COMPATIBILITY LARGE MAT DESIGN TO FOUNDATIO CONDITIONS

BY

J L EHASZ[1] AND P C LIU[2]

SYNOPSIS

MARK -
"COMPATIBILITY OF LARGE MAT
DESIGN TO FOUNDATION CONDITIONS"
WILL GIVE YOU A GOOD BACK-
GROUND OF THE STRUCTURE.
BILL C

cor ... lement
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and I ... on the
west ... about 20 miles west of New Orleans.
It is a 1165 MW PWR nuclear unit. The construction permit was issued by the Atomic Energy Commission in November, 1974, and the plant is scheduled for commercial operation in early 1980.

The plant is designed to have a Nuclear Plant Island Structure, or a Combined Structure which will house all the seismic Class I structures. The seismic Class I structures include the Reactor Building, the Reactor Auxiliary Building, the Fuel Handling Building, and the Essential Cooling System Structures. The Nuclear Plant Island Structure is a rectangular box-like structure on a concrete mat with the Reactor building located near the center, and other buildings located around the reactor building. The Reactor Building is a double containment structure 154 ft. in diameter and 250 ft. above the common mat. The lower two stories of the structure will be below final plant grade.

The Nuclear Plant Island Structure will be supported on a continuous common mat 270 ft. wide, 380 ft. long, and 12 ft. thick. The mat is supported on the Upper Pleistocene clays which underlie the site about 60 ft. below plant grade.

50-382

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B/14

COMPATIBILITY — LARGE MAT DESIGN TO FOUNDATION CONDITIONS

BY

J L EHASZ[1] AND P C LIU[2]

SYNOPSIS

This paper describes the foundation conditions and settlement considerations that dictated the coordinated analysis, design and construction sequencing effort. It considers a design technique for large structural mats on compressible foundations; establishes the influence of the changing subsurface stiffness due to settlement, illustates the redistribution of structural shears and moments within the foundation mat and considers the effects of foundation stiffness on dynamic response.

INTRODUCTION

The Waterford Unit No. 3 power plant owned by Louisiana Power and Light Company is being constructed in St. Charles Parish, on the west bank of the Mississippi River about 20 miles west of New Orleans. It is a 1165 MW PWR nuclear unit. The construction permit was issued by the Atomic Energy Commission in November, 1974, and the plant is scheduled for commercial operation in early 1980.

The plant is designed to have a Nuclear Plant Island Structure, or a Combined Structure which will house all the seismic Class I structures. The seismic Class I structures include the Reactor Building, the Reactor Auxiliary Building, the Fuel Handling Building, and the Essential Cooling System Structures. The Nuclear Plant Island Structure is a rectangular box-like structure on a concrete mat with the Reactor building located near the center, and other buildings located around the reactor building. The Reactor Building is a double containment structure 154 ft. in diameter and 250 ft. above the common mat. The lower two stories of the structure will be below final plant grade.

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For the purpose of minimizing differential settlements between buildings as well as improving the dynamic structural response of the structures, the combined structure is designed according to the floating foundation principle. It is designed to have sufficient buoyancy within the soil to maintain soil bearing pressures on its common mat only slightly greater than the pressure existing at that level prior to construction of the structure.

This paper describes the criteria used in the foundation design and the structural design of the large concrete foundation mat. It discusses and illustrates the effects of variations in soil stiffness considered to achieve static compatibility of the soil-structure system and also considers the effects of soil stiffness on dynamic response.

FOUNDATION DESIGN CONCEPTS

The foundation conditions at the site were determined through an extensive and detailed boring and testing program. The subsurface soil profile is generalized on Figure 1 together with the properties of the various strata. The details of the investigation program and evaluation of the various foundation alternatives considered are described in an earlier paper;[1] however, the final foundation design concept and construction sequencing are significant to the structural analysis and will therefore be further developed in this paper.

The existing soil conditions at the site are evaluated in terms of vertical effective stresses. These stresses are now in the order of 3,300 lb per sq ft. Figure 2 illustrates the various stress conditions during construction. Upon dewatering the stresses briefly go up to 6,750 lb per sq ft. However, at the end of the first construction stage upon completion of excavation to the bottom of mat elevation the effective stress reduces to zero. Next, an intermediate stage of construction is illustrated in which the effective stress at the bottom of the mat is equal to 4000 lb per sq ft. This is due to the weight of the concrete structures with the water table held at some level below the mat. The final stage illustrated is the completed stage, with the buildings completed to the final elevation, the sand backfill completed, and the ground water table back to its initial condition at elevation +8 ft. The final pressures are indicated. It can be seen that the pressures should be 3100 lb per sq ft. This is 200 lb per sq ft. less than the existing effective soil pressures at the site.

The other significant consideration for this foundation design is the settlement induced in the deep soil column of relatively compressible soils. Any considerable increase in effective soil pressure will cause excessive consolidation of the foundation soils, this consideration has led to the adoption of the "floating foundation" design as well as the consideration of variable foundation soil stiffness for the structural design of the foundation mat.

Since this "floating foundation" concept involves the balancing of existing site soil pressures, a soil pressure time history diagram

1. Ehasz, J. and Radin, E., "Foundation Design of the Waterford Nuclear Plant,"
The 2nd Specialty Conference on Structural Design of Nuclear Plant Facilities, Chicago, December 1973.

was developed and is illustrated in Figure 3. This figure details the soil pressures at the bottom of the foundation mat. It begins with the existing soil pressure conditions and develops the pressures during the various phases of the work. After excavation, the pressures are reduced to zero. This is analogous to the phase described earlier. During the concrete construction phases, the pressures begin to increase and continue until a stress of 4000 lb per sq ft. has been applied. This pressure has been determined to be the maximum short term preload pressure that was desirable during reloading. This was based on the reconsolidation characteristics of the soils and was deemed to be a prudent value to maintain during the construction phase. In order to keep the soil pressure at this level or below, the water table will be allowed to rise in accordance with the predetermined plan as indicated in Figure 3. This procedure will reduce the effective soil pressures and maintain the effective pressures below the 4000 lb per sq ft level and ensure that the final effective pressures are established as described above.

Detailed construction phases have been given particularly close attention. Each construction phase corresponds to the phase outlined on the aforementioned soil pressure time history diagram. These phases allow for the various construction features involved during each step of the work including the sand backfilling, saturation of backfill and other construction aspects.

In summary, the detailed foundation design has considered the following principles, rationale and distinct features:

a) The base of the combined mat foundation will be located at elevation -47 ft. resulting in a final average effective soil loading condition of 3100 lb per sq ft. as compared to the existing effective overburden pressures of 3300 lb per sq ft. Minor tendencies of relaxation or rebound will be absorbed within the compacted granular backfill by frictional transfer. This fill will effectively equalize existing pressures and all future loadings which may vary due to water table fluctuations. A compacted filter blanket of locally available shell will be installed under the base of the foundation mat to act as a pore pressure equalizer for the Pleistocene clays.

b) Design criteria have established a margin of overload above the existing effective soil pressures which will be applied only during the construction phase of the work. This is primarily to maintain a margin of pressure below the preconsolidation pressure of the materials with the lower over-consolidation ratios.

c) The excavation of the recent deposits, consisting of soft clays, silts and sands extending to approximate elevation -40 ft. and subsequent excavation of the stiff Pleistocene clays will result in rebounding of the final exposed clay bearing strata during the excavation period. The major portion of the rebound will occur during the final excavation stages of the Pleistocene clays. Control will crete placeme in designated sections of the t in a predetermined sequence to minimize heave.

d) By conforming to the "floating foundation" principle, settlement of the Class I structures will be confined essentially to the recompression range; that is, the range of the amount of movement that the clay surface will experience due to rebound. It is desirable to complete the major portion of the recompression settlement during the construction period. The applied loading sequence has been arranged with this particular aspect in consideration.

e) By applying a maximum effective loading of 4000 lb per sq ft. the major amount of recompression will take place during the construction phase. The phase loading diagram illustrated graphically in Figure 3 shows that, after a total load of 4000 lb per sq ft. has been applied, the granular backfill which will already have been placed and compacted to predetermined elevations, must be saturated in stages in order to achieve buoyancy and permit application of additional total load.

f) During the present construction phase, a dewatering system is installed around the perimeter of the excavation to control underseepage through semi-continuous silt and sand layers in the excavation slopes. In addition, deep wells have been sunk to the silty sand stratum extending from approximate elevation -77 ft to elevation -92 ft to relieve the hydrostatic pressure at this level and minimize heave of the Pleistocene clays. A series of recharge wells will also be located around the perimeter of the mat foundation extending to the filter blanket below the mat. It is concluded that the combination of dewatering and recharge wells will provide additional control, if required, in minimizing heave and recompression respectively. The construction loading sequence has been designed such that the maximum differential loading across the mat does not exceed 1000 lb per sq ft. The addition of compacted granular backfill will surcharge the foundation, thereby increasing bearing capacity, and also assist in control of deformation.

g) Detailed instrumentation, consisting of electrical extensometers, mechanical heave points, pore pressure piezometers and settlement markers, are installed to monitor heave and recompression settlement of the mat foundation. Since the "floating foundation" will induce smaller soil pressures than now exist, and since any recompression will essentially take place during the construction period, it can be concluded that very little, if any, long term settlements will occur. Any such settlements will be less than one inch and would be due to local pore pressure adjustments within the clays.

COMBINATION STRUCTURE MAT DESIGN

As can be realized from the above described foundation design conditions, all of the foundation bearing pressures induced by the structure have been considered to be uniform, that is, the total weight has been averaged across the entire base of the combination structure. There are only a few ways, in reality, that this condition can exist with the unsymetric layout of the various power plant structures. The possibilities reduce to considering the structural mat as being a completely rigid member, which would give uniform bearing pressures on any foundation soil; or by considering the foundation soil as being soft and yielding, which would also give uniform bearing pressures for any structural mat. Obviously, the reality, lies somewhere between these two extremes and the actual bearing pressures and structural shears and moments are a function of both the stiffness (rigidity) of foundation mat as well as how soft or yielding the foundation soils are. The following discussion describes the details of the study involved in going from establishing the structural mat thickness to the final design details of the structure.

THICKNESS DETERMINATION

In order to proceed with the detailed model, described later, the thickness of the foundation mat was studied with respect to foundation soil and concrete mat stiffness. A simplified mat model was developed, and the "EASE" finite element computer program was used. The mat was analyzed as a flat plate on elastic foundation, and the rigidity of superstructural system was not included. The finite element model was represented by 649 triangular plate elements, 270 beam elements, and 365 node points. Beam elements were introduced to input loads transmitted through the structural wall system supported by the mat. The subsoil flexibility was represented by vertical springs at each node point, and they were calculated based on a constant soil subgrade modulus. Two different soil subgrade moduli were studied each for a thickness of 10, 12 and 15 feet.

The representive mat deflection curves, through the North-South cross section for different mat thickness using two soil subgrade moduli are shown in Figure 4. From the mat deflection curves for the same soil subgrade modulus, it was found that the mat did not behave as a rigid structure and that increasing the mat thickness from 10 to 15 ft had very little effect on the relative rigidity. As the soil subgrade modulus was varied the magnitude of mat deflection changed accordingly, but the general pattern of deformation remains without significant change. The mat thickness optimization was based on the results of the mat designed to the corresponding structural loadings. The 12 foot thickness which was finally chosen was an economic compromise between the cost of additional concrete to eliminate shear reinforcing and provision of some shear reinforcing in local areas.

MODELING AND ANALYSIS TECHNIQUES

Once the elastic nature and the thickness of the mat were established the effects of the elastic as well as the plastic nature of the foundation sois were considered. Since interaction between the structure and the foundation is sensitive to the structural stiffness, the modeling of the system included the various buildings, walls and other structural components above the mat level.

Due to the complexity of the structures which will be supported by the common mat, the "STARDYNE" finite element computer program was chosen for the mat stress analysis. The structure was represented by an assembly of 643 beams, 2393 plates and 1087 nodes. The foundation soil was represented by linear springs at every node in the mat. The finite element model was designed to closely represent each part of structure rigidity together with load distribution, in order that the stress and deformation of the mat could be analyzed more accurately. Model simplification was made where minor carry-over effects existed. Structure walls which are directly supported by the mat, and floor slab systems which are supported by the column and beam frame systems on the mat were modeled in detail with little or no simplification.

The technique of utilizing the effective foundation springs, rather than the actual soil modulus of elasticity, was used to represent the structural foundation support since the long term effects of consolidation and settlement were considered. The initial subgrade modulus was calculated utilizing the elastic stress-strain characteristics from laboratory tests of the various soils as well as the geometry of the structure. The modulus was then adjusted to lower values in an iterative process based upon the results of bearing pressures and foundation settlement characteristics.

The analytical procedures were as follows: First the soil bearing pressures and deflections were calculated utilizing the initial subgrade modulus and considering it to be constant over the entire mat area. Next, the stresses were plotted and contours of equal stresses were constructed. These stress plots were utilized to adjust the subgrade modulus to be used in the next iteration. This adjustment was made by comparing the induced bearing pressures with present effective stresses at the foundation mat elevation, and then calculating the settlement that would be caused by the bearing pressures higher than the present stress conditions, and reducing the subgrade modulus accordingly. Thus, the modulus was varied from place to place over the mat area and this procedure was used to iterate the modulus until the resulting foundation bearing pressures were compatible with the anticipated settlements. The variations in bearing pressure contours from the assumed rigid mat condition to the initial constant modulus condition and then to the final variable modulus condition can be seen on Figure 5.

As illustrated on the above plan of pressure contours as well as on profiles A-A and B-B given on Figure 6, the effects of the yielding foundation soils can be recognized. This effect is one of forcing the combined structure and mat to spread the loadings toward achieving a more uniform pressure distribution that approaches the distribution given by the rigid mat analysis also shown on Figure 6.

A particular concern in the design of such a large structural mat is the shear and bending requirements resulting from the redistribution of the soil bearing pressures. As can be realized, from considering the effects of yielding support beneath the mat, the loadings are spread to other areas within the foundation, thereby, increasing the induced bending moments. As can be seen in Figure 7, the shears and moments within the mat are redistributed as the foundation yields and the bearing pressures become more uniform. The importance of the redistribution was observed and the stress changes due to moment redistribution within the structural mat were on the order of a 20%

increase in the more highly stressed areas when comparing the initial subgrade modulus and structural stiffness to the final iterated conditions; that is, concrete stresses increased from 1200 psi to 1400 psi. As can be realized from the moment comparisons there were locations where the stress changes were in excess of 100% but these were in the less stressed areas and of little significance to the design concerns.

In order to establish a conservative design for the structural mat, an envelope of design shears and moments was established for the section studied as indicated on Figure 8. This envelope covers all possible support conditions, ranging from the stiffer support indicated in the initial subgrade modulus to the complete yielding case indicated by the rigid mat consideration.

DYNAMIC ANALYSIS FOR SEISMIC LOADINGS

The earthquake intensity was established for the site through a detailed study of the geology and seismology of the Gulf Coastal Plain in accordance with the Reactor Site Criteria of the U.S. Atomic Energy Commission. A synthetic acceleration time history was developed for the site and site soil column response analysis were performed to establish the dynamic soils modulus and damping that are compatible with the strains induced during the postulated seismic event. These properties together with the structural characteristics of the buildings were used to perform the dynamic analysis of the combined structure.

Mathematical Model

In order to establish the seismic loads of buildings supported by the common mat, the Nuclear Plant Island Structure was modeled by a lump mass system. The model consisted of five individual cantilevers representing the Fuel Handling Building, Shield Building, the Containment Vessel, the Internal Structure and the Reactor Auxiliary Building, respectively. The five cantilevers are founded on the same base which, in turn is supported by foundation springs. For vertical and horizontal excitations, a two dimensional lump-mass spring system was used. For torsional response analysis, a three dimensional lump-mass spring system was used.

The foundation springs utilized for the dynamic analysis were calculated from the methods proposed by Whitman et. al. and incorporated the soil properties obtained from field, laboratory and soil column response studies. Since the soil shear modulus and damping are strain dependant parameters the effective values were established from the strains induced by both the static and dynamic considerations. Statistical methods of analysis were utilized to appreciate the participation of the modulus throughout the time history analysis. Conservative ranges of soil moduli were studied to establish the response of the soil-structure system.

Response Analysis

The structural dynamic analysis was based on the response spectra developed for 5% g (OBE) and 10%g (DBE). The spectrum, acceleration and displacement time histories for the lump-mass model were analyzed using a synthetic acceleration time history at the foundation base.

Parametric studies were performed to determine the relative effects of structural responses due to structure rigidity, and foundation spring constants. It was found that the foundation modulus influences a significant part of the structural response; the relative proportion of structure deflection due to structure rigidity, translation and rocking were approximately 5, 40, and 55% respectively.

By varying the magnitude of soil shear modulus in the dynamic analysis, the maximum structure loads were established and used in the mat design. The maximum structure and soil displacements resulting from the dynamic analysis were used to calculate the earthquake soil pressures used in the mat stress analysis.

The effects of the foundation stiffness on the seismic induced total shears and moments at the mat levelcan be seen on Figure 9. The effective shear modulus from the above studies was determined to be 1000 KSF. As can be seen, both the total shear and moment increase rapidly with increasing foundation stiffness to approximately G = 3000 KSF. Despite the fact that the soil modulus was stiffer than it could ever be, in reality, this value was conservatively used for the combined structure design.

Figure 10 shows the variation in response spectra for varying soil stiffness. The marked shift and change in the acceleration floor response spectrum can be seen to be quite significant.

Figure 11 shows the consistent spectral shift and change at other floor levels and structures within the combined structure. The higher floor levels indicate higher peak accelerations at higher levels, but consistent spectral shifts with changing foundation stiffness.

In order to maintain the consistent conservative design considerations required by the Regulatory Agencies the parametric studies of foundation stiffness were performed and conservative design envelopes for each building and level within the combined structure (Figure 11) were developed for the design floor responses.

DESIGN AND CONSTRUCTION COORDINATION

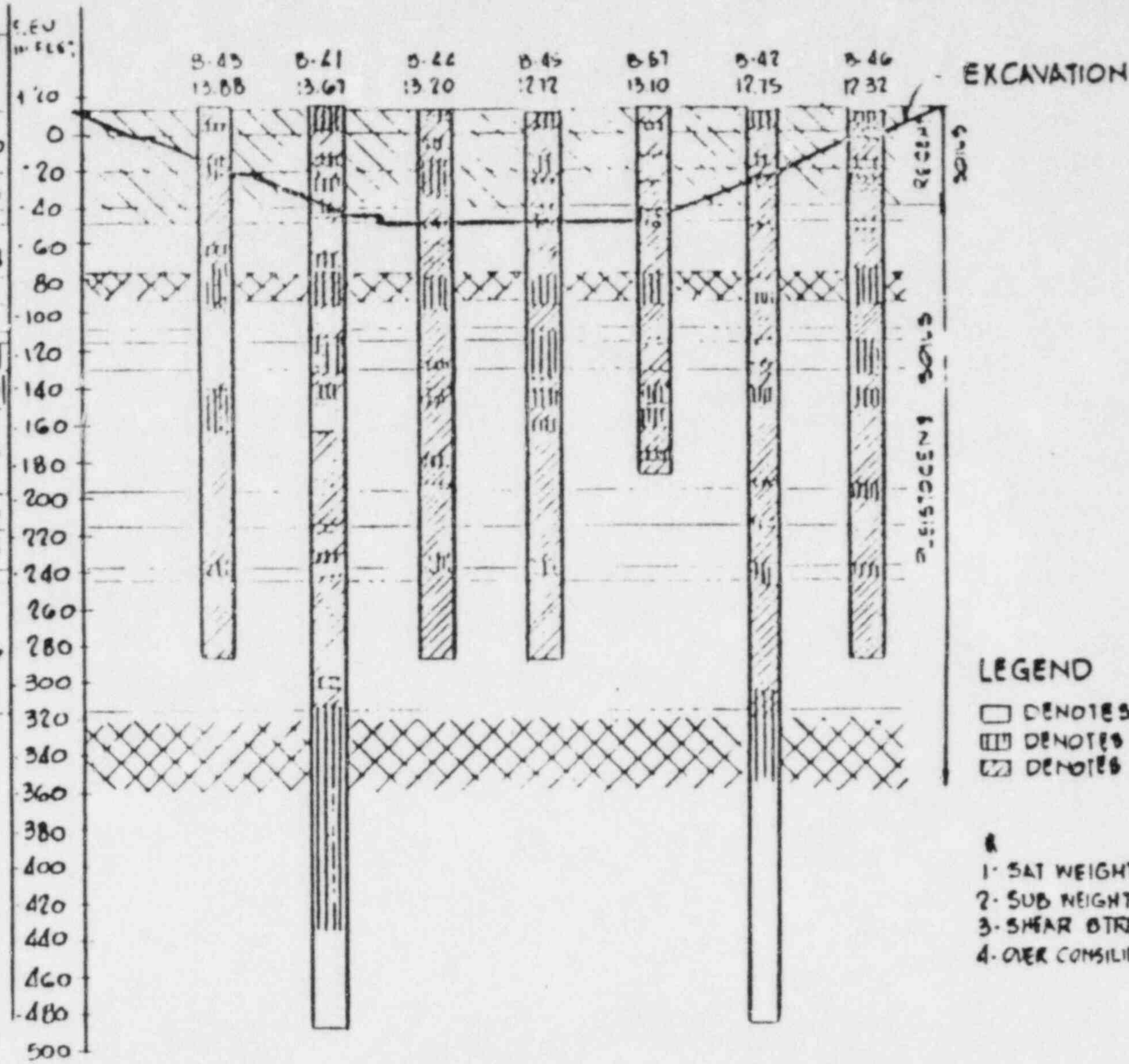
The implementation of the design-construction condition was studied very carefully to eliminate any overstress of the subsoil and to maintain mat stability from differential settlement and tilting. Each construction stage was established to meet the requirements of the net and the allowable differential soil bearing pressures. The critical path of the construction schedule was factored into the design considerations and step by step coordination was made to satisfy both design and construction. The excavation, concrete and backfill sequencing as well as the effects of dewatering and recharging of groundwater, all have been carefully planned as indicated earlier in Figure 3. In addition, the subsurface and structure instrumentation have also been designed to ensure that the subsoils, structure and construction sequencing will perform as planned and designed.

CONCLUSIONS

In conclusion, the design of large structural mats on soil foundations are very much influenced by the relative stiffnesses of mat and its foundation. It was shown that the realistic appraisal of the imposed bearing pressures must consider the loading history of the foundation soils and the compatibility of the foundation settlements as well as the construction sequencing toward completion. The redistribution of structural shears and moments are significant to the design considerations, and a conservative design envelope should be utilized to appreciate the changing conditions during construction and redistribution phases of the foundation soil and structure interaction.

AVERAGE STRATA PROPERTIES

SOIL DESCRIPTION	1	2	3	4
EL 15.00 / YMG EL 8.00				
CLAY WITH SILT AND SAND POCKETS	113	51	C·500	2.0
SILTY CLAY & CLAY WITH SAND LENSES	107	45	C·500	1.5
GRAY SILTY CLAY (FISSURED)	125	63	C·1500	[illegible]
TAN AND GRAY CLAY (FISSURED)	116	54	C·1500	3.6
TAN SILTY SAND	125	63	N·50+	
GRAY CLAY WITH SILT LENSES (PART FISSURED)	12	50	C·1700	.6
DARK GRAY CLAY (ORGANIC)	104	42	C·800	.5
GRAY AND TAN CLAY WITH SAND LENSES	119	57	C·700	2.0
GREENISH-GRAY CLAY AND SILTY CLAY WITH SAND LENSES	119	57	C·2000	1.6
GREENISH GRAY & DARK GRAY CLAY (ORGANIC)	112	50	C·2000	1.5
GRAY CLAY WITH SILTY SAND POCKETS & SHELLS	119	51	C·2000	1.8
GRAY SILTY SAND	125	63	N·50+	
GRAY CLAY WITH SILT AND SHELL FRAGMENTS	119	51	C·2000	1.6
GRAY SILTY SAND	125	63	N·50+	

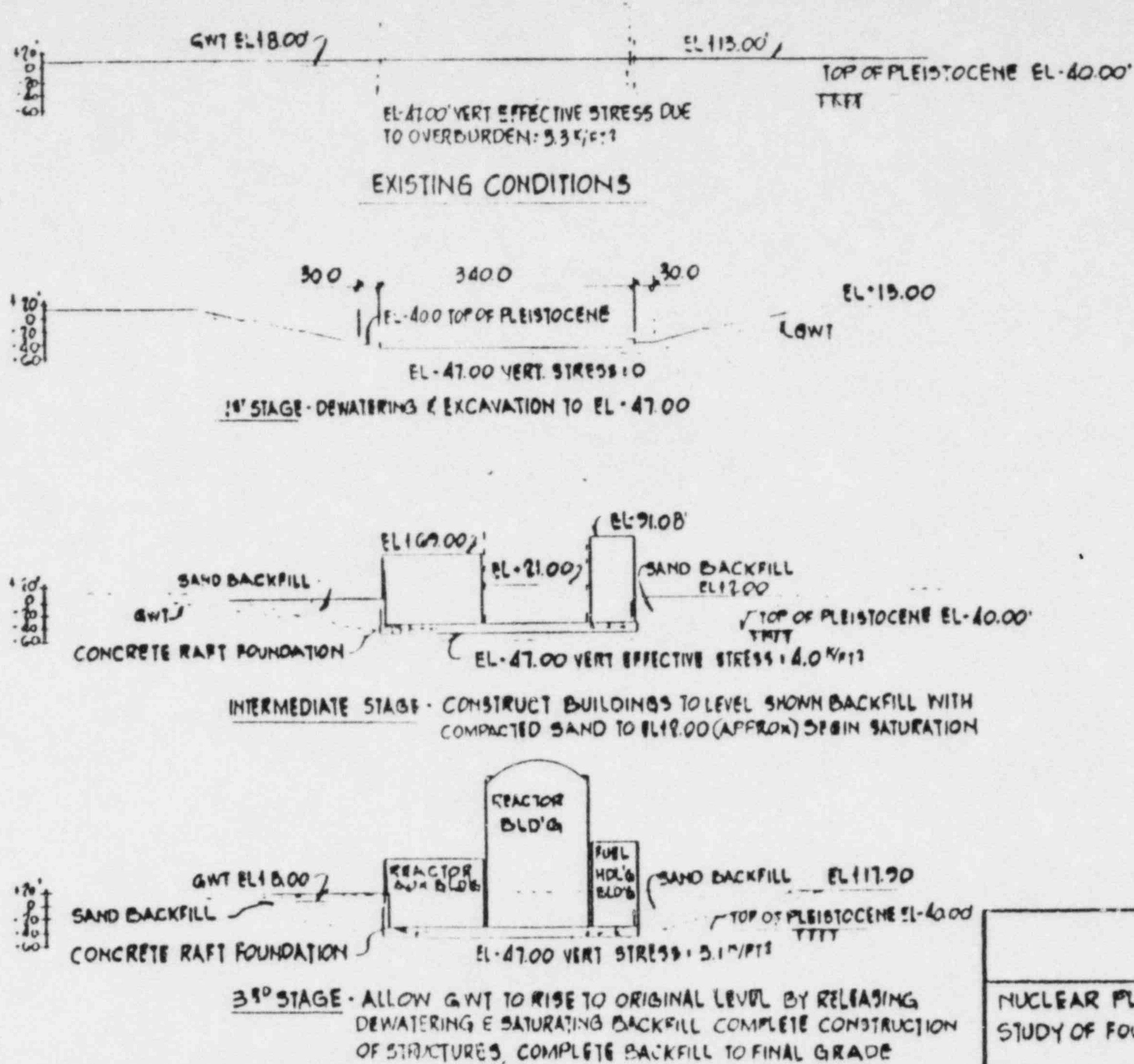


LEGEND
- DENOTES SAND
- DENOTES SILT
- DENOTES CLAY

1- SAT WEIGHT P.C.F
2- SUB WEIGHT P.C.F
3- SHEAR STRENGTH P.S.F.
4- OVER CONSOLIDATION RATIO: Pc/Pn

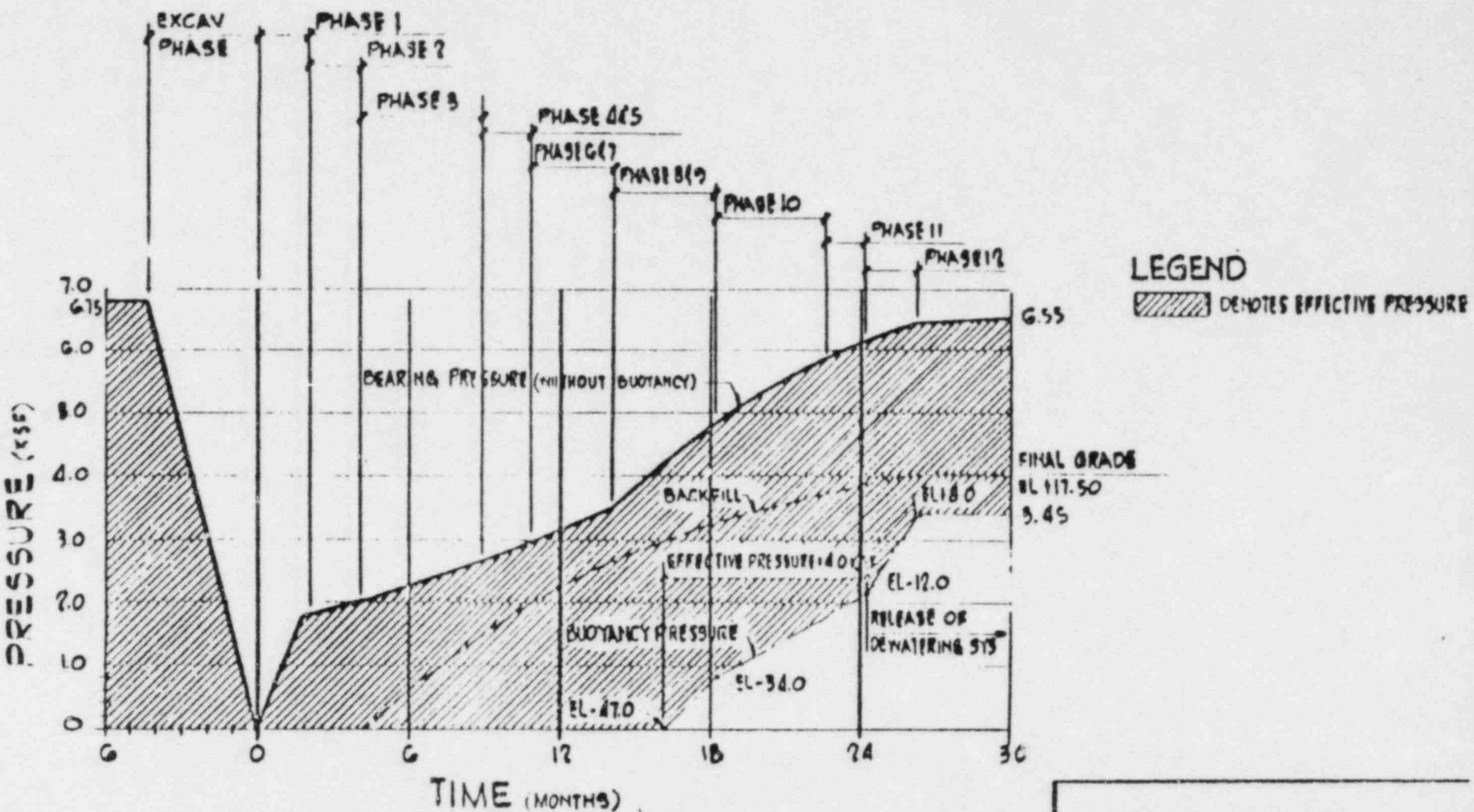
SCALE
VERT. 1"=100'
HOR. N.T.S.

GENERALIZED SOIL PROFILE



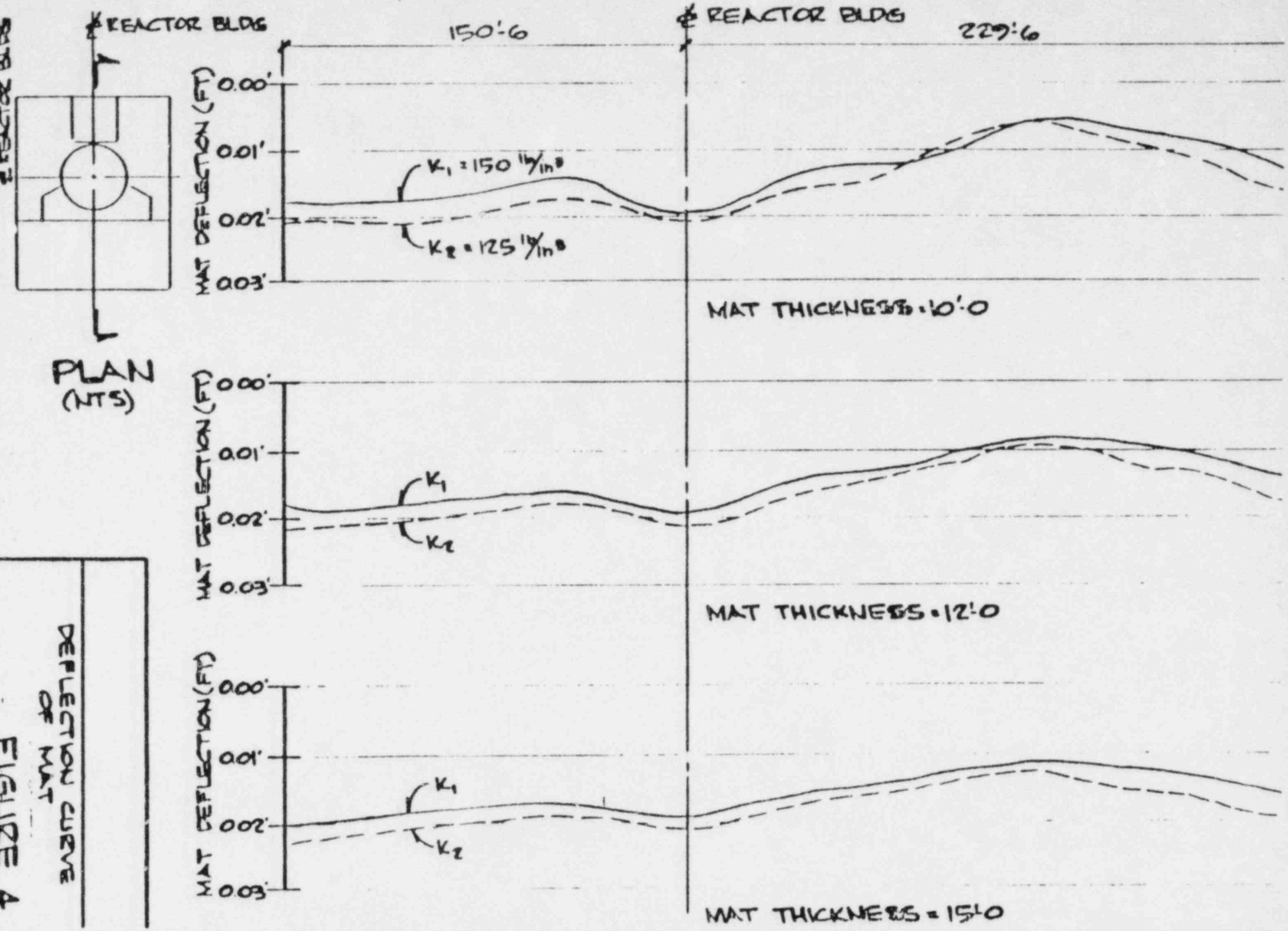
NUCLEAR PLANT ISLAND STRUCT
STUDY OF FOUNDATION PRESSURE

FIGURE 2



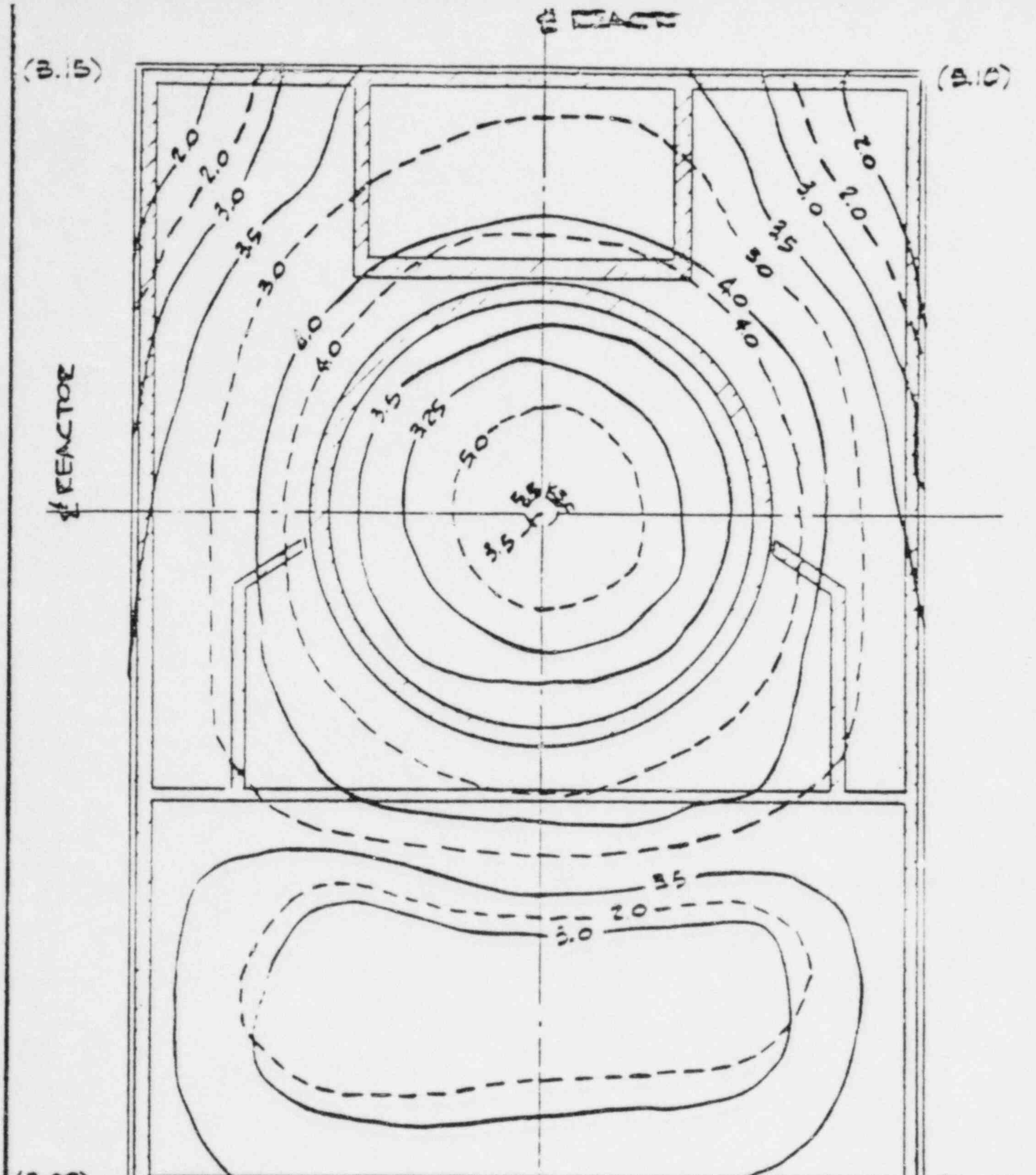
SOIL PRESSURE TIME-HISTORY DIAGRAM

FIGURE 3



DEFLECTION CURVE OF MAT

FIGURE 4

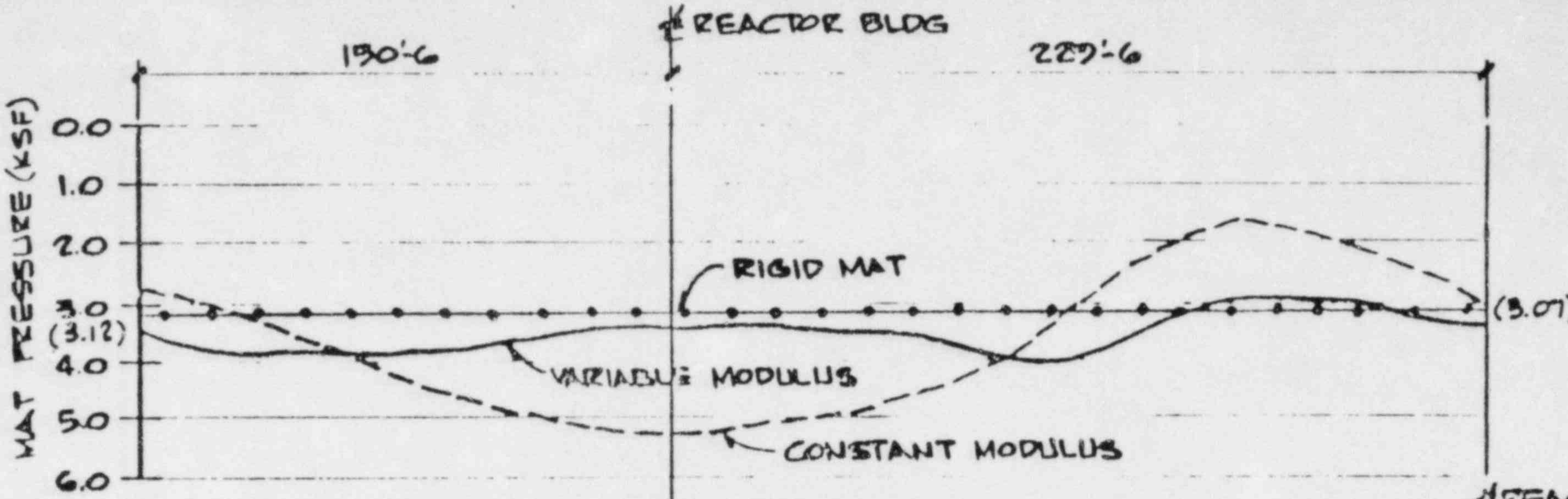


FLEXIBLE MAT ANALYSIS — — — — CONSTANT MODULUS
FLEXIBLE MAT ANALYSIS ——— VARIABLE MODULUS

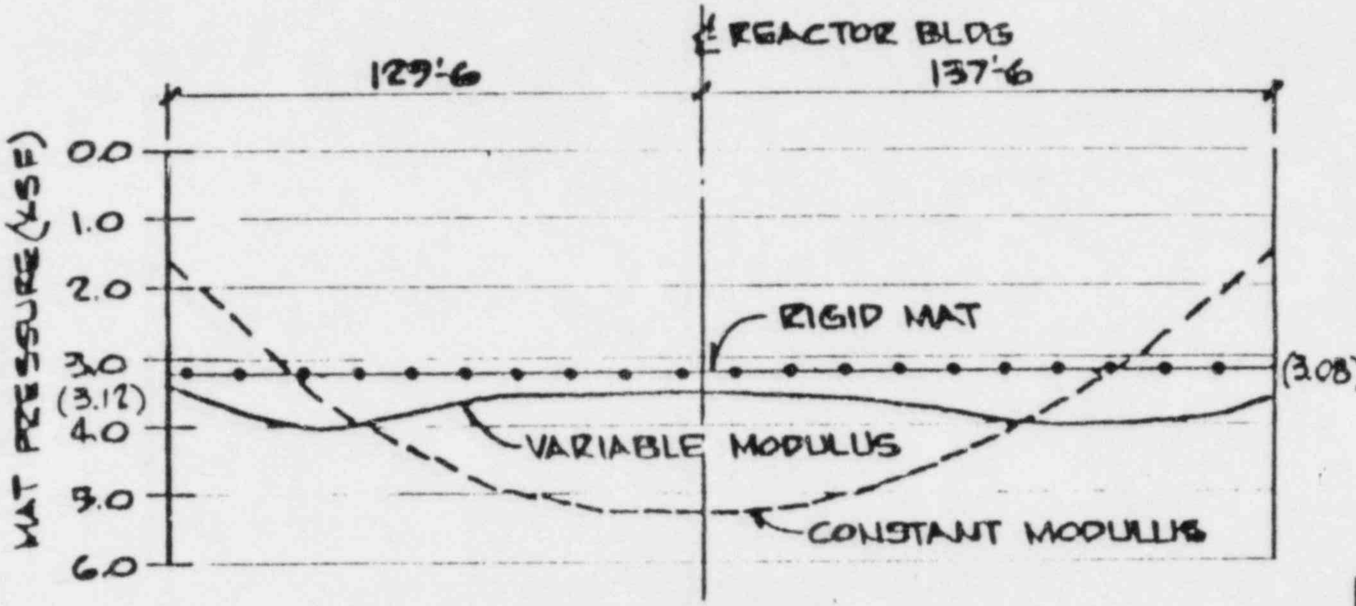
RIGID MAT ANALYSIS (0.00 KSF)

EFFECTS OF SOIL MODULUS ON MAT PRESSURES

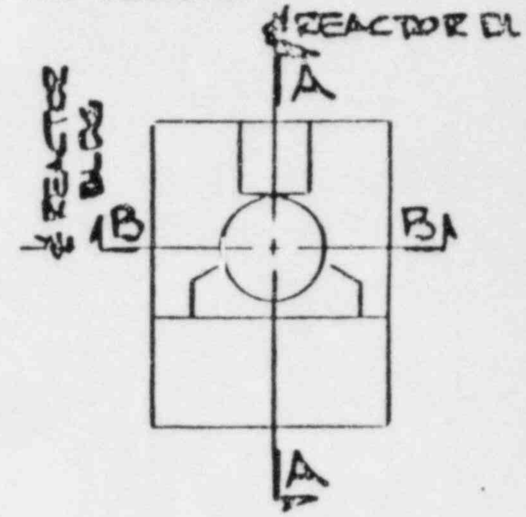
FIGURE 5



SECTION A-A



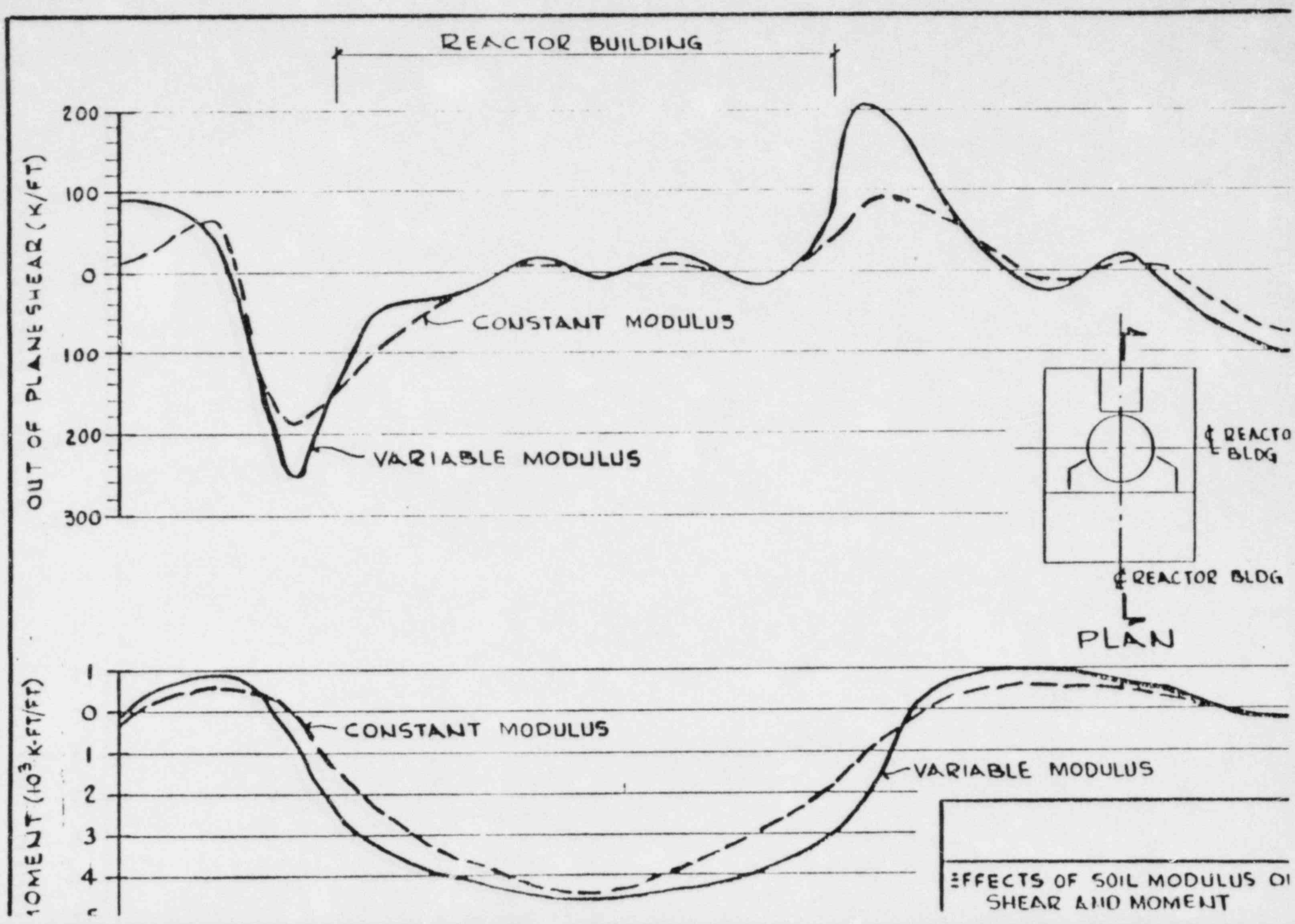
SECTION B-B



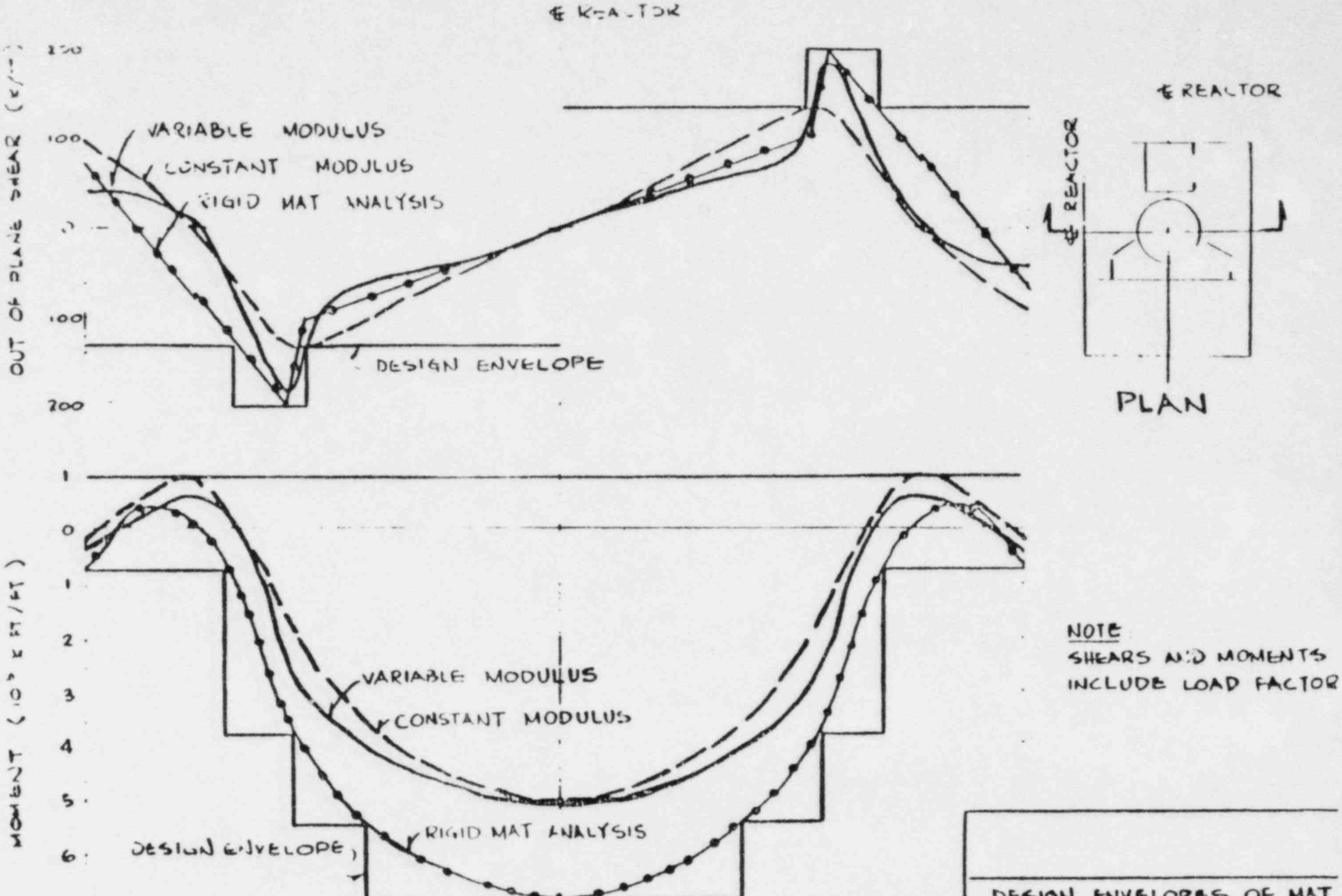
PLAN
(NTS)

EFFECTS OF SOIL MODULI
ON MAT PRESSURES

FIGURE [illegible]



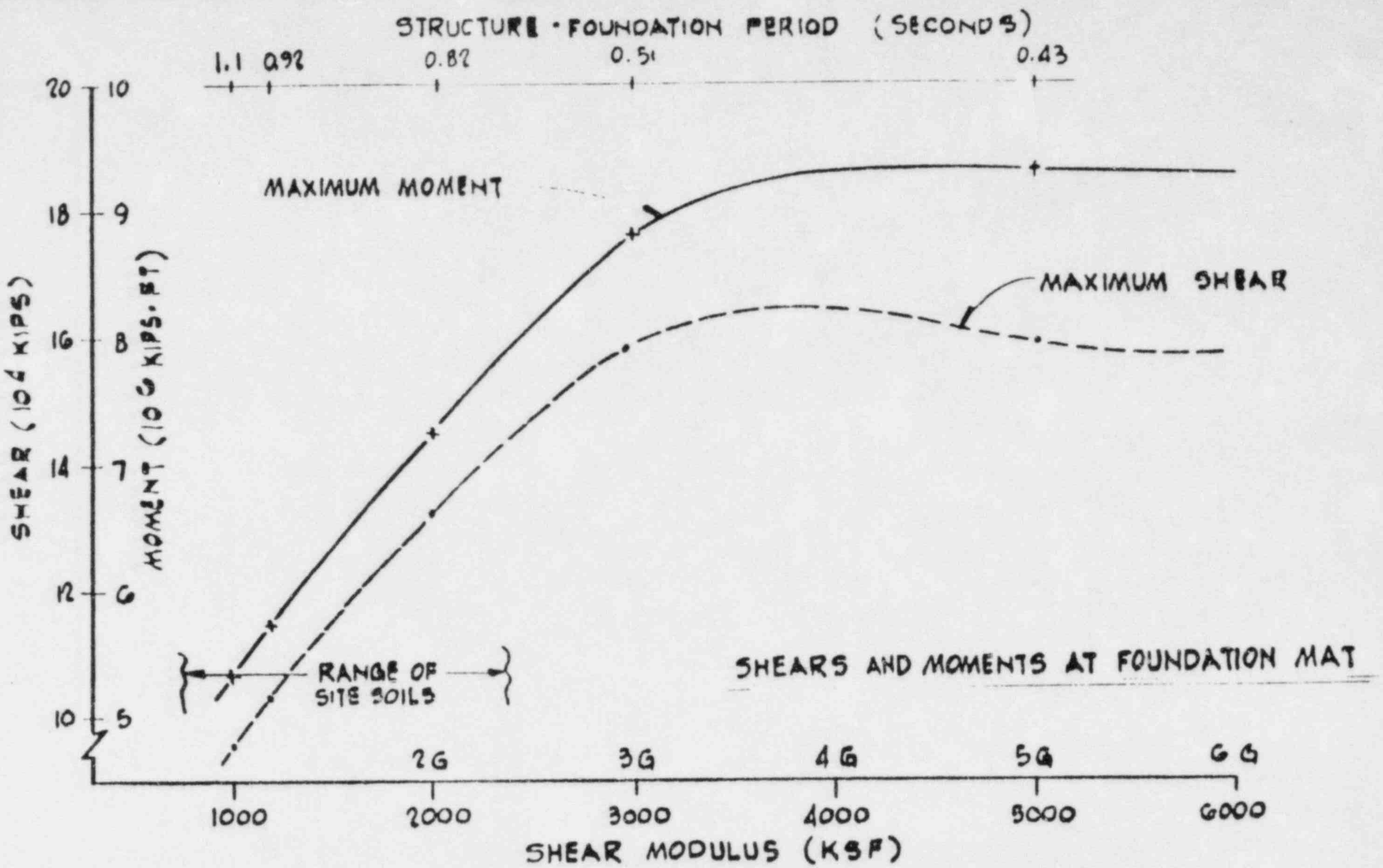
EFFECTS OF SOIL MODULUS OI SHEAR AND MOMENT



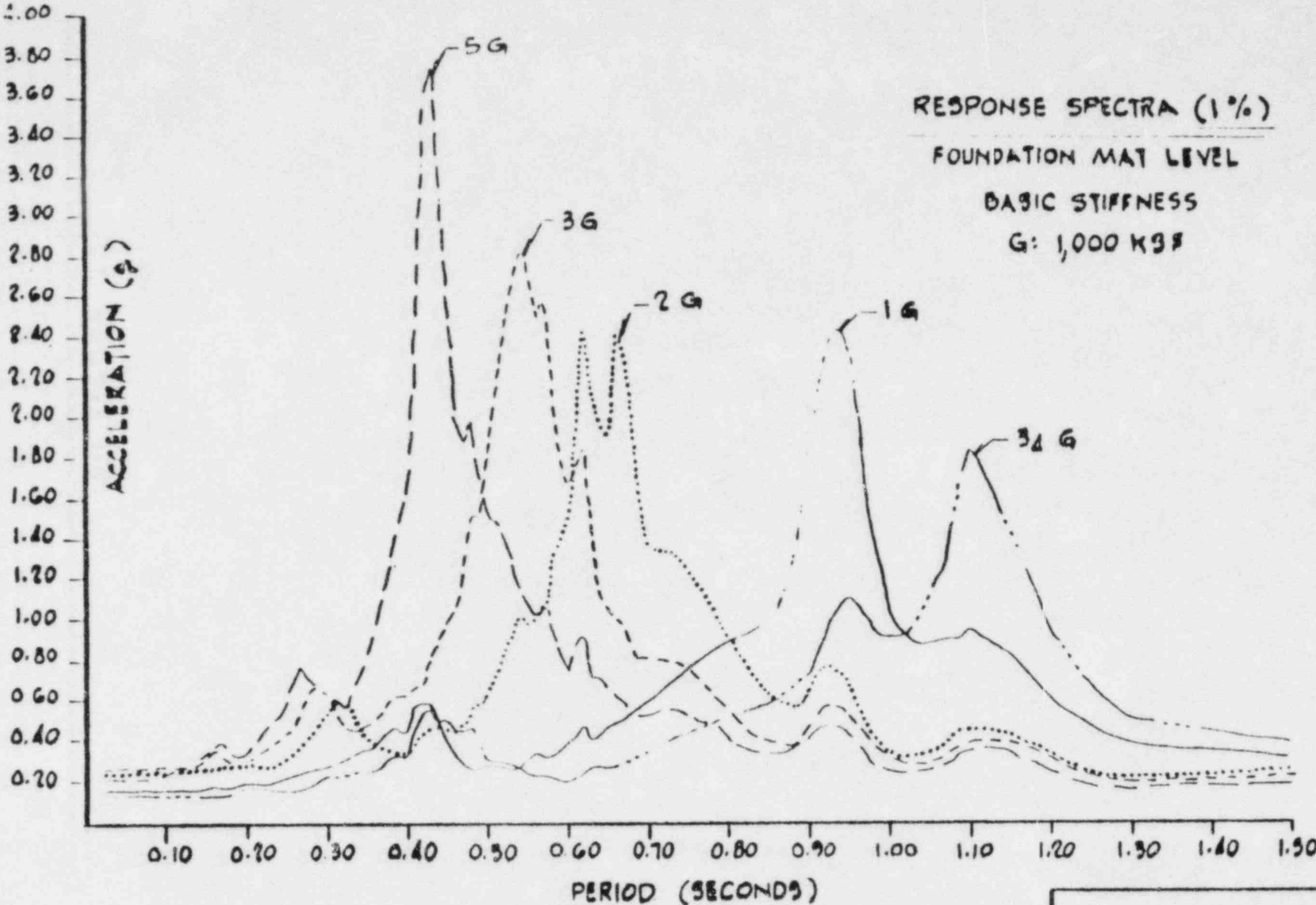
NOTE
SHEARS AND MOMENTS INCLUDE LOAD FACTOR

DESIGN ENVELOPES OF MAT SHEAR AND MOMENT

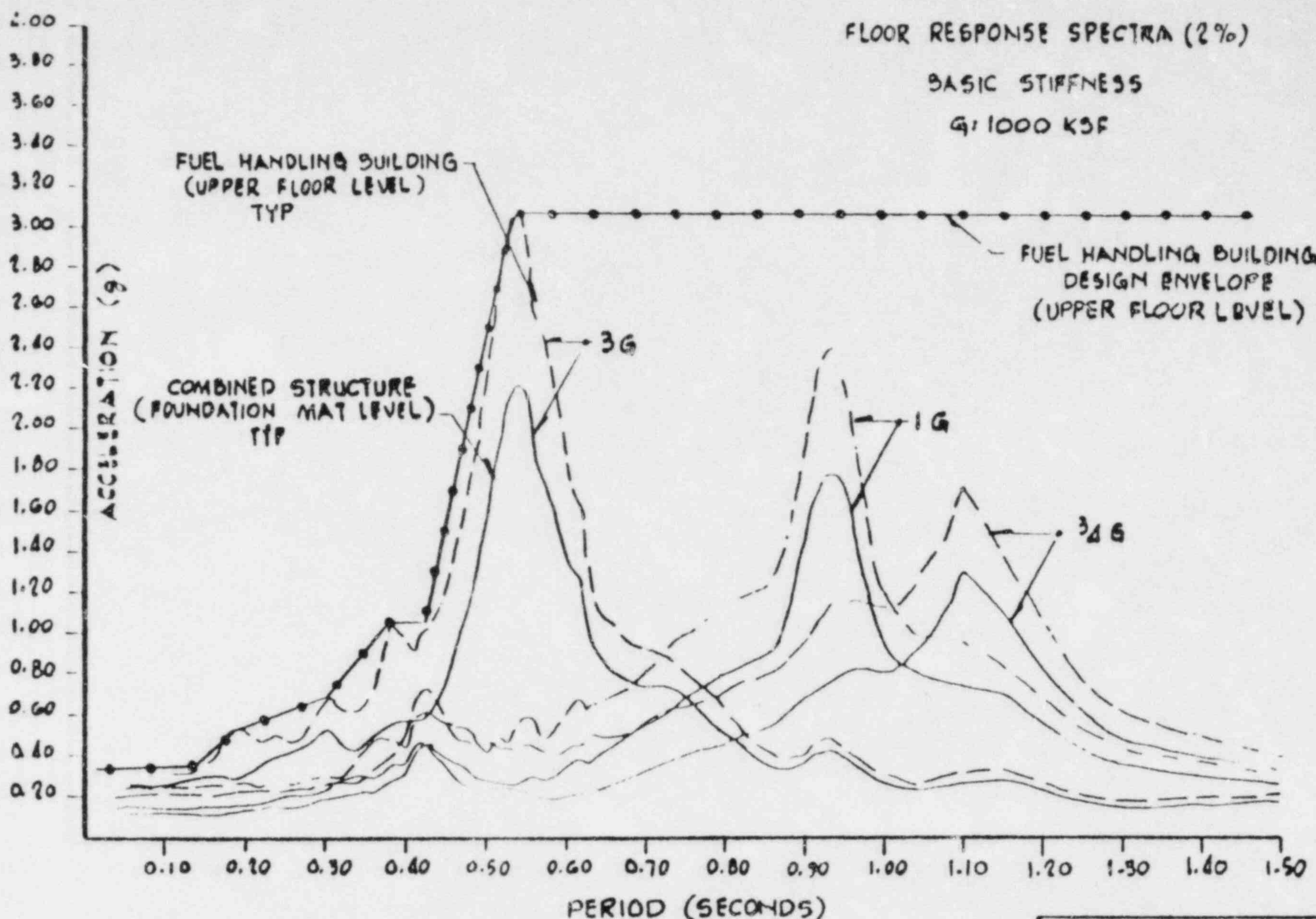
FIGURE 9



EFFECTS OF FOUNDATION STIFFNESS
DYNAMIC SHEARS AND MOMENTS



EFFECTS OF FOUNDATION STIFFNESS ON DYNAMIC RESPON



EFFECTS OF FOUNDATION STIFFNESS ON DYNAMIC RESPONSE AT VARIOUS LEVEL

EBASCO SERVICES
INCORPORATED

INCOMING

3

UTILITY CONSULTANTS - ENGINEERS - CONSTRUCTORS

TWO RECTOR STREET
NEW YORK, N.Y. 10006

CABLE ADDRESS "EBASCOS"

March 15, 1977
LW3-452-77
File 14Q-B-5d

Mr D L Aswell
Manager of Power Production
Louisiana Power & Light Company
142 Delaronde Street
New Orleans, Louisiana 70174

EBASCO SERVICES, INC.
RECEIVED
? 1 1977
? FIELD

Re: WATERFORD SES UNIT NO. 3
TRANSMITTAL OF PSAR CHANGE REQUEST CH-3
FOR REVIEW AND APPROVAL

Dear Mr Aswell:

Enclosed please find PSAR [illegible]
Pressure Prior to Rec[illegible]

The purpose [illegible] [illegible]ble
recompressio[illegible]
soil bearing[illegible]
pounds per sq[illegible]
within the maxi[illegible]
square foot.

MARK - DOCS FOR INCREASE OF RECHARGING PRESSURE FROM 4000 PSF TO 4500 PSF AND SUBSEQUENT FSAR ADDRESS.
BILL C

Evaluation and ap[illegible] [illegible]rformed by
Ebasco in accordan[illegible] [illegible]-3.

Therefore, upon LP&L [illegible]ted in the master copy of
the PSAR until the FS[illegible] [illegible]RC. Ebasco also recommends that
this change be retaine[illegible] [illegible]the convenience of NRC auditors.

Very truly yours,

RK Stampley / [illegible]

R K Stampley
Project Manager

bc: R K Stampley
R L Teal
~~H W Nelson~~
R W McCaffrey
G J Lambrakos
~~L J Sas~~
C Seoane
P V Gvildys
D N Galligan
R A Hartnett
L T Skoblar
J E Moaba
P E Grossman
A Wern
P C Liu
L J Ehasz
G Goodheart

RKS:JJC:mm
Encl.

cc: D L Aswell
L V Maurin
A E Henderson
D B Lester
P V Prasankumar
R Prados
H W Ottillio
C G Chezem
T F Gerrets
F X Shaughnessy
J M Brooks
J O Booth (2)
Power Production Department - Nuclear (3)

B/15

EBASCO SERVICES
INCORPORATED

INCOMING

UTILITY CONSULTANTS - ENGINEERS - CONSTRUCTORS

TWO RECTOR STREET
NEW YORK, N.Y. 10006

CABLE ADDRESS "EBASCOE"

March 15, 1977
LW3-452-77
File 14Q-B-5d

Mr D L Aswell
Manager of Power Production
Louisiana Power & Light Company
142 Delaronde Street
New Orleans, Louisiana 70174

EBASCO SERVICES, INC.
RECEIVED
MAR 21 1977
WATERFORD 3 FIELD

Re: WATERFORD SES UNIT NO. 3
TRANSMITTAL OF PSAR CHANGE REQUEST CH-3
FOR REVIEW AND APPROVAL

Dear Mr Aswell:

Enclosed please find PSAR change request CH-3, "Allowable Soil Bearing Pressure Prior to Recharging", for your review and approval.

The purpose of the subject PSAR change request is to increase the rate of recompression of the foundation soils. Ebasco recommends that the allowable soil bearing pressure prior to recharging be increased from 4,000 to 4,500 pounds per square foot. This additional effective pressure is still well within the maximum allowable soil bearing pressure of 15,000 pounds per square foot.

Evaluation and approval of PSAR change request CH-3 has been performed by Ebasco in accordance with Nuclear Licensing Procedure No. L-3.

Therefore, upon LP&L approval CH-3 will be documented in the master copy of the PSAR until the FSAR is submitted to the NRC. Ebasco also recommends that this change be retained at the site for the convenience of NRC auditors.

Very truly yours,

RK Stampley/pp

R K Stampley
Project Manager

RKS:JJC:mm
Encl.

cc: D L Aswell
L V Maurin
A E Henderson
D B Lester
P V Prasankumar
R Prados
H W Ottillio
C G Chezem
T F Gerrets
F X Shaughnessy
J M Brooks
J O Booth (2)
Power Production Department - Nuclear (3)

bc: R K Stampley
R L Teal
~~H W Nelson~~
R W McCaffrey
G J Lambrakos
~~L J Soe~~
C Seoane
P V Gvildys
D N Galligan
R A Hartnett
L T Skoblar
J E Moaba
P E Grossman
A Wern
P C Liu
L J Ehasz
G Goodheart
M Pavone
C Glia (2)
L-Lo

SAR ER CHANGE REQUEST

CHANGE NO. CN-3

J. Moaba — Lead Licensing Engineer

FROM P. C. Liu — Lead Discipline Engineer

SUBJECT LOUISIANA POWER & LIGHT CO., WATERFORD SES UNIT #3 — Project Title

PSAR/~~FSAR~~ CHANGE RECOMMENDATION
Allowable Soil Bearing Pressure Prior to Recharging

The affected area is:

Page 2.D-4 (Amendment No. 12) Paragraph 3 Line 7

Recommended change and reasons for requesting change:

Change: During the concrete construction phases, the pressures begin to increase and continue until a stress of 4500 lb per sq. ft. has been applied.

Reason: *See page attached.

Notes: Any reference to Figure 2.D-5 concerning the previous allowable stress of 4000 psf will be similarly changed to 4500 psf in the forthcoming FSAR.

Submitted P C Liu — DISCIPLINE ENGINEER — Date 3/11/77

Approved [signature] — DISCIPLINE SUPERVISOR — Date 3/11/77

Reviewed RW McCaffrey — PROJECT ENGINEER — Date 3/11/77

Approved J E Moaba — LICENSING ENGINEER — Date 3/14/77

Disposition

Signature ______ DISCIPLINE ENGINEER (IF REQUIRED) Date ______

Signature ______ LICENSING ENGINEER (IF REQUIRED) Date ______

TO BE RETAINED IN LICENSING DEPARTMENT FILES

Attachment Sh 1/1

Reason:

The recompression of the foundation soils have been progressing at a slower rate than anticipated, primarily due to the long and extended period of partial excavation and final excavation. In order to increase the rate of recompression the allowable bearing pressure prior to recharging should be increased from 4000 to 4500 psf. The response of the foundation soils are being monitored continuously and the time and magnitude of loading prior to recharging will be predicated on the actual recompression being experienced. The objective is to essentially recompress the foundation soils to their preconstruction condition; namely, overload the soils until the heave experienced during the excavation phase has been compensated by induced settlement (recompression).

This additional effective stress is still safety within the maximum allowable soil bearing pressures of 15,000 psf. The factor of safety against any bearing failure under the increased loading is still in excess of 3.

INCOMING

LOUISIANA POWER & LIGHT / 142 DELARONDE STREET P. O. BOX 6008 • NEW ORLEANS, LOUISIANA 70174 • (504) 366-2345
MIDDLE SOUTH UTILITIES SYSTEM

March 23, 1977

LPL 6635
3-A1.04
3-A1.02
Q-3-A28.14

EBASCO SERVICES, INC.
RECEIVED
MAR 25 1977
WATERFORD 3 FIELD

Mr. R. K. Stampley
Ebasco Services, Inc.
Two Rector Street
New York, N. Y. 10006

SUBJECT: Waterford SES Unit No. 3
PSAR - Soil Bearing Pressure Limit

Dear Mr. Stampley:

Attached, for your information, is a copy of a documentation of a telephone conversation.

Yours very truly,

D L Aswell

D. L. Aswell
Manager of Power Production

DLA:AEH:jhl

Attachment

cc: Ebasco (2), J. M. Brooks, J. O. Booth (2)√ D. L. Aswell, L. V. Maurin, A. E. Henderson, D. B. Lester, P. V. Pransankumar, H. W. Otillio, F. X. Shaughnessy, L. Biondolillo, T. F. Gerrets, C. G. Chezem, D. N. Galligan, C. J. Decareaux.

DOCUMENTATION OF
TELEPHONE COMMUNICATIONS

DATE: March 23, 1977 TIME: 2:50 ~~A.M.~~, P.M.

PARTY CALLING: A. E. Henderson ael by RCH (Name) Louisiana Power & Light Company (Company)

PARTY ANSWERING: W.G. Hubacek (Name) NRC Reactor Insp. (Company)

SUBJECT: Waterford SES Unit 3

PSAR - Soil Bearing Pressure Limit

FILE: 3-A1.04

3-A1.02

Q-3-A28.14

SUMMARY: (INCLUDING DECISIONS AND OR COMMENTS)

Reported to the NRC that a potential significant deficiency exists at the construction site. "The soil bearing pressure prior to recharging will exceed the 4,000 psf as stated in the PSAR."

Explained that Ebasco Engineering had requested that the limit be raised to 4,500 psf which still gives a safety factor of 3. Hubacek suggested (could not tell us what to do) that NRC licensing be made aware of this.

ACTION REQUIRED:

Keep Mr. Hubacek informed.

DISTRIBUTION:

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March 24, 1977

LPL 6640
Q-3-A35.02.01
Response Req'd: Yes
By: April 5, 1977

Mr. R. K. Stampley
Project Manager
Ebasco Services, Inc.
Two Rector Street
New York, N. Y. 10006

EBASCO SERVICES, INC.
RECEIVED
MAR 28 1977
WATERFORD 3 FIELD

SUBJECT: Waterford SES Unit No. 3
NRC Audit - March 2 - 4, 1977

Dear Mr. Stampley:

Attached is a copy of a letter dated March 21, 1977, from the NRC Office of Inspection and Enforcement - Region IV together with a copy of the NRC Inspectors Report concerning the audit conducted on March 2 - 4, 1977.

Please refer to the paragraph in the letter relative to proprietary information. According to the letter, LP&L is to notify the NRC within twenty (20) days if any information contained in the report is considered to be proprietary.

If any information in this report is considered proprietary, your written response must be handled in an expeditious manner. Our response to the NRC must be made before Friday, April 8, 1977. If you do not contact us by April 5, 1977, we will assume that you consider none of the information contained in the report to be proprietary.

By copy of this letter to Mr. W. Mawhinney, we are asking CE to respond to this request in like manner.

Yours very truly,

D L Aswell

D. L. Aswell
Manager of Power Production

DLA/OPP/jhl
Attachment
cc: Ebasco (2), J. M. Brooks, J. O. Booth (2), D. L. Aswell, L. V. Maurin, A. E. Henderson, D. B. Lester, P. V. Pransankumar, H. W. Otillio, F. X. Shaughnessy, L. Biondolillo, T. F. Gerrets, C. G. Chezem, D. N. Galligan, C. J. Decareaux, W. Mawhinney, O. P. Pipkins

3. Common Foundation Mat Loading and Subsurface Recharge

The common foundation mat is founded on Pleistocene clays at an elevation 47 feet below mean sea level (MSL). The PSAR, Appendix 2.D and Ebasco Design Specification LOU 1564.461 505, Section VII, "Foundation Properties," specify maximum allowable net soil bearing pressure is 4.0 kips per square foot (ksf). The maximum allowable pressure differential across the mat is 1 ksf. (For periods of less than 2 months, maximum differential loading is 2 ksf.)

Review of the Ebasco computer print-out, "Accumulative Summary of Placement Stress," indicated that the current soil bearing stresses of the mat, as of February 16, 1977, (week #70) were 3.921 ksf maximum (Northwest corner) and 2.895 ksf minimum. The predicted bearing stresses for weeks #72 and #74 were 3.947 ksf maximum, 2.958 ksf minimum, and 4.001 ksf maximum with 3.114 ksf minimum, respectively.

Redesign of the non-safety related turbine building foundation requires the placement of structural backfill (Class B) from the Pleistocene layer to an elevation 14.5 feet above MSL, in lieu of pilings. The excavation and backfill activities in the area of the turbine building may delay the schedule for recharge of ground water to effectively maintain the net maximum foundation mat bearing pressure at or below 4.0 ksf. Ebasco representatives indicate that consideration is being given to increasing the maximum allowable net soil pressure from 4.0 to 4.3 or 4.5 ksf.

No discrepancies were noted during this portion of the inspection.

4. Structural Backfill - Class A

The backfill around the common foundation mat and safety related structures is divided into seven (7) fill areas (#1 through #7). Records dated from October 4, 1976, to January 25, 1977, for inspection and testing of backfill were reviewed for the following areas:

Fill Areas	No. Days Reviewed
1	2
3	3
5	3
6	5
7	3

The following records were reviewed for each of the days listed above:

J. A. Jones	Daily Backfill Inspection Report
Ebasco	Borrow Material Inspection Report
Ebasco	Excavation and Stripping Inspection Report
Ebasco	Daily Backfill Inspection Report
Ebasco	Backfill Acceptance Report

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MIDDLE SOUTH UTILITIES SYSTEM

March 25, 1977

LPL 6644
Q-3-A28.14

EBASCO SERVICES, INC.
RECEIVED
MAR 28 1977
WATERFORD 3 FIELD

Mr. R. K. Stampley
Ebasco Services, Inc.
Two Rector Street
New York, New York 10006

SUBJECT: Waterford SES Unit No. 3
Soil Bearing Pressure

Dear Mr. Stampley:

Attached is a copy of a Documentation of Telephone Communication for your information.

Yours very truly,

D L Aswell

D. L. Aswell
Manager of Power Production

DLA:LVM:gmw

Attachment

cc: Ebasco (2), J. M. Brooks, J. O. Booth (2), D. L. Aswell, L. V. Maurin, A. E. Henderson, D. B. Lester, C. G. Chezem, F. X. Shaughnessy, H. W. Otillio, P.V. Prasankumar, T. F. Gerrets, L. Biondolillo, D. N. Galligan, C. J. Decareau, F. J. Drummond

DOCUMENTATION OF
TELEPHONE COMMUNICATIONS

DATE: March 23, 1977 TIME: 3:45 ~~XXXX~~ P.M.

PARTY CALLING: L. V. Maurin (Name) LP&L (Company)

PARTY ANSWERING: Robert Benedict (Name) NRC (Company)

SUBJECT: Soil Bearing Pressure FILE: 3-A1.04

3-A1.02

Q-3-A28.14

SUMMARY: (INCLUDING DECISIONS AND OR COMMENTS)

I called Mr. Benedict to inform him that the Soil Bearing Pressure, specified not to exceed 4000 lbs. per square foot in the PSAR, would actually exceed 4000 psf but not 4500 psf. I informed Mr. Benedict that Region IV Inspection and Enforcement had been notified of this fact and it had classified this situation as being a "Potential Significant Incident". If it develops that this incident is not significant, Region IV I&E will be so notified by phone. Should it develop that this incident is significant then Region IV I&E will be given a written justification within thirty days.

I pointed out to Mr. Benedict that the increased effective stress is still safely within the maximum allowable soil bearing pressure of 15000 psf, and that the factor of safety against any bearing failure under the increased loading is still in excess of 3.

Mr. Benedict expressed satisfaction with this report and felt that, since Region IV I&E was aware of the situation, everything was in order.

ACTION REQUIRED:

DISTRIBUTION:

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MIDDLE SOUTH UTILITIES SYSTEM

March 25, 1977

LPL 6639
Q-3-A28.14
Response Req'd: Yes
By: April 11, 1977

Mr. R. K. Stampley
Ebasco Services, Inc.
Two Rector Street
New York, New York 10006

EBASCO SERVICES, INC.
RECEIVED
MAR 28 1977
-WATERFORD 3-FIELD

SUBJECT: Waterford SES Unit No. 3
Allowable Soil Bearing
Pressure Limit

REFERENCE: (1) Letter LW3-452-77 dated March 1-
(2) Letter LPL 6635 dated Ma-
(3) Letter LPL 6640 dat-

Dear Mr. Stampley:

THIS LETTER WAS RELEASED TO RIDENHOUR BY LP&L. BILL C

We have review- ... ation that the soil ... from 4,000 to 4, ... of safety agains ... in excess of 3,b ... 0 pounds per squ ... nd included i

Reference ... tween LP&L and t ... reported as a Potential ... essure prior to recharging ... ot as stated in the Waterford 3 ... This information was also communicated ... sing Branch by LP&L. In this regard we ask Eb ... detailing the reason why the Soil Bearing Pressu ... pounds per square foot will be exceeded and justifying ... nded change in the Soil Bearing Pressure Limit to 4,500 pounds ... square foot. This report should be provided in a suitable format for submission to the NRC.

We request that you advise LP&L of Ebasco's recommendations for handling this potential deficiency. Should it be treated as a reportable deficiency or should the NRC be provided a written report for information only.

IE Inspection Report No. 50-382/77-03 which was forwarded to you by reference 3 addresses the Turbine Building Foundation Design Change as an item of concern. We recommend that Ebasco consult this reference prior to responding to the above requests.

Please note that LP&L must respond to the Potentially Reportable Deficiency with thirty (30) days.

Yours very truly,

D L Aswell

D. L. Aswell
Manager of Power Production

DLA/FJD/dd

cc: Ebasco (2), J. M. Brooks, J. O. Booth (2), D. L. Aswell, L. V. Maurin, A. E. Henderson, D. B. Lester, P. V. Prasankumar, H. W. Otillio, F. X. Shaughnessy, L. Biondolillo, C. G. Chezem, T. F. Gerrets, D. N. Galligan, C. J. Decareaux, F. J. Drummond

SUMMARY OF FINDINGS

I. Enforcement Action

A. Items of Noncompliance

None

B. Deviations

None

II. Licensee Action on Previously Identified Enforcement Matters

A. Items of Noncompliance

1. Violations

None

2. Infractions

76-11/I.A.2 Certification of QC Inspector

This item remains open pending review of the licensee's corrective action. (Details I, paragraph 4.)

3. Deficiencies

None

B. Deviations

None

III. New Unresolved Items

77-04/III Potential Significant Construction Deficiency Related to Soil Bearing Pressures

On March 23, 1977, the licensee reported to RIV a potential significant construction deficiency related to the possibility of exceeding the maximum soil bearing pressures under the common foundation mat allowed by the PSAR and specifications. The licensee is currently evaluating this matter. (Details I, paragraph 6.)

IV. Status of Previously Reported Unresolved Items

None

5. Status of 50.55(e) Incidents

Initial pressure grouting of foundation mat section 499S03-19 has been completed. Additional cores will be taken to explore areas of excess grout "take" on the north boundary of the affected area. The licensee anticipates that repairs will be completed on schedule and the final report will be submitted by July 22, 1977.

Repairs to wall G-570S03-51B are essentially complete. The licensee is preparing the final report of this incident and plans to submit it to NRC by April 22, 1977.

6. Potential Significant Construction Deficiency Related to Soil Bearing Pressures

On March 23, 1977, the licensee informed RIV of a potential significant construction deficiency related to the possibility of exceeding the maximum allowed soil bearing pressure under the common foundation mat. The PSAR and Ebasco Specification LOU 1564.401 S05 both state that the maximum allowed soil bearing pressure under the mat is 4000 pounds per square foot (p.s.f.). At the time of the inspection, the maximum soil pressures had not yet exceeded the allowed 4000 p.s.f. Further, a licensee representative informed the inspector that NRR had been contacted with regard to changing the maximum allowed soil bearing pressure from 4000 p.s.f. to 4500 p.s.f. as recommended by Ebasco. Documentation supporting the recommended change from 4000 p.s.f. to 4500 p.s.f. was available for review by the inspector. (See Details III, paragraph 3.)

The inspector informed the licensee that this matter will be considered unresolved pending the licensee's evaluation of its significance in accordance with the requirements of 10 CFR 50.55(e).

DETAILS III

Accompanying Inspector: ______________________
J. I. Tapia, Reactor Inspector Intern
Engineering Support Section

Reviewed by: ______________________
R. E. Hall, Chief, Engineering Support Section

1. Persons Contacted

   a. Louisiana Power and Light Company (LP&L)

      O. P. Pipkins, QA Engineer

   b. Ebasco Services Incorporated (Ebasco)

      G. F. Goodhart, Site Soils Engineer

2. Scope of Inspection

   The scope of this inspection was limited to a review of the licensee approved increase in soil bearing pressure and to the review of quality assurance records relative to Category I Structural Backfill. This inspection was performed under the supervision of the principal inspector.

3. Soil Bearing Pressure Limit Increase

   Ebasco PSAR change request CH-3, "Allowable Soil Bearing Pressure Prior to Recharging," was reviewed by the inspector. This report justifies an increase in allowable bearing pressure on the basis that an increase to 4000 pounds per square foot would actually be favorable in recompressing the foundation clay to its preconstruction condition. LP&L letter of concurrence number LPL 6639, dated March 25, 1977, documents licensee approval of the change request and requires a report detailing the recommended change.

   No discrepancies were noted during this portion of the inspection.

(Closed) Unresolved Item (382/77-04): Potential significant construction deficiency related to soil bearing pressures. The inspector was informed that this matter has been determined by the licensee to be not reportable in accordance with requirements of 10 CFR 50.55(e). Design change No. CH-3 has been approved which documents changing the PSAR limit for soil bearing pressure under the common foundation mat from 4000 pounds per square foot (PSF) to 4500 PSF.

3. Site Tour

The inspectors walked through various areas of the site to observe construction activities in progress and to inspect housekeeping, equipment protection and adherence to fire protection requirements.

The inspectors noted that protection for installed mechanical equipment in the auxiliary and reactor buildings, while adequate, appeared to be deteriorating and maintenance appeared necessary to prevent further deterioration.

While the inspector was observing concrete batch plant operations for concrete production for placements 556S01-11 and 593S02-10, an equipment malfunction occurred that necessitated switching concrete production from the main batch plant to the backup plant. It was observed that the backup plant could not be put into operation in a timely manner because its associated ground hopper contained untested aggregate which remained from production for a previous placement. The necessity for emptying the hopper of aggregate prior to recharge with acceptable material and problems encountered with an admix dispenser contributed to delays in resuming concrete production This delay caused the above placements to be terminated short of completion. The inspector noted that QA Corporation procedure 1.36.1, Section 6.2.2 requires that the ground hopper of the backup plant is to be empty except while in use.

This finding represents noncompliance with the requirements for adherence to procedures in 10 CFR 50, Appendix B, Criterion V and QA Corporation procedure 1.36.1.

4. Significant Construction Deficiencies Reported by the Licensee

The inspector reviewed licensee action related to items which were previously reported as significant or potentially significant construction deficiencies in accordance with the requirements of 10 CFR 50.55(e).

a. Foundation Mat Placement 499S03-19

Twelve verification cores have been drilled in the mat 19 placement after grouting. Two of the 12 cores had indications

The inspector selectively witnessed the stress relief activities for conformance with the CB&I Procedure and ASME B&PV Code, Sections III and VIII, 1971 edition including Code Case 1493 requirements. The inspector and Licensee's QA Technician prepared a time-temperature plot of the vessel stress relief cycle to assess conformance with the following requirements as specified in the ASME B&PV Code and the CB&I Procedure:

(1) Heating rate above 600°F - 100°F/hr

(2) Maximum gradient in 15' on vessel, heating and cooling - 250°F

(3) Holding period - 1150°F (+75 - 50°F) - 2½ hours maximum temperature gradient - 125°F

(4) Maximum allowed temperature - 1225°F

(5) Cooling rate above 600°F - less than 125°/hr

A segment of the plot, Figure 1, is included showing the data recorded for heating, hold and cooling of the vessel. The inspector observed CB&I personnel monitoring instrumentation, burner operation, support equipment, thermal expansion and insulation integrity on a regular basis. A maximum temperature of 140°F was measured at the inner surface of the concrete shield wall.

No items of noncompliance or deviations were identified.

12. Foundation Interaction

The inspector reviewed the results of twenty-three In-Place Density Tests, five Particle Size Analyses, and thirteen Daily Backfill Inspection Reports for the randomly selected dates of April 11 and 12, 1977. All records reviewed were representative of the area beneath the Turbine Generator Building and were found to be in accordance with Ebasco Specification LOU 1564.482, "Filter and Backfill," Rev. 3 and Ebasco Quality Control Instruction QCIP-2, "Soils Control," Issue G.

The inspector reviewed the Ebasco computer print-out entitled, "Accumulative Summary of Placement Stress," which indicated that the common mat bearing stress as of June 22, 1977, was 4,117 pounds per square foot. The allowable soil bearing pressure prior to recharging, which is now 4,500 pounds per square foot, was increased by 500 pounds per square foot in accordance with the recommendations in the Ebasco report which was reviewed by the inspector entitled, "Allowable Mat Bearing Pressure," April 1977.

The inspector discussed the redesign of the Turbine Generator Building foundation from piles to spread footings with the cognizant Ebasco design engineer. He stated that the redesign does not affect the translational spring constant representing the compressibility of the soil on the south wall of the Reactor Auxiliary Building. The projected final design bearing pressures are 6,000 pounds per square foot for the Turbine Generator Building and 3,200 pounds per square foot for the common foundation ma. of the Containment and Auxiliary Buildings. The design engineer informed the inspector that, due to a fifty-one foot difference in elevation and an eighty-five foot lateral separation of the foundations, there would be no amalgamation of the respective Boussinesq stress distributions.

No items of noncompliance or deviations were identified.

13. Unresolved Items

Unresolved items are matters about which more information is required in order to ascertain whether they are acceptable items, items of noncompliance or deviations. The following two unresolved items were disclosed during this inspection regarding the piping erection contractor's QA program and control of personnel access to warehouses:

Identifier	Title	Reference
77-06-1	QA Program Inadequacies	Paragraph 4
77-06-2	Personnel Access Control	Paragraph 10

14. Exit Interview

The inspectors met with licensee representatives (denoted in paragraph 1) at the conclusion of the inspection on June 10, 17 and 23, 1977. The inspectors summarized the purpose and the scope of the inspection findings. A licensee representative acknowledged the statements of the inspectors concerning the unresolved items (paragraphs 4 and 10).

a) Static Earth Pressure

The combined structure was designed for at-rest pressure and hydrostatic loading. The at-rest earth pressure coefficient (K_o) of 0.5 and a buoyant unit weight of 65.5 pounds per cubic ft. (pcf) were used for the backfill material. Two hydrostatic loading conditions were used. The water level was taken at +8 ft. MSL for normal conditions and +30 ft. MSL for flood conditions. The pressure distribution used to design the below grade structure walls is shown on Figure 2.5-100. Refer to Subsection 2.5.4.6 for a discussion of the groundwater conditions at the site. For a complete description of earth pressure load combinations used in conjunction with other foundation loads refer to Section 3.8.

b) Dynamic Earth Pressure

A dynamic lateral earth pressure analysis was performed for all seismic Category I structures using the following criteria:

1) Effective displacement of structural wall relative to the soil was the arithmetic sum of the movement of the wall obtained from the dynamic analysis and the maximum relative soil displacement in the free field as determined by the SHAKE computer analysis.

2) The strain was computed from the wall movement at a particular depth divided by the horizontal component of length of the Rankine failure surface at that depth.

3) The lateral pressures were obtained by a relationship between coefficient of earth pressure vs. strain, as determined from laboratory tests (Figure 2.5-87) discussed in Subsection 2.5.4.5.3.

The dynamic earth pressure distribution used for design of the below grade structure walls is presented in Figure 2.5-101. Hydrostatic pressure under SSE loading was taken as +5 ft. MSL, i.e. low water level condition.

2.5.4.11 Design Criteria

The existence of the slightly overconsolidated Pleistocene clays at elevation -92 ft. MSL, indicated that significant long term and differential settlements could be expected for heavily loaded structures founded on individual spread footings. To eliminate differential and long term settlement considerations the heavy loads were compensated by a combined foundation structure with the Reactor Building, Reactor Auxiliary Building, and Fuel Handling Building (seismic Category I structures) located on a common mat foundation. The floating foundation principle was utilized and the combined foundation will apply an effective load to the bearing stratum clays which is approximately equal to the existing overburden pressure.

All seismic Category I structures are founded in the Pleistocene formation on a common mat with a bottom elevation of -47 ft. MSL. At this level the mat bears in the upper stiff, tan and gray clays of the Pleistocene formation. The objective of the common mat foundation is illustrated in Figure 2.5-102. This figure illustrates the various soil conditions and pressures during four stages of construction shown, beginning with initial soil conditions and finishing with the completed structures and backfill in place.

The soil conditions at the site were evaluated in terms of vertical effective stresses at the mat bearing level (-47 ft. MSL). These stresses initially were 3300 psf prior to construction. The first construction stage illustrates the pressure upon completion of excavation t the bottom of mat elevations thereby reducing the stress to zero. Next, an intermediate stage of construction is illustrated in which the effective stress at the bottom of the mat approaches 4500 psf. This is due to the weight of the concrete structures with the water table lowered below the mat. The final stage illustrated is with the buildings completed, the sand backfill completed, and the groundwater table back to its initial condition of +8 ft. MSL. The mat level bearing pressures for the completed stage will be 3100 psf. This is 200 psf less than the initial soil pressures at the site. For this reason, settlements will not be a concern with this type of foundation.

Since this foundation concept involves the balancing of existing soil pressures, a time history diagram of soil pressure was developed and is illustrated on Figure 2.5-103. This figure details the soil pressures at the bottom of the foundation mat. It begins with the initial soil pressure conditions and develops the pressures during the progressing phases of construction. After excavation the pressures were reduced to zero. This is analogous to the phase described earlier in Figure 2.5-102. During the concrete construction stages, the pressures increased and continued until a pressure of nearly 4500 psf was applied. This pressure was predetermined to be a maximum pressure that is desirable with this type of foundation concept. This is based on the reconsolidation characteristics of the soils and was deemed to be a prudent value to maintain during the construction phase. In order to keep the soil pressure at this level or below, the water table will be allowed to rise thus compensating for further pressure increases, as shown on Figure 2.5-103. This procedure reduces the effective soil pressure and maintains the effective pressures below the 4500 psf level and establishes final effective pressures as described above. Detailed construction stages are given on Figure 2.5-104 thru 2.5-111. Each diagram corresponds to the phase outlined on the aforementioned bearing pressure time history diagram. These figures illustrate the various construction features involved during each phase of the work including the sand backfilling, saturation of backfill, and other construction aspects.

In particular, the detailed foundation design considers the following principles, rationale and distinct features:

a) The base of the combined mat foundation is located at elevation -47 ft. MSL resulting in a final effective soil loading condition of 3100 psf as compared to the initial effective overburden pressure of 3300 psf.

b) Design criteria have established a 1200 psf overload above the existing effective soil pressures which may be applied only during the construction phase of the work. This is primarily to maintain a margin of pressure below the preconsolidation pressure of the materials with the lower OCR's.

c) The excavation of the Recent deposits, consisting of soft clays, silts and sands extending to approximate elevation -40 ft. MSL and subsequent excavation of the stiff Pleistocene clays results in an elastic rebound and heave of the final exposed clay bearing strata. Refer to Subsection 2.5.4.13 for a discussion of measured foundation heave and settlement. Heave is minimized by excavating in increments and by rapid concrete placement in designated sections of the mat in a predetermined sequence to optimize recompression.

d) By conforming with the floating foundation principle, construction settlement of the seismic Category I structures is confined essentially to the recompression of the rebound and heave experienced by the Pleistocene materials with an additional preconsolidation for the higher backfill imposed loading. It is desirable to complete the major portion of this settlement during the construction period therefore the applied loading sequence is arranged with this particular aspect in consideration.

e) By applying a maximum effective loading of nearly 4500 psf the major amount of recompression takes place during the construction phase.

f) During the construction phase a dewatering system is installed around the perimeter and within the excavation to control underseepage through silt and sand layers in and below the excavation slopes. Refer to Subsection 2.5.4.5.2 for a discussion of the dewatering system used at the site. A series of twelve recharge wells are also located around the perimeter of the mat foundation extending into the compacted shell filter blanket under the mat. The locations of these wells are shown in Figure 2.5-83. These recharge wells assist in introducing hydrostatic uplift forces to compensate for additional construction-imposed foundation loads beyond the 4500 psf allowable pressure.

In order to ensure meeting the design objectives, detailed excavation specifications and drawings were prepared. Figures 2.5-81 and 2.5-82 detail configuration of the excavation. The slopes presented on these drawings were established based on the soil properties determined from laboratory and field tests. The excavation specification detailed the construction of the concrete mat foundation such that it minimized the exposure of the stiff clays at the base of the foundation. In order to assure uniform pore pressure distribution in the clays beneath the mat upon relieving the dewatering system, a filter media consisting of compacted shell was utilized. Detailed instrumentation, consisting of electrical extensometers, mechanical heave points, pore pressure piezometers and settlement plates, were installed to monitor heave and recompression settlement of the mat foundation. Refer to Subsection 2.5.4.13 for a complete discussion of the instrumentation system. A plot plan showing the instrumentation systems which monitor foundation responses is presented on Figure 2.5-112.

The criteria for selection of design parameters and the design methods and associated factors of safety are based upon established soil mechanics procedures and have been noted in the relevant sections. References have been cited where applicable.

2.5.4.12 Techniques to Improve Subsurface Conditions

In order to improve conditions within the plant area and to prevent liquefaction around the NPIS all Recent material (initial plant grade to -40 ft. MSL) was excavated and replaced with compacted sand backfill. Further, to prevent excessive long-term consolidation settlement and differential settlement a floating foundation principle was utilized including a carefully monitored construction dewatering system to maintain foundation pressures as close as possible to their in situ state. Refer to Subsections 2.5.4.5 and 2.5.4.11 for discussions of the excavation-backfill program and the floating foundation principle respectively.

No grouting, vibroflotation rock bolting etc. beneath the NPIS was required.

MARK -

THERE ARE TWO INCIDENTS OF WATER BLEEDING THROUGH THE COMMON MAT.

THE FIRST WAS IN 1977 AND WAS OF NECESSITY REPAIRED/SEALED SINCE IT WAS WITHIN THE RINGWALL WHERE CONCRETE WAS TO BE PLACED TO SUPPORT THE CONTAINMENT VESSEL.

THE SECOND WAS IN 1983 WHEN THESE WERE IDENTIFIED IN THE GAS SURGE TANK ROOM, WASTE COMPRESSOR ROOM B AND SOME OTHERS AT -35' EL (TOP OF THE COMMON MAT).

TWO NCR'S COVERING THESE ARE ATTACHED.

THE THREE AREAS OF CONCERN THAT WE HAD WERE:

1. STRUCTURAL INTEGRITY OF THE MAT.
   THIS WAS ADDRESSED BY E. GALLAGHER IN HIS "EVALUATION OF NCR (Suppl #3) 8/26/77
2. CORROSION OF REINFORCING STEEL AND CONTAINMENT VESSEL. ADDRESSED BY PEABODY AND OLIVEIRA. "CORROSION OF REINFORCING STEEL, etc 8/5/77.
3. STEEL CONTAINMENT STABILITY
   SAME TITLE PC LIU 5/24/83

THE CRACKING WITHIN THE RINGWALL WAS ~~REPORTED~~ AS "POTENTIALLY REPORTABLE IN ACCORDANCE WITH

50.55(e) AS LP&L CONSTRUCTION INCIDENT NO. 8.
IT WAS LATER DETERMINED NOT TO BE REPORTABLE
AND WE CONCURRED.

HOWEVER, WE REVIEWED THE CORRECTIVE
ACTION AND REPORTED IN IE REPORTS
50-382/77-07 AND 77-08 (PAGES ATTACHED).

6009-11412-75

REPORT NO. W3-535-Supp #1 & 2

EBASCO SERVICES INCORPORATED

QUALITY ASSURANCE

NONCONFORMANCE REPORT

Distribution:
White - PQAE or Site QA Supervisor
Yellow - Organization recommending disposition
Pink - Initiator of NCR

INSTRUCTIONS: (See back of form)

CLIENT OR PROJECT (2)
Waterford SES Unit #3

DRAWING NO./SPEC NO. (3)
PSAR Section 5.2.2.10

SUPPLIER, CONSTRUCTION QC OR CONTRACTOR (4)
Construction

P.O. NO. (5)

DESCRIPTION OF COMPONENT, PART OR SYSTEM (6)
Common Foundation Mat

I. DESCRIPTION OF NONCONFORMANCE (7) (Items Involved, Specification, Code or Standard to Which Items Do Not Comply. Submit Sketch if Applicable)

The top of the mat beneath the containment structure contains a number of cracks which were discovered to be weeping water. The rate of weeping is generally enough to s[illegible] the crack and to moisten the surrounding concrete. ~~It appears that these radial cracks are the result of the concave shape which the material has assumed due to differential settlement.~~ [initials] 7-28-77

NAME AND SIGNATURE OF PERSON REPORTING NONCONFORMANCE (8): [signature] R. A. Hartnott
TITLE: Q. A. Site Supervisor
DATE (9): 7-28-77

II. RECOMMENDED DISPOSITION (10) (Submit Sketch if Applicable)

SEE ATTACHED SHEET.

SEE PPI No. 8

REPORTABLE	YES	NO
10CFR50.55(e)	☐	☑
10CFR21	☐	☑
Reviewed		

NAME AND SIGNATURE OF PERSON RECOMMENDING DISPOSITION (11): W.C. GRIGGS W.C. Griggs
TITLE: RES. ENG'R. - FIELD
DATE (12): 7/29/77

III. EVALUATION OF DISPOSITION BY EBASCO, REASON FOR DISPOSITION (13)

After the attached test method for repair of cracking is performed a detailed test report is to be presented to Engrg for review & final evaluation. A detailed work procedure for final repairs shall then be written prior to ~~commencing~~ work. [initials] 7-29-77

☒ ENGINEERING	☐ QUALITY ASSURANCE	☐ CONSTRUCTION	☐ OTHER AUTHORIZED PERSONNEL
NAME (SIGNATURE): EJ Gallagher	NAME (SIGNATURE):	NAME (SIGNATURE):	NAME (SIGNATURE):
DATE: 7-29-77	DATE:	DATE:	DATE:
☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS

V. VERIFICATION OF DISPOSITION ☒ REQUIRED ☐ NOT REQUIRED (14)

(15) BY: Joe Gutierrez (EBASCO WATERFORD QA ENGINEERING) SIGNATURE: Joe Gutierrez TITLE: Civil QA Engr DATE: 1-5-77

Refer to Supp #3

EBASCO SERVICES INCORPORATED

QUALITY ASSURANCE

NONCONFORMANCE REPORT

REPORT NO. (1) W3-535 - Supp #3

Distribution:
White - POAE or Site QA Supervisor
Yellow - Organization recommending disposition
Pink - Initiator of NCR

INSTRUCTIONS: (See back of form)

CLIENT OR PROJECT (2): Waterford SES - Unit No 3

DRAWING NO./SPEC NO. (3): PSAR Section 5.2.2.10

SUPPLIER, CONSTRUCTION QC OR CONTRACTOR (4): Construction

P.O. NO. (5):

DESCRIPTION OF COMPONENT, PART OR SYSTEM (6): Common Foundation Mat Within the RCB Wall

I. DESCRIPTION OF NONCONFORMANCE (7) (Items Involved, Specification, Code or Standard to Which Items Do Not Comply. Submit Sketch if Applicable)

This supplement provides additional information on the crack pattern and documents the crack patterns on the attached Field Sketch No 1564-4.1-G-28.

REPORTABLE	YES	NO
10CFR50.55(e)	☐	☑
10CFR21	☐	☑

Reviewed by: [signature] Date: 9/6/77

NAME AND SIGNATURE OF PERSON REPORTING NONCONFORMANCE (8): R A Hartnett [signature]

TITLE: Q A Site Supervisor

DATE (9): 8-25-~~66~~ 77

II. RECOMMENDED DISPOSITION (10) (Submit Sketch if Applicable)

REFER TO ENG FOR EVALUATION and DISPOSITION

NAME AND SIGNATURE OF PERSON RECOMMENDING DISPOSITION (11): [signature] FRED D. ROSE

TITLE: SR QC Sup.

DATE (12): 8/26/77

III. EVALUATION OF DISPOSITION BY EBASCO, REASON FOR DISPOSITION (13)

See attached supplement #3 to NCR W3-535 for evaluation

☐ ENGINEERING	☐ QUALITY ASSURANCE	☐ CONSTRUCTION	☐ OTHER AUTHORIZED PERSONNEL
NAME (SIGNATURE): E J Gallagher	NAME (SIGNATURE):	NAME (SIGNATURE):	NAME (SIGNATURE):
DATE: 8-26-77	DATE:	DATE:	DATE:
☐ ACCEPTED ☐ REJECTED ☑ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS

IV. VERIFICATION OF DISPOSITION ☑ REQUIRED ☐ NOT REQUIRED (14)

(15) BY Joe Gutierrez (EBASCO VENDOR QA OR QA ENGINEERING) SIGNATURE Joe Gutierrez TITLE Civil-QA Engr. DATE 1-5-77

NONCONFORMANCE REPORT
CLOSURE VERIFICATION

NCR No. W3-535
and Supp #1 #2 #3

REINSPECTION: [X] Required [ ] Not Required

Repair or rework to be witnessed by Ebasco's Q.C. Inspector [X] Yes [ ] No

Corrective Action Taken (Use sketch if necessary)

The subject surface cracks on the mat within the Reactor Bldg have been satisfactorily repaired in accordance with supplement to NCR W3-535 dated Aug 3, 1977. Visual inspection has been performed & indicates successful repair of any weeping of water from the cracks. -

Q.C. to verify the above & placement 7 thru 14 may proceed - EJGallagher 8/18/77

Cracks that were identified after the 8/18/77 were also repaired in accordance with the Ebasco disposition attached to NCR W3-535 & found satisfactory. 9/13/77 EJGallagher

[ ] Accept

[ ] Reject

Contractor's Q.C. Inspector N/A Date —

Ebasco's Q.C. Inspector ______ Date ______

See reverse side. Ole Cochran 9/13/77 →

Form No. ASP-III-7-4 (3-16-76)

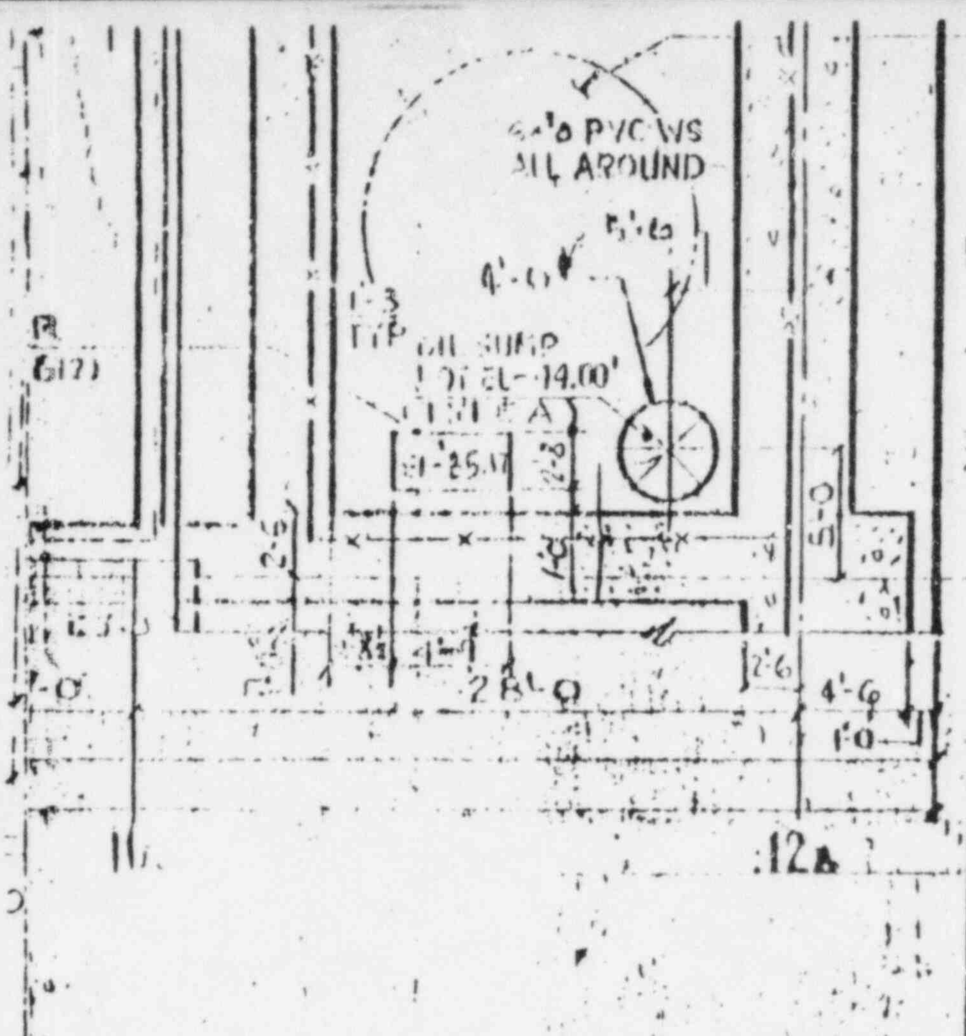
EVALUATION OF DISPOSITION TO NCR SUPPL. #3 W3-535

The newly identified cracks which are indicated by the dashed line on the attached sketch, are to be sealed and repaired according to the Supplement #2 attached to NCR W3-535. All such cracks beneath a specific concrete placement must be sealed and dry prior to concrete placement. These cracks, after being repaired, will not cause any further effect on the structural capabilities of the foundation mat. If any of the construction joints indicate leakage, the entire construction joint is to be sealed until all leakage ceases.

Quality Control should carefully inspect the cracks prior to placement to verify that no cracks have been missed due to surface dust or placement equipment and that the cracks that have been repaired are not continuing to leak.

E J Gallagher

E. Gallagher 8-26-77
Site Concrete-Hydraulics Engineer



RECEIVED

SEISMIC CLASS

LOUISIANA POWER & LIGHT COMPANY WATERFORD S. E. S. UNIT NO. 3 1980-1165 MW INSTALLATION		
RCB FOUNDATION CRACK MAP		
EBASCO SERVICES INCORPORATED FIELD		
SCALE 1/8" = 1'-0	RELEASED	DATE 8-17-77
DIV. CONSTRUCTION DR. AAJ CH. WR/MB		FIELD SKETCH No. SK 1564 - 4.1-G-28

DATE	REVISION	BY	CH.	RELEASED
2-21-77	ADD ADDITIONAL [illegible]	MB	MB	
5-23-77	ADD ADDITIONAL CRACK [illegible]	MB	MB	

EBASCO SERVICES
INCORPORATED

UTILITY CONSULTANTS - ENGINEERS - CONSTRUCTORS

TWO RECTOR STREET
NEW YORK, N.Y. 10006

CABLE ADDRESS "EBASCO"

File

August 24, 1977
LW3-1617-77
14Q-R-12

Mr D L Aswell
Vice President-Power Production
Louisiana Power & Light Company
142 Delaronde Street
New Orleans, Louisiana 70174

Re: WATERFORD SES UNIT NO. 3
CONSTRUCTION INCIDENT NO. 8
CONCRETE FOUNDATION MAT CRACKING
BENEATH THE CONTAINMENT

As requested by Mr A E Henderson, we are forwarding one copy of our file on Construction Incident No. 8. This contains the bases for our opinion that this incident is considered to be of the non-reportable type.

If you have any questions or require additional information, please advise us.

Very truly yours,

RK Stampley
R K Stampley
Project Manager 8/26/77

RKS:PG:ej
Att:

cc: D L Aswell
L V Maurin
A E Henderson (w/encl)
D B Lester
P V Prasankumar
C J Decareaux
Power Production Dept-Nuclear (3)
H W Otillio
C G Chezem
T F Gerrets
F X Shaughnessy
J M Brooks
J O Booth (2)

b. Wall 499S04-11A4 and Shield Wall Reinforcing Steel

The inspector observed the repaired section of wall 11A4 which still had the forms in place. The licensee informed the inspector that the reinforcing steel at the top of the shield wall had been repaired and that the nonconformance report had been closed out. Final documentation of the repairs will be reviewed during a subsequent inspection. This item will remain open pending review of the final documentation.

c. Concrete Foundation Mat Cracking Beneath the Containment

The inspector reviewed the status of a potentially significant construction deficiency relating to cracking in the foundation mat which was reported to RIV on August 1, 1977. The cracks are located beneath the containment and are identified by water seepage. Review of correspondence indicated that Ebasco is requiring that the cracks be sealed prior to placement of [illegible] concrete beneath the containment vessel. The inspector observed the sealing of cracks with Sikadur "High-mod LV." This item will remain open pending review of the results of the sealing during a subsequent inspection.

8. Safety Related Structural Steel

The inspector observed structural steel erection by American Bridge in the area of the cooling towers. Specifically observed were the bolting and torque testing of four joints. These work activities were found to be in accordance with American Bridge Procedures No. 4 and 10.

Qualification records of the QC inspector were reviewed. These records indicated that the QC Inspector was qualified in accordance with ANSI N45.2.6.

The inspector reviewed calibration records for torque wrench No. 9498 and the Skidmore-Wilhelm Bolt Tester SN. 3055. The torque wrench was found to be calibrated in accordance with Procedure No. 10. The bolt tester was found to be calibrated by Pittsburgh Testing Laboratory on March 14, 1977; however, the tester is not specifically included in the calibration program as part of the procedures. This and similar omissions of equipment requiring certification had been identified in the Ebasco audit of American Bridge, Report No. JG-77-7-1, dated July 29, 1977. This matter will be resolved through the close out of the Ebasco audit. Resolution will be verified during a subsequent inspection. This item is considered an unresolved item pending review of final closeout of the audit findings report.

No items of noncompliance or deviations were identified.

During the inspection of the above fillet welds, it was noted that the zinc-rich paint applied to the welds as a protective coating in accordance with Fischbach and Moore procedure CP-203, Rev. 2. contained cracks. The inspector reviewed construction procedure CP-203 and quality control instruction QCI-101W3 to determine the requirements defining an acceptable painted surface and could not ascertain well defined acceptance criteria. The inspector discussed the matter with the licensee's Quality Assurance Technician and the contractor's Project QC Manager and was informed that painting was inspected during the final inspection of the installed supports. The inspector expressed concern to the licensee regarding the definition of quality requirements for the zinc coating. The licensee committed to redefine the quality requirements for the coating and review the components already painted to insure the coatings were not cracked.

This item is considered unresolved and will be reviewed during subsequent inspections.

10. Significant Construction Deficiencies Reported by the Licensee

The inspector reviewed licensee action related to the following items which were previously reported as significant or potentially significant construction deficiencies in accordance with the requirements of 10 CFR 50.55(e).

a. Common Foundation Mat Cracks

16

After an unsuccessful attempt at pressure injection of Concressive 1380 epoxy into hairline cracks caused by mat flexure, a more effective procedure was initiated to control the leakage of water through the cracks. This procedure consisted of chipping a one inch deep trench along the length of the crack, roughening and cleaning of the surface as well as a one foot strip on either side of the crack, and filling of the trench with SIKADUR Hi-Mod-LV epoxy. All repairs beneath placement 5[illegible]-6 were monitored for one day and no indication of water leakage was observed. The inspector viewed the results of further sealing operations performed in anticipation of future concrete fill placements which should, when placed, reverse the flexure and minimize the cracks. [redacted] THIS MATTER IS CONSIDERED CLOSED

b. Excessive Air Entrainment

7

Additional borings in wall placements G-571-S01-5B and -8B have identified the area where concrete compressive strength is less than the design strength of 4000 pounds per square inch as an area from one to four feet below the top of wall 5B and up to and including thirteen feet from the extreme west end of this placement. The total area involved is therefore approximately fifty-two square feet out of a total wall area of eight hundred and eighty-two square feet. The wall is three feet thick.

EBASCO SERVICES INCORPORATED

QUALITY ASSURANCE

NONCONFORMANCE REPORT

REPORT NO. W3-6212

Distribution:
White - POAE or Site QA Supervisor
Yellow - Organization recommending disposition
Pink - Initiator of NCR

INSTRUCTIONS: (See back of form) TRND TREND CODE: 2000.60.69 SUS# 99C

CLIENT OR PROJECT (2): WATERFORD SES UNIT NO. 3

DRAWING NO./SPEC NO. (3): F.S.A.R. -

SUPPLIER, CONSTRUCTION QC OR CONTRACTOR (4): J.A. JONES CONSTRUCTION CO.

P.O. NO. (5): W3-NY-4

DESCRIPTION OF COMPONENT, PART OR SYSTEM (6):

I. DESCRIPTION OF NONCONFORMANCE (7) (Items Involved, Specification, Code or Standard to Which Items Do Not Comply, Submit Sketch if Applicable)

There are concrete cracks in the base mat of the Reactor Auxiliary Building. This is evidence by the percolation of water in small amounts, up through these cracks. These cracks are located in the Gas Surge Tank Room, Waste Gas Tank Room, and Waste Gas Compressor "B" Room, all at elevation -35.00. See attached F.S.A.R. requirements for supplemental information. NOTE: These are examples of where cracks were found.

ITEM NO: 43

NAME AND SIGNATURE OF PERSON REPORTING NONCONFORMANCE (8): S. Horton Sam Horton

TITLE/COMPANY: Q.A. Surv. Engr./Ebasco

DATE (9): 5-11-83

II. RECOMMENDED DISPOSITION (10) (Submit Sketch if Applicable)

ACCEPT AS IS.

SEE ATTACHMENT III.

REPORTABLE	YES	NO
10CFR50.55(e)	☐	☑
10CFR21	☐	☑

Date: 5/25/83

NAME AND SIGNATURE OF PERSON RECOMMENDING DISPOSITION (11): George Ed Hill

TITLE/COMPANY: QA - EBASCO

DATE: 5-25-83

III. EVALUATION OF DISPOSITION BY EBASCO, REASON FOR DISPOSITION (13)

5/25/83

IV. CORRECTIVE ACTION (14) ☐ Required ☒ Not Required

5/25/83

V. ☒ ENGINEERING	☐ QUALITY ASSURANCE	☐ CONSTRUCTION	☐ OTHER
NAME SIGNATURE: Brian Kent	NAME SIGNATURE: E. Claussen	NAME SIGNATURE	NAME SIGNATURE
DATE: 5/25/83	DATE: 5/25/83	DATE	DATE
☒ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☒ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS	☐ ACCEPTED ☐ REJECTED ☐ ACCEPTED WITH COMMENTS

VI. VERIFICATION OF DISPOSITION ☐ REQUIRED ☒ NOT REQUIRED 5/25/83

BY ______ SIGNATURE ______ TITLE ______ DATE ______

NCR QE-6212
Page 1

ATTACHMENT III

The effect of postulated widespread hairline cracking of the basemat has been investigated by Civil Engineering for stability of the Containment Vessel against flotation and overturning under buoyant conditions caused by postulated groundwater intrusion and by Corrosion Engineering for groundwater induced corrosion of reinforcing steel and Containment Vessel bottom head. THESE WERE THE ONLY POTENTIAL PROBLEM AREAS IDENTIFIED IN THE INVESTIGATION. 5-26-83

Based on their findings that there are no stability or corrosion problems it is concluded that no corrective action is required.

See attached memorandums:

1. Memorandum COR-LW3-77-55M from A.W. Peabody/M.D. Oliveira to P. Grossman, dated August 5, 1977.

2. Memorandum from P.C. Liu to B. Grant dated May 24, 1983.

5-25-83

NCR W3-6212
Page 2

August 5, 1977
COR-LW3-77-55M

To: P Grossman

From: A W Peabody/M D Oliveira

Subject: LOUISIANA POWER & LIGHT COMPANY
WATERFORD SES UNIT 3
CORROSION OF REINFORCING STEEL AND
STEEL CONTAINMENT VESSEL PLATES IN CONTACT WITH WATER

In accordance with your telephone request, we have analysed a possible situation in the common mat where supposedly ground water weeping from concrete cracks found on the surface of the mat could corrode the reinforcing steel and the outside bottom plates of the Steel Containment Vessel.

It is a proven fact that concrete by its alkaline nature passivates carbon steel embedded in it.

It is also known that water in contact with concrete becomes alkaline and consequently its corrosivity to steel decreases considerably.

In addition to these factors, assuming that ground water is left inside the crack network to a certain extent, this water will be near stagnant and without replenishment of oxygen. Consequently, the rate of corrosion under the above circumstances, if any, will be negligible. This applies to the reinforcing rebars as well as to the outside of the vessel bottom plates, in case the repairs presently being conducted do not fully prevent the water from reaching the vessel.

MDO/hm
cc: R K Stampley
J O Booth/B D Fowler
D N Galligan
L Skoblar
W F Gundaker

MAY 25 1982

CR-W3-6212
Page 3

Rec'd
5/24/83

SH To: B. Grant ... Call x277
From: P C Liu

EBASCO

Interoffice Correspondence

DATE May 24, 1983 FILE REF. File: 6-S-20

TO B Grant

OFFICE LOCATION Waterford Site

FROM P C Liu

OFFICE LOCATION 87 WTC

SUBJECT LOUISIANA POWER & LIGHT COMPANY
WATERFORD SES UNIT NO. 3
STEEL CONTAINMENT STABILITY

This is to confirm our conversation that the steel containment stability has been reviewed for an imaginally condition that the exterior of the containment would subject to subsurface water up to EL-1.50 ft. The results of the review have concluded that under such a condition the stability of the containment will not be compromised. The stability calculations will be included in Volume II, PSAR Design Input - 6W12-PSAR-002.

PCL:dg

cc: G A Kanakaris
S S Kowalski
P C Liu
Project File

RECEIVED
MAY 25 1983