April 24, 1984

NOTE FOR : D. Jeng

FROM : J. Ma

SUBJECT : WATERFORD 3 BASE MAT

Since the meeting did not take place yesterday as it was scheduled, I am sending you my revised draft for evaluation and consideration. The revised draft had the benefit of reading J. Chen's report on the evaluation of Waterford 3 foundation soil and further discussions with him.

Last week I told you that I had contact an engineering firm it is capable of performing the type of nondestructive testing as recommended in my revised draft. Please advise me of your decision.

John Ma

cc: G. Lear
L. Heller
J. Chen

FOIA-84-455
E/B. 29

~~DRAFT~~
~~D. Jeng~~
~~16 Apr 84~~

Enclosure 2

STRUCTURAL ADEQUACY AND SAFETY

EVALUATION OF WATERFORD 3 BASE MAT

By John Ma
April 23, 1984

1. This report provides the structural ~~Structural and Geotechnical Engineering Branch's~~ 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

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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 spent approximately four hours to ~~also~~ review~~ed~~ construction records and interview~~ed~~ some people who participated in the actual construction of the ~~nuclear island foundation and~~ concrete base mat.

3. Analysis and Design of the ~~Concrete~~ Base 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 judgment 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 proporation to the deformation shape of the mat. The top and bottom reinforcing steel bars that resist the negative and positive bending moments, respectively, were proportioned 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
~~Based upon my review, I have~~ 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, if one can assume that the foundation soil behaves as predicted in the analysis and that construction was carried out properly.

However, an evaluation by the geotechnical engineering staff has indicated that the soil foundation directly beneath the concrete mat was far from being uniform, as it was assumed in the two static soil-structure analyses, and the distribution of variable soil spring constants ued in the third static soil-structure analysis was opposite to the actual situation. The licensee has not refined its static soil-structure analyses to account for actual soil conditions. Thus, the validity of bending moments and shears obtained from the three static soil-structure analyses and used for design (proportionting sectional strength) is in question. The licensee is required to either justify that the original assumptions of analyses will lead to a more conservative structural design of the mat than the actual foundation soil conditions being used, or perform additional analyses to account for the actual foundation soil conditions.

4. Specific Calculation of Key Block Mat Capacities.

Since shear cracks in the reactor shield building~~s~~ and ~~a~~ 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

Yet, there are not enough evidence to draw the same conclus for the mat under SSE loads simply by comparing the calculate shear stress of 210 $^{K}/ft$ with the calculated shear strength of 274 $^{K}/ft$. This is because that the shear strength was calculated basing on ideal conditions, i.e. no cracks and voids, and that cracks and voids in concrete do reduce its shear strength. Nondestructive testing metho are recommended to obtain information on cracks and void in the concrete mat so that a realistic assessment of shear strength of the mat can be performed.

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 Louisianna Power and Light (LP & L) quality assurance ~~group~~ QA 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. ~~These problems are believed to be the main contributors to concrete cracking and honeycombing observed during construction in late 1975 and early 1976.~~ Deficiency notes were written for the cracking and honeycombing ~~, but no non-conformance report (NCR) was initiated~~ and the cracking pattern indicates the concrete might suffer curing proble. 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 ~~paved~~ poured and hardened concrete to the staff knowledge.

There were construction problems in the first three placement of concrete blocks. However, the degree of deficiency is unclear. Reviewing construction records is being undertake by other NRC engineering staff members. Their input together with the nondestructive testing results of cracks and voids is needed to form a basis for an realistic assessment of the structural adequacy of the concrete base mat.

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

"ADVANCE COPY"
May, 1984

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

MEMORANDUM FOR: Dennis Crutchfield,
Waterford Team Leader
Division of Licensing

FROM: Richard H. Vollmer, Director
Division of Engineering

SUBJECT: WATERFORD 3 BASEMAT EVALUATION

On April 27, 1983, a report entitled, "Safety Evaluation of the Structural Adequacy of the Waterford 3 Basemat" was sent by George Lear to you. As stated in your May 8, 1984 memorandum-to-file, the April 27, 1983 report "may be revised". Accordingly, upon the receipt of additionalinformation from the NRC consultants (BNL and R. Philleo) and after discussions with the various members of your review team, a revision of the earlier report was prepared and is enclosed.

Included in the report are affidavits prepared by the staff's geotechnical engineer reviewer, Dr. John Chen and the structural engineer reviewer, Dr. John Ma. Also, enclosed either as a part of the document or referenced therein are the reports by staff consultants, BNL and R. Philleo, as well as the Chemical Engineering Branch.

Briefly, the conclusions common to the reviewers and consultants are: 1) the basemat can perform its intended function; 2) a surveillance (monitoring) program will be needed to assure its continued adequacy; and 3) additional response from the applicant for confirmation of certain issues and the preparation of acceptable technical specifications for the surveillance program are needed. Should a hearing be required or should the applicant be unable or unwilling to do certain confirmatory studies, then additional funding for further participation by BNL may be needed. We are ready to meet with the applicant to initiate resolution of confirmatory issues and to develop the technical specifications.

Richard H. Vollmer, Director
Division of Engineering

Enclosure: As stated

cc: J. Knight
L. Shao
T. Novak
S. Turk
G. Lear

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E/B.30

WATERFORD 3 BASEMAT EVALUATION

STRUCTURAL & GEOTECHNICAL ENGINEERING BRANCH

DIVISION OF ENGINEERING

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, Division of Engineering (SGEB, DE) was assigned the task of re-evaluating the structural adequacy of the basemat structures at the Waterford Nuclear Power Plant. Concern was focused on the effect of cracks which had occurred in the concrete during 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 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 staff, the DE staff and its consultants have completed evaluations of the structural adequacy of the basemat.

These evaluations are found in the affidavits of the SGEB reviewers for geotechnical enginering and structural engineering (Attachments 1 and 2). The Chemical Engineering Branch, Division of Engineering, has provided an evaluation (Attachment 3) of corrosion potential. The SGEB consultant, Brookhaven National Laboratory (BNL), has provided an independent evaluation (Attachment 4) with conclusions that are supportive of those of the NRC staff. Robert E. Philleo, a consultant to Mr. Larry Shao, (NRC supervisor for allegations research and resolution of other civil/mechanical issues at Waterford) has also prepared a report which has been considered and included as a reference in the SGEB structural engineer's affidavit.

Briefly, the conclusions common to all of the reviewers or consultants are:

1. The basemat can provide its intended function.
2. An acceptable surveillance (monitoring) program is needed to assure continued adequacy of affected structures.
3. Additional response from the applicant for confirmation of certain issues and the preparation of acceptable technical specifications for the surveillance program are needed.

Details of these conclusions with related recommendations are presented in the attached documents.

Attachment 1

Geotechnical Engineering Evaluation of Concrete Cracking in the Basemat Waterford No. 3

John T. Chen, Geotechnical Engineer

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

In July 1977, a number of east-west oriented cracks were discovered at the top of the mat within the ringwall for the containment structure (Ref. 3 & 4). Weeping water was reported to be low and just 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 ringwall.

In May 1983, new cracks (not reported in 1977) and accompanying weeping water were discovered in the base mat outside the containment structure (Ref. 3). Some of those cracks were found to extend to vertical walls and to extend up those walls by an NRC investigation team (Lear, Ma, Jeng and Chen) in March, 1984.

This evaluation of the geotechnical engineering related causes which may have contributed to the observed cracking presents foundation conditions and anticipated future behavior of the mat and was based on the review of the referenced documents, field observation, and meetings held with the applicant on March 23 and 27, 1984. Other possible causes of the observed cracks are discussed elsewhere (Ref. 8). The subsurface conditions and significant soil characteristics were presented in the Waterford SER Section 2.5.4.1. The construction sequence was presented in SER Section 2.5.4.2.

2. EVALUATION

The plant, as stated in Reference 1, was designed to give a net reduction, by about 200 psf, of the applied effective soil loading at foundation level (El.-48 ft.). 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 should be negligible. The ultimate bearing capacity was calculated to be 15,000 psf, thus, there is no potential for bearing type failure and the bearing capacity is adequate.

During construction, the insitu vertical stresses were controlled by lowering the groundwater level simultaneously with the excavation 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 total and 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 the subsurface soil conditions were somewhat different than anticipated. Numerous construction difficulties, encountered during construction, may have caused some differential settlements which may have contributed directly or indirectly to the observed cracking of the foundation mat; those difficulties encountered during construction included:

(a) Dewatering:

As discussed in Waterford SER·Section 2.5.4.2 (Ref. 1), 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, all the wells did not completely lower the groundwater level in the foundation soils to below El. -48, as evidenced by some of the piezometric readings (Ref. 6). 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 (Ref. 5).

(b) Variable foundation soil conditions:

The foundation mat was founded at elevation-47 on the upper Pleistocene clay. These clays were considered to be fairly uniform and over-consolidated in the design and construction of the mat (Ref. 1 & 7). 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 (Ref. 6). The measured heave at various locations was 2 to 4 times the anticipated maximum heave used in the mat design; this indicates that the differential settlements of the mat during construction would be greater than anticipated and the induced stresses might be significant enough to cause concrete cracking.

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

The compaction procedures, using a vibratory roller for 10 passes, were selected based on the results of a test fill program (Ref. 1 & 5). However, due to the variability of the supporting soil and groundwater conditions, despite occasional greater effort up to 40 passes, the degree of compaction achieved in these shell filter strips varied widely, from 80 to 98 percent (Ref. 5). Compaction of fill (shells) over a spongy subgrade is more difficult than over a solid subgrade. 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 about 120 sq. ft. occurred in strip number 2 during compaction. Filter strip number 4, 48.5 feet long, was compacted to 98 percent. All filter strips were to be 1 foot in thickness.

These variable degrees of shell compaction reflect the condition and consolidation of the underlying foundation soils indicating that the subgrade moduli varied among these strips. Settlements of the mat due to uniform structural loads would be expected to vary accordingly; strip number 2 would be expected to settle more than strip number 1 while strip number 4 would settle less. The resulting differential settlement may have induced bending stresses in the mat and caused east-west oriented cracking in the newly placed foundation mat. Subsequently, differential settlements would be experienced by the superstructure founded over different strips with variable soil properties and rates of consolidation.

(d) Foundation mat construction sequence:

As stated above, from December 1975 to May 1976 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 on the subgrade due to pouring of the first block of concrete caused a measured settlement about 3/4 of an inch and, later, some additional consolidation settlement (Ref. 6 & 7). After the second and third blocks were poured adjacent to the first block, differential settlements between the top of the completed 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. If the residual stresses were large enough, they may have caused concrete cracking or may have caused preexisting cracks to expand further.

(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 (Ref. 6). 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 stresses in the concrete when it was still in the process of curing, contributing to the concrete cracking.

The plant foundation design, the "compensated" foundation concept, is a sound one. The cracks which may have been initiated due to thermal stresses or shrinkage (Ref. 8), in the foundation mat appear to have been affected significantly by the differential settlements experienced and, to a lesser degree, by superstructure loads as they were applied during construction. The differential settlements were caused mainly by the variable soil conditions, high groundwater levels, variable compaction of the shell filter strips, and foundation mat construction sequence. 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 applicant performed a detailed soil-structure interaction analysis to evaluate the effects of changes in the values of the subgrade modulus used in the design of the concrete mat (Ref. 2 &

7). However, those difficulties encountered during construction and mentioned above have not been considered in the applicant's analysis. To evaluate the potential for future cracking, the effects of differential settlements during construction should be determined so that the current state of stresses in the base mat can be better assessed. The soil shear moduli to be used in such an analysis should reflect more closely the soil conditions that existed during construction, when the foundation soil was in the process of being consolidated.

The future settlement is expected to be negligible because of the "compensated" foundation design. The results of the current settlement monitoring program show that the overall settlement of the mat has been essentially stable since 1979, with some minor movements (about ½ inch) due to seasonal groundwater level fluctuations (Ref. 6). The cracks reported in 1983 and vertical wall cracks discovered in 1984 seem to indicate that movements of the foundation mat and growth of cracks are continuing. The current settlement monitoring program reveals that the mat moves in conjunction with fluctuation of groundwater levels. Unfortunately, the scope and accuracy of the current monitoring 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 this relationship.

The scope of the applicant's current monitoring program should be expanded to collect more useful and accurate information about the differential settlements in the mat and about the precise growth of all prominent cracks. 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 should provide information about the development of new cracks and the propagation of the cracks, particularly those cracks that extend to the vertical walls.

3. CONCLUSION AND RECOMMENDATION

Based on the information reviewed, it is concluded that:

(a) The plant foundation design, the "compensated' foundation concept, is sound and acceptable. The soil bearing capacity is adequate and the future settlement should be negligible.

(b) The east-west oriented cracks in the foundation mat and structural walls may have been caused or further aggravated by differential settlements that occurred mainly during construction.

(c) These differential settlements resulted from complicated soil conditions, high groundwater levels, variable compaction of shell filter strips and foundation mat construction sequence.

(d) Movements of the foundation mat, probably less than an inch, as the mat rises and falls in conjunction with seasonal groundwater level fluctuation, will continue. In addition the cracks may be expected to continue.

(e) A more refined analysis using the soil conditions disclosed during construction should be performed to determine the effects of past and future differential settlement on the potential for cracking of the concrete mat.

(f) In order to better examine and evaluate differential settlement and possible cracking of the foundation mat, it is recommended that the currently proposed 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 monitoring program.

References

1. Safety Evaluation Report (SER) Related to the Operation of Waterford Steam Electric Station, Unit No. 3 (NUREG-0787, July 1981) (2.5.4);

2. Letter from the Applicant to the NRC Staff dated June 24, 1981 (Subject: Response to SER Open Item 49, "Reevaluate Foundation Mat for Changes in Value of Subgrade Modulus");

3. Harstead Associates, Inc., Waterford III SES Analysis of Cracks and Water Seepage in Foundation Mat, Report No. 8304-1, September 19, 1983;

4. Amended and Supplemental Motion to Reopen Contention 22, December 12, 1983, Atomic Safety and Licensing Appeal Board;

5. Nonconformance Report W3-5997, Clam Shell Filter Blanket Under the Nuclear Island, LP&L, June 23, 1983.

6. LP&L's Draft Responses to NRC's Question on Waterford 3 Basemat, March 26, 1984;

7. Affidavit of R. Pichumani on the Stability of the Foundation Underlying the Concrete Mat at Waterford 3, Nov. 1983;

8. R. E. Philleo, Evaluation of Concrete in the Basemat, Waterford 3, May 8, 1984.

Attachment II

Structural Engineering Evaluation
of Concrete Cracking in the Basemat
Waterford Unit No. 3
John Ma, Structural Engineer

1. INTRODUCTION

The adequacy of the Waterford Unit 3 foundation base mat in light of the discovery of concrete cracking and water seepage in the mat was assessed and documented in my earlier affidavit.[1] The adequacy of the same mat is reassessed in light of new information. The new information was obtained from: (a) observation during a one day site visit, (b) a geotechnical engineering staff report prepared by Dr. J. Chen,[2] (c) a report prepared by Mr. Robert E. Philleo[3], and (d) data furnished by Ebasco Services, Inc. (Enclosure 1).

In the evaluation, the observation of cracks on concrete surfaces, the review of records, and the interviews of various individuals during the site visit are described first. The possible existence of diagonal tension cracking inside the mat is then hypothesized. The adequacy of the analysis and design methods used for the mat is reexamined in light of the new information. Surveillance (monitoring) programs are discussed. Conclusions and recommendations are finally stated.

2. CRACK OBSERVATION, RECORD REVIEW, AND INTERVIEW

I visited the Waterford 3 site on March 27, 1984, and observed cracks on the ring wall and wet cooling tower walls. These cracks had not been mapped or brought to my 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. This type of crack seems to be more complicated and severe than the flexural cracks on top of the mat as previously reported.

At the site, I also reviewed construction records and interviewed various individuals who participated in the construction of the foundation base mat. Based on the review of construction records and interviews, I found that despite the effort of Applicant's quality assurance organization, the first three blocks of concrete placement, where major cracks occurred, did have quality control problems. These problems included (1) dropping concrete beyond 5' height at times, (2) using a concrete vibrator improperly and providing insufficient vibration, as well as (3) one instance of sledge hammering a reinforcing bar to make room for a concrete-placing elephant trunk, thus transmitting shock waves to the concrete below through vertical reinforcing bars. Such action could lead to cracking concrete or creating voids around reinforcing bars. Deficiency notes were written for observed cracking and honeycombing detected in vertical walls of the concrete blocks on concrete surface, and the records showed that the deficiencies were repaired. These quality control problems were later evaluated and reported by Mr. R. Philleo[3] as not significant enough to impair the structural integrity of the foundation base mat. Action to eliminate such deficiencies resulted finally in a stop work order, issued by LP&L after the concrete placement of the first three blocks. When the construction was resumed, quality control was improved.

3. THE HYPOTHESIS OF DIAGONAL TENSION CRACKING

The most dominant cracking pattern observed on the top face of the mat is the numerous parallel cracks generally running in the East-West direction. The lengths of these cracks almost extend to the entire width of the mat. This type of cracking pattern suggests that one-way slab (beam) action in the longitudinal (North-South) direction is predominant. Diagonal tension cracking associated with this type of beam action is possible and is believed to have the most potential to affect the integrity of the mat.

The mechanism of forming diagonal tension cracking is fairly well understood, having been studied in the laboratory as well as theoretically. An element at the neutral plane (axis) of the mat in the longitudinal direction would be subject to a shear stress but would not be subject to flexural stress. Along the 45° line (diagonal) with the neutral plane, tensile (diagonal tension) stress with a magnitude equal to the shear stress will develop. When the tensile (diagonal tension) stress exceeds the tensile strength of concrete, a crack of 45° slope opens. This type of crack is termed a diagonal tension crack. When the crack propagates away from the neutral plane, the slope of the crack changes gradually due to the influence of flexural stress. Since diagonal tension stress is related to shear stress, and shear stress is normally computed, but diagonal tension stress is not, in structural analysis, shear stress has long been and is still being used as a measure of diagonal tension stress. Therefore, shear capacity in concrete beams or one-way slabs usually means diagonal tension capacity, and the reader is reminded that the shear capacity referred to in Enclosure 1 is actually diagonal tension capacity. For better understanding, a diagonal tension crack in a test beam and its development is shown in Enclosure 2, which is an excerpt from a text book "Reinforced Concrete Fundamentals" 4th edition by P. M. Ferguson.

Diagonal tension stress was introduced in the foundation base mat during construction, even before any external load was applied. There were three contributing factors to the diagonal tension stress during construction and all related to foundation soil.

The concrete placement of the mat was poured one block at a time. Each newly poured block experienced an immediate settlement of about 3/4 of an inch while the existing blocks adjacent to it then settled to a much lesser extent. The restraint provided by means of a shear key and reinforcing bars at the interface between the old and new blocks created shear stress and, in turn, diagonal tension stress. Concrete placed on top of the foundation soil under strip number 2 tended to settle more than the concrete in strip number 1, due to the differences in foundation soil stiffness under these strips (discussed in the goetechnical engineering evaluation). This uneven settlement would have generated diagonal tension stress . Significant hydrostatic pressure changes (discussed in the geotechnical engineering evaluation) in conjunction with the non-uniform nature of foundation soil underneath the mat, during concrete placement of the mat, similarly would have produced diagonal tension stress. Whether these factors acted alone or in combination in causing diagonal tension cracking within the mat is unknown, because the cracks are not visible on the surface, except as flexural cracks. None of these factors was included in the design analyses performed for the mat. Thus, the effect of these factors in contributing to the diagonal tension cracking has not been quantified.

If a diagonal tension crack does exist in the mat, it is possible and likely to join the flexural cracks near the top of the mat and to extend to the flexural cracks likely to be present at the bottom of the mat. My experience in testing has indicated that the joining of a diagonal crack and a flexural crack is not only possible and likely, but also almost certain is this case. This type of through crack would permit the ground water, under hydrostatic pressure, to seep up through the mat. The effect of this type of crack is discussed in the next section.

4. REASSESSMENT OF THE ADEQUACY OF ANALYSIS AND DESIGN

In my previous affidavit, I had determined that the analysis and design of the mat were adequate. I further stated that any conclusion was not altered by the concrete cracking that had been discovered. This was because the cracks were reported as "hairline" in size and were believed to be flexural cracks. This type of flexural crack was considered in the (ultimate) strength design method, which was used by Ebasco Services, Inc. in designing the mat. However, certain quality control problems experienced during concrete placement, the new information on differential settlements of foundation soils, and the new discovery of additional floor cracks which extend to and up the wet cooling tower wall and ring wall, point to the need for a re-examination of the adequacy of the analysis and design of the mat.

Concrete quality control problems were evaluated by Robert E. Philleo, an NRC staff consultant on concrete construction adequacy. His report[3] indicates that the assumed concrete compressive strength of 4000 psi in design was attained. He also indicates that the assumed transfer of force from one reinforcing bar to the adjacent one through caldwelding may be assumed to have been attained, and that the bond between the concrete and reinforcing steel was attained. In short, the quality control problems were not significant enough to invalidate the original reinforced concrete base mat analysis and design.

The new information on uneven settlements of foundation soil and differential ground movements raise other concerns. When one portion of the mat is pushed upward or settles down relative to another portion of the mat, shear stress (diagonal tension stress) is created. These particular types of movements and associated stresses had not been included in the original design analysis, and thus were not specifically designed for. It is not clear as to whether the diagonal tension capacity of the mat can accommodate the additional shear stress (diagonal tension stress).

To permit further evaluation of the diagonal tension capacity of the mat, Enclosure 1 provides diagonal tension capacity and stresses in the mat in two regions where I believe that diagonal tension cracks were most likely to occur. Shear stress calculations shown in Enclosure 1 do not include those induced due to uneven settlement of foundation soil and differential ground movements. It is reported in Enclosure 1 that shear stresses along major crack in Block 5A (see Enclosure 3) were 64 k/ft for normal operating loads (no earthquake) and 166 k/ft for loading combinations including Design Basis Earthquake (DBE) which is equivalent to Safe Shutdown

Earthquake (SSE), while in Block 1 (see Enclosure 3) they are 52 k/ft for normal operating loads and 210 k/ft for loading combinations including SSE. It is also reported that the shear capacity was 274 k/ft for both blocks with shear reinforcing steel contributing 98 k/ft and concrete 176 k/ft. Based on the above numbers, it is shown that the diagonal tension capacity is substantially greater than the diagonal tensile stresses under normal operating loads. Under the DBE (SSE) condition, there is still some margin left between the shear stress generated under DBE and the ultimate shear (diagonal tension) capacity. The lack of physical information on the potential existence of diagonal tension cracks in the mat combined with a yet uncalculated diagonal tension stress due to uneven settlement of foundation soil and differential ground movements, as noted before, makes it difficult to draw conclusions on the adequacy of the above noted margin. Therefore, additional analysis, which accounts for the actual soil condition during concrete-placement, should be performed and non-destructive testing should also be used to detect and locate any major diagonal tension cracks. The information, thus obtained, should provide a high level of confidence in assessing the adequacy of the cracked mat.

At present, the adequacy of the mat in terms of diagonal tension can only be judged based on the information contained in Enclosure 1, and the pattern and size of surface cracks. Since the diagonal tensile stress is much less than the diagonal tension capacity under normal operating loads and the widths of surface cracks are small, the mat is safe under operating loads. However, there are not enough data or information to predict, with a great confidence, the adequacy of the margin to a diagonal tension failure under DBE (SSE). When the diagonal tension capacity is exceeded within a partial mat width or over an entire mat width, one portion of the mat may slide downward, and/or rotate relative to the other along the face of the crack. If this failure mode were to occur, the sliding movement of the mat itself will be gradual, limited, and not catastrophic because the foundation soil underneath it has adequate bearing capacity. The vertical shear (diagonal tension) reinforcing steel may yield and the horizontal flexural reinforcing steel may form a kink, but none will break. Although the mat will not collapse even when the diagonal tension capacity of the mat is exceeded, the response of the mat under DBE (SSE) may deviate from what was originally predicted in elastic analysis assuming the mat was a monolithic piece.

The degree of such deviation depends on the size of the diagonal tension crack and the length across the width of the mat. However, the current knowledge can not provide a quantative relationship. If the response of the mat deviates from its original prediction as a result of diagonal tension failure, the response of Category I structures, safety class equipments and piping which are supported by the mat will also deviate from their original predictions. This situation should not be allowed to occur and it must and can be prevented by providing localized prestressed tendons to tighten the diagonal tension cracks. Moreover, repair to the mat may be difficult and costly after the zigzagged type of a crack surface (interface) is destroyed by the hypothesized sliding action. From engineering and economic point of view, the sooner the non-destructive testing and additional analysis data are available the better for LP&L. However, these data are not required prior to licensing, because (1) it is believed that the mat possesses enough diagonal tension capacity against DBE (SSE) although the confidence level of this believe is not confortably high due to information yet to be obtained from the non-destructive testing and additional analysis as described earlier and (2) in the event of the DBE, that the diagonal tension failure was to occur, the mode of failure will be gradual and limited.

5. SURVEILLANCE (MONITORING) PROGRAMS

In my previous affidavit, I recommended a surveillance program for the ground water. It is now apparent that a surveillance program for the concrete cracks should also be instituted.

There are two types of causes of concrete cracking, namely volume change and external load. Thermal contraction and shrinkage of concrete both belong to the type of volume change. Concrete contracts following the dissipation of the heat of hydration as the concrete hardens. Concrete shrinks when it loses moisture by evaporation. If restrainted, the concrete strain due to contraction or shrinkage may cause cracking. This type of cracking, if it develops, would have occurred during the concrete construction stage, and would not occur to the mat now or in the future. The other type of cracking is related to external loads. The pattern of cracking, the width of the cracks, and the propagation of the cracks reflect how a structure responds to external loads.

The surveillance program for concrete cracking should include the marking and recording of the length of a crack and its propagation against time. For those cracks which appear to have greater impact on the structural integrity than others, the width of those cracks should also be recorded as a function of time. The result of the non-destructive testing should be used as a basis to modify the crack surveillance program.

6. CONCLUSION AND RECOMMENDATION

The adequacy of the Waterford Unit 3 foundation base mat was reassessed in light of new information presented. It is concluded that the as-built mat is adequate to serve its intended purpose.

The most likely failure mode of the mat is believed to be of the diagonal tension type in one-way slab action in the longitudinal direction. The concrete placement sequences, uneven settlements of foundation soil, and differential ground movements under the mat have all contributed to potential diagonal tension problems. None of these contributions was included in Ebasco's design analysis. Therefore, the additional analysis earlier described, which accounts for the actual soil conditions during concrete placement should be performed.

Since diagonal tension cracks are not visible from the surface of the mat, non-destructive testing should be conducted to detect and locate such cracks. The data obtained from non-destructive testing may be used in conjunction with the results from the additional analysis, thereby providing useful information as to whether there is a need for strengthening diagonal tension capacity.

Diagonal tension failure in the mat is judged to be unlikely but possible under the design loads and, should it occur, will not be a catastrophic one, but a gradual and limiting sliding, and/or rotating movements between the two faces of a crack. This is because the foundation soil underneath it has adequate bearing capacity. However, this type of failure must and can be eliminated with a great confidence by providing localized prestressed tendons to tighten the cracks.

The Applicant must develop and implement a surveillance (monitoring) program for ground water and concrete cracking.

References

1. Affidavit of John S. Ma, Before the Atomic Safety and Licensing Appeal Board in the matter of Louisiana Power and Light Company (Waterford Steam Electric Station Unit 3). NRC Docket No. 50-382.

2. "Geotechnical Engineering Evaluation of Concrete Cracking in the Basemat, Waterford No. 3" by J. T. Chen.

3. "Evaluation of Concrete Construction Adequacy in the Basemat, Waterford Unit No. 3", by R. E. Philleo, May 18, 1984.

ENCLOSURE 1

REQUEST FOR ADDITIONAL INFORMATION
WATERFORD NPP - STRUCTURE ENGINEERING

J. Ma's Question on 4/4/84

Provide shear capacity and design shear stress in the mat in two regions:

A. Bounded by column line 12M and 7FH in N-S direction and T2 and R in E-W direction. This shear stress and shear capacity is measured along the 45° line from R column line toward column 12M.

B. Bounded by column line 12M and 7FH in N-S direction and column line R and P. This shear capacity and stress should be E-W direction.

<u>Response</u>

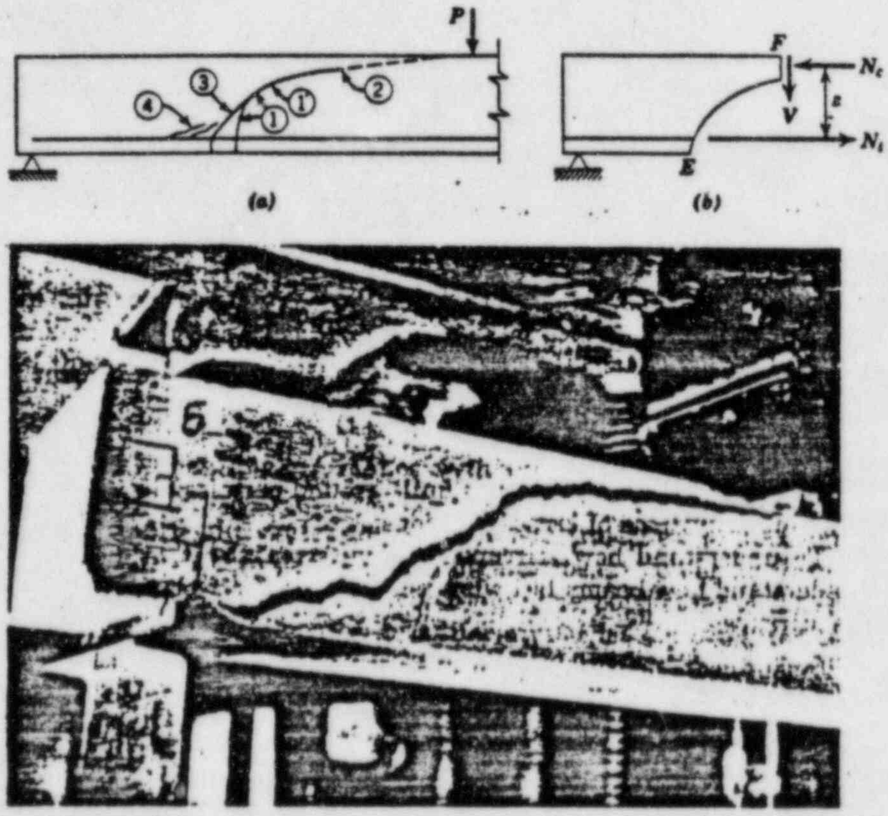
A. The design shear stress under normal operation condition (Load Factor = 1.0) along the 45° line as defined is 64 K/ft. The ultimate shear capacity of the mat is 274 K/ft which includes 98 K/ft. from shear reinforcement (#11 @ 3'-0 center each way), and 176 K/ft. from concrete. The allowable concrete unit shear stress is calculated based on $2\phi\sqrt{fc'}$, where $\phi = 0.85$ and fc' are 4,000 psi.

   The design shear stress under DBE loading combination is 166 K/ft.

B. The design shear stress under normal operation condition is 52 K/ft, and the ultimate shear capacity same as "A", 274 K/ft.

   The design shear stress under DBE loading combination is 210 K/ft.

ENCLOSURE 2



(c)

FIGURE 5.2 Development of a diagonal tension crack when loads and reactions are far apart. (*a*) Diagram showing sequence in crack formation. (*b*) Equilibrium sketch for portion of beam. (*c*) Failure of beam. The failure crack developed from the flexural crack faintly seen about one beam depth from the end. This crack turned gradually into the diagonal crack, as at 1 in the sketch. The final wide crack is comparable to 2–1–3–4 in (*a*) (The failure picture has been inverted to make this comparison easier).

ATTACHMENT III

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

APR 12 1984

NOTE TO: George Lear, Chief, SGEB

FROM: Victor Benaroya, Chief, CMEB

SUBJECT: CORROSION EFFECTS ON BASEMAT REBAR AT WATERFORD III

We have 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.

We considered the following factors in our evaluation:

1. 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).

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

3. 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.

4. The corrosion rate of steel by alkaline water is low.

On the basis of our evaluation, we find that there is reasonable assurance that the basemat rebar will not be significantly corroded by the penetration of groundwater of the acidity and chloride content observed at the Waterford site.

The board required monitoring the quality of groundwater at the Waterford site. The licensee has prepared a Limiting Condition for Operation requiring the analysis of a sample of groundwater at least once per 92 days to verify that the chloride content does not exceed 250 ppm. On the basis of the above evaluation, where the time element is not critical, we conclude that the proposed Limiting Condition for Operation is acceptable.

Vic Benaroya

Victor Benaroya, Chief
Chemical Engineering Branch

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