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

August 2, 1984

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U.S. Nuclear Regulatory Commission
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In the Matter of
LOUISIANA POWER AND LIGHT COMPANY
(Waterford Steam Electric Station, Unit 3)
Docket No. 50-382 OL

Dear Members of the Appeal Board:

Certain facts have come to my attention during the past few days, concerning Harstead Engineering Associates, Inc., consultant on base mat issues to Louisiana Power & Light Company (LP&L), as to which you should be advised.

First, Mr. Gunnar Harstead (HEA Principal) participated as a member of the NRC Staff's structural audit team during a one-week structural audit performed by the Staff at the Waterford facility in April 1981; this audit appears to have involved all Category I structures, and included consideration of the foundation base mat. To the best of my knowledge, Mr. Harstead served as a consultant to the Staff's structural audit team and has not been involved subsequently in the Staff's reviews of the Waterford application. Mr. Harstead prepared notes in this regard, which he provided in July 1983 to a member of the Office of Inspection and Enforcement's Inquiry Team as background material related to the Inquiry Team's assessment of base mat cracking and water seepage; a copy of those notes is attached to this letter.

Second, Mr. Harstead has continued to provide consulting services to the Staff until very recently, and may be continuing to do so, in connection with other facilities. These services were provided under a personal services contract which, I am informed, expired on June 30, 1984. As far as I have been able to ascertain, these services were provided in connection with the Staff's review of the Midland, Seabrook, Byron, and River Bend applications, and it is possible that other facilities were involved, as well.

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While we believe that these facts should be brought to your attention, we do not consider that they affect the Staff's review of the Waterford base mat or the Appeal Board's determinations in this proceeding. First, the I&E Inquiry Team did not place any reliance on the work performed or the notes provided by Mr. Harstead; on the contrary, the Inquiry Team identified various concerns related to the base mat and recommended that LP&L undertake a comprehensive independent evaluation in order to resolve those concerns. (See Board Notification BN-83-133, September 15, 1983). In addition, Dr. John Ma, whose affidavit was submitted to the Appeal Board in November 1983, performed an independent structural audit of LP&L's analysis and design of the foundation base mat in support of his conclusions as to the base mat's adequacy. (See Affidavit of John S. Ma, filed November 28, 1983, at 1-2). Further, the Staff has now performed a second assessment of base mat adequacy in light of the cracking that has been discovered, the results of which will be forwarded to the Appeal Board on August 7, 1984. This most recent assessment of the base mat relies to a significant extent on an independent evaluation prepared by the Structural Analysis Division of the Brookhaven National Laboratory, whose efforts are unaffected by any relationship between the Staff and Mr. Harstead.

I regret that these facts were not brought to your attention sooner. However, as discussed above, the Staff does not consider that they affect the Staff's review efforts or the Appeal Board's determinations in this proceeding.

Sincerely,

Sherwin E. Turk

Sherwin E. Turk
Deputy Assistant Chief
Hearing Counsel

Attachment: As Stated

cc: See Page 3

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My personal notes of Waterford Structural Audit during Week of April 6, 1981. Gunnar Rec'd 7/1/83

1.0 Introduction

A week long structural audit was conducted for the Waterford 3 Nuclear Power Station at the headquarters of Ebasco Services Corp. the designers of the plant.

The members of the NRC team are as follows:

F. Rinaldi

P. Huang

J. Matra

G. Harstead

The audit covers the structural design criteria and design procedures used in the design. The information contained herein was supplied by Ebasco personnel.

2.0 General Description

All Seismic Category I Buildings and Structures are located on a common mat. The containment structure is a steel vessel enclosed with the reactor shield building. A four foot annulus was provided between the cylinders of the steel containment and the reinforced concrete shield building.

The stated reason for buildings on a single mat was to avoid the possibility of significant differential settlement of the buildings.

3.0 Geotechnical Investigation

The geotechnical work was performed by LAW Engineering. Field testing consisted of determining shear wave velocities by means of cross hole seismic testing. Laboratory testing consisted of resonant column tests and triaxial tests. Apparently a Soil Shear Modulus was determined to be about 6400 psi.

Law Engineering also developed the artificial time history ground motion based upon the criteria site response spectrum. The site response spectrum used is lower than that required by NRC Reg. Guide 1.60; however, the spectrum calculated from the artificial ground motion generally exceeds that required by NRC Reg. Guide 1.60. Where the calculated spectrum is below that required by NRC Reg. Guide 1.60 the difference does not appear to be significant.

4.0 Mat Design

The structural mat is 12'-0" thick and has been termed as a "floating" foundation mat. The term floating is; however, an inappropriate term in that hydrostatic pressures acting on the bottom surface of the mat will not exceed the dead and permanent live loads of the structures and mat supported by the saturated soil.

The construction and design concept of the mat was described as follows:

1. The in-situ soil pressure at El. 47'-0" is 3.3 KSF
2. The site is dewatered, increasing soil pressure at

El. 47'-0" to 6.5 KSF. This will consolidate the soil.

3. The site is excavated to El. 47'-0" and constuction proceeds.

4. The dead load increases pressure to about 4 KSF. Additional dead load is counterbalanced by gradually lessening the dewatering. A constant soil pressure of about 4 KSF was maintained during construction.

5. Upon completion of the construction and removal of dewatering the soil bearing pressure is 3.1 KSF.

6. The fact that the final net soil bearing pressure at El. 47'-0" is 3.1 KSF compared to in-situ soil bearing pressure of 3.3 KSF gave rise to the floating mat terminology.

7. Even during the maximum flood there remains a net soil bearing pressure at El. 47'-0", ensuring that the plant will not float down the Mississippi River.

The analysis of the mat was performed using a finite element program. The stiffening effects of shear walls was included in the model. Two cases were examined, one, using a constant subgrade modulus of 150 lb/in^3, and two, where the subgrade modulus was 70 lb/in^3 within the reactor building area, 110 lb/in^3 surrouding the reactor building, and 150 lb/in^3 elsewhere. These adjustments were made in order to account for the fact that the subgrade modulus would decrease for increaing soil strain.

In addition, a rigid mat analysis was performed. The fundamental assumption of a rigid mat analysis is that the soil bearing pressure is uniform. The moments and shears in the mat are calculated for both the applied dead and live loads and the uniform soil bearing pressure. This method generally leads to conservative results.

Therefore, three sets of results were obtained for the mat. The mat was reinforced for moment and shear for the envelope of these three sets of results.

5.0 Dynamic Analysis

5.1 Mathematical Model

5.1.1 Ebasco Model

The model is one that is usually refered to as a stick model, i.e. lumped masses connected by massless springs. Five cantilevers represent the Containment Vessel, Reactor Shield Building, Fuel Handling Building, Reactor Auxiliary Building, and the Combined Structure. The five cantilevers are joined at a node representing the base mat. The cantilevers are not mathematically tied together at any other point and they are all located at the same vertical axis.

The base mat is attached to the rigid base by means of soil springs. These spring constants are dependent upon the Soil Shear Modulus and geometry of the base mat. The formulas are taken from standard references. Due to uncertainity of the soil spring calculation runs were made with three sets, spring constants based upon of soil Shear Modulii of 5800, 800, and 16050 psi.

5.1.2 Stardyne Model

In order to ascertain the effect of eccentricity of masses with respect to the shear center of each cantilever as well as eccentricity of each cantilever to the shear center of the soil springs, a model was prepared taking these eccentricities into account. This introduces a torsional degree of freedom for the model.

No torsional soil spring was added; therefore, this degree of freedom did not appear.

5.1.3 Comparison of Results

The runs were both made using soil springs calculated from the greatest value of Soil Shear Modulus. Although this value of Soil Shear Modulus was more than double the value recommended by Law Engineering, the system is still somewhat flexible. The fundamental period, T, equals 0.6 seconds, which is not within the peak acceleration range of the spectra specified by Reg. Guide 1.60. However it does appear that a period of 0.6 seconds will for the spectra developed by Law Engineering, result in value of acceleration which will exceed the specified spectra of Reg. Guide. 1.60.

A comparison of the two runs indicated by resulting accelerations at selected mass points were in the same range. Two different programs were used with possibly different methods of calculating model dumping and the fact that no torsional soil spring was used in the torsional model. Therefore the specific purpose of determining differences due torsional effects, was not satisfactorially achieved. Even though the exterior walls do provide a structural tie between the Fuel Building and Auxiliary Building, this was not accounted for in the model.

5.1.4 General Comments

a. Mode Shapes

A review of mode shapes of the two computer runs was made. It appeared that the first two modes of Stardyne run indicated a response similar to a rigid block supported by a horizontal spring and a rocking spring. A study of the mode shapes of the Ebasco program didn't seem to exhibit this type of response. However, studies of the mode shapes from the computer output print out was somewhat unwieldy, plots of mode shapes are recommended.

b. Earthquale Combinations

Earthquake motions were considered independently as follows:

North-South
East-West
Vertical

Three separate mathematical models were used. The models for horizontal earthquake motions did not include a vertical degree of freedom. Similarly, the model for the vertical earthquake motion did not include horizontal degree of freedom. Both models included rotational degrees of freedom. The vertical ground acceleration is

2/3 of the horizontal acceleration.

In the structural design of the plant, the "g" loads calculated from each earthquake direction are applied independently in order to calculate stresses or stress resultants, which are combined as the absolute sum of the N-S direction and vertical and then the E-W direction and vertical.

5.1.5 Discussion of Subgrade Modulus and Vertical Soil Spring

The subgrade modulus was used in the static analysis of the mat, while the vertical soil spring was used in the dynamic analysis of the earthquake.

An Ebasco representative contended that there was no relationship between these two parameters. In the static analysis the soil stiffness was represented in the classic treatment referred to as a Winkler Spring, except modified to place linear vertical springs at mode points rather than uniform as in the classical treatment. The vertical spring for the dynamic analysis is refered to in numerous tests and papers, The basic formulation is presented by Timoshenko and Gere for a disk on an elastic half-space. Inasmuch as this formulation is static rather than dynamic, a consideration of the two parameters representing vertical soil stiffness is indeed appropriate. The indications are that the two parameters representing vertical soil stiffness is indeed appropriate. The indications are that the soil stiffness is much more flexible in the dynamic analysis than in the static analysis of the mat.

If one assumes that the subgrade modulus of 100 psi is too stiff, it is likely that the moment and shear results would fall somewhere between the results for the subgrade modulus of 100 pci and the results of the rigid analysis. Therefore, these results will fall within design envelops.

If the soil spring for the dynamic analysis were stiffer, the fundamental period would decrease, however, since one of the three values of soil shear modulus was very close to the peak and the change would not be very great.

Due to the conservatism of the mat design and the fact that the calculated fundamental period is 0.6 seconds, it appears that the value of soil stiffness is not sensitive.

5.1.6 Damping Values

The values for structural damping are less than those specified in Reg. Guide 1.61 and the soil damping value were selected as 7.5%. This value of soil damping is much less than values generally recommended for soil structure interaction analyses.

5.1.7 Hydrodynamic Soil Effects

The soil hydrodynamic effects were ignored which is general practise. In general, the neglect of hydrodynamic soil effects is conservative; however, the fundamental period will be effected. Because one of the assumed values of Soil Shear Modulus was very close to the peak range (Reg. Guide 1.60), the seismic analysis is considered to be conservatie.

The peak ground seismic acceleration for design is 0.05g and 0.10g for OBE and DBE respectively. The artificial ground spectrum developed by LAW Engineering, was for 20 seconds using an interval of 0.01 seconds. The record was not base line corrected. Base line correction has little effect on acceleration.

5.1.8 Masonary Walls

No Seismic Category I equipment or structures are supported on masonary walls and it has been determined that these walls will not collapse under DBE. (i.e. SSE)

6.0 Reactor Shield Building

This building consists of a cylinder of a48' Ø with walls 3'0" thick and a spherical dome 2'6" thick of radius 112'0". The basic reinforcing pattern is #11 @ 12" o-c, E.F., E.W. This reinforcement is greater than that for the reactor building for the St. Lucie nuclear power plant, which was designed for tornado missiles.

7.0 Reactor Containment Shell

The containment vessel was designed and fabricated by CB&I in accordance with ASME Sec. III, Subsection NE 1971 updated by 1972 Winter addenda. The material is ASTMA 516 Grade 70. The thickness of the cylinder is approximately 2". Post-weld heat treatment was applied after the entire vessel was erected by heating the interior by means of heat applied at the penetrations. The design pressure is +39.6 psig and -0.15psig. The major penetrations are as follows:

Construction Hatch	32'-0" Ø
Maintenance Hatch	14'-0" Ø
Personnel Lock	6'-0" Ø
Personnel Escape Lock	5'-9" Ø

The design of penetrations used the area replacement rule and an analysis was made using WRC Bulletin 107.

Inasmuch as the R/t ration for the containment vessel exceeds the limit specified in WRC Bulletin #107, Ebasco will provide additional information concerning the back-up on the extrapolation to an R/t ratic of 600. The seismic "g" load varied from 0.1 at the base to 0.37 at the top for OBE. SSE was double these values.

8.0 Missile Shield Grating

The structure provides for tornado missile protection and consists essentially of a highway grating. The calculations were made by establishing an equivalent plate. This is adequate for the local bending effects; however, this would be unconservative for local shear. A calculation made during the audit indicated that the shear was acceptable. This should be made part of the calculation record.

9.0 Internal Structures of the Containment

The structures consist of the reactor cavity, the steam generator and pump enclosures, and the secondary shield wall. The reactor vessel is supported on the reactor cavity. The steam generator support system is a sliding base which is keyed so as to accommodate thermal growth but is keyed to resist reactions due to pipe break. Bolts are provided for uplift forces. State-of-the-art analyses and design of these structures was employed.

10. Spent Fuel Storage Pool

The spent fuel pool liner is 3/16" thick for the walls and 1/4" thick for the floor. The stainless steel is ASTM A-167 Type 304. Embedded wall stiffeners are provided at 17"0.c., except for the upper 13'-0" which is at 8½" o.c. The floor stiffeners are 8B24 members at 2'-7½" spacing. The construction sequence was such that the base liner was welded to the top of the 8B24 members and a non-shrink grout was used to fill the space between the top of the concrete pour and the floor liner. The grout used was Master Builders 636. Resulting gaps between the liner and grout of up to 5/16" were considered acceptable.

The spent fuel storage rocks were provided by Wachter Associates and are designed for high density storage. The racks rest on the floor without any structural connection. The rack module is attached to each other near their bases. Horizontal restraint is provided by extensions from the perimeter of racks to the fuel pool wall. The walls were designed for horizontally applied loads of 19 kips/ft. According to the calculations this value was not exceeded. (Tipping of the racks under seismic was not covered by this audit).

11. Turbine Missiles

Turbine missile criteria was not considered in the structural design. An analysis of the turbine effects concluded that the high trajectory missiles had a low probability of striking the Category I structures. The low trajectory missiles were considered to have a probability of striking the Reactor Shild Building. The results of using the NDRC and BRL showed incipient penetration and penetration respectively. Even though the turbine missile penetrates the Reactor Building, the missile was found not to perforate the reactor containment. From the values that were presented this was not obvious and Ebasco will provide additional data and information.

12. General Conclusions

The methods used for the structural analysis of this plant appear to be conservative. The parameters or range of parameters are not sensitive, in that, small variations would have caused increases in calculated results.