ENCLOSURE 1

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

MAR 2 1985

Docket No. 50-382

LICENSEE: LOUISIANA POWER AND LIGHT COMPANY

FACILITY: WATERFORD 3

SUBJECT: BASEMAT - MEETING SUMMARY

A meeting was held on February 13, 1985 in room P-422 of the Phillips Building in Bethesda, Maryland to discuss the licensee's common foundation basemat programs. Louisiana Power and Light (LP&L) was accompanied by representatives of EBASCO Services Incorporated. The NRC was represented by members of the Division of Licensing and the Division of Engineering as well as Brookhaven National Laboratory. A list of attendees is included as Enclosure 1.

The purpose of the meeting was to discuss the Confirmatory Analyses and Surveillance Programs for the Waterford Steam Electric Station, Unit 3 common foundation basemat.

The licensee first presented their Program to Perform Confirmatory Analyses. The purpose of the program is to provide a more detailed structural analysis which addresses:

1. dynamic coupling between the reactor building and the basemat for seismic stresses resulting from the vertical earthquake input,

2. dynamic effects of lateral soil/water loadings,

3. artificial boundary constraints in finite element model, and

4. fineness of basemat finite element mesh.

LP&L's program is included as Enclosure 2.

The NRC staff originally requested a fifth analysis addressing the origin of cracks in the vertical walls. LP&L believes the fifth analyses has been adequately answered by the NDT studies performed on the walls. These cracks have been identified as being shallow and probably resulting from shrinkage. They are not related to the cracks in the basemat. The staff agrees with the licensee's argument and will not require additional analysis.

After LP&L's presentation, the staff made the following requests and recommendations. These remarks are provisional pending final review of the licensee's proposed programs by the staff.

LP&L's program, as presented, does not take into consideration the actual stresses caused by the differential settlements of the mat during construction. The staff has requested that LP&L evaluate these stresses taking into account the actual measured basemat settlements and the construction sequence.

In accomplishing the first objective of the new analysis, dynamic coupling, the staff suggested that the licensee only use the FLUSH Computer code rather than both FLUSH and the STARDYNE model. The staff also advised the licensee that if they intend to use the SUPER-FLUSH computer code rather than FLUSH they should discuss its merits with the staff considering the code's lack of QA documentation.

The staff also requested that the mass of the Turbine building be taken into consideration when the FLUSH analyses are performed.

In addition, the staff felt that the finite element mesh, as presented at the meeting, required additional modification to further improve the fineness of the grid.

In the afternoon, LP&L presented their Basemat Monitoring Program. The purpose of the program is to provide overall assurance that changes in observable and measurable phenomena will be detected and that sufficient data is available to evaluate the causes and effects of the changes with respect to the basemat integrity. The program elements are:

1. Basemat settlement,

2. Ground Water Chemistry,

3 Seasonal Variation of Ground Water, and

4. Crack surveillance.

The program is included as Enclosure 3.

LP&L's program, as presented, does not include surveillance or mapping of the cracks in the vertical walls. The staff requested at the meeting that the program be modified to include these cracks.

The staff also requested that the tolerances of the measurements be specified and that the allowable tolerances be interpreted in terms of a mat response parameter (i.e. mat stresses).

The staff also suggested that, as well as being mapped, photographs should be taken of the cracks for historical data.

Finally, the staff reminded LP&L that both the Confirmatory Analyses and Monitoring Programs must be submitted formally prior to exceeding 5% power.

Lisamarie Lazo

Lisamarie Lazo, Project Manager
Licensing Branch #3
Division of Licensing

Enclosures:
As stated

cc w/enclosures:
See next page

ENCLOSURE 1
to meeting summary

MEETING ON COMMON FOUNDATION BASEMAT

FEBRUARY 13, 1985

LIST OF ATTENDEES

NRC

J. Knight
G. Knighton
L. Lazo
R. Bosnak
G. Lear
J. Ma
J. Chen

BNL

M. Reich
C. Miller
C. Costantino

LP&L

K. Cook
R. Burski
W. Cross
D. Buschbaum
T. Smith
M. Holley

EBASCO

A. Wern
P. Lin
J. Castello
D. Nuta
S. Chen

ENCLOSURE 2
to meeting summary

for information only

PROGRAM
TO
PERFORM CONFIRMATORY ANALYSES

NUCLEAR PLANT ISLAND STRUCTURE BASEMAT
AT
WATERFORD STEAM ELECTRIC STATION-UNIT NO 3

LOUISIANA POWER AND LIGHT COMPANY

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February 8, 1985

PROGRAM
TO
PERFORM CONFIRMATORY ANALYSES

NUCLEAR PLANT ISLAND STRUCTURE BASEMAT
AT
WATERFORD STEAM ELECTRIC STATION-UNIT NO 3

LOUISIANA POWER AND LIGHT COMPANY

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NUCLEAR PLANT ISLAND STRUCTURE BASEMAT
AT
WATERFORD STEAM ELECTRIC STATION-UNIT NO 3

LOUISIANA POWER AND LIGHT COMPANY

I. INTRODUCTION

This describes the program which Louisiana Power and & Light Company proposes to undertake to resolve the concerns raised by the Nuclear Regulatory Commission concerning the analysis of the basemat for the Nuclear Plant Island Structure (NPIS) at Waterford SES-Unit 3. The methods to be used, the computer programs which will be utilized and the sources of data regarding the material properties which will be used are all included.

II. PURPOSE OF THE CONFIRMATORY ANALYSES

The staff of the Nuclear Regulatory Commission, in their review of the basemat cracks recommended that a more detailed, confirmatory analysis be performed for portions of the basemat structural analysis for the Waterford 3 plant. The staff requested that confirmatory analyses be performed that will address:

1. dynamic coupling between the reactor building and the basemat for seismic stresses resulting from the vertical earthquake input

2. dynamic effects of lateral soil/water loadings

3. artificial boundary constraints in finite element model

4. fineness of basemat finite element mesh

5. origin of cracks in vertical walls.

The fifth analysis requested by the NRC staff has been adequately answered by the NDT studies performed on the walls. These cracks have been identified as being shallow and probably resulting from shrinkage. They are not related to the cracks in the basemat. Brookhaven National Laboratory, in Attachment F to the December affidavit agreed that.."(cracks in the vertical walls are no longer considered a problem)." Therefore the concerns which led to the request for the fifth analysis will be considered as adequately answered and the analysis will not be pursued any further.

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III. ANALYSIS METHODOLOGY

A. DYNAMIC COUPLING OF THE REACTOR BUILDING AND BASEMAT

1. GENERAL DESCRIPTION OF ANALYSIS

The subject of dynamic coupling between the reactor building and the basemat for stresses resulting from the vertical earthquake input is interpreted by LP&L to mean the possible effect of the mat flexibility on vertical seismic responses and the sensitivity of the mat stresses to vertical seismic accelerations which reflect the mat behavior.

To address this subject, LP&L proposes to undertake an analysis which will confirm that the vertical seismic accelerations obtained under the rigid mat assumption, as described in FSAR Section 3.7.2.1 (Appendix A), are conservative and form an acceptable design basis. The study will show that the stresses in the mat are not significantly affected and are within the Code allowables when the vertical accelerations are factored into the design.

Specifically the proposed confirmatory analysis will consist of the following:

a. Performance of a static analysis of the mat and superstructure complex which incorporates the maximum vertical acceleration obtained from the seismic analyses described in FSAR Section 3.7.2.1 (Appendix A). The 0.175g maximum vertical acceleration indicated in Table 3.7-9 of the FSAR (Appendix B) will be applied to all the structural masses and the forces will be combined with other concurrent loads. The static analysis will be performed with the STARDYNE Computer code and the finite element model as used for the original analysis modified by the use of the Martin element in place of the original element used. This analysis is identified in the table in IV. B as Old Loads/Old Model.

b. Establish, using state-of-the-art techniques, a conservative estimate of material and non-hysteretic damping which are reasonable for use in the vertical seismic analyses described in FSAR Section 3.7.2.1 (Appendix A). Experts in the field of soil dynamics will be consulted. The soil damping will be limited to 20 percent.

c. Perform vertical seismic dynamic analyses using the model shown in FSAR Figure 3.7-10 (Appendix C), incorporating soil damping which reflects material and non-hysteretic (radiation) damping, and utilizing the DYN 2037 Computer Code, as described in FSAR Section 3.8.3.4.1.1 (Appendix E).

The maximum vertical acceleration will be compared to the previous maximum of 0.175g to establish the reduction in the predicted responses associated with the use of more realistic soil damping.

d. Perform a literature search to confirm that the maximum variation of vertical seismic responses due to assumptions related to mat flexibility (ie; mat is rigid vs mat is flexible) for nuclear structures is ±20%.

2. ANALYSIS EXTENSION - IF WARRANTED

It is believed that the above exercises in stress analysis will be sufficient to confirm the validity and conservatism of the design of the basemat. However, in the event that the results of the vertical seismic analyses using the more realistic soil damping do not indicate a decrease in the maximum responses that is sufficient to cover possible response variations associated with mat flexibility, LP&L will perform more extensive analyses. These would include finite element soil-structure interaction analyses using the FLUSH or SUPER-FLUSH Computer code to establish more precise values of vertical seismic accelerations.

Two dimensional analyses utilizing the existing lumped mass structural models (as shown in FSAR Figure 3.7-10 Appendix C) with modifications made to include a finite element representation of the mat and the soil beneath and surrounding the Nuclear Plant Island Structure will be performed.

Material properties will be derived as defined in III.B.3.

Parametric studies will be performed to determine the sensitivity of the model chosen to the various assumptions required for the performance of the analysis.

The results to be obtained from these analyses will be a listing of the amplified accelerations at each level in the various buildings supported on the basemat.

The accelerations obtained will be used to recompute the basemat internal forces caused by the vertical earthquake. This will require a rerun of the STARDYNE model used to evaluate the basemat internal forces. These runs will be for the DBE case for N-S and E-W earthquake directions only and will include the other loads normally included in such loadcases.

B. DYNAMIC EFFECTS OF LATERAL SOIL/WATER LOADINGS

1. GENERAL DESCRIPTION OF THE ANALYSIS

This analysis will be performed to evaluate the maximum and minimum membrane forces and bending moments exerted on the basemat by the lateral soil and water pressures on the end walls of the NPIS during a seismic event. The original calculation of these forces was a static approximation utilizing a knowledge of the deformations of the soil and building during earthquake and applying these deformations to known soil properties.

LP&L proposes to perform the following confirmatory work:

a. finite element soil-structure interaction seismic analyses under DBE horizontal earthquake input in order to establish dynamic soil pressures.

b. establish dynamic water pressures using classical (closed form) solutions.

c. finite element static analysis of the NPIS complex incorporating the dynamic soil and water pressures and appropriate concurrent loads.

2. SEISMIC SOIL STRUCTURE INTERACTION ANALYSES

These analyses will be performed using the FLUSH computer code or the SUPER-FLUSH code. Specific features of both programs are:

- they are implicit finite element codes using the frequency domain approach.
- the non-linear soil behavior is approximated by an equivalent linear approach by iterating the stiffness and damping values for each element consistent with average values of strain occuring during the analysis.
- the only form of seismic input allowed is that of rigid "bedrock" shaking.
- the codes have both continuum and plain strain elements.
- deconvolution analyses are incorporated directly into the programs.
- the codes incorporate viscous dashpot boundaries used to simulate 3-D effects, and energy transmitting boundaries which can be used to minimize the number of finite elements required.

In conjunction with these programs two-dimensional models utilizing the existing lumped mass structural models and augmented with a finite element representation of the soil beneath and alongside the lateral walls, will be developed.

Specifics regarding the FLUSH or SUPER-FLUSH analyses under horizontal DBE effects are as follows:

- two dimensional models representing the mat and side walls as rigid elements and incorporating the lumped-mass models shown in FSAR Fig. 3.7-9 (Appendix D) and a soil element mesh will be used.

- input motion will be specified as applicable at the bottom of the mat level (El.-47.0 ft). Only DBE analyses will be performed. North-south and east-west motion will be considered separately.

- the horizontal time history for analyses will be applied at the lower rigid boundary, the location of which will be established by performing parametric studies. This driving time history will be established using deconvolution techniques. If the location of the lower boundary is such that the size of the soil finite element model becomes too large, the compliant base available in SUPER-FLUSH, consisting of viscous dashpots at the base of the model to absorb reflected waves from the surface, will be used.

- vertically propagating shear waves will be assumed.

- a finer soil mesh will be used against the vertical structural walls and around the basemat edges, where the rocking effects are most pronounced, in order to account for the weakening of the soil locally due to large strains. The soil finite element mesh will extend to about the edge of the backfill where energy transmitting boundaries will be used.

- lateral out-of-plane viscous boundaries will be used to simulate out-of-plane radiation effects.

- the vertical dimension of the soil elements will be kept smaller than one-fifth of the smallest wavelength (associated with the highest frequency) of interest. For this soft site, a cutoff frequency of 12Hz will be used.

- the computation of the Fourier transform of the input motion will be performed using a number of time and frequency increments which will allow for frequency components of the input motion up to 12Hz to be accurately reproduced.

- the effective embeddment depth (i.e. the area over which connectivity between lateral walls and soil is assumed) will be varied. Soil-structure connectivity will be assumed on both sides of the 2-D models.

- the analyses will consider a range of shear modulus vs strain curves including average, average x 1.5 and average/1.5.

- time history of lateral soil forces at all points of connectivity will be obtained.

3. MATERIAL PROPERTIES

The material properties for the soil will be derived from material presented in Section 2.5 of the FSAR. Concrete and steel material properties will be normally accepted values. The structural properties of the structural spring/lumped mass model, as described in FSAR Section 3.7.2 (Appendix A) will be used.

The material soil damping and the non-hysteretic (radiation) soil damping values will be established by utilization of known site soil properties, literature values, state of the art analytical techniques and consultation with experts in the field. The ranges of shear strain vs modulus will be derived from literature and consultation with experts in the field.

4. PARAMETRIC STUDIES

Parametric studies will be performed to determine the sensitivity to various assumptions required in the performance of the analysis. The parametric studies will consist of:

- a range of shear modulus vs strain curves as described above.

- studies to establish the location of the lower rigid boundary.

- studies to establish the adequacy of the soil finite element mesh.

- studies to establish the effect of the assumed effective embeddment depth.

Ref. (1) Westergaard, N. M. (1933), "Water Pressures on Dams During Earthquakes," Transactions of the American Society of Civil engineers, Volume 98.

5. DYNAMIC LATERAL WATER PRESSURES

The dynamic water pressure will be established using the Westergaard theory as described in Ref. 1. The soil porosity will be used to establish if lower dynamic water pressures, reflecting the fact that water is entrapped in the soil, may be used.

6. FINITE ELEMENT STATIC ANALYSES

The dynamic lateral soil and water pressures will be incorporated in static finite element analyses using the STARDYNE computer code and the mat-superstructure representation used in the original basemat analyses.

7. RESULTS TO BE OBTAINED FROM COMPUTER RUNS

The results to be obtained from this analysis will be a definition of the maximum and minimum membrane forces in the basemat and the maximum and minimum bending moments applied to the basemat by the lateral soil forces.

8. APPLICATION OF RESULTS TO THE CONCERNS RAISED

The forces and bending moments will be compared to the forces and bending moments from these sources in the original basemat STARDYNE analysis to provide assurance that the basemat stresses are within code allowables under seismic loading. In particular, attention will be paid to areas where the bending moments due to the lateral forces diminish the gravity load bending moments causing tension at the top surface of the basemat.

C. <u>ARTIFICIAL BOUNDARY CONSTRAINTS IN FINITE ELEMENT MODEL</u>

1. GENERAL DESCRIPTION OF THE ANALYSIS

This analysis will be performed to demonstrate the effect on basemat stresses when the artificial boundary constraints used in the STARDYNE analysis are altered to more closely match physical conditions.

2. DESCRIPTION OF THE MODEL

The STARDYNE model used for the basemat analysis will be altered so that each node point will be restrained by two horizontal springs, along with the vertical springs already used, connected to the node point by a stiff stick. This stick will extend from the middle of the mat (the plane of the finite element representation of the mat) to the bottom of the mat(6'). The horizontal and vertical springs will be placed at the base of the sticks. The horizontal springs will represent a distributed frictional resistance due to contact with the soil.

3. COMPUTER PROGRAMS TO BE USED

The STARDYNE program used in the original basemat analysis will be used modified by the use of the Martin element in place of the original element used.

4. MATERIAL PROPERTIES

The properties of the springs will be based upon the soil properties obtained from soil testing at the time of the PSAR along with textbook interpretations of soil stiffness. The vertical springs of the old model will be used for the new model. The horizontal springs will represent the basemat base friction and subsoil deformation characteristics under unbalanced horizontal seismic loads. The base friction is assumed to be equal to the subsoil cohesion, 1500 psf or 10.4 psi, since it is a cohesive soil. The amount of subsoil deformation is assumed to be equal to the relative displacement between the basemat and subsoil, which ranges from 0.5 to 3.0 inches. Therefore, the horizontal spring constant can range from 20.8 to 3.5 lb/inch per square inch of basemat area. These values will be confirmed.

5. PARAMETRIC STUDIES

The STARDYNE runs will be made utilizing all of the loads as originally used for the basemat analysis and the modified constraints as defined above. This will define the effect of the modification of the boundary constraints on the basemat loads.

Prior to the STARDYNE runs, a sensitivity study will be made for the effect of the spring coefficient of the horizontal springs. The modified constraint model will be analyzed using one load combination, DBE with east-west earthquake, with both the 3.5 and the 20.8 lb/cubic inch spring constant. The horizontal reactions at the springs along with the flexural moments within the basemat will be evaluated for these two conditions. The spring constant which yields the greater moments within the mat or the greater peak reaction will be selected for the STARDYNE runs. If the differences caused by varying the spring constant are small and negligible, a spring constant of 20.8 lb/cubic inch will be used for the computer runs.

The STARDYNE runs will be made for the DBE load combination with both east-west and north-south earthquakes used. The loads as originally defined will be applied to the modified artificial boundaries models.

6. RESULTS TO BE OBTAINED FROM COMPUTER RUNS

The results to be obtained from this analysis will be a complete listing of basemat internal forces with the old loads and with the new boundary constraints.

7. APPLICATION OF RESULTS TO THE CONCERNS RAISED

The basemat stresses with the new boundary constraints will be computed from the internal forces and will be compared to code allowable stresses to assure compliance with the code under seismic loading conditions. An illustration will be prepared to demonstrate the effect of distributing the boundary constraints on the internal forces.

D. FINENESS OF BASEMAT FINITE ELEMENT MESH

1. GENERAL DESCRIPTION OF THE ANALYSIS

The existing STARDYNE finite element model will be altered by reducing the element size to provide additional elements between supports. In general, at least four elements between supports will be provided, except where supports have formed a corner. The element size of superstructures affected will be modified accordingly.

2. DESCRIPTION OF THE MODEL

The existing STARDYNE model of the basemat will be modified as necessary to incorporate the finer element sizes. The areas which will be modified are areas in the vicinity of the Reactor Shield Building wall and areas forming the junction between the exterior walls of the NPIS and the basemat. Figure 1 shows the proposed modifications to the basemat finite element model mesh.

3. COMPUTER PROGRAMS TO BE USED

The STARDYNE computer program used in the original basemat analysis will be utilized modified by the use of the Martin element in place of the original element used.

4. MATERIAL PROPERTIES

Material properties as utilized for the original analysis will be used.

5. PARAMETRIC STUDIES

STARDYNE runs with the finer mesh will be made for the loads and support conditions as originally used.

Prior to the STARDYNE runs, a mesh evaluation will be made using only the normal operation load combination. Typical moment and shear diagrams in the modified areas will be studied for a reasonable presentation of stress gradient and the mesh will be modified to assure a fineness sufficient to allow a reasonable definition of the stress gradient.

6. RESULTS TO BE OBTAINED FROM COMPUTER RUNS

The results to be obtained from this analysis will be a listing of internal forces (shears and moments) for each element for the old and new element sizes for the old applied loads. The results obtained in this study will be those of load combinations cases:

- Normal Operation
- DBE east to west motion
- DBE north to south motion.

7. APPLICATION OF RESULTS TO THE CONCERNS RAISED

The internal forces will be translated into basemat unit stresses and compared to code allowable stresses to verify that they are within the allowable limits. An illustration will be assembled to demonstrate the effect that making a finer finite element mesh had on the internal forces.

IV. SUMMARY OF COMPUTER RUNS

A. FLUSH/SUPER-FLUSH

1. Lateral Soil Pressure (North-South and East-West)
2. Vertical acceleration only if warranted.

B. STARDYNE(Each run comprises a north-south and an east-west run when lateral loads are involved). Load conditions: Normal Operation and DBE.

LOADS	MODEL		
	OLD	NEW CONSTRAINTS	NEW MESH
OLD	X	X	X
NEW VERTICAL	X		
NEW LATERAL	X		

V. SCHEDULE

The schedule commitment is to have the work completed and submitted to the NRC staff prior to start-up following the first refueling.

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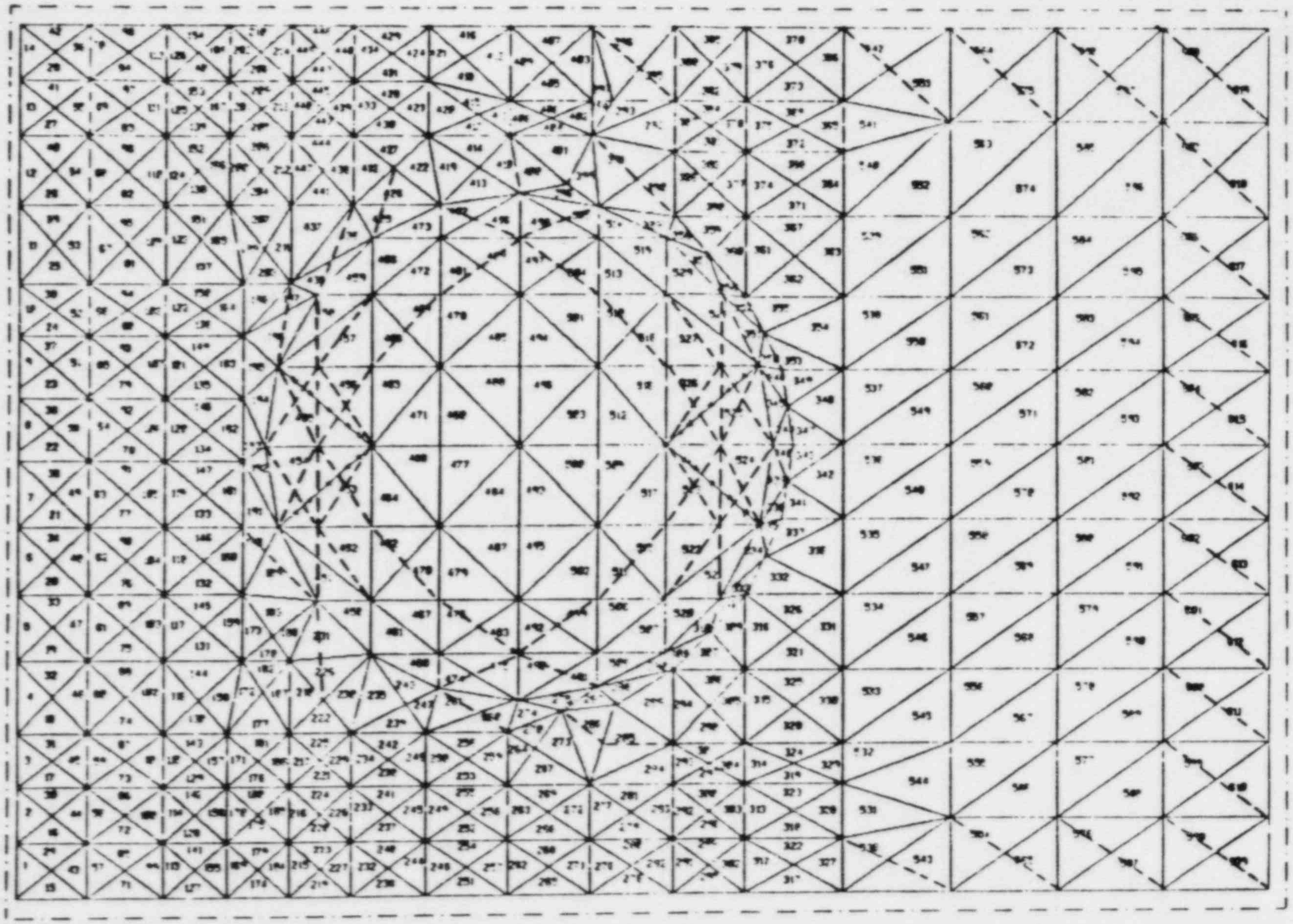

LEGEND:

- - - - - ADDED ELEMENT BOUNDARY

—•—•— ELIMINATED ELEMENT BOUNDARY

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FIGURE 1

APPENDIX A

Frequency Range (hertz)	Increment (hertz)	No. of Frequencies Used
0.2 - 3.0	0.10	37
3.0 - 3.6	0.15	7
3.6 - 5.0	0.20	10
5.0 - 8.0	0.25	14
8.0 - 15.0	0.50	16
15.0 - 18.0	1.00	3
18.0 - 22.0	2.00	4
22.0 - 34.0	3.00	9
		100

Similar design response spectra and time history spectra were made utilizing 200 computed period points within the above frequency range, which verified the above results.

3.7.1.3 Critical Damping Values

The damping ratios, expressed as percent of critical damping, which are used in the analysis of seismic Category I systems and components are presented in Table 3.7-1. These damping values both for the SSE and OBE are equal to or more conservative than the values recommended by NRC Regulatory Guide 1.61. Damping values utilized by the NSSS are given in Subsection 3.7.3.1.2.

The damping value for the soils at the site are selected on a conservative basis from the strains induced by the earthquakes. Individual damping versus strain curves are presented in Subsection 2.5.4.

Since damping values are strain-dependent, the single values used in design were compatible with the actual strains developed during earthquakes. An equivalent linear variable-damping lumped-mass solution, similar to that developed by Idriss and Seed[18], was utilized. In this analysis, damping and shear moduli values were assumed and were a portion of the input to the computer. The output included a profile of calculated shear strain versus depth. On the first run, the calculated shear strain value did not correspond to the initially assumed value. The shear modulus was adjusted accordingly using Figures 2.5-77 and 2.5-78 and successive iterations made until the calculated shear strain and the assumed strain converged. The point of convergence occurred at 0.04 percent strain for the Recent alluvium and 0.08 percent strain for the upper Pleistocene sediments. Therefore, the following design values were utilized: | 1

	DAMPING percent
Recent Alluvium (+13 to -40 ft. MSL)	8
Pleistocene Sediments (-40 to -317 ft. MSL)	7.5

3.7.1.4 Supporting Media For Seismic Category I Structures

All seismic Category I structures are founded at elevation - 47 ft. MSL on a one ft. thick compacted shell filter blanket on top of the Pleistocene clay. The Reactor Building, Reactor Auxiliary Building, Fuel Handling Building and the Component Cooling Water System structures are supported on a common foundation mat, 267 ft. wide and 380 ft. long, which is embedded 64.5 ft. below finished plant grade, in the stiff gray and tan clays.

Table 3.7-2 provides a tabulation of the foundation elevation and total structural height of the seismic Category I structures supported on common foundation mat. The plant grade elevation is +17.5 ft. MSL.

The soil layering characteristics and soil properties are discussed in Subsection 2.5.4.

3.7.2 SEISMIC SYSTEM ANALYSIS

This subsection includes discussion of seismic analysis of all seismic Category I structures. Seismic analysis of seismic Category I piping systems and components including the Reactor Coolant System is discussed in Subsection 3.7.3.

3.7.2.1 Seismic Analysis Methods

The seismic analyses of all seismic Category I structures were performed using either the normal mode time history technique or the response spectrum technique.

In the case of seismic Category I structures, the seismic response was determined by the response spectra developed for the OBE (0.05 g) and the SSE (0.10 g), as described in Subsection 3.7.1.1.

3.7.2.1.1 Seismic Category I Structures

3.7.2.1.1.1 Mathematical Model

As all seismic Category I structures are founded on a common foundation mat, described in Section 3.8, the mathematical modeling involves construction of a single composite model for each directional seismic analysis.

The model comprises five individual cantilevers, representing the Reactor Building, the containment vessel, the reactor internal structure, the Reactor Auxiliary Building and the Fuel Handling Building. The Component Cooling Water System is not separately identified and is included in the Reactor Auxiliary Building and Fuel Handling Building cantilevers. The five cantilevers are founded on the same base, which is in turn supported by foundation springs. For each cantilever, the distributed masses of the structure are lumped at certain select points and connected by weightless elastic bars representing the stiffness of the structure between the lumped masses. In determining the stiffnesses, the deformation due to bending, shear and joint rotation are considered throughout.

Typical mathematical models for horizontal and vertical excitation analysis are shown on Figures 3.7-9 and 3.7-10, respectively. The input data used for these models for seismic analyses are summarized in Tables 3.7-3 and 3.7-4.

Equivalent soil springs, as described in Subsection 3.7.2.4, and damping values, as described in Subsection 3.7.1.3, are used in the analysis.

Every mass point of the two dimensional horizontal model is allowed two degrees of freedom, namely, translation and rotation. For the vertical model, only one translational degree of freedom is considered. A mathematical model for torsional effects is described in Subsection 3.7.2.11.

3.7.2.1.1.2 Equations of Motion

Once the mathematical model is established, the motion of each lumped mass under any external excitation may be written in the matrix form as follows:

$$[M]\left\{\ddot{\Delta}\right\} + [c]\left\{\dot{\Delta}\right\} + [K]\left\{\Delta\right\} = \left\{F\right\} \qquad (1)$$

where: $[M]$ = square mass matrix

$[K]$ = square matrix of stiffness coefficients including the shear and bending deformations

$\left\{\ddot{\Delta}\right\}$ = column matrix of acceleration vectors

$\left\{\dot{\Delta}\right\}$ = column matrix of velocity vectors

$\left\{\Delta\right\}$ = column matrix of lateral displacement and joint rotation vectors

$\left\{F\right\}$ = column matrix of external load vectors

$[c]$ = damping matrix

The stiffness matrix $[K]$ is formulated by computing the stiffness coefficients for each joint of the original structure and assembling them in the proper sequence to form the complete square matrix. In the computation of the stiffness matrix, it is assumed that all joints at the same level have the same displacements (i.e., translations and rotations).

The cantilever connecting two lumped masses is considered as a beam element
and the effects of bending and shear deformation are included in computing
the stiffness coefficients. The effects of equivalent soil springs are also
included in the formation of the stiffness matrix $[K]$. As shown in Figure
3.7-9, there are three soil springs, two translational and one rocking being 1
considered for horizontal excitations. The first translational spring Kx
represents the shear effect between the common foundation mat and the soil
and it is applied at the bottom of the mat, while the second translational
spring Kxx represents the bearing effect between the mat and the soil and
it is applied at the mid height of the mat side surface. The rocking
spring $K\phi$ is considered acting at the rotation center of the mat. The 2
method used to account for torsional response is discussed in Subsection
3.7.2.11.

The effect due to relative displacement between interconnected mass points
are also considered. The connecting members between mass points are modeled
as beams and springs and their effects to the structural response are modeled 1
porated in the stiffness matrix. In the design of seismic Category I systems
and components, the maximum relative displacement time histories of supports
obtained from structural responses are utilized.

3.7.2.1.1.3 Natural Frequencies and Mode Shapes

In calculating the natural frequencies and the mode shapes, the damping term $[c]\left\{\dot{\Delta}\right\}$ is ignored and the external load vector in equation (1) is set to zero, the displacement vector $\{\Delta\}$ is assumed to take the form of simple harmonic motion:

$$\{\Delta\} = \{\phi\} \operatorname{Sin} \omega t \qquad (2)$$

where: $\{\phi\}$ = Relative amplitude of mode shape vector

ω = Natural frequency of vibration

After substituting into equation (1) and simplifying, the equations of motion are reduced to the following form:

$$[K]^{-1} [M] \{\phi\} = \frac{1}{\omega^2} \{\phi\} \qquad (3)$$

Solution to this eigenvalue problem exists only for particular values which correspond to the natural frequencies of vibration of the structure. Equation (3) is solved by the Jacobi method to obtain values of natural frequency of vibration (ω) and their corresponding mode shape vectors $\{\phi\}$.

3.7.2.1.1.4 Modal Analysis

After all natural frequencies and their mode shapes are determined, the method of modal analysis is employed to calculate the structural responses. This method actually simplifies the analysis of a multidegree of freedom system into an analysis of several equivalent single degree systems, one corresponding to each normal mode. The governing equation of motion is shown in the following:

$$\ddot{A}_n + 2\beta_n \dot{A}_n + \omega_n^2 A_n = \frac{-\ddot{Y}_{so} f_a(t) \sum_{x=1}^{m} M_x \phi_{xn}}{\sum_{x=1}^{N} M_x \phi_{xn}^2} \tag{4}$$

where: A_n = displacement of any one arbitrarily selected mass (usually the topmost mass) for the nth mode

β_n = damping coefficient = $\lambda_n \omega_n$

λ_n = percentage of critical damping of the nth mode

ω_n = natural frequency of the nth mode

$\ddot{Y}_{so}$ = maximum ground acceleration

$f_a(t)$ = time function of ground motion

M_x = mass at the xth level

m = number of masses subjected to inertia $M_x \ddot{Y}_{so} f(t)$

ϕ_{xn} = normalized displacement of the mass M_x of the nth mode

N = total number of degrees of freedom

If the two summations on the right-hand side of the equation (4) are denoted by P_n, which is defined as the modal participation factor of the nth mode, then

$$\ddot{A}_n + 2\beta_n \dot{A}_n + \omega_n A_n = -P_n \ddot{Y}_{so} f_a(t) \tag{5}$$

Since the values of β_n, ω_n and P_n are already known for each normal mode, equation (5), which is actually "n" independent equations, can be solved separately using the method developed by NC Nigen and PC Jennings(1).

The total displacement is the summation of the displacement of each normal mode, that is:

$$Y_x(t)_{max} = \sum_{n=1}^{N} P_n \phi_{xn} A_n \tag{6}$$

In spectral analysis, A_n's are spectral values from the design spectral curves. The algebraic sum of equation (6) gives the upper limit of the displacement of any mass. However, all the maximum displacements of all normal modes do not necessarily occur at the same time. For the purpose of design, the root-mean-square method is adopted from the statistical point of view:

$$Y_{x\,max} = \left[\sum_{n=1}^{N} (P_n \phi_{xn} A_n)^2 \right]^{1/2} \qquad (7)$$

3.7.2.2 Natural Frequencies and Response Loads

A summary of natural frequencies for significant modes is presented in Table 3.7-5. A summary of structural responses determined by the seismic analysis for major seismic Category I structures is presented in Tables 3.7-6 through 3.7-9.

3.7.2.3 Procedure Used for Modeling

Major seismic Category I structures that are considered in conjunction with foundation media in forming a soil-structure interaction model are defined as "seismic systems." Other seismic Category I structures, systems, and components that are not designated as "seismic systems" are considered as "seismic subsystems."

The procedure used to calculate the lumped masses at designated floor levels consisted of combining the floor weights, equipment weights and one-half of the wall and column weights from the adjacent upper and lower floors. In solving the mathematical model for vertical excitation, similar lumping of masses was used.

3.7.2.4 Soil-Structure Interaction

The free-field motion of the site, during a seismic event, is locally affected by the presence of the buildings. The effects of dynamic interaction between soil and buildings can be such that the free-field response of the soil is either amplified or attenuated in some portions of the frequency range of interest. To evaluate the modifying effect of soil-structure interaction on the free-field motion (at the foundation level), a simplified lumped-mass soil spring analysis has been performed. The rationale of using lumped-mass spring method instead of finite element method for the interaction study is as follows:

a) The soil conditions, immediately underneath the plant foundations are fairly uniform and a hard rock boundary is not present in the immediate vicinity. Both these conditions dictate the use of a simplified approach for conservatism.

b) The effects of variations in soil shear modulus with strain have been considered and effective values were established from strains induced by both the static and dynamic considerations. Statistical methods of analysis were utilized to determine the participation of shear modulus throughout the time history analysis. A range of soil moduli was studied to establish the responses of soil-structure system (see Appendix 3.7-A).

c) All seismic Category I structures are located on a single common mat foundation. By virtue of this arrangement, the effects of adjacent structures on the soil-structure interaction response are automatically eliminated, leading to a simplified analysis.

The soil-structure interaction model for vertical and horizontal excitations consisted of a two dimensional lumped-mass spring system, representing the seismic Category I Nuclear Plant Island Structure and typical site geology. A three dimensional lumped-mass spring system was used for torsional response analysis. The basis for selection of a simplified soil spring approach is discussed in Appendix 3.7A. The foundation springs for horizontal excitation consisted of one rotational spring and two translational springs as shown on Figure 3.7-9. The foundation springs for vertical excitation are shown in Figure 3.7-10. The rotational and translational spring constants were calculated using the following formulae by Whitman and Richart[2], and Barkan[3]:

Rotation (or rocking) $K_\phi = \frac{G}{1-\mu} \beta_o BL^2$ 1

Sliding (or shear) $K_x = 2(1+\mu) G \beta_x \sqrt{BL}$

Bearing (or compression) $K_{xx} = \frac{G \beta_z}{1-\mu} \sqrt{A}$

where: G = shear modulus of soil

μ = Poisson's ratio of soil

B = width of rectangular foundation

L = length of rectangular foundation

A = bearing area

β_o, β_x and β_z = site constants dependent on B/L ratio

The values of shear modulus and Poisson's ratio were obtained from laboratory testing and field geophysical analysis (see Subsection 2.5.4.2).

Since shear moduli are strain-dependent, the single values used in design were compatible with the actual strains developed during earthquakes. An equivalent linear variable-damping lumped-mass solution, similar to that developed by Idriss and Seed[18], was utilized. In this analysis, damping and shear moduli values were assumed and were a portion of the input to the computer. The output included a profile of calculated shear strain versus depth. On the first run, the calculated shear strain value did not correspond to the initially assumed value. The shear modulus was adjusted accordingly using Figure 2.5-77 and 2.5-78 and successive iterations made until the calculated shear strain and the assumed strain converged. The point of convergence occurred at 0.04 percent strain for the Recent alluvium and

0.08 percent strain for the upper Pleistocene sediments. Therefore the following design conservative values were utilized:

	SHEAR MODULUS psi
Recent Alluvium (+13 to -40 ft. MSL)	3400 (490 KSF)
Pleistocene Sediments (-40 to -317 ft. MSL)	5800 (830 KSF)

Refer to Appendix 3.7A for the results of a parametric study of shear modulus where it was varied from 5800 psi to 16,050 psi.

3.7.2.5 Development of Floor Response Spectra

A time history method of analysis is used to develop floor response spectra, as described in detail in Subsection 3.7.2.1.

3.7.2.6 Three Components of Earthquake Motion

The seismic analysis of seismic Category I structures, systems or components does not consider simultaneous action of three components of design earthquake nor the calculation of responses by square root of the sum of the square of corresponding maximum values of the response as recommended in Regulatory Guide 1.92, Combination of Modes and Spatial Components in Seismic Response Analysis, December 1974. Instead the maximum value of response in each element is determined by considering each horizontal and vertical component of an earthquake separately.

For each structural element, the two responses related to one horizontal and one vertical earthquake components are combined using the absolute sum method. The comparisons of the maximum response used in the plant structural design and that obtained using square root of the sum of the squares (SRSS) are provided in Tables 3.7-18 to 20. They are made for three randomly selected elements of the Reactor Shield Building at elevations +184.0, +61.0 and 0.0 ft. MSL, respectively. They indicate that the maximum response used is larger than the maximum response obtained using SRSS. Thus, the design approach in obtaining the maximum earthquake is equivalent to that obtained in accordance with Regulatory Guide 1.92. 1

3.7.2.7 Combination of Modal Responses

When the spectrum method of modal analysis is used, the modes are combined by the square root of the sum of the squares (SRSS), without taking into consideration the effect of spacing of modes, as recommended by Regulatory Guide 1.92 (refer to Subsection 3.7.2.6).

3.7.2.8 Interaction of Noncategory I Structures With Seismic Category I Structures

The structural frames of nonseismic structures are designed to withstand seismic motion such that nonseismic structures will not collapse and impair the integrity of seismic Category I structures or components.

3.7.2.9 Effects of Parametric Variation on Floor Response Spectra

The following conservative assumptions are included in the calculation of the floor response spectra:

a) The expected actual earthquake time histories are enveloped by a smooth ground response spectrum for design use. This has conservative effects on modal analysis because it treats the modes in the maximum acceleration range as though they all had the same amplification factor as the most strongly amplified mode.

b) The time history used to calculate the floor response spectra produces a ground response spectrum which envelopes the design ground response spectra. In order to do this, it has spectral peaks which are substantially higher than the design spectra.

c) The building and soil damping values used in the analysis are near the lower bound of the available damping data. The actual values of damping are expected to be much higher than the values used in the analysis.

d) The yield strengths used in the analysis are based on the minimum values and are considerably lower than expected values.

e) The additional strength and damping that are available when materials are stressed beyond yield are neglected when using linear elastic analytical methods.

In order to maintain the consistent conservative design objective, parametric studies of foundation stiffness were also performed using a range of shear modulus from 5,800 psi to 16,050 psi. As a result of these studies, conservative design envelopes for all mass points and levels within the seismic Category I structures were developed for the design floor responses.

Figures 3.7-11 through 3.7-20 show the variation in floor responses (SSE with one percent damping) for shear modulus values of 5,800, 8,000 and 16,050 psi and the design envelope for related mass points and levels. Each design envelope encompasses all the spectral peaks occurring within the above range of soil shear modules and results in extremely conservative equipment and piping design at respective floor levels.

3.7.2.10 Use of Constant Vertical Load Factors

A vertical seismic system multi-mass dynamic analysis is used to account for vertical response loads (refer to Subsection 3.7.2.1.1.1).

3.7.2.11 Method Used to Account for Torsional Effects

The effects of torsional modes of vibration are analyzed by a three-dimensional lumped-mass system using the MRI/Stardyne computer program (refer to Subsection 3.8.3.4). Each mass point of the system is given two orthogonal horizontal degrees of freedom and a third rotational degree of freedom in the same plane, as shown in Figure 3.7-21. The mass points are then idealized as a rigid diaphragm with three degrees of freedom, two translational and one rotational. In this analysis, torsional effect results from the translational seismic inputs because of the eccentricity between the mass center and the shear center of each floor (mass polar moment of inertia).

Soil structure interaction is considered by including translational and rotational springs at the base of the lumped-mass mathematical model as discussed in Subsection 3.7.2.4. In addition, a torsional spring is also considered.

The maximum increase in acceleration due to torsional modes of vibration is found to be less than five percent from a case without torsional mode of vibration, as shown in Table 3.7-10. The structural design takes into account the torsional effect. An additional 5 percent to or a subtraction of 5 percent from actual eccentricity has been found to have a negligible additional effect on structural acceleration responses. 19

3.7.2.12 Comparison of Responses

In order to provide a check on the seismic analysis of seismic Category I structures, an analysis using both the modal analysis response spectrum method and time history method has been conducted. Tables 3.7-6 through 3.7-9 give the response at selected points for major seismic Category I structures using both these methods. These responses illustrate approximate equivalency between the two methods.

3.7.2.13 Methods for Seismic Analysis of Dams

There are no seismic Category I dams associated with Waterford-3.

3.7.2.14 Methods to Determine Category I Structure Overturning Moments

The seismically induced overturning moments in the seismic Category I structures are obtained from the seismic dynamic analysis discussed in Subsection 3.7.2.1.

The bearing pressures arising from two horizontal orthogonal components of seismic motion, are combined algebraically and further combined with buoyancy and other applicable loads in accordance with the load combinations discussed in Subsection 3.8.4.3.

In calculating factors of safety against overturning, the moments due to two horizontal orthogonal components of seismic motion are combined by the SRSS method. The factor of safety against overturning for the Nuclear Plant Island Structure is 2.77 as shown in Figure 3.7-22.

3.7.2.15 Analysis Procedures for Damping

The structural and foundation material damping ratios considered in the seismic analyses are those specified in Subsection 3.7.1.3.

Composite damping in the mathematical models is determined by first evaluating the mode shapes of the system and identifying the relative participation of all portions of the system for each of these modes. Where the response participation is primarily from a single material type, the assumed damping is appropriate to that material. Where no single material can be identified as primary to the response, the damping is computed as a weighted average of the different material damping ratios based on the relative participation of each material in the mode shape. Using this procedure, modal damping ratios representing the composite damping characteristics are determined for each mode of response for use in the normal mode time history technique.

The procedure used to find the equivalent modal damping ratios for the natural modes of a structure having composite materials or substructures with different damping ratios is as follows:

$$D_n = \frac{\sum_{i=1}^{m} d_i S_{ni}}{S_n}$$

where: D_n = percentage of critical damping ratio for the n^{th} mode

d_i = percentage of material damping ratio for the i^{th} structural component

S_{ni} = strain energy of the i^{th} structural component in the n^{th} mode = $\sum_l \sum_j \phi_{ln} K_{lj}^{(i)} \phi_{jn}$ where l and j are limited to the component only.

S_n = total strain energy of structure in the n^{th} mode = $\sum_l \sum_j \phi_{ln} K_{lj} \phi_{jn}$ where l and j are covered for the whole structure.

m = number of structural components

TABLE 3.7-3

INPUT DATA FOR SEISMIC ANALYSIS
HORIZONTAL EXCITATIONS

	Mass Point	Length (ft.)	Area Moment of Inertia ($ft.^4$) N-S	Area Moment of Inertia ($ft.^4$) E-W	Effective Area ($ft.^2$) N-S	Effective Area ($ft.^2$) E-W	Weight (Kip)
Shield Building	1	27.73	2,554,000		401		7,010
	2	21.7	4,058,000		711		4,959
	3	19.7	4,058,000		711		4,318
	4	20.0	4,058,000		711		4,104
	5	25.0	4,058,000		711		4,446
	6	25.0	4,058,000		711		6,242
	7	20.0	4,058,000		711		4,446
	8	22.0	4,058,000		711		4,104
	9	19.0	4,058,000		711		5,301
	10	18.0	4,058,000		711		2,822
	11	17.0	11,782,470		2,262		10,173
Containment Vessel	12	21.5	257,500		98		354
	13	22	527,500		129		376
	14	22	1,031,000		213		376
	15	22	1,420,000		287		668
	16	22	1,723,000		416		1,735
	17	22	1,420,000		287		755
	18	22	1,420,000		287		755
	19	22	1,420,000		287		755
	20	22	1,420,000		287		755
	21	11	1,420,000		287		755
Reactor Bldg. Internal Structure	22	7.3	540,000	190,600	962	494	1,295
	23	7	540,000	190,600	962	494	2,167
	24	11	1,770,000	1,317,000	1,519	670	8,060
	25	12	1,770,000	1,317,000	1,519	670	5,782
	26	14.5	1,876,000	1,353,000	1,737	1,105	9,538
	27	12.5	2,095,820	1,364,900	2,102	2,070	8,855
	28	7	2,080,000	1,607,000	2,096	2,580	7,802
Fuel Handling Bldg.	29	44.5	764,130	1,561,810	292	524	6,853
	30	24.5	1,118,940	2,512,750	725	1,373	10,240
	31	20.0	12,545,150	45,558,660	2,110	2,160	25,010
	32	36.0	15,630,050	53,700,752	2,262	2,676	33,670
Reactor Auxiliary Building	33	15.5	42,650	10,400	164	68	428
	34	15.5	158,800	16,050	270	68	1,029
	35	23.0	4,009,200	10,607,934	531	660	17,637
	36	25.0	14,056,450	24,867,658	1,017	1,472	34,965
	37	25.0	27,605,870	50,543,260	3,177	3,055	49,093
	38	31.0	38,109,290	71,336,276	3,832	3,973	59,499

TABLE 3.7-3 (Cont'd)

Foundation Mat

Shape	Length (ft.)	Width (ft.)	Thickness (ft.)	Weight (Kips)	Mass Moment of Inertia (K-ft^2) N-S	Mass Moment of Inertia (K-ft^2) E-W
Rectangular	380	267	12	293,100	3.4440×10^9	1.6244×10^9

Soil Spring Constants

K_{H2} Bearing Spring Const (K/ft.) N-S	K_{H2} Bearing Spring Const (K/ft.) E-W	K_{H1} Sliding Spring Const (K/ft.) N-S	K_{H1} Sliding Spring Const (K/ft.) E-W	Rocking Spring Const (ft.-K/radian) N-S	Rocking Spring Const (ft.-K/radian) E-W	E (K/ft.^{2})	μ
127,500	156,500	865,000	881,000	38.4×10^9	24×10^9	2764.8	0.5

E: Young's Modulus of Soil

μ: Poisson's Ratio of Soil

K_{H1}: Horizontal or translational spring constant for soils below base mat

K_{H2}: Horizontal or translational spring constant for soils against side faces of base mat**

** By including K_{H2}, the natural period of the structure decreased approximately 7.5%, thereby moving toward the peak response region of the response spectrum. Therefore, it is conservative to include this spring constant in the analysis.

Physical Properties for Structural Materials

A. Concrete

Modulus of Elasticity:

$E_c = W^{1.5}\ 33\sqrt{f'_c} = 5.11 \times 10^5$ KSF

where W = 140 lb./ft.^{3}, f'_c = 4,000 psi

$G_c = E_c\ /\ 2(1+\mu) = 2.16 \times 10^5$ KSF

where $= \sqrt{f'_c}/350 = \sqrt{4,000}/350 = 0.18$

B. Soil

Modulus of Elasticity:

Pleistocene Sediments:

$\mu = 0.5$, G_1 = 6,400 psi = 921.6 KSF

$E_1 = 1.5 \times 2 \times 921.6$ = 2,764.8 KSF

Recent Alluvium:

$\mu = 0.5$, G_2 = 2,300 psi = 331.2 KSF

TABLE 3.7-4

INPUT DATA FOR SEISMIC ANALYSIS
VERTICAL EXCITATIONS

	Mass No.	Cross-Sectional Area ($ft.^2$)	Weight (Kips)	Member Length (ft.)	Floor Stiffness (k/ft.)	Floor Mass Point No.
Shield Building						
	1	802	7,010	27.73		
	2	1,423	4,959	21.7		
	3	1,423	4,319	19.7		
	4	1,423	4,104	20.0		
	5	1,423	4,446	25.0		
	6	1,423	6,242	25.0		
	7	1,423	4,445	20.0		
	8	1,423	4,104	22.0		
	9	1,423	5,301	19.0		
	10	1,423	2,822	18.0		
	11	1,423	10,173	17.0		
		4,524				
Containment Vessel						
	12	195	354	21.5		
	13	259	376	22.0		
	14	426	376	22.0		
	15	575	668	22.0		
	16	832	1,735	22.0		
	17	575	755	22.0		
	18	575	755	22.0		
	19	575	755	22.0		
	20	575	755	22.0		
	21	575	755	11.0		
Reactor Building Interal Structures						
	22	1,250	1,295	7.3		
	23	1,250	2,167	7.0		
	24	2,111	7,973	11.0		
	25	2,111	5,682	12.0		
	26	2,623	9,438	14.5		
	27	3,945	8,855	12.5		
	28	3,353	7,802	7.0	20.6×10^6	29
Fuel Handling Building						
	30	840	6,853	44.5		
	31	2,357	10,240	24.5		
	32	2,441	25,017	20.0		
	33	2,408	33,670	36.0		
Reactor Auxiliary Building						
	34	232	428	15.5		
	35	338	1,029	15.5		
	36	1,191	17,637	23.0		
	37	2,489	34,965	25.0		
	38	4,247	49,093	25.0		

TABLE 3.7-4 (Cont'd)

Foundation Mat	Mass No.	Weight (Kips)	Vertical Spring K_z (K/FT)
	40	291,110	1.5076×10^6

Soil Spring Constants

The vertical spring constant considered in the present Waterford - 3 studies consists of two parts: one due to normal stress over the base area; another due to shear stress around the side areas.

a) Bearing Spring Constant: K_{z_1} (Vertical spring constant for soils below base mat)

$$K_{z_1} = \frac{G}{1-\mu} \beta_z \sqrt{BL}$$

$G = 6{,}400$ psi $= 921.6$ KSF
$\mu = 0.5$
$L = 380'$, $B = 267$

Shear modulus and Poisson's ratio for pleistocene sediments

$L/B = 380/267 = 1.43$
$\beta_z = 2.15$

$$K_{z_1} = \frac{921.6}{0.5} \times 2.15 \times \sqrt{380 \times 267}$$

$= 1{,}260{,}988$

$= 1.260988 \times 10^6$ K/ft.

(Reference: "Design Procedures for Dynamically Loaded Foundations," R V Whitman and F E Richart, Jr Journal of the Soil Mechanics and Foundation Division, 1967)

b) Sliding Spring Constant: K_x (Vertical spring constant for soils against side faces of base mat)**

$K_x = 2(1 + \mu)\, G \beta_x \sqrt{BL}$

$G = 2{,}300$ psi $= 331.2$ KSF for recent alluvium

$\mu = 0.5$

L is the length of rectangular foundation in the direction of acting force; for side effects L is equal to the thickness of the mat.

$L = 12'$, $B_1 = 380'$, $B_2 = 267'$

$L/B_1 = 12'/380' = 0.0316$ $\beta_{x_1} = 1.0$

** See Table 3.7-3 for the similar reasons to include K_x in the analysis.

$L/B_2 = 12'/257' = 0.045 \quad \beta_{x_1} = 1.0$

$$K_x = 2\left[2(1 + 0.5) \times 331.2 \times \sqrt{12 \times 380} + 2(1 + 0.5) \times 331.2 \times \sqrt{12 \times 257}\right]$$

$$= 6(331.2 \times 67.5 + 331.2 \times 56.6)$$

$$= 6 \times 41,100 = 246,610 \text{ K/ft.}$$

Vertical Soil Spring Constant:

$$K_z = 1,261,000 + 246,600$$

$$= 1,507,600$$

$$= 1.5076 \times 10^6 \text{ K/ft.}$$

Lumped Mass Weight of Foundation Mat

$$W = 297.110^K$$

Consider Mat as a one degree of freedom structure, the natural period is:

$$T = 2\pi\sqrt{\frac{297,110}{32.2 \times 1.5076 \times 10^6}} = 0.492 \text{ sec.}$$

Consider the whole mathematical model as a one degree of freedom structure, the natural period for W = 645.930 = 200.60 x 10^2 k - sec.^{2}/ft. is:

$$T = \frac{2\pi}{100}\sqrt{\frac{200.60}{1.5076}} = 0.724 \text{ sec.}$$

If the shear modulus G increases to 3G, 5G, then   becomes

$$T = \frac{0.722}{\sqrt{3}} = 0.418 \text{ sec. (for 3G)}$$

$$T = \frac{0.722}{\sqrt{5}} = 0.324 \text{ sec. (for 5G)}$$

TABLE 3.7-4 (Cont'd)

Pressurizer:

Floor Stiffness:

$K = 870\ E\ I_a/a^2$ $a/b = 1$ pg. 167, Norris

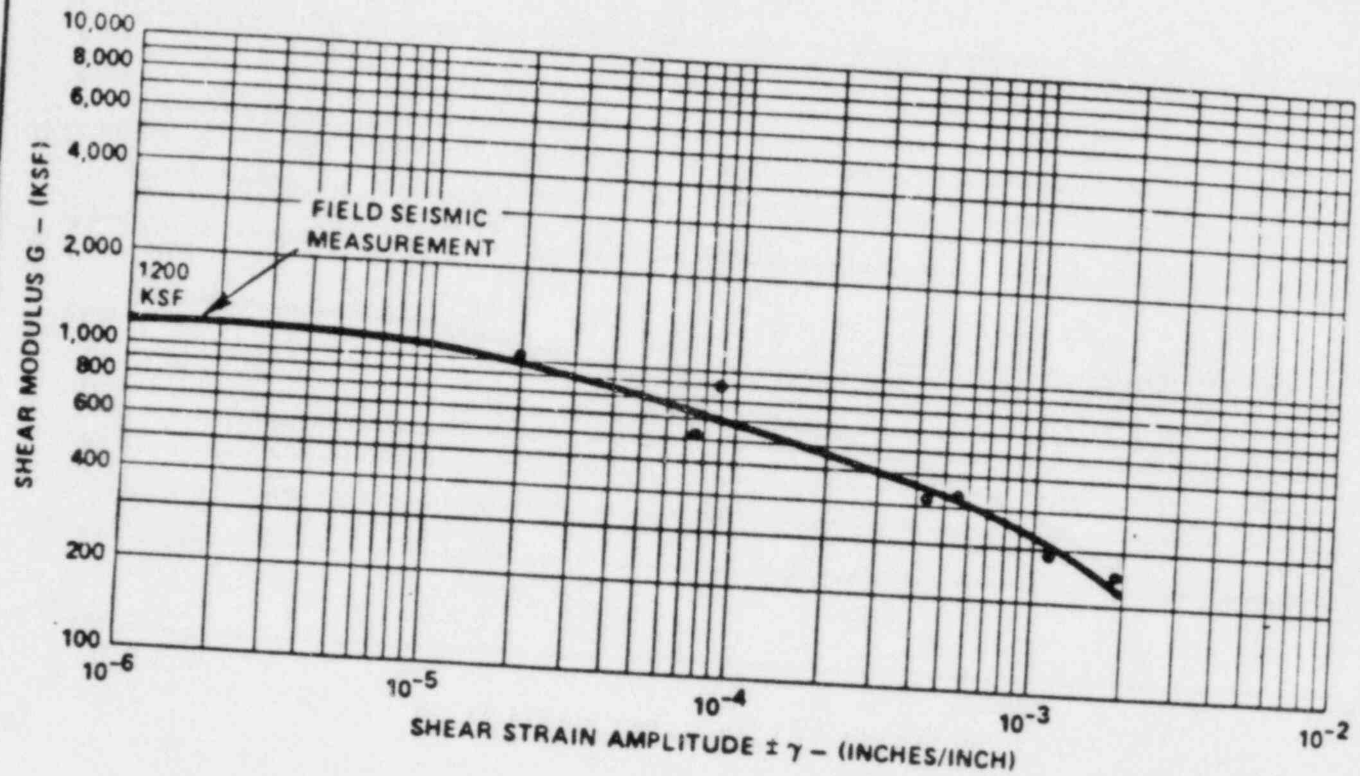
I_a is the moment of inertia per unit width.

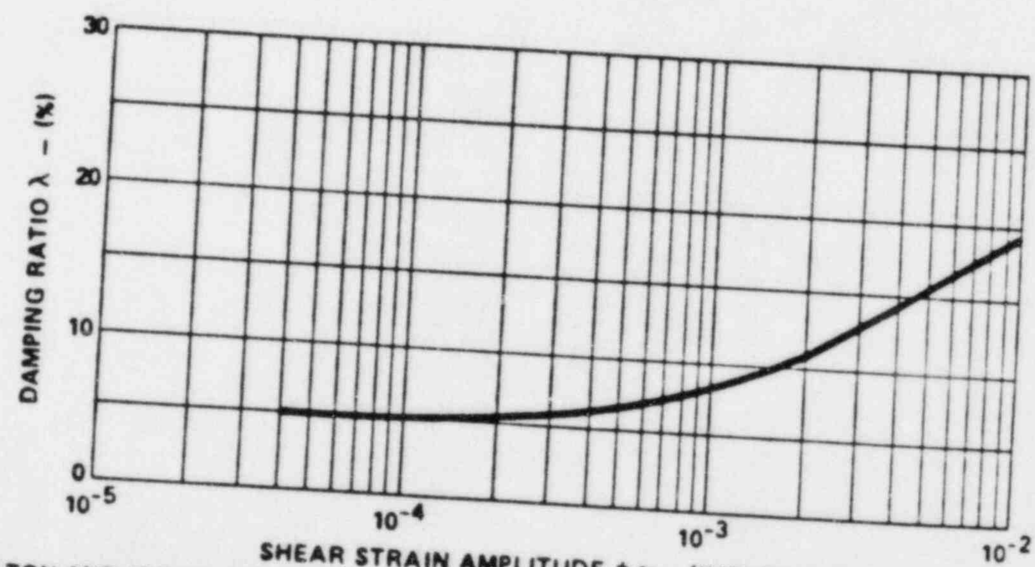
$I_a = \frac{h^3}{12} = \frac{5^3}{12} = \frac{125}{12}$, $a = 15$

$K = 870 \times 511{,}000 \times \frac{125}{12} \times \frac{1}{15^2} = 2.06 \times 10^7$ K/ft.

$W = 287^K$

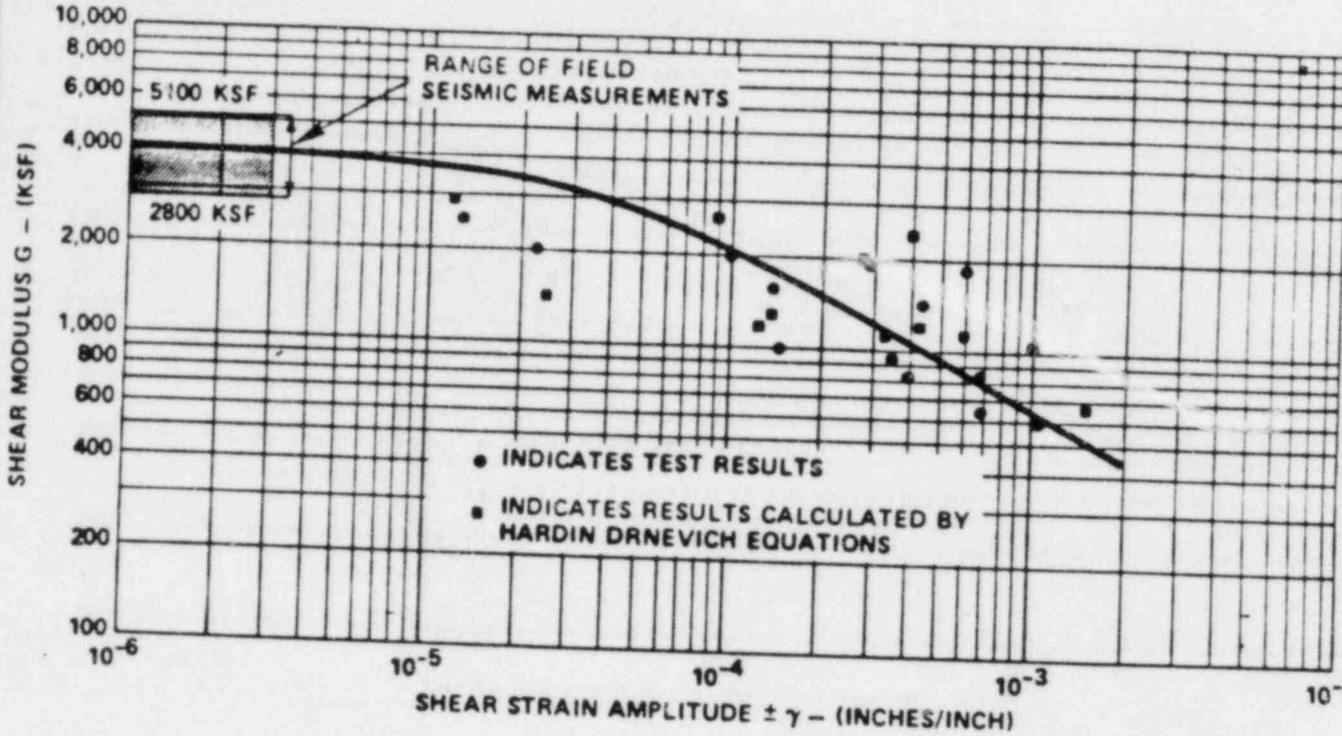
Reference: Structure Design for Dynamic Loads, Charles H Norris

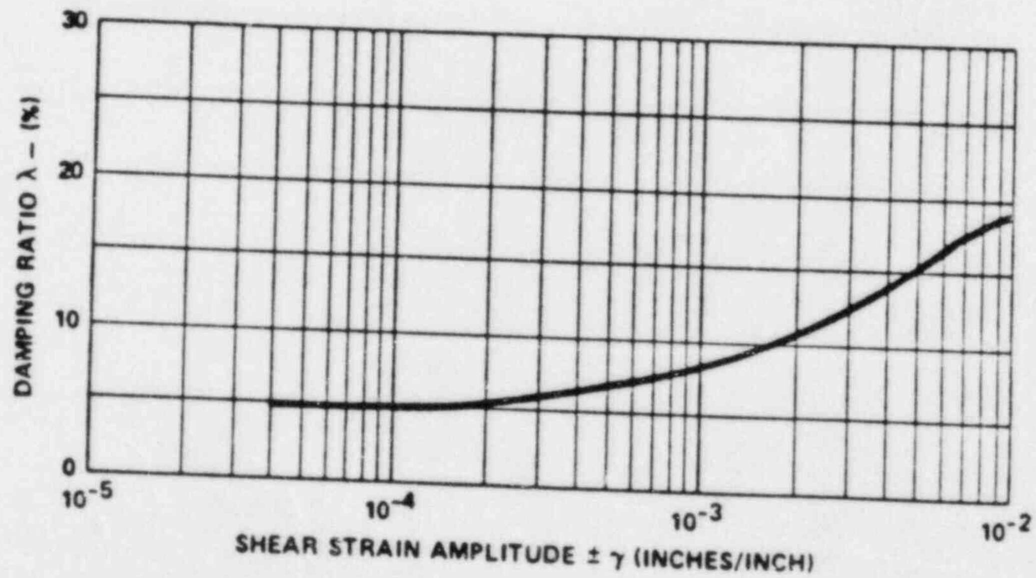




TAKEN FROM SEED, H. BOLTON AND IDRISS I.M. (1969)
"THE INFLUENCE OF SOIL CONDITIONS ON GROUND
MOTIONS DURING EARTHQUAKES"

LOUISIANA POWER & LIGHT CO. Waterford Steam Electric Station	AMENDMENT NO. 33 (9/83) SHEAR MODULUS & DAMPING VS STRAIN RECENT MATERIAL (GRADE TO -40 FT. MSL)	Figure 2.5-77





TAKEN FROM SEED, H. BOLTON AND IDRISS, I.M. (1967) "THE INFLUENCE OF SOIL CONDITIONS ON GROUND MOTIONS DURING EARTHQUAKES"

LOUISIANA POWER & LIGHT CO. Waterford Steam Electric Station	SHEAR MODULUS & DAMPING VS. STRAIN – UPPER PLEISTOCENE (–40 FT. MSL TO –317 FT. MSL)	Figure 2.5–78

APPENDIX B

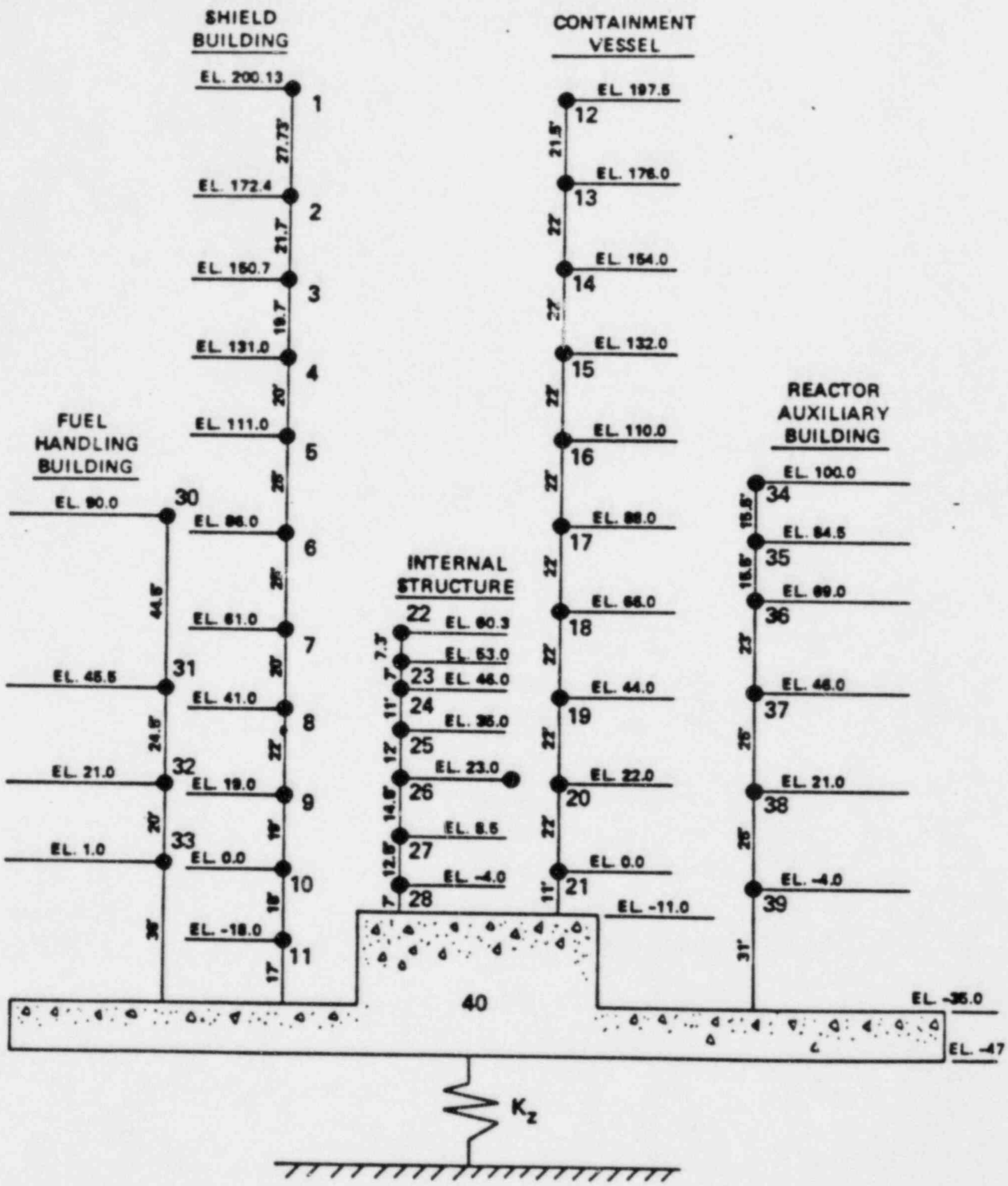
TABLE 3.7-9

COMPARISON OF ACCELERATION FOR SEISMIC CATEGORY I STRUCTURES
USING RESPONSE SPECTRA AND TIME HISTORY METHODS

SSE
SOIL SHEAR MODULUS = 16050 psi

	Mass No.	Elevation (Ft)	Response Spectrum Method (5%)			Time History Method		
			E-W Accel (G)	N-S Accel (G)	Vert Accel (G)	E-W Accel (G)	N-S Accel (G)	Vert Accel (G)
Shield Bldg.	1	200.13	0.498	0.432	0.180	0.546	0.448	0.175
Containment Vessel	12	197.50	0.362	0.314	0.173	0.387	0.320	0.16[illegible]
Reactor Bldg. Internals	22	60.3	0.256	0.245	0.172	0.235	0.217	0.168
FHB	29	90.0	0.276	0.267	0.176	0.262	0.245	0.167
RAB	33	100.0	0.291	0.274	0.177	0.284	0.254	0.170
Mat.	39	-37.24	0.200	0.210	0.171	0.197	0.197	0.167

APPENDIX C

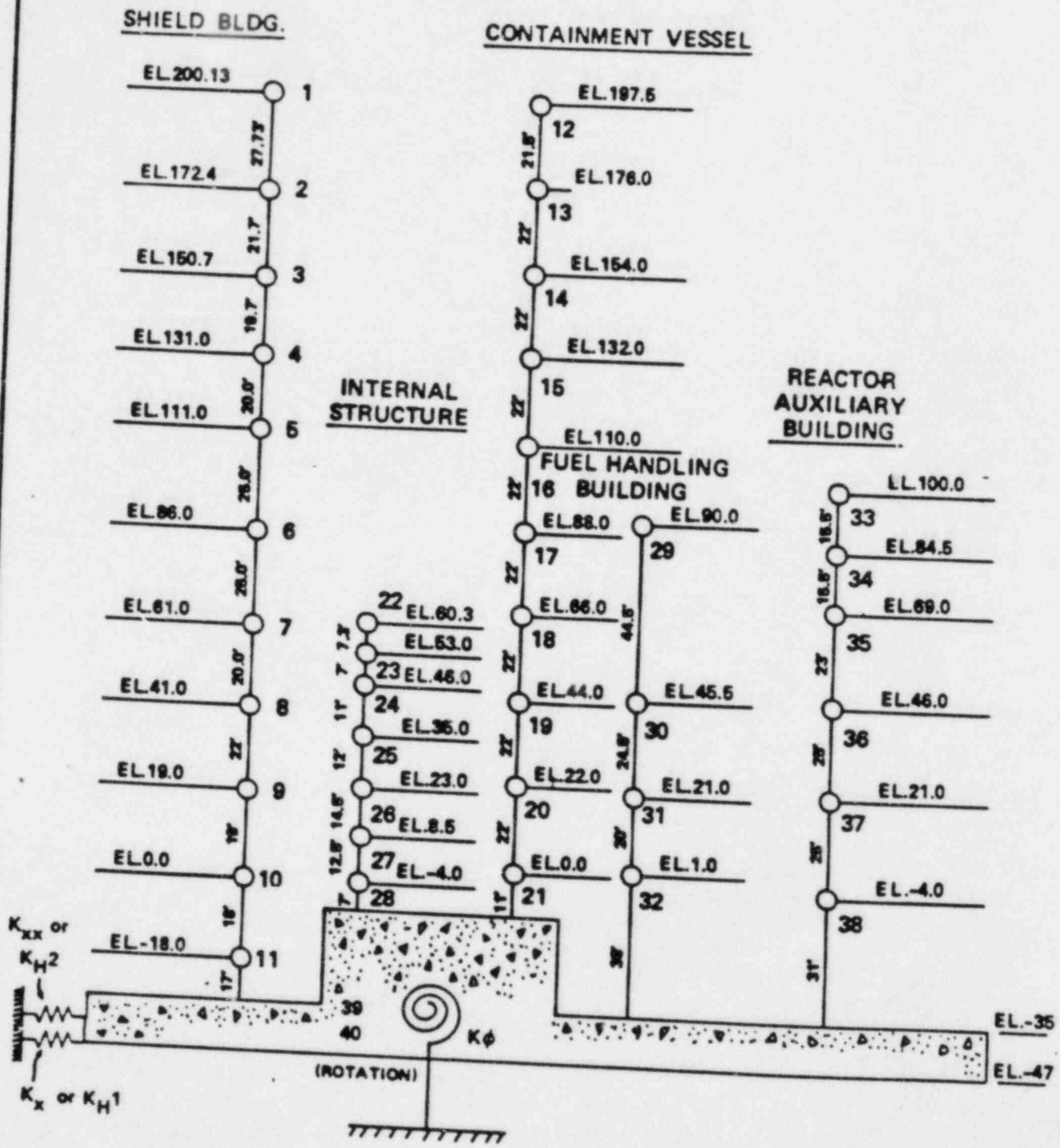


LOUISIANA POWER & LIGHT CO.
Waterford Steam Electric Station

MATHEMATICAL MODEL FOR SEISMIC ANALYSIS (VERTICAL EXCITATION)

Figure 3.7-10

APPENDIX D



LOUISIANA POWER & LIGHT CO.
Waterford Steam Electric Station

AMENDMENT NO. 2 (3/79)

MATHEMATICAL MODEL FOR SEISMIC ANALYSIS (HORIZONTAL EXCITATIONS)

Figure 3.7-9

APPENDIX E

Structural steel is designed in accordance with basic working stress design methods. Increased allowable stresses are used for the accident condition.

The final designs of the interior structures and equipment supports are reviewed to assure that they can withstand applicable design pressure loads, jet forces, pipe reactions, and earthquake loads without loss of function. The deflections or deformations of the structures and supports are checked to ensure that the functions of the containment and safety feature systems are not impaired.

3.8.3.4.1.1 Computer Programs Utilized for Structural and Seismic Analyses

The following computer programs have been used in structural and seismic analyses to determine stress and deformation responses of seismic Category I structures. A brief description of each program and the extent of its use are given below:

FIXMAT 2037

FIXMAT 2037 is an Ebasco in-house computer program which operates on BURROUGHS 6700 and handles the dynamic analysis of lump-mass-spring type models. It provides results of natural periods of vibration, mode shapes participation factors and structural responses. Both methods of time history and response spectrum can be specified. The program also generates floor response spectra.

This program was used for all seismic analysis of seismic Category I structures and to calculate all floor responses and their spectra curves.

STARDYNE 2 AND NASTRAN 22

STARDYNE 2 AND NASTRAN are public domain computer programs designed to analyze static and dynamic problems of linear elastic structural systems using finite element techniques.

The programs are capable of a) computing structural deformations and member loads and stresses caused by an arbitrary set of thermal and mechanical applied loads and/or prescribed displacements, and b) dynamic response analyses for transient, steady state, harmonic, random and shock spectra excitation type loading conditions. The results are presented as displacements, accelerations or velocities and/or as internal member loads/stresses. 22

EAC/EASE

The EAC/EASE (Elastic Analysis for Structural Engineering) is a public domain computer program developed by Engineering/Analysis Corporation (Redondo Beach, California) which provides static structural analyses of linear, three-dimensional systems, subjected to sets of arbitrarily prescribed mechanical and thermal loads and displacement boundary conditions. The program is capable of modelling with three distinct types of structural elements, beams, membranes, and plates, which can be used separately or together in assembling a three-dimensional array. The program computes joint displacements, reactive forces, beam forces moments and stresses.

Rigid Frame 2117

Rigid Frame 2117 is an Ebasco in-house computer program which analyzes a two dimensional single or multi-story rigid frame under vertical or horizontal loads. This is accomplished by using a stiffness matrix approach with a Gaussian elimination method. This program was used for frame analysis of all seismic Category I structures.

FIXMAT 2037 program was developed by Ebasco. Since this program is not a recognized program in public domain, a comparison with STARDYNE (version 4/1/72) and NASTRAN, both proven programs in public domain, is made in Tables 3.8-23 to 3.8-30 to demonstrate its validity and applicability.

Rigid Frame 2117 is also an Ebasco program and operates on a Burroughs 6600 machine. Due to the relatively simple nature of the program, comparison of results were made by solving several sample problems with known solutions to demonstrate its validity and applicability.

As discussed above, CDC/STARDYNE and EAC/EASE programs are proven programs existing in the public domain and therefore no comparison of results with other programs is presented.

3.8.3.4.1.2 Analysis and Design Procedures

a) Dynamic Analysis

Analytical techniques for the seismic dynamic analysis are described in Section 3.7.

Analytical techniques for the protection against dynamic effects associated with the postulated pipe rupture are described in Section 3.6.

Analytical technique for the protection against missiles is described in Section 3.5.

b) Design Procedures

All the structural elements of the internal structures are analyzed statically based on a LOCA loading combination described in Subsection 3.8.3.3. The equivalent static load resulting from the application of the accelerations at various levels obtained from the above mentioned dynamic analysis are included.

NUCLEAR PLANT ISLANDS STRUCTURE

COMMON FOUNDATION BASEMAT MONITORING PROGRAM

GENERAL

The monitoring program for the Nuclear Plant Island Structure (NPIS) Common Foundation Basemat has been established to provide continuing assurance of basemat integrity. The program provides for data collection and trending such that information will be available to conduct a detailed evaluation and correlation of data should this become necessary or desirable. The elements monitored were chosen to reflect relationships among the parameters. For example, cracking could result from induced stress caused by differential settlement of the foundation. Should an unexpected indication be observed, the data can be used to identify potential causes, and allow an accurate assessment of the structural integrity of the basemat.

PROGRAM OVERVIEW

The Basemat Monitoring Program established to demonstrate continued integrity is divided into four major areas. The criteria will provide overall assurance that changes in observable and measurable phenomena will be detected and that sufficient data is available to evaluate the causes and effects with respect to the basemat integrity. The program elements are:

A. Basemat Settlement
B. Ground Water Chemistry
C. Seasonal Variation of Groundwater Level
D. Crack Surveillance

The program is implemented using approved Plant Operating Manual procedures to conduct the necessary surveillances.

SURVEILLANCE METHODOLOGY

A. Basemat Settlement. This portion of the program is essentially an extension of the data taken during the past several years. Elevation data is taken on selected monitoring points and differential settlement is checked between key monitoring points. FSAR Figure 2.5-117 shows the previously used monitoring points and the associated settlement. Prior to fuel load, some monitoring point locations were revised and additional points added. Several sets of concurrent data on the old and new monitoring points were taken to provide correlation data between the points. The monitoring points were revised to facilitate measurements during plant operation considering accessibility from an ALARA and Security standpoint. Enclosure (1) provides an overview of the selected monitoring points and the calculations made to determine differential settlement. As shown in the enclosure a one inch criteria is used as a threshold beyond which additional evaluation is required. This criteria is relative to the baseline data taken prior to fuel load.

Presently the elevation data is taken through surveys conducted on a quarterly basis. Similar to other equipment monitoring programs such as Steam Generator Tube Inspection (Technical Specification 3.4.4) and Snubbers (Technical Specification 3.7.8) the monitoring interval will be lengthened provided no significant changes are observed and no adverse or unexplained data has been observed. Three consecutive, satisfactory surveillances are required to extend the interval to the next interval stated below. The intervals are: (as used within Technical Specifications)

Q At least once per 92 days

SA At least once per 184 days

A 12 months

R At least once per 18 months

B. Groundwater Chemistry. Actual corrosion in the groundwater surrounding the basemat is highly unlikely given the normal groundwater chemistry found in the vicinity of Waterford 3, and the minimal contact between the water and rebar. Nonetheless, water samples are taken and analyzed for chloride content from wells provided for this specific purpose. Enclosure (2) shows the locations of the wells with respect to the basemat. A conservative threshold of 250 ppm chloride has been established beyond which more extensive water analyses and/or evaluation is required to determine the potential impact on rebar corrosion.

Samples are presently being taken and analyzed each quarter. Several samples have shown that chloride content is well below the 250 ppm threshold and stable around 30 ppm. It is intended to extend the interval of chemical samples in the same manner as the basemat settlement provided the chloride content is below the threshold and shows no significant change from the previous sample. This provides assurance that long term natural changes are detected as well as groundwater contamination from an external source.

C. Seasonal Variation of Groundwater Level. Groundwater level measurements will be taken and maintained to provide data in the event that evaluation of other observed basemat phenomena becomes necessary. These measurements will be taken on a quarterly basis. The wells established for groundwater sampling provide a means to determine the groundwater level.

D. Significant Cracking. All currently observable cracks in the basemat have been mapped, although due to inaccessibility and floor finish some existing cracks may still be undetected. State-of-the-art NDT inspections, calculations, and evaluations have determined that existing cracking does not imply any degradation of the designed structural integrity. To provide further assurance that basemat integrity is not degraded from some unanticipated mechanism or postulated event from this time on, a program associated with basemat cracks has been established. The program includes obtaining quantitative data on changes in crack width.

The quantitative program will consist of taking precision measurements on representative cracks that are chosen based on visual appearance, crack depth and accessibility. These cracks will be "the most significant cracks" for comparison purposes. These cracks will be instrumented similar to that shown in Enclosure (3) which allows detection of any changes in crack width. A change in crack width, should any occur, will be used in two ways.

The crack monitoring activities also include a visual inspection of the previously mapped cracks and inspection of accessible areas of the basemat for additional cracks. Additional cracks and changes to existing cracks are updated on the crack maps.

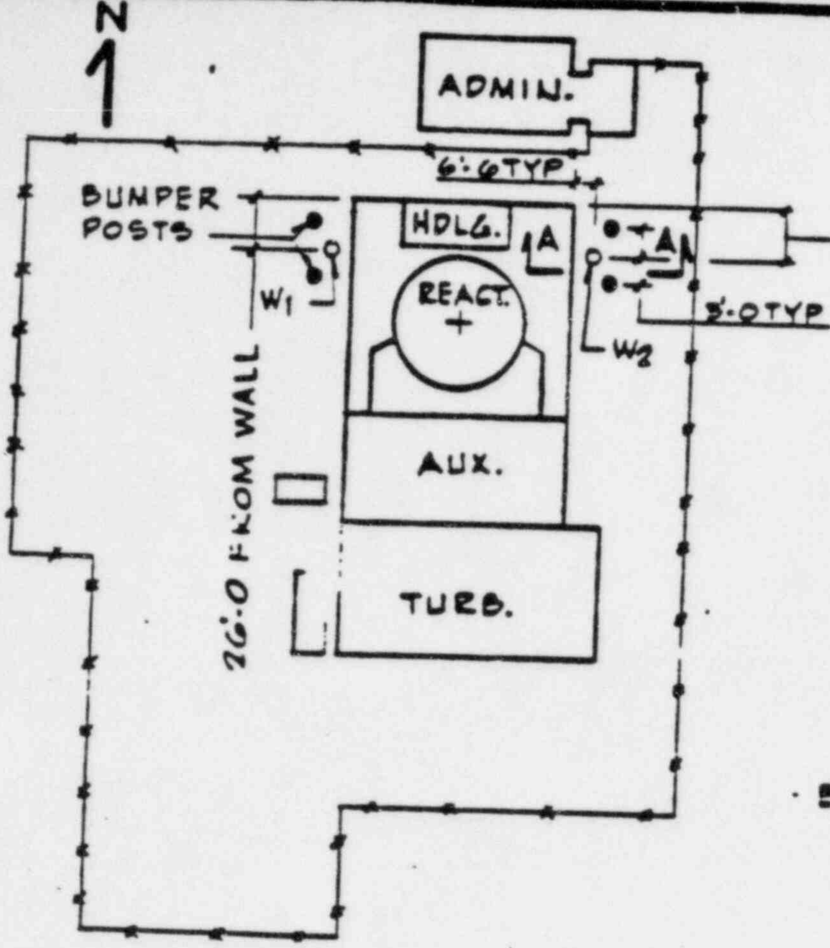
The cracks in the vertical walls were investigated by the Non-Destructive Examination (NDE) program using ultra sound. These cracks were identified as being shallow and probably resulting from shrinkage. They are not related to the cracks in the basemat. Brookhaven National Laboratory (BNL) agrees that "...cracks in the vertical walls are no longer considered a problem." Therefore, LP&L does not propose to either map the cracks in vertical walls or to monitor their length, width, or other characteristics.

FOUNDATION BASEMAT DIFFERENTIAL SETTLEMENT MONITORING

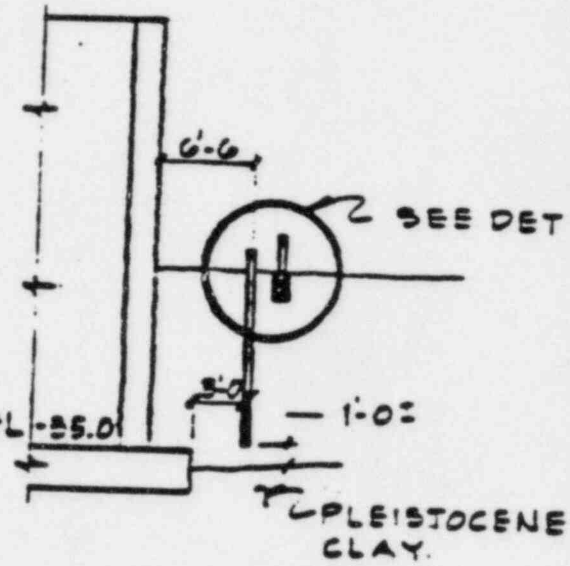
	Monitoring* Points	Differential Calculation** Basemat Edge Relative to Shield Building
Shield Building Monitoring Points	A (East Side) B (West Side)	
Basemat Corner Monitoring Points	NE (Northeast)	[A-NE]
	SE (Southeast)	[A-SE]
	SW (Southwest)	[B-SW]
	NW (Northwest)	[B-NW]
Basemat Edge Monitoring Points	E1 (East)	[A-E1]
	E2 (East)	[A-E2]
	W1 (West)	[B-W1]
	W2 (West)	[B-W2]

*Monitoring points may be located on the Basemat or on the walls above the Basemat to facilitate measurements. Monitoring points may be relocated after original baseline measurements provided the correlation of the new and old monitoring points is measured and recorded to enable comparison to the baseline data.

**Baseline Calculations shall be taken prior to initial unit operation. Subsequent calculations shall be compared to the baseline calculation data. Changes from the baseline calculation of less than or equal to one inch are acceptable.

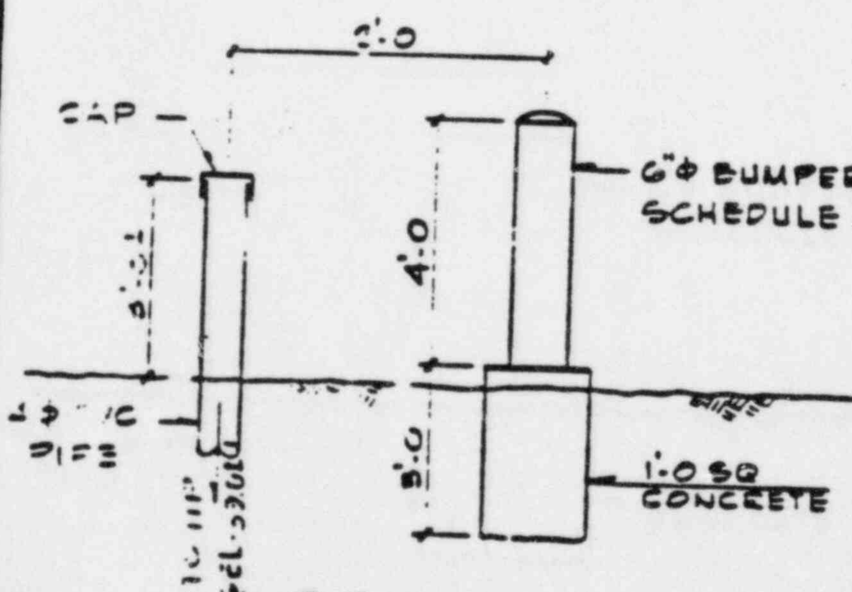


PLAN



SECT A

NOTE
6" ± MAXIMUM HORIZONTAL DEVIATION OF GROUNDWATER SAMPLING WELL BETWEEN: POINT OF ENTRY AT GRADE LEVEL AND BOTTOM OF WELL.



DET 1

LOUISIANA POWER & LIGHT COMP
WATERFORD S.E.S UNIT NO. 3
1983-1165 MW INSTALLATION

GROUNDWATER SAMPLING WEL

EBASCO SERVICES INC.-FIELD

SCALE	RELEASED	DATE
DIV. / DR. / CH.		FIELD SK SK 1864

NO.	DATE	REVISION	BY	CH.	RELEASED

ASSEMBLY FOR MONITORING THE PROPAGATION OF THE CRACK WIDTH