These variable degrees of shell compaction reflect the condition of the foundation soils. Settlements of the mat due to uniform structural loads would vary significantly; strip number 2 would settle more than strip number 1 while strip number 4 would settle less. Thus, differential settlements would be experienced by structures founded over different strips. The resulting differential settlement may induce bending stresses in the mat and cause east-west oriented cracking in the foudnation mat.

d) Foundation mat construction:

As discussed in 3(c), the foundation mat was constructed in 28 blocks with a thickness of 12 feet and an area which varied from 2000 to 5000 square feet. The load due to pouring of the first block of concrete caused an immediate settlement about 3/4 of an inch, and later, some additional consolidation settlement. When the second and third blocks were poured adjacent to the first block, differential settlements between the blocks were observed. This type of settlement pattern occurred for all later constructed blocks. These differential settlements may have induced some residual stresses in the concrete and may have caused concrete cracking.

e) Significant hydrostatic pressure change:

During the construction of the concrete mat and superstructures, the groundwater levels were changed significantly three times, ranging from 20 to 30 feet. These changes in hydrostatic

pressure changed the effective stresses in the foundation soils and caused movements of the foundation soils and the concrete mat. Because of the non-uniform nature of the foundation soils, differential movements within the mat would be expected. These differential movements may have induced strain in the concrete when it was still in the process of curing.

The plant foundation design, the "compensated" foundation concept, is a sound one. The cracks in the foundation mat appear to have resulted mostly from the differential settlements experienced and, to a lesser degree, as superstructure loads were applied during construction. These differential settlements were caused mainly by the variable soil conditions, high groundwater levels, and the variable compaction of the shell filter strips and concrete mat construction procedures. The hydrostatic pressure changes, affecting the effective stress state in supporting soils, may have aggravated the growth of the cracks after the mat was completed.

The future settlement should be limited and "stable" becuase of the "compensated" design. However, the cracks discovered in 1983 and vertical wall cracks discovered in 1984 seem to indicate that the movements of the foundation mat and the growths of the cracks are continuing. The current settlement monitoring program provided some useful information indicating that the mat would move in conjunction with fluctuation of groundwater levels. But the scope

and the accuracy of the current program, are not sufficient to provide accurate information to assess and relate the actual differential settlements to the growths of the cracks in the mat. Sensitive measurements are essential to determine the future behavior of the concrete mat.

The scope of the current monitoring program should be expanded to collect more accurate information about the differential settlements in the mat and about the precise growth of new and old cracks. The more accurate differential settlement monitoring can be achieved by installing additional monitoring points on the mat with increased monitoring accuracy. The added points can be located on the outside walls of the mat. The crack monitoring program would provide information about the development of new cracks and the propagation of the cracks. Specifically, those cracks that extend to the vertical walls should be monitored. Leachate on the cracks should be cleaned out to expose the cracks. Brass pins or other means should be used to identify the extent and progression of the cracks.

5. CONCLUSION AND RECOMMENDATION

Based on the information reviewed to date and such other matters as in our judgement are pertinent, it is concluded that:

a) The plant foundation design, the "compensated" foundation concept is sound and acceptable.

b) The cracks in the foundation mat and structural walls were probably caused by differential settlement that occurred mainly during construction.

c) These differential settlements resulted from complicated soil conditions, high groundwater levels, compaction of shell filter strips and the concrete block construction procedures.

d) Movements of the foundation mat and the growth of the cracks will continue.

e) Seasonal groundwater level fluctuation will cause some movement of the foundation mat.

f) In order to examine and evaluate the future performance of the foundation, it is recommended that the current monitoring program be expanded to enable more accurate measurements of differnetial settlements and crack growths. All prominent cracks should be mapped and included in the program.

Enclosure 2

SAFETY EVALUATION OF THE STRUCTURAL ADEQUACY OF WATERFORD 3 BASE MAT

1. This report provides the structural safety evaluation of the "as-built" Waterford 3 mat. Specific conclusions and recommendations to be incorporated as part of the OL license for the plant are also listed herein.

2. Inspection of Base Mat Structure Foundation and Review of Mat Construction Records

The SGEB staff visited the Waterford 3 site on March 27, 1984. Staff observed cracks on the ring wall and wet cooling tower walls. These cracks had not been specifically mapped and brought to the NRC/SGEB staff attention until the March 27, 1984 visit. Some of the cracks were inclined to the vertical axis (perpendicular to the mat) and were joined by a crack on the mat. Thus, these cracks were believed to be shear cracks. Other cracks on the walls and on the mat appeared to be shrinkage or flexure cracks.

At the site, the Structural Engineering staff also reviewed construction records and interviewed some people who participated in the actual construction of the nuclear island foundation and base mat.

3. Analysis and Design of the Concrete Mat

The applicant's analysis of the base mat utilized finite element methods and generally recognized formulas presented in a textbook written by R. J. Roark; these approaches are fundamentally independent of each other. The use of finite element methods in conjunction with electronic computers permits solutions of structures having complex geometry, loading and boundary conditions, such as the Waterford Unit 3 base mat, although correct use of this method is rather difficult. The use of textbook formulas permits solutions for ideal loading and boundary conditions, but must be utilized in conjunction with engineering judgement to obtain solutions for actual (non-ideal) conditions.

In its application of pertinent formulas, the applicant calculated positive bending moment in the mat under the reactor building by assuming a 20% edge fixity of a circular plate under the shield building, and a uniform soil pressure beneath the mat. The applicant calculated negative bending moment under the shield building by assuming a 50% edge fixity and uniform soil pressure under the mat.

In its finite element analysis, the applicant calculated two bending moments in the mat, by using actual loading conditions and two separate soil conditions: constant soil modulus, and variable soil modulus in which the modulus varies in rough proportion to the deformation shape of the mat. The top and bottom reinforcing

steel bars that resist the negative and positive bending moments, respectively, were porportioned in a manner such that a surplus bending moment capacity is always provided. This fact was verified by comparing the three design bending moments calculated for a given location: one derived from use of the formulas and two derived from the finite element analyses. In each of these three analyses, the estimated dead load on top of the mat was multiplied by a factor of 1.5 before being used in calculating the required design bending moments, thus providing the 50% margin (surplus) in load capacity referred to above.

The shear capacity of the base mat was calculated and provided in a manner similar to the bending moment treatment described above: a surplus shear capacity is always provided. Again, this fact was verified by comparing the design shear forces obtained in each of the three calculations. As before, the estimated dead load was multiplied by a factor of 1.5 prior to being used in calculating the required design shear resistance.

The structural engineering staff determined that the procedures and approaches utilized in the applicant's analysis and design of the base mat are sufficiently conservative and are acceptable. The sum of the top and bottom reinforcing steel bars and the vertical shear reinforcing bars have provided adequate strength for the mat to resist the load imposed by the reactor and shield buildings, assuming that the foundation soil behaves as predicted in the

analysis and that construction was carried out properly. However, as discussed in our geotechnical engineering evaluation (enclosure 1), the foundation soil did not behave as predicted in the original analysis. This may indicate that the concrete mat design may be inadequate because it was designed based on ideal conditions. As a confirmatory item, additional analyses using the actual foundation soil conditions are required to validate the adequacy of the foundation mat design.

4. Specific Calculation of Key Block Mat Capacities

Since shear cracks in the reactor shield building and concrete walls were detected during the staff site visit on March 27, 1984, the applicant was requested to perform calculations to obtain shear stresses under operating and SSE conditions, and also shear capacity (strength) for base mat Blocks 5A and 1, where the shear cracks occurred. It was reported by Ebasco via telephone that shear stresses along the crack in Block 5A were 64 k/ft for normal operating loads and 166 k/ft for SSE loads while in Block 1 they are 52 k/ft for operating loads and 210 k/ft for SSE loads. Shear capacity computed in accordance with applicable ACI Code provisions was 274 k/ft for both blocks with shear reinforcing bars contributing 98 k/ft and concrete 176 k/ft. The shear cracks did not appear to present a challenge to the structural integrity of the mat under operating conditions. This is because the shear reinforcing bars alone have provided more than adequate resistance to the computed shear stress. Yet, there is not enough evidence to

draw the same conclusion for the mat under SSE loads by comparing the calculated shear stress of 210 k/ft with the calculated shear capacity of 274 k/ft. This is because the shear capacity was calculated based on ideal conditions, i.e., no cracks and voids. Nondestructive testing methods are recommended to obtain information on cracks and potential voids in the concrete mat so that a realistic assessment of their effect on shear capacity of the mat can be performed. The Waterford NPP is located in a low seismicity area and as a consequence there is a very low likelihood of occurrence of an SSE and associated effects. Moreover, the inherent safety margin in the original design of base mat and related Category I structures, as yet unquantified (because of cracking effects and other questions) seems to be sufficiently adequate to permit the performance of a confirmatory evaluation in the near future. Therefore, the confirmatory requirements may be accomplished during the final licensing stage and after issuance of the OL, except where otherwise specified.

5. Construction Problems

Construction problems described here are limited to the first three blocks of concrete placement where major cracks occurred. Based on the review of construction records and interviews, we find that Louisiana Power and Light (LP&L) quality assurance group did try to make its program a success. Nevertheless, the first three blocks of concrete placement did have quality control problems. These problems included dropping concrete beyond 5' height at times,

using a concrete vibrator improperly (providing insufficient vibration) as well as sledge hammering reinforcing bars to create openings thus transmitting shock waves to the concrete below through vertical reinforcing bars. Deficiency notes were written for the cracking and honeycombing, and the cracking pattern indicates the concrete might have suffered curing problems. A stop work order was issued by LP&L after the concrete placement of the first three blocks, but no drilled cores or nondestructive testing techniques were used to verify the quality and strength of the 5074 cubic yards of poured and hardened concrete to the staff's knowledge.

6. Conclusions and Recommendations

A. The mat is not currently in distress based on the crack observation.

B. Verification of shear capacity under SSE needs to be done. As part of this verification program, nondestructive testing and evaluation are recommended to obtain information on cracks and potential voids and their effect on the concrete mat.

C. The licensee is required to either justify that its original analyses are still adequate in light of the NRC geotechnical engineering staff evaluation mentioned above, or perform additional analyses to account for the actual foundation soil conditions.

D. A general surveillance (monitoring) program is recommended for all the cracks. For shear cracks, the length and size of a crack and its propagation against time should be marked and recorded.

E. Significant corrosion of reinforcing bars due to the ground water is believed to be unlikely at the site. Nevertheless, a surveillance program is recommended.

Enclosure 3

REVIEW OF WATERFORD III BASE MAT ANALYSIS

BY

Brookhaven National Laboratory

April 16, 1984

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.

Because of the very short time interval assigned for the review and preparation of this report (i.e., April 4-13, 1984), it was decided to concentrate the BNL efforts 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 run contains 9 load cases and their various combinations. The input/output printout alone consists of roughly two thousand pages of information and thus only selected portions could be reviewed with some detail. The other sections were however reviewed from an engineering judgement view point. 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.

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

	ELEMENT	Mx D	Mx B	My D	My B	Mxy D	Mxy B	Normal Pressure Side Pressure Mx	Normal Pressure Side Pressure Mx	Normal Pressure Side Pressure 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 x 12 x 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 x 24 x 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 shielded 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 obtained from relatively crude analyses which are based on some generalized analytic solutions available for rigid mats on the surface of elastic soils. The actual design of the mat was based on a series of interactive 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 in that the long term consolication 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 the conversation with HEA engineers, it was told 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 BNL is not clear whether an amplification factor due to vertical mat frequency was used or not. A quick check by the reviewers indicates that this factor could have some influence on the results.

Horizontal earthquake effects were input into the HEA finite element analysis as an equivalent bending moment and in plane (fx_2) shear acting on the pertinent nodes of the foundation mat. The reviewers however, are not certain whether the dynamic interaction effects between the superstructure and the mat were accounted for in the analysis, nor are they certain about its importance in effecting 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 infact 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.

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. Since the lateral pressures have a major impact on the computation stresses in the mat the dynamic effects can significantly in fluence the stresses computed in load combination studies.

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 must 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 condition 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 EBASCO analysis are constant curvature elements so that the computed moments will be constant within each element. The steep moment gradient in the elements listed indicates that a finer mesh would be required to obtain a better representation of element stresses. A similar effect was also noted when investigating the elements forming the junction between the lateral earth retaining walls and the base mat. In general, it is felt that the finite element grid used for the structural modeling is too coarse.

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. Tha plant island is supported by relatively soft over consolidated 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 sellements 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 nuclear power plant structures were employed.

(c) Accepting the information pertaining to loadings, geometries of the structures, material properties and finite element idealization as correct, it is the judgement of the reviewers:

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

(ii) that 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, would have occured after the construction of the superstructure but before the placement of the backfill. Their growth would 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) The validity of the BNL conclusions depend mainly on the information supplied by EBASCO, HEA and LPL, either verbally, in reports or in computer outputs. While some checks for accuracy and engineering approach were made pertaining to the supplied information some open questions still remain, especially those mentioned in the text under topics 4 thru 7 under the heading, "Structural Analysis Topics Reviewed". It is recommended that the particular issues raised under these items be resolved.

Since the Waterford plant is located in a low seismicity zone, there is a low likelihood of occurrence of an SSE and its associated effects. Thus, although the inherent safety margins in the design of the basemat are as yet unquantified (due to cracking effects and the other items mentioned above), they seem to be sufficiently adequate to permit the performance of a confirmatory evaluation for their resolution in the near future.

APPENDIX A-1
LIST OF CONTRIBUTORS

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