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

DOCKET NUMBER
PROD. & UTIL. FAC. 50-382-OL

SEP 4 1984

DOCKETED
USNRC

'84 SEP 10 P12:15

MEMORANDUM FOR: Martin G. Malsch
Deputy General Counsel

FROM: Guy H. Cunningham, III
Executive Legal Director

SUBJECT: GUNNAR HARSTEAD -- WATERFORD

Enclosed are affidavits prepared by Michael S. Callahan, Kellogg V. Morton, Morris Reich, Frank Rinaldi, and Robert E. Shewmaker, in response to your memorandum of August 9, 1984, requesting responses to five questions concerning the above matter.

The other offices which were asked to furnish responses to your memorandum have not provided information beyond that which is covered by the enclosed affidavits.

Guy H. Cunningham, III
Executive Legal Director

Enclosures: As stated

cc: Waterford Service List
Gunnar Harstead

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PDR ADOCK 05000382
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OFFICE OF SECRETARY
DOCKETING & SERVICE
BRANCH

COUNTY OF MONTGOMERY)
) SS
STATE OF MARYLAND)

AFFIDAVIT OF MICHAEL S. CALLAHAN

I, Michael S. Callahan, being duly sworn, do depose and state:

1. Since December 1982, I have been employed as Chief, Staffing Section, Staffing and Position Evaluation Branch, Division of Organization and Personnel, Office of Administration, U.S. Nuclear Regulatory Commission. As part of my official duties, I am responsible for maintenance and safe-keeping of official personnel records for NRC employees, incuding special Government employees.

2. At the request of the Director of the Division of Organization and Personnel, I undertook a review of NRC personnel files in order to ascertain the employment status of Dr. Gunnar Harstead. My review disclosed that Dr. Harstead was engaged by the NRC as a special Government employee (expert) under an appointment commencing on February 15, 1980 (No. AT-(49-24)-1353); this appointment expired on June 30, 1984.

3. Dr. Harstead's personnel records do not indicate the particular matters on which he has worked; that type of information is not maintained by my office, but is normally available in the offices of the technical

staff with whom an individual has worked, such as the Office of Inspection and Enforcement or the Office of Nuclear Reactor Regulation. However, NRC payroll records indicate that Dr. Harstead submitted vouchers for payment for work he performed as a special Government employee during calendar years 1980-1984 as follows:

CY 1980 (commencing in June 1980)	52 days
CY 1981	90 days
CY 1982	85 days
CY 1983	0 days
CY 1984 (last voucher, April 10, 1984)	9 days

M. Callahan
Michael S. Callahan

Subscribed and sworn to before me
this 4th day of September, 1984.

Malinda L. McDonald
Notary Public

My commission expires: 7/1/86

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'84 SEP 10 P12:16

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BRANCH

COUNTY OF MONTGOMERY)
) SS
STATE OF MARYLAND)

AFFIDAVIT OF KELLOGG V. MORTON

I, Kellogg V. Morton, being duly sworn, do depose and state:

1. I am employed as Chief, Technical Contracts Branch, Division of Contracts, Office of Administration, U.S. Nuclear Regulatory Commission. As part of my official duties, I am responsible for supervising the performance of the total contracting function, including selection, negotiation, administration and close-out activities, in support of technical assistance and confirmatory research needs of NRC Offices and Divisions.

2. A review of NRC contract files (not including personnel files) has been conducted under my direction, in order to ascertain whether the NRC has contracted for the services of either Harstead Engineering Associates, Inc. (HEA) or Dr. Gunnar Harstead. This review has disclosed that no contracts (as distinct from personal service or consultant agreements) have been issued by the NRC directly to either HEA or Dr. Harstead.

3. The contract file review referred to above has further disclosed that HEA serves or has served as a subcontractor to other entities with whom the NRC has or has had a contractual relationship. To the best of my knowledge, these instances are as follows:

Contractor/ NRC Contract No.	Facility	Days Worked/ Calender Year	Approximate Cost
Parameter, Inc. NRC-05-82-249	Byron	115 (CY 1983)	$80,100
	Comanche Peak	2 (CY 1983)	900
WESTEC Services, Inc. NRC-05-84-151	Byron	5 (CY 1984)	3,900
	Diablo Canyon	11 (CY 1984)	7,800
	Perry	76 (CY 1984)	51,900
	River Bend	203 (CY 1984)	131,200

4. Except as described above, I have no record or knowledge of any other matters in which the NRC utilized the services of HEA or Dr. Harstead.

Kellogg V. Morton

Subscribed and sworn to before me
this 4th day of September, 1984

Edythe S. Becker
Notary Public

My Commission expires: July 1, 1986

COUNTY OF SUFFOLK)
STATE OF NEW YORK) SS
)

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BRANCH

AFFIDAVIT OF MORRIS REICH

I, Morris Reich, being duly sworn, do depose and state:

1. I am employed as Head of the Structural Analysis Division, Department of Nuclear Energy, Brookhaven National Laboratory (BNL). As part of my official duties, I headed a team of BNL technical consultants to the Structural and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Regulatory Regulation, U.S. Nuclear Regulatory Commission, concerning the foundation base mat at Waterford Unit 3.

2. At the request of the Office of the Executive Legal Director, I prepared a summary of all contacts between BNL and Harstead Engineering Associates, Inc. (HEA) personnel concerning the Waterford Facility; this summary is set forth in a letter from me to Sherwin Turk, Esq. dated August 31, 1984, a copy of which is attached to this affidavit as Attachment 1.

3. To the best of my knowledge and belief, the summary set forth in Attachment 1 hereto provides a true and correct description of all contacts between BNL and HEA personnel concerning the Waterford Facility.

Morris Reich

Subscribed and sworn to before me
this 4th day of September, 1984.

Notary Public

My commission expires:

DONNA M. GALLAGHER
NOTARY PUBLIC, State of New York
No. 4777833, Suffolk County
Term Expires March 30, 19__

ATTACHMENT 1 TO
AFFIDAVIT OF MORRIS REICH

BROOKHAVEN NATIONAL LABORATORY
ASSOCIATED UNIVERSITIES, INC.

Department of Nuclear Energy
Building 129

Upton, Long Island, New York 11973

(516) 282
FTS 666 2448

August 31, 1984

Mr. Sherwin Turk
Room No. 9604
Maryland National Bank Building
U.S. Nuclear Regulatory Commission
7735 Old Georgetown Road
Bethesda, MD 20814

Dear Mr. Turk:

With reference to the memorandum from Martin G. Malsch to Guy H. Cunningham III, dated August 9, 1984, subject: Gunnar Harstead - Waterford, please be advised of the following:

All contacts between BNL and Harstead Engineering Associates (HEA) personnel were concerned with the following HEA reports and the associated finite element computer output;

"Analysis of Cracks and Water Seepage in Foundation Mat", Report 8304-1, September 19, 1983.
"Analysis of Cracks and Water Seepage in Foundation Mat", Report 8304-2, October 12, 1983.
Stardyne Computer Output for Waterford Basemat, run-date, October 4, 1983.

Specific dates, locations of meetings, personnel present, are listed in the table below.

Meeting Location	Date	Personnel Present
Bethesda, MD	March 26, 1984	NRC, BNL (M. Reich, S. Sharma, P.C. Wang), HEA (G.A. Harstead, A.V. du Bouchet, A.I. Unsal), EBASCO, LPL
Waterford Site	March 27, 1984	"
EBASCO Inc., N.Y.C.	April 4, 1984	NRC, BNL (M. Reich, S. Sharma, P.C. Wang, C. Miller, C. Costantino), HEA (A.V. du Bouchet, A.J. Unsal), EBASCO

Meeting Location	Date	Personnel Present
EBASCO Inc., N.Y.C.	July 2, 1984	BNL (M. Reich, S. Sharma, C. Miller, C. Costantino), HEA (A.V. du Bouchet, A.I. Unsal), EBASCO

In addition to the meetings, approximately 6 phone conversations were held with either A.V. du Bouchet and A.I. Unsal of HEA during the period April-June, to clarify the format and various aspects of the computer outputs.

If there is any other information that I can provide to clarify the above, please do not hesitate to contact me.

Sincerely yours,

Morris Reich, Head
Structural Analysis Division

MR/dv

COUNTY OF MONTGOMERY)
) SS
STATE OF MARYLAND)

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AFFIDAVIT OF FRANK RINALDI

I, Frank Rinaldi, being duly sworn, do depose and state:

1. I am employed as a Structural Engineer in the Structural and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission.

2. At the request of the Director of the Division of Engineering (DE), I prepared a response on behalf of DE to the questions raised by OGC concerning Dr. Gunnar Harstead. This response is attached as an Enclosure to a Memorandum from Richard H. Vollmer to Guy Cunningham, III, dated August 29, 1984, a copy of which Memorandum is attached hereto as Attachment 1.

3. In three respects, the Enclosure to Attachment 1 hereto should be revised. First, in response to Question 1, the date of July 1979 reflects a period in which NRR had communications with Dr. Harstead concerning his potential employment with the NRC; NRR Contract No. AT-(49-24)-1353 actually commenced at a later date. Second, in response to Question 5, the work performed by Dr. Harstead on the Midland project was performed in part as a special Government employee (NRC) and in part as a consultant or special Government employee of the Naval Surface Weapons Center; at this time, I do not have readily available to me a breakdown of how these services were provided. Third, it should be

noted that the number of days worked by Dr. Harstead, as reflected in the Enclosure to Attachment 1 hereto, is an approximation.

4. Except as indicated above, to the best of my knowledge and belief the information set forth in the Enclosure to Attachment 1 hereto provides a true and correct response to the questions raised by OGC.

Frank Rinaldi

Frank Rinaldi

Subscribed and sworn to before me
this 4th day of September, 1984.

Malinda L. McDonald

Notary Public

My commission expires: 7/1/86

ATTACHMENT 1 to
AFFIDAVIT OF FRANK RINALDI

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

AUG 29 1984

Joe Scinto

MEMORANDUM FOR: Guy Cunningham, III
Executive ~~Legal~~ Director

FROM: Richard H. Vollmer, Director
Division of Engineering
Office of Nuclear Reactor Regulation

SUBJECT: GUNNAR HARSTEAD - WATERFORD

Your memo dated August 15, 1984, on the above subject presented a request for answers to five questions set forth in M. G. Malsh's memo of August 9, 1984. The enclosure provides the answers to the five questions by the Office of the General Counsel. The enclosure has been prepared by Frank Rinaldi, SGEB.

Knight for

Richard H. Vollmer, Director
Division of Engineering
Office of Nuclear Reactor Regulation

Enclosure:
As stated

cc: J. Scinto
J. Knight
G. Lear
P. Kuo
J. Ma
J. Chen
F. Rinaldi

ENCLOSURE

ANSWERS TO QUESTIONS RELATED TO G. HARSTEAD SERVICES

Prepared by F. Rinaldi, SGEB-DE

Q.1 For what time period did the NRC employ or contract for the services of Harstead on matters relating to the Waterford facility?

A.1 Dr. Harstead was retained by NRC Structural Engineering Branch under NRR Contract AT-(49-24)-1353 on July 1979, to provide consulting services in the field of structural engineering. The contract was renewed up to June 30, 1984, at which time the contract expired.

Dr. Harstead's involvement on matters related to the Waterford facility, as staff consultant, was limited to the period of March through May 1981.

Dr. Harstead has worked as a subcontractor on tasks sponsored by NRC Office of Inspection and Enforcement (I&E) on various nuclear power plants, but not on the Waterford facility.

Q.2 Please describe the work Dr. Harstead has performed for the NRC staff regarding Waterford?

A.2 Dr. Harstead served on an audit team that audited the structural engineering design of Category I structures for the Waterford facility at the offices of Ebasco Services Inc. Dr. Harstead's involvement commenced during March 1981 and ended in May 1981 after the issuance of the Safety Evaluation Report (SER) from the Division of Engineering to the Division of Licensing. The audit took place during the week of April 6, 1981.

Dr. Harstead became familiar with the scope of the structural audits performed by the Structural Engineering Branch and the applicable sections of the Final Safety Analysis Report on the Waterford facility. Dr. Harstead participated in this audit as technical expert for the NRC staff. Dr. Harstead participated in raising various issues requiring technical clarification, as documented in the minutes of the audit, and in the evaluation of additional material provided by Ebasco Services Inc. to answer staff concerns. No other work on Waterford has been performed by Dr. Harstead for the staff following the issuance of the SER in May 1981.

Q.3 What is the relationship between the work he performed for the staff and the Waterford licensing proceedings?

A.3 During the audit the staff checked, as per staff guidelines, the design of all Category I structures for adequacy and general agreement with the design requirement identified in the Waterford FSAR and with the staff requirements identified in the applicable sections of the NRC Standard Review Plan. The recent proceedings have focused on the reinforced concrete basemat for the Category I structures. The recent concern specifically is the effect of the possible through-cracks of the basemat. The April 1981 audit evaluated the design of the basemat but did not consider the cracks discussed during the recent proceedings. The structural staff was not aware of any cracks at the time they conducted the audit.

Q.4 What, if any, oral or written communications on behalf of Louisiana Power & Light Company (LP&L) or any other private entity has Mr. Harstead made to the NRC staff, the Licensing Board, or the Appeal Board regarding the Waterford facility. Please describe any such communications in detail and provide pertinent documents. Describe meetings, if any, with the NRC staff that Mr. Harstead attended as a representative of LP&L or HEA.

A.4 Dr. Harstead made one oral presentation to the NRC staff on behalf of LP&L on March 26, 1984, related to the basemat integrity for the Waterford facility. Dr. Harstead provided the following three reports addressing the Waterford basemat:

(1) Harstead Engineering Associates Report 8304-1, September 19, 1983

- o Evaluated effects of cracks on basemat integrity
- o Mapped basemat cracks
  (Cracks were so small as to be undetectable by standard inspection techniques)

- o Reviewed significant events during construction
    - o Stop Work Order No. 1
    - o Placement difficulties - Placements 10B & 19

- o Reviewed settlement plan and data

- o Evaluated corrosion potential

- o Evaluated steel containment vessel stability

- o Performed a general review of basemat engineering design and construction.

(2) Harstead Engineering Associates Report 8304-2, October 10, 1983

- o Performed an independent structural analysis of the basemat.

(3) Harstead Engineering Associates Report 8304-3, January 9, 1984

- o Review of construction documentation to evaluate whether design objectives were met.

Q.5 How many days per calendar year has Dr. Harstead worked for the NRC on Waterford and other matters for each year from 1981 to 1984? Was he hired as a special government employee or as a contractor?

Dr. Harstead has worked on the following tasks:

Plant	Purpose	Period	Estimated Days
D. C. Cook	Evaluation of Ultimate Capacity of the Containment	April 1980 to May 1981	30
Waterford	Audit of Category I Structures	March 1981 to May 1981	10
Midland	Audit of Category I Structures/Evaluation of Underpinning/ Evaluation of DGB	April 1981 to June 1984	100
Comanche Peak	Audit of Category I Structures	May 1981	10
Dresden	Effects of Fuel Racks during seismic events	Sept. 1981 to July 1982	90
Byron 1	Subcontractor to Parameter	May 1983 to present	Unknown
Seabrook	Subcontractor to EG&E	Sept. 1983 to present	Unknown
River Bend	Subcontractor to West Tech	March 1984 to present	Unknown
Perry	Subcontractor to West Tech	July 1984 to present	Unknown

The Midland project is the only one that has required extended involvement by Dr. Harstead. The reported 100 days can be subdivided as follows:

Fiscal Year	Estimated Days
1981	25
1982	35
1983	25
1984	15

Dr. Harstead was hired on a special government employee status for work performed for NRR and as a subcontractor for work performed for I&E.

COUNTY OF MONTGOMERY)
) SS
STATE OF MARYLAND)

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AFFIDAVIT OF ROBERT E. SHEWMAKER

I, Robert E. Shewmaker, being duly sworn, do depose and state:

1. I am employed as a Senior Civil-Structural Engineer in the Engineering and Generic Communications Branch of the Office of Inspection and Enforcement (IE), U.S. Nuclear Regulatory Commission.

2. As part of my official duties, I served as a member of the IE Waterford Inquiry Team in June-July 1983, following the May 1983 discovery of cracks in the Waterford foundation base mat. I also served as a member of the special Task Force which was organized to review allegations concerning the Waterford facility (including allegations related to the foundation base mat), in April-May 1984, and thereafter was involved in preparing written evaluations and summaries of the Task Force findings.

3. At the request of the Director of the Division of Emergency Preparedness and Engineering Response, IE, I prepared a response to the questions raised by OGC concerning Mr. Gunnar Harstead, which response is attached as Enclosure 1 to a memorandum from Edward L. Jordan to Guy H. Cunningham, III, dated August 31, 1984; in addition, I prepared

a "Chronology of Events Related to Gunnar Harstead and Waterford and Other Projects," which is attached as Enclosure 2 to the above mentioned Memorandum. A copy of that Memorandum, together with Enclosures 1 and 2 and related attachments, is attached to this Affidavit as Attachment 1.

4. To the best of my knowledge and belief, the information set forth in Enclosures 1 and 2 to Attachment 1 hereto is true and correct.

Robert E. Shewmaker

Robert E. Shewmaker

Subscribed and sworn to before me
this 4th day of September, 1984.

Edythe S. Becker

Notary Public

My commission expires: 7/1/86

ATTACHMENT 1 to
AFFIDAVIT OF ROBERT SHEWMAKER

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

AUG 31 1984

Docket No. 50-382

MEMORANDUM FOR: Guy H. Cunningham, III, Director
Office of the Executive Legal Director

FROM: Edward L. Jordan, Director
Division of Emergency Preparedness
and Engineering Response
Office of Inspection and Enforcement

SUBJECT: GUNNAR HARSTEAD-WATERFORD

This is in response to your memorandum dated August 15, 1984 relative to the above subject. Dennis Allison sent you a note earlier indicating our full reply would not be available until August 31, 1984 after Bob Shewmaker returned from annual leave. We have now assembled the facts in order to respond to your request.

Enclosure 1 provides responses for IE to the five (5) questions set forth by OGC on the issue. Attachment A is provided to Enclosure 1 to supplement the information and provide a sample of the conflict of interest considerations made for contractors on Integrated Design Inspections.

Enclosure 2 and its attachments (A thru H) provide a commentary prepared by Bob Shewmaker which provides more detail on the technical interaction between the staff and Mr. Harstead during his involvement as a consultant to IE. It was during this time that Mr. Harstead and his company, HEA, were working for Louisiana Power and Light Company on the Waterford 3 facility basemat issue.

If we can be of further help in the preparation of necessary information or affidavits, please contact Bob Shewmaker (x-27432).

Edward L. Jordan, Director
Division of Emergency Preparedness
and Engineering Response
Office of Inspection and Enforcement

Enclosures:
1. Response to OGC Questions
w/Attachment A
2. Chronology of Events
w/Attachments A thru H

cc: see page 2

cc: w/enclosures
R. DeYoung, IE
R. Vollmer, NRR
D. Eisenhut, NRR
J. Collins, RIV
S. Schwartz, IE
J. Grace, IE
E. Halman, ADM
L. Shao, RES
S. Turk, ELD
J. Knight, NRR
G. Lear, NRR
T. Ankrum, IE
R. Baer, IE
A. Dromerick, IE
J. Milhoan, IE
M. Peranich, IE
R. Shewmaker, IE

Enclosure 1

RESPONSE TO OGC QUESTIONS
OF AUGUST 9, 1984

1. IE did not employ or contract for the services of Mr. Harstead on any matters related to the Waterford 3 facility.

2. Mr. Harstead performed no work for IE on the Waterford 3 facility.

3. Same as (2).

4. IE has not received any oral or written communications from Mr. Harstead on behalf of Louisiana Power and Light Company or any other private entity regarding the Waterford 3 facility except as submitted by Louisiana Power and Light Company on the 50-382 docket. Other written material the IE staff had in its possession consisted of notes prepared by Mr. Harstead in 1981 while working as a consultant to the NRC-NRR (Refer to details in Enclosure 2). To the best of our knowledge, the only member of the IE staff who attended any meetings in which Mr. Harstead participated as a representative of HEA or LP and L relative to the Waterford 3 facility is Mr. Mark Peranich. There were two such meetings which Mr. Peranich attended for a portion of the meeting. The first was held by NRR in the Maryland National Bank Building on October 26, 1983 to discuss a series of 10 preliminary staff questions dated October 17, 1983 which had been sent to the licensee. The second meeting was held by NRR in the Landow Building on March 26, 1984 and addressed a series of 32 staff questions relative to the Waterford 3 basemat.

5. Mr. Harstead has not worked for IE on the Waterford 3 facility.

   Mr. Harstead has worked for IE on other matters, specifically in the area of the Integrated Design Inspection program effort as listed below:

   Byron IDI - Period of 4/25/83-10/1/83. Intermittent for a total of 51 man days and then 2 man days on 7/30 and 7/31/84 for open items resolution.

   Seabrook IDI - Period of 10/19/83 - 2/18/84. Intermittent for a total of 66-1/2 man days. Of this 66-1/2 man days approximately 49 man days were for 1983 and the remaining approximately 17-1/2 man days were in 1984.

   River Bend IDI - Period of 3/23/84-7/31/84. Intermittent for a total of 55-1/2 man days.

   Perry IDI (currently underway) - Period prior to 7/31/84 a total of 1 man day. August 1984 hours have not as yet been reported but Mr. Harstead's work has been nearly full time during August 1984.

Mr. Harstead's services were obtained in all cases as a consultant through a contractor in the following methods:

Byron IDI - As a consultant/subcontractor for Parameter, Inc. under NRC Contract No. NRC-05-82-249 with Parameter.

Seabrook IDI - As a consultant/subcontractor for EG&G Idaho, Inc., operators of the National Laboratory in Idaho for the Department of Energy under an NRC work order (FIN A6178).

River Bend IDI- As a consultant/subcontractor to WESTEC Services, Inc. which is providing consulting services under NRC Contract No. NRC-05-84-151.

Perry IDI - Same as River Bend IDI.

In addition, Attachment A to this enclosure (Enclosure 1) provides an example of the criteria used in screening IDI consultants from the standpoint of conflicts of interest. The statement from HEA, Mr. Harstead, for the Byron station is also provided as a sample.

ATTACHMENT A

Parámeter, Inc.

Since 1964

Consulting Engineers • Mechanical Design and Analysis

13380 WATERTOWN PLANK ROAD, ELM GROVE, WISCONSIN 53122 414-786-7580

July 28, 1983

Mr. Wolfgang Laudan, Project Officer
Engineering & Generic Communications Branch
Office of Inspection & Enforcement
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Copy: D. Allison, NRC/IE

Reference: NRC Contract No. 05-82-249
PAR: NRC/IE-82/83, Task 36

Subject: Integrated Design Inspection
@ Byron Station

Dear Mr. Laudan:

For your files we are forwarding copies of letters from consultants assigned to this project which were received in response to Enclosure 1 given to them by NRC/IE during the initial briefing sessions.

Also for completeness of your files, we are transmitting a copy of Enclosure 2, Proprietary Information Agreement, which was executed during the meetings and left with your staff.

No action is required with regard to this submittal.

Very truly yours,

PARAMETER, Inc.

Richard A. Lofy

Richard A. Lofy
President

RAL:mak
Enclosures

ENCLOSURE 1

Information Concerning Selection Criteria and Potential Conflicts of Interest

The competence of the individuals is the primary screening factor in the selection.

With respect to conflicts of interest, the individuals may not have had any direct previous non-NRC involvement with the matters that they will be reviewing for a given plant.

In addition, the factors listed below will be considered on a case by case basis in evaluating the question of potential conflicts of interest for an individual.

1. Whether the individual has been previously hired by the licensee to do similar design work.

2. Whether the individual has been previously employed by the licensee (and the nature of the employment).

3. Whether the individual owns or controls significant amounts of licensee stock.

4. Whether members of the present household of the individual are employed by the licensee.

5. Whether any relatives are employed by the licensee in a management capacity.

In the above discussions, licensee should be construed to mean the licensee, the architect-engineer or the NSSS vendor for the plant to be inspected or a design contractor to one of the above.

It should be noted that additonal factors 1 through 5 above do not necessarily disqualify an individual but rather are to be considered on a case by case basis. For instance, in some circumstances previous employment with the architect engineer firm may not raise any significant potential for conflict of interest and may be a positive factor with respect to qualifications.

/

[illegible] MAY 2 1 1983

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.

169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656 • Phone: (201) 391-2115

May 19, 1983

Mr. Richard A. Lofy, P.E.
Parameter, Inc.
Consulting Engineers
13380 Watertown Plank Road
Elm Grove, WI 53122

Dear Mr. Lofy:

This letter is in regard to my participation in the inspection of Byron Station currently being conducted by the NRC Office of Inspection and Enforcement.

Dennis Allison of that office and the designated team leader of this inspection team, has requested that team members provide any information that could reveal a potential conflict of interest.

To assist the team members in providing this information, Mr. Allison distributed a summary of the pertinent factors to be reviewed by each team member, a copy of which is enclosed for your information.

While my response to the enclosure is unequivocably negative, I wish to bring out all previous contacts, no matter how indirect or routine in nature, as follows:

1. As a consultant to NRR of NRC I reviewed the applicant submittals concerning the effect of spent fuel racks on the floor of the fuel pool. This work culminated in my offering testimony at an NRC hearing in June of 1982.

2. Harstead Engineering Associates, Inc. in January 1983 applied to CECO for a listing on their list of approved consultants. At the time we were told the decision process takes about eight months.

3. Harstead Engineering Associates, Inc. were engineering consultants to Southern Boiler and Tank Works who provided a Flow Diverter to Illinois Power for the Clinton plant. Sargent and Lundy were the plant A/E's. This work was completed over three years ago; however, there have been

a few FCR's submitted for our approval during this period of time.

4. Scot & Harstead Associates, of which I was part until July 1979, were engineering consultants to Leslie & Eliot who had a contract with Illinois Power to provide spent fuel storage racks. The work stopped in early 1980 and all outstanding fees have been paid.

In summary, no conflict of interest exists concerning my assignment with I & E on the Byron IDI.

Sincerely yours,

HARSTEAD ENGINEERING ASSOCIATES, INC.

Gunnar A Harstead

Gunnar A. Harstead
President

GAH/tm

Enclosure 2

Agreement

For proprietary and potentially proprietary information that is disclosed to me in connection with my work on the NRC's Integrated Design Inspection of the Byron Plant, I agree:

1. Not to make further disclosures.
2. Not to make further copies.
3. To return my copies to the NRC Team Leader upon completion of the Byron inspection project.

Gunnar A Harstead 5/17/83

Signature Date

Enclosure 2

CHRONOLOGY OF EVENTS RELATED
TO GUNNAR HARSTEAD AND WATERFORD AND OTHER

PROJECTS (By R. E. Shewmaker, August 28, 1984)

Beginning in May 1983, Mr. Gunnar Harstead, President of Harstead Engineering Associates, Inc. worked in my civil-structural group as a member of the Integrated Design Inspection (IDI) Team for the Byron Project. This work was done as a consultant/subcontractor to Parameter, Inc. under NRC Contract Number NRC-05-82-249 with Parameter, Inc. During the last week of field effort which was being completed at the offices of Sargent and Lundy, architect/engineer for the Byron facility, I was informed by the Byron IDI Team leader, Mr. Dennis Allison, on June 21, 1983 that it was possible I would have to leave immediately to begin to address allegations related to cracking and leakage of the basemat at Waterford 3. A series of phone calls took place between myself and IE management and the designated Waterford Inquiry Team leader, Mr. Mark Peranich, on June 21 and 22, 1983. A decision was made that I would remain through June 24, 1983 (a Friday) to complete field work on the Byron IDI and began on the Waterford Inquiry Team effort on Monday, June 27, 1983.

In the course of time between June 21 and June 24, 1983, Mr. Harstead and I discussed the design concepts of the Waterford base mat since we both had some previous knowledge and professional interest in the "floating foundation concept" which was used to address the specific site conditions along the Mississippi River. During that time my preliminary interest was what design conditions and assumptions had been used to proportion the base mat's reinforcing steel to account for positive and negative moments. Mr. Harstead indicated to me that he had worked for NRR, specifically the Structural Engineering Branch in 1981, when a structural audit was performed for the Waterford 3 facility associated with NRR's licensing review. That week long audit addressed the structural design criteria and design procedures used by EBASCO in the design of the Waterford 3 facility. Mr. Harstead indicated to me that he believed that he still had a copy of the notes he had made and provided to the Structural Engineering Branch in NRR and that if he could locate them when he returned to his office the week of June 27, 1983, he would send me a copy. He also indicated that the principal designers of the basemat from EBASCO had written and presented several papers in the professional journals and meetings related to the design and he might have a copy of one of those which he would also send.

On June 27, 1983, I initiated in-office study and preparation for a site visit to the Waterford 3 facility and to participate in the conduct of an interview with a newspaper reporter. From June 28, 1983 until June 30, 1983, I was in New Orleans or at the Waterford 3 facility. Upon returning to my office on July 1, 1983, I found that Mr. Harstead had indeed sent me a copy of his notes of his work with NRR as a member of the SEB audit (see Attachment A). In addition, he sent a copy of an article entitled, "Foundation Design of the Waterford Nuclear Plant," J. L. Ehasz and E. Radin[1] (Attachment B). I read both of these documents along with material which I obtained from the docket

[1]Presented at the Second Specialty Conference on the Structural Design of Nuclear Plant Facilities, Chicago, December 1973.

files or had been assembled by Region IV personnel and given to me by Mr. Eric Johnson, also a member of the Inquiry Team and from Region IV. The information from RIV included two additional technical articles which were published on the Waterford 3 facility related to the basemat and the foundation and structural design concepts. These articles were "Foundation Movements--Prediction and Performance", J. L. Ehasz and M. Pavone (Attachment C) and "Compatibility of Large Mat Design to Foundation Conditions," J. L. Ehasz and P. C. Liu (Attachment D). Additionally there were project reports including "Allowable Mat Bearing Pressure," M. Pavone and J. L. Ehasz, April 1977 (Attachment E) and "Review of Site Settlements," M. Pavone and J. L. Ehasz, September 1978 (Attachment F) and the FSAR and other related project documents.

As a result of my site visit on June 30, 1983 and documents reviewed, the Waterford Inquiry Team provided a report dated July 14, 1983 which included, as Item 4, the issues defined relative to the basemat cracking and leakage (see Attachment G). In addition I prepared a write up of additional information, dated July 12, 1983 for use by the NRC group which would be responsible for the resolution of the issues (Attachment H). The judgments and conclusions provided in these two documents were independent and were the result of my professional experience and the facts I gathered for this assignment. My major work on this assignment was basically completed on July 8, 1983 except for document typing, editing and signing of Enclosures 1 and 2 which were completed the week of July 11, 1983.

Beginning the week of July 11, 1983, I returned to the task of writing and consolidating input from civil-structural team member, Mr. Harstead, for the Byron project for which field work had been completed on June 24, 1983. This work with Mr. Harstead continued into July and August of 1983 with some minor work in September of 1983. The Byron IDI report was completed and issued on September 30, 1983.

To the best of my memory, some time early in the month of July 1983, in conversation with Mr. Harstead concerning the Bryron IDI report, I became aware of the fact that he had been contacted by Louisiana Power and Light Company, the licensee for the Waterford 3 Project, relative to some possible independent consulting work associated with the basemat. I had no further discussion with Mr. Harstead on the Waterford 3 project until later in July 1983 in conversation with Mr. Harstead relative to the Byron IDI report, I learned that Harstead Engineering Associates, Inc. were under contract to Louisiana Power and Light Company for studies into the basemat. At that point in time I informed Mr. Harstead that we could no longer discuss the Waterford 3 issues. Since that point in time Mr. Harstead and I have not discussed the resolution of the basemat issues for the Waterford 3 project. A docketed report[2] by the licensee indicate that Harstead Engineering Associates, Inc. personnel were involved with the Waterford 3 project at least as early as July 15, 1983 when HEA Trip Report No. 1, W3-HE-LP-001 was documented. Other trip reports documented in Reference 2 are as follows:

[2]Harstead Engineering Associates, Inc., Report 8304-1, September 19, 1983.

HEA Trip Report No. 2, W3-HE-LP-002, August 1, 1983
HEA Trip Report No. 3, W3-HE-LP-003, August 22, 1983
HEA Trip Report Nos. 4&5, W3-HE-LP-004, August 24, 1983
HEA Trip Report No. 6, W3-HE-LP-006, September 6, 1983

On September 28, 1983, I was informed that a decision had been made that the resolution of Item 4 of the NRC Inquiry Team's report of July 14, 1983 would be the responsibility of NRR. I had several discussions with Dr. John Ma of the Structural Engineering Branch in NRR who was assigned responsibility for the resolution of the issues in the civil-structural area. I also provided him relevant documents related to the basemat issue.

During the month of October 1983, I was again in contact with Mr. Harstead relative to another IDI effort. The project in this instance was the Seabrook project. For this effort Harstead Engineering Associates, Inc. provided services to IE through EG&G of Idaho Falls, Idaho. This task was conducted during October, November, and December of 1983 with the field work concluding on December 21, 1983. Mr. Harstead again worked in my civil-structural group. Report writing and related efforts extended into January and February of 1984 with the report being issued on April 2, 1984.

During March of 1984, I was involved in assisting in the planning and logistics for the preparation of the River Bend IDI which started field work on April 9, 1984. Harstead Engineering Associates, Inc. was involved in this effort. In this case as a subcontractor to WESTEC Services, Inc. who was under contract to the NRC for IDI support work. Mr. Harstead was a member of the IDI team in that effort. The field work was basically completed on May 18, 1984. The report was issued on August 29, 1984.

On April 5, 1984, I was assigned to the Waterford Task Force and arrived at the Waterford 3 site on April 9, 1984 as the discipline leader for the civil-structural area. The task was to review allegations and evaluate them for safety including those related to the basemat issues which I had been associated with in 1983. Site work was completed on May 25, 1984 and the NRC staff documentation work proceeded during June and July of 1984.

On July 30 and 31, 1984, Mr. Harstead and I worked together in Chicago on closeout issues related to the Byron IDI effort which was begun in 1983.

During July and August of 1984, I was also involved with the preparation of an affidavit for the Waterford 3 project. During this time I noted to Mr. Sherwin Turk, the ELD attorney for the Waterford 3 project, that I had notes from Mr. Harstead which he had prepared while working with NRR on a structural audit of the Waterford 3 facility at EBASCO during 1981. I was requested to provide the notes to Mr. Turk which I provided near the end of July 1984.

I am also aware that Harstead Engineering Associates, Inc. and Mr. Harstead are participating in another IDI for the Perry project with the same contractural arrangements that existed for River Bend. This project began on July 23, 1984 for Mr. Harstead with field work commencing on August 6, 1984. The inspection effort is currently under way.

During the described period Mr. Harstead has made no oral statements to me on behalf of Louisiana Power and Light Company nor any other private entity regarding the Waterford 3 facility except as noted herein. Neither have any written communications been provided to me on this subject by Mr. Harstead. I have not been in any meetings where Mr. Harstead attended as a representative of HEA or LP&L on the Waterford 3 project.

ATTACHMENT A

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

1.0 Introduction

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

The members of the NRC team are as follows:

F. Rinaldi

P. Huang

J. Matra

G. Harstead

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

2.0 General Description

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

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

3.0 Geotechnical Investigation

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

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

4.0 Mat Design

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

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

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

calculation runs were made with three sets, spring constants based upon of soil Shear Modulii of 5800, 800, and 16050 psi.

5.1.2 Stardyne Model

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

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

5.1.3 Comparison of Results

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

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

5.1.4 General Comments

a. Mode Shapes

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

b. Earthquale Combinations

Earthquake motions were considered independently as follows:

North-South
East-West
Vertical

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

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

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

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

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

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

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

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

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

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

5.0 Dynamic Analysis

5.1 Mathematical Model

5.1.1 Ebasco Model

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

The base mat is attached to the rigid base by means of soil springs. These spring constants are dependent upon the Soil Shear Modulus and geometry of the base mat. The formulas are taken from standard references. Due to uncertainity of the soil spring

2/3 of the horizontal acceleration.

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

5.1.5 Discussion of Subgrade Modulus and Vertical Soil Spring

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

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

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

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

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

5.1.6 Damping Values

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

5.1.7 Hydrodynamic Soil Effects

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

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

5.1.8 Masonary Walls

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

6.0 Reactor Shield Building

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

7.0 Reactor Containment Shell

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

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

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

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

8.0 Missile Shield Grating

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

9.0 Internal Structures of the Containment

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

10. Spent Fuel Storage Pool

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

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

11. Turbine Missiles

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

12. General Conclusions

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

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FOUNDATION DESIGN OF THE WATERFORD NUCLEAR PLANT

BY

J L EHASZ[1] AND E RADIN[2]

SYNOPSIS

This paper describes the soils' conditions and evolution of the foundation design from a simple spread footing design to a combined plant island foundation structure that "floats" in the ground. The compressible soils and settlement considerations dictated extensive studies and design effort to minimize the imposed soils pressures from the massive concrete structures. The construction is sequenced from excavation to completion with foundation soil loadings as the primary concern, especially during the construction phases.

INTRODUCTION

The Waterford No. 3 Generating Station owned by Louisiana Power and Light Company will be constructed in St. Charles Parish, on the west bank of the Mississippi River about 20 miles west of New Orleans. The Nuclear Plant Island Structure of this 1165 MW nuclear plant of the PWR type which was scheduled for operation in 1977 consists basically of three buildings, the Fuel Handling Building, the Reactor Building and the Reactor Auxiliary Building. These buildings are all incorporated into a common structure called the "Nuclear Plant Island Structure" which is supported on a common foundation mat. This mat in turn is supported on the Upper Pleistocene clays which underlie the site about 50 ft. below grade.

The Nuclear Plant Island Structure is a rectangular box-like structure 380 ft. long, 267 ft. wide and extends about 60 ft. below grade. The central portion of the structure consists of the reactor building including the reactor internal structure which is within a free standing steel containment and concrete shield structure whose outside diameter is 154 ft., and top elevation of the dome is 197 feet above grade. Finished grade is at about 17.5 ft. above mean sea level and the top of the foundation mat is 35 ft. below mean sea level.

The Nuclear Plant Island Foundation is designed according to the "Floating Foundation" or "Compensated Foundation" principle, that is, it is designed to have sufficient buoyancy to maintain soil pressures on its foundation mat which are approximately equal to and which are no greater than the existing overburden pressure (the soil pressures which existed at that level prior to the start of construction).

This paper describes the investigation programs which were undertaken to study the properties of the soils which exist at the site and which governed the design. It discusses the criteria used in the foundation design and briefly recounts several of the preliminary design concepts which were considered during the early stages of the design.

1. J L Ehasz, Supervising Soils Engineer, Ebasco Services Incorporated
2. E Radin, Principal Engineer, Ebasco Services Incorporated, New York, N.Y.

SITE INVESTIGATIONS

Extensive boring, sampling and testing of the soil conditions at the site were performed in two stages. The first phase of the work was completed in 1970 and details of that phase, including boring logs and representative laboratory tests were submitted to the Atomic Energy Commission in May of 1971. The second phase of the work concentrated essentially on soil deposits underlying the proposed Nuclear Plant Island Structure. Borings and samples were taken in late 1971, the associated laboratory testing was completed in January 1972.

Basically, the initial study program for the evaluation of the site soil properties consisted of:

a) A Boring Program

b) Soil Tests

c) Seismic Traverses

d) Electric Logging

e) Piezometer Installations

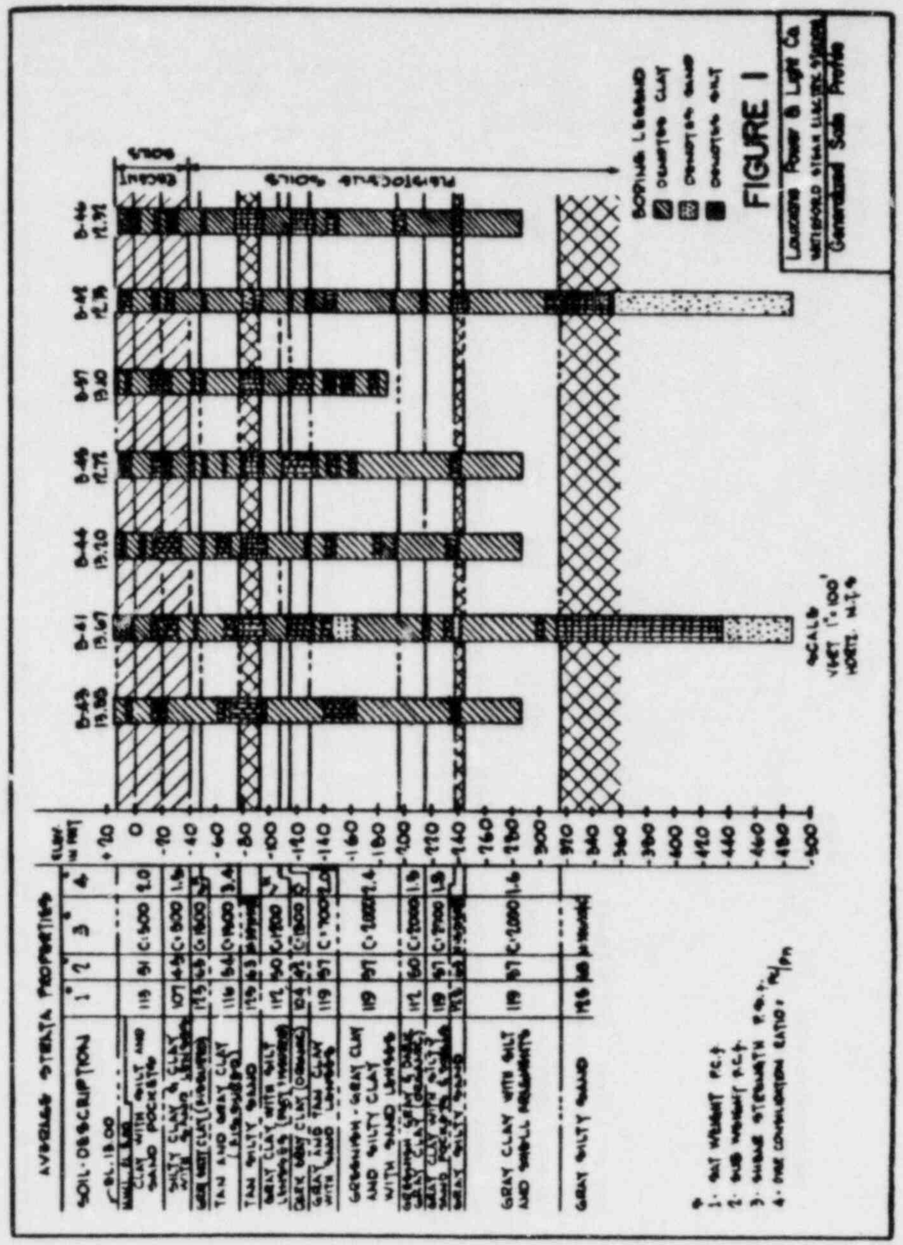
Boring program consisted of 35 borings, 23 to a depth of 100 ft, 6 to a depth of 200 ft, 4 to a depth of 300 ft, and 2 to a 500 ft depth. Most borings were concentrated in the general building area while others were taken to define the entire site. Previous knowledge of this site indicated that a minimum boring depth of 100 ft was advisable. The 300 ft deep borings are about twice as deep as the Reactor Building width and were judged to be the extent of significant soils behavior. The 500 ft borings were taken for added assurance at depth and to supply geological information.

Static soil tests were performed on selected samples in accordance with ASTM Standards. Seismic wave velocity traverses were conducted throughout the site to ensure the continuity of the soils strata for some distance from the proposed structure locations.

Fourteen piezometers were set in the several sand layers of various scattered bore holes. Piezometer readings and river stages were monitored at frequent intervals. The responses of these piezometers to the Mississippi River levels gave insite as to the continuity of the various soil strata.

The logs of the soil borings were studied on soil profiles which revealed the different strata of soils which underlie the site. Basically, five distinct zones of material were established which had to be considered in the foundation design as follows:

Zone 1, approximately the top 50 feet, is an unconsolidated mixture of soft fine grain materials, (clay, silt, sand of recent geological age).



Zone 2, from approximately 50 to 90 feet below grade, is a fairly uniform, stiff to very stiff clay, of Pleistocene (Geological) age with occasional sand lenses.

Zone 3, from about 90 to 105 feet below grade, is a medium dense sand with some clay.

Zone 4, which extends in depth from about 105 to 330 feet below grade, is essentially a repetition of Zone 2, but of somewhat stiffer clay. However, at the top of this zone, there exists a soft clay stratum, which was studied in detail as part of the second phase of the soils investigation. It was the consolidation in this soft clay which caused to the adoption of the rather severe criteria for the foundation design which are discussed later in this Study.

Zone 5, from about 330 feet to 500 feet below grade is a very dense silty sand. At elevations lower than 500 feet below grade, the geology and seismology reports indicate similar dense sands and clays for perhaps thousands of feet.

An extensive laboratory test program was conducted on the recovered samples. This program consisted of:

1) Unconfined compression tests, on samples of various diameter both in horizontal and vertical direction.

2) Triaxial tests on selected representative unconsolidated-undrained and consolidated-undrained samples from particular strata.

3) Consolidation tests on samples at specific elevations to determine:

a) Preconsolidation Pressures (P_c).

b) Overconsolidation Ratios (OCR)

c) Rebound (C_s)

d) Recompression indices (C_R) to simulate excavation and reloading sequence which will take place during construction.

e) Coefficient of consolidation (C_v) to establish time-settlement characteristics.

Continuous profiles of moisture content, density and Atterberg Limits were also obtained for individual borings. Drained and undrained triaxial tests were also conducted on samples of batture and pumped river sand, which represent the most likely source of borrow material for the required compacted granular backfill.

The upper 50 feet of materials are the recent alluvial deposits described as soft clays and silty clays with occasional sand lenses or pockets. The average cohesion is 500 lb per sq ft. At approximately 50 ft of depth, or elevation -40 ft, and extending to great depths, there is a marked change in soil strata indicating the top of the Pleistocene Age soils. The upper part of these soils are stiff, gray and tan clays with occasional silt lenses and varying degrees of fissuring. The average cohesion varies from 1500 lb per sq ft in the uppermost strata to 2000 lb per sq ft at depth.

Historically, in this southern Louisiana area, Zone 1 material has supported roads, railroads and light construction. However, the Zone 1 material is not suitable for the support of major structures, since the materials are weak and the anticipated settlements would be large and variable. All heavy industrial construction in the area is founded on Zone 2 or lower material and it was necessary that Zone 2 should be the support level for the Seismic Class 1 structures which comprise the Nuclear Plant Island.

A second phase soil study program was then undertaken with the objective of confirming the conditions described above and establishing detailed soils information. Particularly, verification of the decision to found the buildings of the Nuclear Plant Island Structure in the Zone 2 Material and additional information was sought regarding the Zone 2 material strength and long term settlement characteristics.

The detailed investigation consisted of eleven 3-inch diameter borings and seven 5-inch diameter borings extending to depths ranging from 100 - 200 feet. Continuous Shelby tube samples were taken in three borings, with the normal sample interval generally 3 feet in depth in the cohesive deposits. Split-spoon samples and standard penetration resistance values were obtained in granular or coarse-grained deposits, and occasionally in transition materials. Samples were retained for laboratory analysis. These clays extend to about elevation -320 ft and contain only two significant and continuous silty sand strata. One is at about elevation -77 ft to elevation -92 ft. The other is at elevation -235 ft to elevation -245 ft. These silty sands are dense to very dense as can be seen by high standard penetration test results. The strata below the stiff clays, from elevation -320 ft to at least elevation -500 ft (the deepest elevation penetrated), is a very dense grey silty sand, as can also be seen from penetration resistances.

The following is a brief summary of the major Pleistocene soil strata indicated on the generalized soil profiles; shown in Figure 2.

a) Elev -40 ft to Elev -77 ft (Stiff tan and grey fissured clay)

A significant increase in strength is noted in this strata indicating the line of demarcation between recent and Pleistocene deposits. The relatively high over consolidation ratio (OCR) value of 3.4 also represents a stage when the clay stratum was subjected to the combined effect of desiccation and overloading. Evidence of fissuring was observed in the majority of the recovered samples although generally minute silt and sand lenses were noted. Test data generally indicated no significant decrease in strength with increasing diameter of sample. The cohesion value of this stratum was found to be 1500 psf.

b) Elev -77 ft to Elev -92 ft (very dense tan silty sand)

This strata was encountered in all borings and generally averaged approximately 15 ft in depth. Penetration resistance values, "N", indicated that the silty sand was in the dense to very dense range; the "N" values were greater than 30 blows/foot. Occasionally lower values were recorded. However, on investigation, they represented transition zones between overlying and underlying clays. Extensive cyclic triaxial testing was conducted to verify the dynamic strength of these transition materials and studies concluded that these materials will not liquify from the dynamic stresses imposed by the design earthquake.

c) Elev -92 ft to Elev -108 ft (Medium stiff grey clay with silt lenses).

A noticeable decrease in average OCR (1.4) and slight reduction in strength (C = 1200 psf) is apparent, when compared with material overlying the sand stratum. Locally isolated clay samples had indicated OCR values as low as 1.1. These values are most probably related to changes in ground water level, limited desiccation due to the overlying sand stratum and the geologic history of the deposition.

d) Elev -108 ft to Elev -116 ft (stiff dark grey clay, organic)

Organic content determinations in this layer ranged from 3 to 16 percent. Noticeable increase in strength (C = 1800 psf) and OCR (1.7) are related primarily to effects of desiccation.

e) Elev -116 ft to Elev -127 ft (Medium stiff gray and tan clay with sand lenses)

Reduction in strength noted is a result of saturated silt and sand pockets and organic content of overlying material.

f) - Elev -127 ft to Elev -317 ft (Very stiff clays with silts and sands).

Deposits characterized by relatively high uniform strength (C = 2000 psf) and low to medium over consolidation ratios (1.5 - 2.4). Sand deposits are generally in the very dense range. Organic content determinations in a layer of dark grey clay extending from Elev -197 to Elev 217 ranged from 4 to 7 percent.

g) - Elev -317 ft to Elev -500 ft (Very dense sands and silty sands)

This strata consists of uniform coarse-grained deposits generally in the very dense range with "N" values greater than 50 blows/foot.

Of particular concern from a construction and dewatering standpoint was the continuity of the various strata, in particular the sandy strata, and possible communication with the Mississippi River. By reviewing many profiles as well as additional soils information from the adjacent Waterford Unit 1 and 2 site, just north of the subject site, it was established that the upper soils are quite discontinuous; the sandy strata could not be correlated along profiles of the borings. However, the Pleistocene soils were found to be continuous and predictable. These facts were further confirmed by piezometers placed in various silty sand strata and monitored for 12 to 18 months. It was found that the sandy strata within the recent soils were not responsive to river level fluctuations. However, the piezometric levels in the silty sands within the Pleistocene were directly affected by the Mississippi River level. Thus, it is certain that these soils are continuous and connected to the river at this location.

FOUNDATION DESIGN CONCEPTS AND STUDIES

As discussed earlier and based on the results of the soil study program, it was necessary to found the buildings of the Nuclear Plant structures on the stiff Pleistocene clays which underlie the site. Several foundation design concepts were investigated and discarded as their shortcomings became evident. Among those design concepts were:

a) Pile Supported Foundations

b) Individual mat foundation under each building

c) Combined mat foundation supporting independent buildings

Each of these concepts is briefly discussed below as follows:

A - Pile Supported Foundation

Heavy industrial construction in the area is founded upon piles which extend into the Pleistocene Clays of Zone 2 or lower. The Waterford Units 1 and 2, which are fossil-fueled power plants adjacent to this site, are so founded. However, this type of construction is not readily adaptable for nuclear power reactor foundations in the Mississippi River Delta. The horizontal forces which must be considered in the Seismic design of nuclear power structures are so large as to be incapable of being resisted by bending of vertical piles especially with the loss of support from the recent soils surrounding the piles during earthquake conditions. These forces if carried axially in battered piles would require an extraordinary amount of batter piles, if indeed, sufficient piles could be placed at all to resist these loads. For these reasons, the piled foundation concept was discarded for the nuclear plant structures. The pile foundation concept was the only one examined which would allow at grade construction of the plant. All others required excavation to the top of the pleistocene and construction of a substantial portion of the plant below grade.

B - Individual Mat Foundations Under Each Building

This was the initial approach to which serious consideration was given. The original design specifications which were established for the foundation design and based upon the results of the Phase I soils investigation contained the following criteria:

1) Top of Pleistocene sediments - Elevation -35.5 ft
2) Ultimate Soil Bearing Pressure - 15 KIPS/sq ft
3) Maximum Design Bearing Pressure for any loading combination = 11.25 KSF
4) Maximum Total Allowable Settlement - 3 inches
5) Maximum Relative Settlement Between the Buildings = 1 inch
6) Maximum Allowable Angularity (out of plumbness) between adjacent structures = 1/20 degree

The severe requirements of Criteria 4, 5 and 6 were necessary because the buildings of the Nuclear Plant Island Structure are in fact inter-related and interconnected with piping, fuel transfer canals, etc. It was not possible to meet these criteria by means of individual foundations under each building. As a matter of fact, preliminary computations indicated that differential settlements from 3 to 6 inches could be expected for structures on this type of foundation because of the existence of the slightly over-consolidated pleistocene clays at deeper elevations. These anticipated large, long term, differential settlements led to the elimination of the concept of individual foundations under each building and led to development of the combined mat considerations.

C - Combined Foundation Mat Supporting Independent Buildings

This concept consists essentially of a large flat common reinforced concrete foundation slab under the independent structures supported upon it. It is usually designed to be rigid enough to act as a unit to insure uniform settlements and sufficient mass of concrete is included to provide factors of safety against sliding and overturning. Reinforcing steel is provided to resist the bending moments and shear stresses caused by the net soil bearing pressure acting against the underside of the mat. This was the concept when the early attempts were made at the design of a common foundation mat.

The common foundation mat concept was adopted in an attempt to minimize the differential settlements which were anticipated for individual footings under each building. It became necessary to establish new design criteria which were applicable to the common foundation concept, select a trial shape for the new foundation, and check this trial shape (by both hand and computer computations) to determine whether the newly established criteria were met. It was also necessary to establish a construction sequence for the Nuclear Plant Island structures in order to assure that the design criteria were not violated during any construction stage.

The most critical aspect of the design criteria was the requirement that maximum and minimum soil bearing pressures should not differ by more than two kips per sq ft for the construction conditions and one kip per sq ft for the final condition. This was established in order to control tilting or uneven settlement of the foundation mat and was an attempt to translate into allowable soil bearing pressures the previous criteria for individual foundations under each building which pertained to maximum allowable relative settlements between buildings and maximum allowable angularity or out of plumbness between adjacent buildings. In other words, the new criteria sought to assure as uniform a settlement as possible by requiring as uniform a soil pressure distribution as possible.

The first attempts at design of a common foundation mat with independent buildings revolved around a six-sided shape. The assumption was made that the mat was rigid and maximum and minimum static soil bearing pressures were calculated. Various arrangements of buildings were studied with respect to soil bearing pressures computed for both this shape and a rectangular shaped foundation under various assumptions regarding backfill, buoyancy and the position of the reactor building on the mat. None of these trial shapes were able to satisfy the established limit of one kip per square foot on the differential between maximum and minimum soil bearing pressures. This was primarily due to eccentricities which were generated by two factors as follows:

1 - The arrangement of the buildings was such as to cause a sizeable eccentricity between the center of gravity of the vertical loads acting down on the mat and the centroid of the mat.

2 - The arrangement of the buildings was such that either for the six-sided or rectangular shape, large eccentricities were caused by the weight of the backfill on that portion of the mat not covered by the buildings.

Several other trial shapes were attempted, for example, extending the mat in an attempt to bring the centroid of the mat closer to the center of the loads, but all such trials proved to be unworkable because of the soil pressure differentials caused by the eccentricity due to backfill. In short, it was not practically possible, using the necessary general arrangement of the buildings to determine a mat configuration which would reduce the eccentric loads due to backfill to values small enough so that the established criteria regarding soil pressure differentials could be met. Finally, the "Floating Foundation" was the only solution which could satisfy the rigid design requirements. Simply stated, the philosophy was this: If the eccentric loads due to backfill are intolerable, then keep the backfill loads off the foundation. In order to accomplish this, a rectangular mat was designed, exterior walls were included buttressed by counter forts and a series of buoyancy chambers were created. By strategic saturation of the backfill around the perimeter of the foundation structure, it was possible to keep the effective pressure at the base of the mat close to the existing overburden pressure.

Computer Analyses of Mat Foundation

As has been previously mentioned, early analyses of the various foundation mats revolved around flat plate type of considerations. The early computer analyses which were performed showed that the mat did not behave as a rigid structure and that increasing the mat thickness had no effect on the relative rigidity. Trial computations had been run for 10 foot, 12 foot and 15 foot thicknesses; the 12 foot thickness which was finally chosen was an economic compromise between the cost of additional concrete to eliminate shear reinforcing and allowing some shear reinforcing.

Later on, as the criteria were more closely defined, and the design evolved into the floating foundation concept consisting of a rectangular foundation mat, buttressed walls and a roof slab, it became evident that for such a complex structure, only a finite element model of the combined structure could adequately demonstrate the interaction between buildings, walls and mat.

The Stardyne finite element computer program was chosen because out-of-plane shears were required and because it had no limitation on the matrix bandwidth. The structure was represented by an assembly of 1082 beams, 2079 plates and 940 nodes. The maximum capacity of the program was 1000 nodes. The soil under the mat was represented by linear springs at every node having a stiffness equivalent to 150 pounds per cubic inch. An iteration proceedure involving soil pressure and local settlement considerations to establish a realistic foundation modulus or spring was utilized. This procedure was continued until compatability of the anticipated recompression settlements and foundation modulus were obtained.

An analysis of the in-plane shears showed that extra reinforcing (up to 1.65 square inches per foot) above that necessitated by the rigid mat analysis was required in selected areas of the combined structure. The bending and shear reinforcement in the retaining walls, the slab at elevation 21.0 and the buildings was determined by separate analysis or local bending under more severe load conditions.

FINAL FOUNDATION DESIGN SUMMARY

The existing soil conditions at the site are evaluated in terms of vertical effective stresses at the present time. These stresses are now in the order of 3300 lb per sq ft. Figure 2 illustrates the study of various construction and stress conditions. The first construction stage illustrates the pressures upon completion of excavation to 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 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 of elevation +8 ft. The final pressures are indicated. It can be seen that the pressures will be 3100 lb per sq ft. This is 200 lb per ft less than the

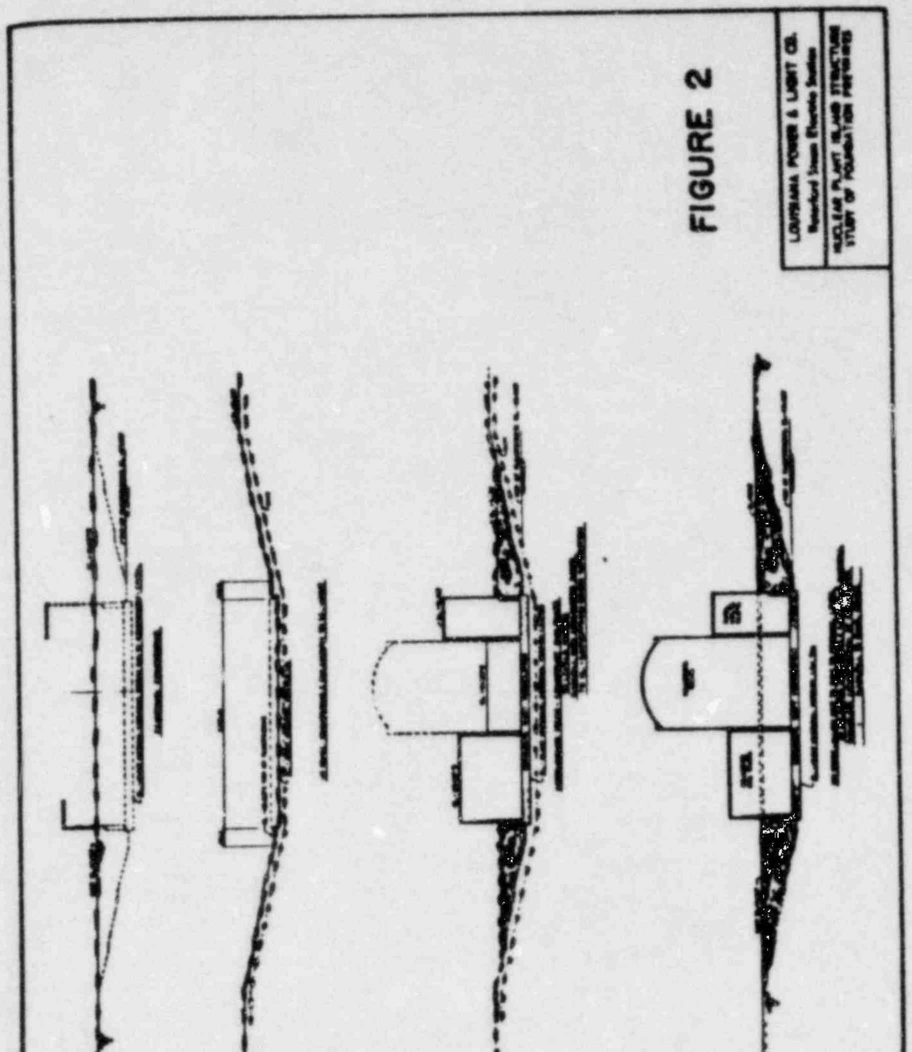


FIGURE 2

existing soil pressures at the site.

Since this "floating foundation" concept involves the balancing of existing site soils pressures, a soil pressure time history diagram 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 a maximum pressure that is desirable for short term considerations during reloading. This is based on the reconsolidation characteristics of the soils and is 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 reduces the effective soil pressures and maintains the effective pressures below the 4000 lb per sq ft level and establishes final effective pressures as described above.

Detailed construction phases are given particular close attention. Each construction phase corresponds to the phase outlined on the aforementioned soil pressure time history diagram. These phases involve 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 considers 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 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 filter blanket of locally available compacted 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 1000 lb per sq ft overload over 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, and subsequent excavation of the stiff Pleistocene clays will result in a heaving of the final exposed clay bearing strata amounting

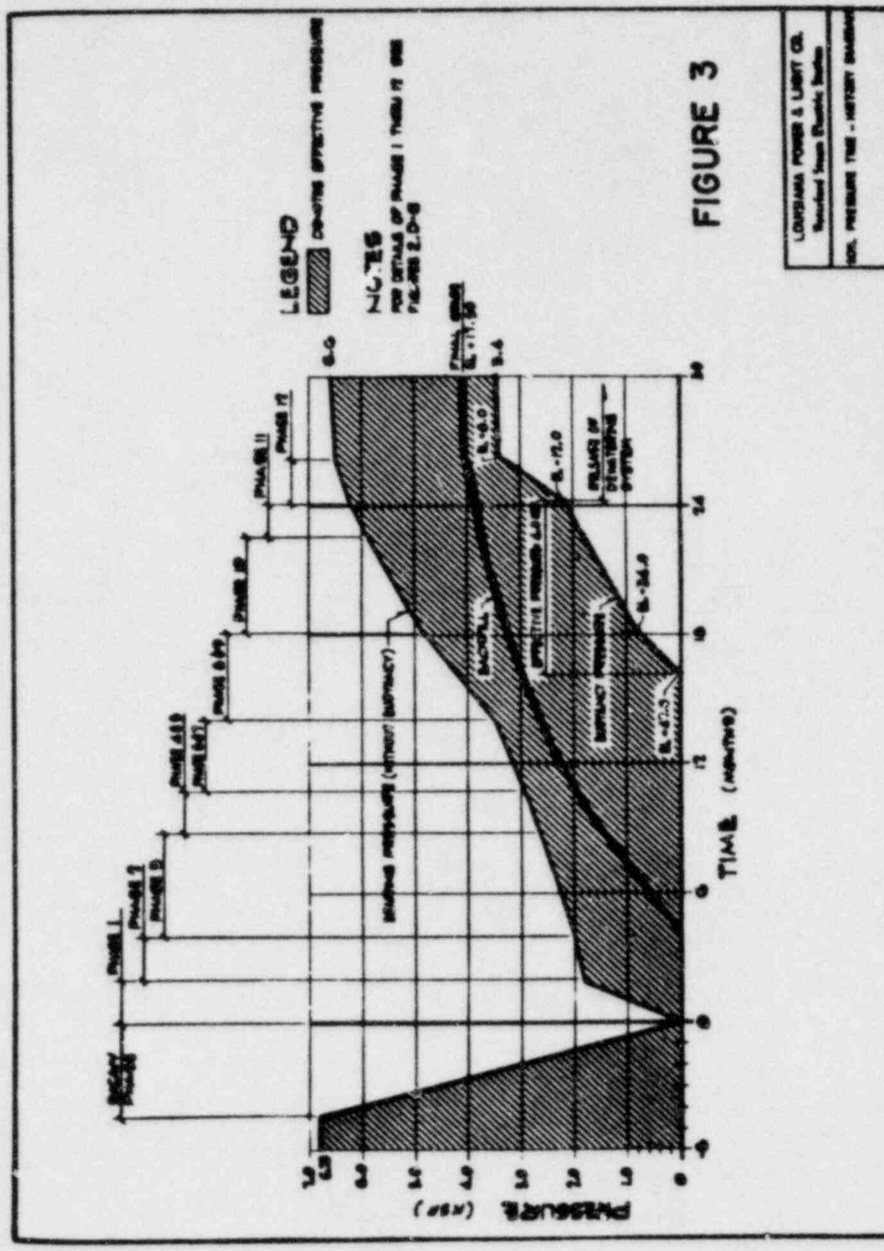


FIGURE 3

to approximately 6 in. over the excavation period. The major portion of the rebound will occur during the final excavation stages of the Pleistocene clays. Control will be achieved by excavating in increments and by rapid concrete placement in designated sections of the mat in a predetermined sequence to minimize heave.

d) By conforming with the floating foundation principle, settlement of the Class 1 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 and heave. 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 loading diagram illustrated graphically in Figure 3 shows that, after a total load of 4000 lb per sq ft has been applied, granular backfill which may have already 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 construction phase, a dewatering system will be 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 will be located in the silty sand stratum extending from approximate elevation -77 ft to elevation -92 ft to relieve the hydrostatic pressure at this level.

A series of recharge wells will also be located around the parameter of the mat foundation extending through underlying clays and silty sand stratum. It is concluded that the combination of dewatering and recharge wells will provide additional control, if required, in minimizing heave and recompression. 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 plates, will be installed to monitor heave and recompression settlement of the mat foundation.

In order to assure meeting the design objectives, detailed excavation specifications and drawings have been prepared. The slopes have been established based on the soil properties determined from laboratory and field tests. The excavation specification details the construction of the concrete mat foundation such that it minimizes the exposure of the stiff clays at the base of the foundation. In order to assure uniform pore pressure redistribution in the clays beneath the mat, upon relieving the dewatering system, a filter media composed of compacted shell will be utilized as a working surface beneath the concrete mat construction. In addition an instrumentation system will monitor soil conditions. This instrumentation system involves heave points, piezometers, observation wells, slope indicator type instruments and alignment markers to monitor the entire excavation.

Since the floating foundation concept will not induce differential soil pressures, and 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.

CONCLUSION

This paper has described in some detail the soil conditions at the Waterford No. 3 site and the evaluation of the foundation design. The essential point to be made here is that Nuclear Plant Structures are by their nature massive structures and demand special attention with respect to soils and foundation design. In particular, the minimizing of imposed soil loadings is essential when rigorous requirements must be maintained to ensure limited displacements of interrelated foundations and structures. This attention has only been directed toward static considerations in this paper; however, a careful and detailed dynamic analysis of both foundation and structure have been considered.

ATTACHMENT C

Rec'd 6/23/83
from Eric Johnson RIV

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Foundation Movements — Prediction and Performance

Les Déplacements de Fondations — la Prédiction et l'Observation

J.L. EHASZ Chief Consulting Engineer, Ebasco Services Incorporated, New York
M. PAVONE Engineer, Ebasco Services Incorporated, New York

SYNOPSIS This paper addresses the accuracy of predicting foundation heave and settlement and evaluates the influence of changing construction activities on these predications. An actual case history will be presented in which foundation movements were computed during design and then subsequently measured during construction. Compensating design modifications, during construction, which minimized the influence of construction activities and obtained a closer agreement between predicted and actual measured foundation performances are also discussed.

INTRODUCTION

The case history utilized is a completed nuclear power plant located on a deep soil site in the southeast United States adjacent to the Mississippi River. In situ soil conditions consisted of an upper fifteen meters of soft clays and silty clays geologically categorized as Recent material. This Recent material was subsequently excavated for the plant construction and replaced with compacted sand backfill. Beneath the Recent material, located at a constant depth of fifteen meters, is the foundation support stratum, geologically categorized as Pleistocene soil. The upper Pleistocene soil primarily consists of stiff clay to a depth of one hundred meters interrupted at a depth of thirty meters by a dense silty sand stratum. Beneath the stiff clays, a continuous dense sand is present to a depth of at least one hundred fifty-five meters (the depth penetrated by the deepest boring).

The typical subsurface profile is shown on Figure 1.

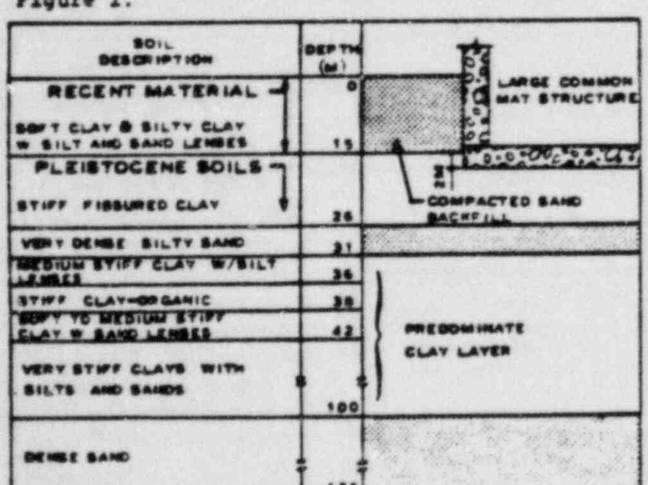


Fig. 1 Subsurface Profile

The existance of soft to medium stiff clays at a depth of thirty-eight meters indicated that significant long term and differential settlements could be expected for heavily loaded structures founded on individual spread footings. To eliminate potential post-construction settlement considerations, the design concept utilized the large common mat floating foundation shown on Figure 2.

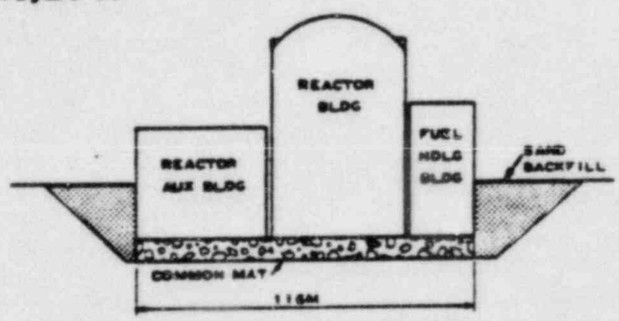


Fig. 2 Floating Foundation Sectional View

This concept resulted in effective vertical stresses at the mat bearing level that are essentially equal to pre-construction in situ stresses, thus precluding any long term settlements.

The common mat has dimensions 116 meters long by 82 meters wide and is embedded two meters into the Pleistocene soil which is approximately seventeen meters below grade (see Figures 1 and 2).

PREDICTED FOUNDATION MOVEMENTS

Predicted foundation movements discussed herein are essentially limited to those that occurred during construction of the plant. Since the design dictated that post-construction foundation effective stresses not exceed the in situ effective stresses there were no predictions made for long-term movements.

In order to fully appreciate the complexity of predicting construction-related foundation movements for this project a brief discussion of the assumed construction sequence and resulting effective stresses at the mat bearing level will be discussed. These assumed effective stresses as well as the assumed construction sequence are shown on Figure 3.

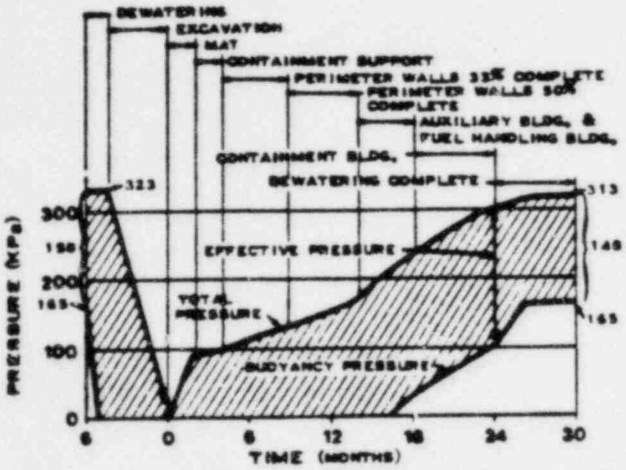


Fig. 3 Assumed Foundation Stresses

The in situ effective stress at the mat bearing level was 158 kPa. With the completion of excavation to the bottom of the mat level (Time T=0 months) this stress was assumed to become zero. For the following construction period of approximately sixteen months it was assumed that the various structural components as well as backfilling around the combined structure would be placed while the site remained dewatered. This was to have resulted in an effective stress of 192 kPa or a 34 kPa overload above in situ conditions. The overload was to be maintained throughout a significant remaining portion of the construction until Time T=24 months through careful manipulation of buoyancy forces. The intent of the overload period was to recompress any heave of the foundation soils associated with the initial excavation. The final effective stress magnitude of 148 kPa (10 kPa less than in situ) was assumed to occur 30 months after the completion of excavation.

The foundation conditions at the site were determined through an extensive boring and testing program, the results of which are generalized on Figure 1. Results of the borings performed in the plant area during the investigation program indicated that the subsurface profile was uniformly horizontal as indicated on Figure 1 so it was therefore decided to utilize this idealized profile in performing heave and settlement computations.

Several one-dimensional consolidation tests with unload-reload cycles were performed in each stratum excluding the sand strata. The various consolidation parameters from each test were reviewed and averaged for each stratum. Based on this review it was decided to utilize consolidation tests performed on samples taken in each stratum from one boring as being representative of the entire soil column.

In performing the analysis no long-term movement was assumed to occur in the sands. It was also assumed that these sand strata and the sand backfill replacing the Recent material around the plant structure would act as drainage layers during consolidation.

A Boussenesq stress distribution analysis was performed to evaluate the reduction of vertical stress at the centers of each stratum due to the excavation.

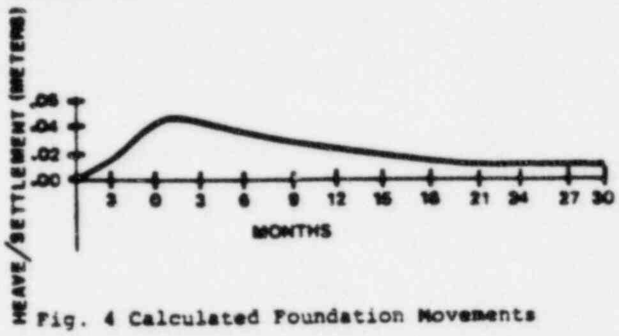


Fig. 4 Calculated Foundation Movements

Anticipated displacements were arrived at by first performing one-dimensional heave/settlement analyses for total movements. With these values determined, a time-settlement analysis was then performed to determine the percentage of the total movements that would be experienced for the assumed construction schedule shown on Figure 3. Within this construction time period the displacements represent approximately twenty percent of the total calculated movements.

Summarized on Figure 4 are the results of these heave and settlement computations. The results indicate a foundation soil heave of 0.05 meters as a result of excavation followed by a recompression of 0.04 meters due to the placement of structural and backfill loads. This results in a net calculated foundation displacement 0.01 meters upward.

MEASURED FOUNDATION RESPONSE

To monitor and control foundation movements during construction and to implement the construction loading criteria a comprehensive instrumentation program was implemented prior to the start of construction. Movements of the structures during construction were monitored from survey points on the common mat as well as on each building as construction advanced. To measure foundation soil movement, nine Burros anchor heave points were installed around the periphery of the structure and two extensometers were installed beneath the structure. Twenty-seven piezometers were used to monitor piezometric pressures in the Recent material and in the Pleistocene sediments.

Presented on Figure 5a is a complete plot of actual measured foundation soil movements for the construction period. The plot is of heave numbers H1 thru H4. These particular instruments were shown because of their being representative and because they are the only devices

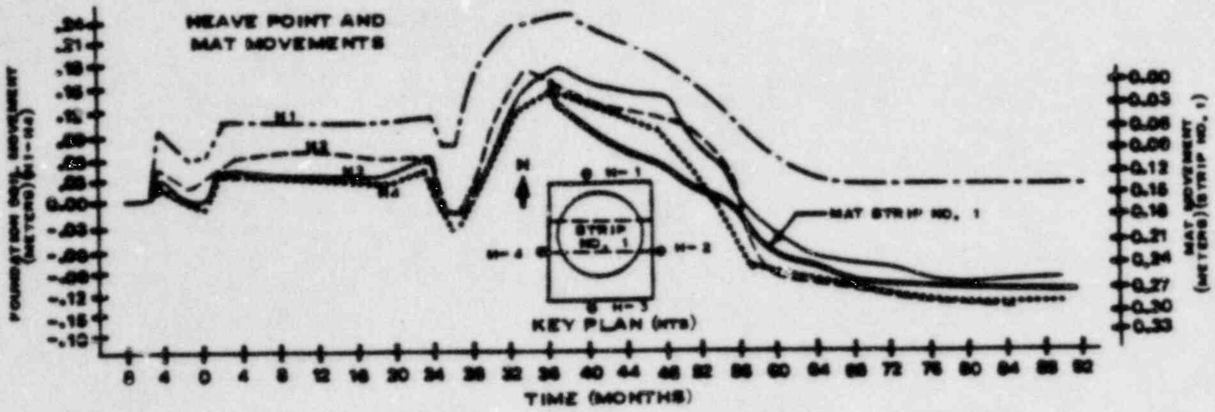


Fig. 5a Measured Foundation Movements

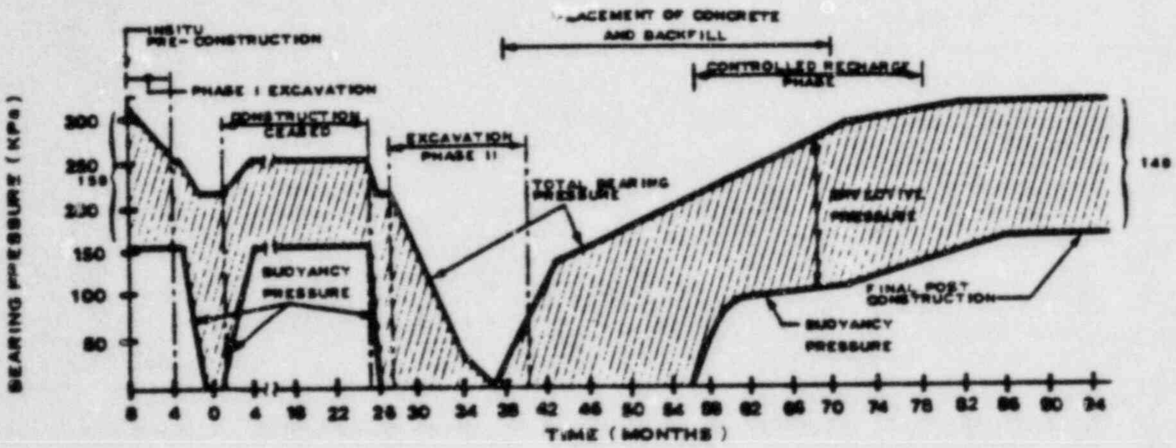


Fig. 5b Actual Foundation Stresses

that were in place prior to the start of any construction activity and remained operative throughout the entire construction period. Also shown on Figure 5a is average movement experienced by the structure and surrounding foundation soils.

Figure 5b shows the actual stress history experienced at the mat bearing level which produced the measured movements in contrast to the assumed stress history shown on Figure 3.

With the start of the Phase I excavation the foundation materials exhibited between 0.03 and 0.09 meters of heave. This heave was the result of excavating 6 meters of material without the beneficial effective stress balancing by site dewatering. With the subsequent start-up of dewatering, approximately 0.03 meters of this heave was recompressed. However, the dewatering system was not in operation long enough to balance the Phase I excavation and therefore, the release of the dewatering system due to an unanticipated cease of construction caused elastic foundation rebound to its predewatering position. The post-shutdown heaves stabilized and remained between 0.03 and 0.09 meters until the resumption of full dewatering in month 24. Between months 24 and 25 the dewatering system was fully operative again and consequently, approximately 0.05 meters of previous foundation heave was recompressed. In month 27, the remaining excavation work (Phase II) was started and continued until month 40. As a result, foundation heaves increased to between 0.122 and 0.244 meters. The commencement of concrete placement and backfilling in month 38 resulted in recompression of the heaves incurred during the excavation phases.

The average heave readings at the site were recompressed to their initial zero readings by month 55. The continued maximum running of the dewatering system until month 56 resulted in an average controlled movement of the heave points to an average net settlement of 0.03 meters. Subsequently the rate of settlement of these heave points was reduced and further controlled by the throttling down of the dewatering system (designated on Figure 5b as the controlled recharge phase). The settlement rates were further diminished throughout the recharge phase which lasted until month 78 at which time the foundation movements virtually leveled off. The post-construction status of foundation movements consists of a net downward displacement which averages 0.122 meters.

DISCUSSION

The predicted post-excavation heave was initially calculated to be 0.05 meters occurring across the site followed by a nearly complete recompression of this heave back to the "zero" initial position. Experienced, however, was a two to four fold increase of actual heave followed by a complete recompression of this heave and ultimately, an overall net downward displacement of 0.122 meters.

These differences are attributed to several factors. In general, the predicted heave was based on an assumed uninterrupted excavation period of approximately five months followed by a rapid (thirty month) application of load on the foundation soils as a result of construction and backfill placement. The actual construction progress occurred over a considerably longer period of approximately ninety months including a twenty-four month interruption due to project shut-down (see Figure 5b). This unanticipated extended construction period allowed a considerably higher percentage of the ultimate heave to occur.

Several more specific factors caused the predicted movements to be underestimated. As previously discussed, the initial Phase I excavation, which consisted of removing 6 meters of soil without the beneficial lowering of piezometric levels, decreased the effective pressures at the mat bearing level. The original predictions envisioned a compensation of effective pressures at the start of excavation by assuming a completely operating dewatering system prior to the start of excavation. This would have greatly reduced or even eliminated the actual heave associated with Phase I excavation.

An additional reason for the higher-than-anticipated heaves, particularly during the Phase II excavation, is felt to be attributed to the possible existence of thin continuous drainage seams of silty sandy soil in the 70-meter-thick predominately clay layer shown on Figure 1. Silty sandy soils were encountered in some of the borings but their occurrence was not conclusive enough to assume that they formed continuous drainage layers. It is reasoned that if these apparent discontinuous seams were acting as drains within a stratum that was assumed to have drainage only at the top and bottom, the heave process would have been accelerated. During the excavation time period a greater percentage of the ultimate heave than the twenty percent calculated would have occurred, perhaps on the order of seventy to eighty percent.

Negligible differential movements were predicted for the entire site because excavation, construction and piezometric effects were assumed to act uniformly across the site. Subsurface conditions were also assumed to be laterally uniform. The higher differential heave experienced at the north end of the site (heave point H1) is attributed to the excavation procedure which essentially handled the material from north to south, as well as the fact that the grade along the south, east and west sides of the excavation was raised 1.5 meters for construction facilities. The north side was not surcharged with the additional fill which in essence, permitted more heave along the north. Additionally, the piezometric pressures along the north were always somewhat higher due to the recharge from the adjacent Mississippi River.

The overall net downward movement of 0.122 meters below the initial "zero" position was intentional, carefully controlled, and was the result of revised thinking during construction. Since a considerably greater amount of heave than anticipated was experienced, it was decided to maintain a completely dewatered site four months longer than originally planned while construction proceeded. This essentially overloaded the foundation soils approximately 48 kPa. It was felt that more control could be exercised over the final leveling of foundation movements if the site was preconsolidated to an overall net settlement position prior to recharging piezometric pressures. It was therefore decided while the site was experiencing 0.03 meters of settlement (Time T = 56 months) to start releasing the dewatering system. The extent of the release was carefully coordinated with further increases in the construction and backfill loading so an overload was maintained, but of continually decreasing magnitude until Time T = 79 months. Beyond that time, no further settlement or heave was experienced at the site, and in fact, the actual differential settlement across the common mat never exceeded 0.05 meters.

CONCLUSION

The importance of maintaining and closely monitoring a comprehensive instrumentation system is to be emphasized. The measurements were performed by qualified engineering technicians with a technical understanding of functioning of the instruments. An evaluation of the measurements immediately followed by engineers thoroughly familiar with the foundation design concept. A general frequency of monitoring the instrumentation was continuously modified and increased to closely measure specific construction activities.

Performing heave/settlement calculations using classic one-dimensional consolidation theory were important as a guide to the engineer and provided a general appreciation of the range of movements to be expected. The magnitude of the movements were within the range of the calculations; however, both the time-related consolidation and heave functions as well as the drainage conditions were extremely difficult to define.

The ultimate assurance that the foundation design concept was being satisfied was through the ability to control the physical environment within which the system operated and to adjust the foundation construction to meet the intent of the design.

ATTACHMENT D

COMPATIBILITY OF 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, illustrates 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.

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.

1. J L Ehasz, Supervising Soils Engineer, Ebasco Services Incorporated, N.Y., N.Y.
2. P C Liu, Principal Engineer, Ebasco Services Incorporated, N.Y., N.Y.

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 be achieved by excavating in increments and by rapid concrete placement in designated sections of the mat 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 soils 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 (DBG). 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

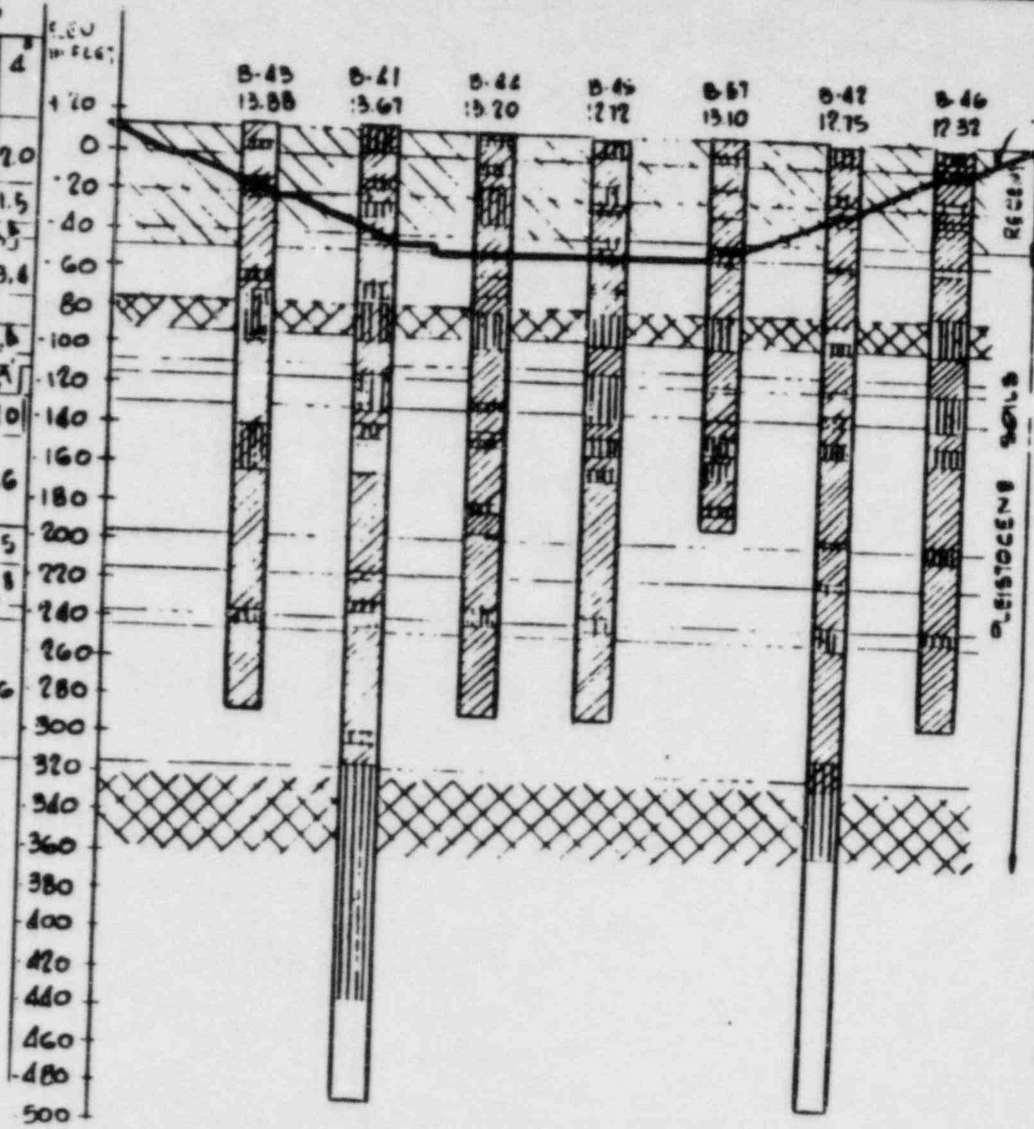
The implementation of the design-construction condition was studied very carefully to eliminate any overstress of the subsoil and to maintain mat stability from differential settlement and tilting. Each construction stage was established to meet the requirements of the net and the allowable differential 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 13.00				
MWL 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	1.5
TAN AND GRAY CLAY (FISSURED)	116	54	C:1500	3.4
TAN SILTY SAND	125	63	N:50+	
GRAY CLAY WITH SILT LENSES (PART FISSURED)	112	50	C:1900	1.6
DARK GRAY CLAY (ORGANIC)	108	42	C:1800	1.3
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	57	C:2000	1.8
GRAY SILTY SAND	125	65	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.

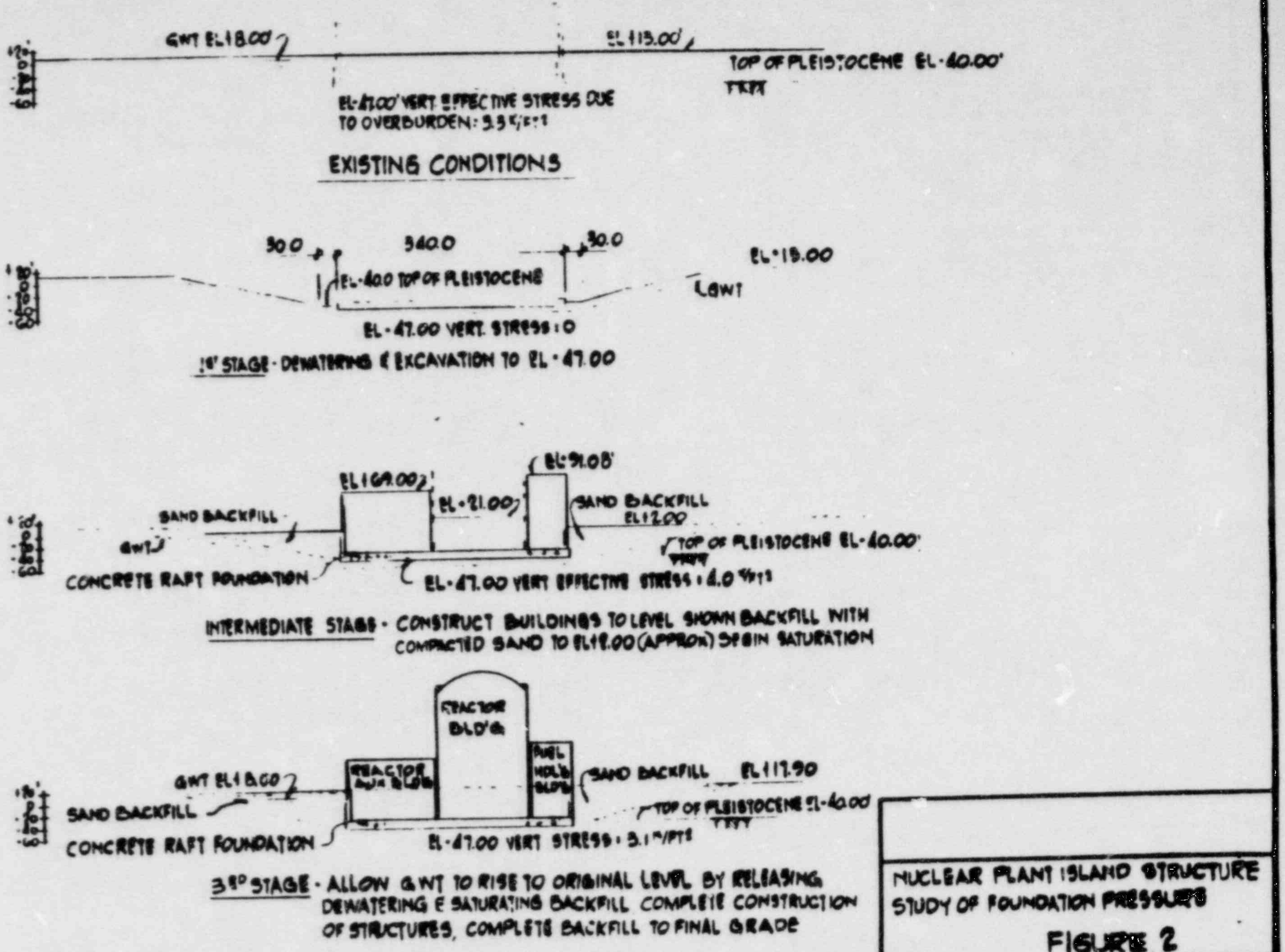
3- SHEAR STRENGTH P.S.F.

4- OVER CONSOLIDATION RATIOS P_c/P_o

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

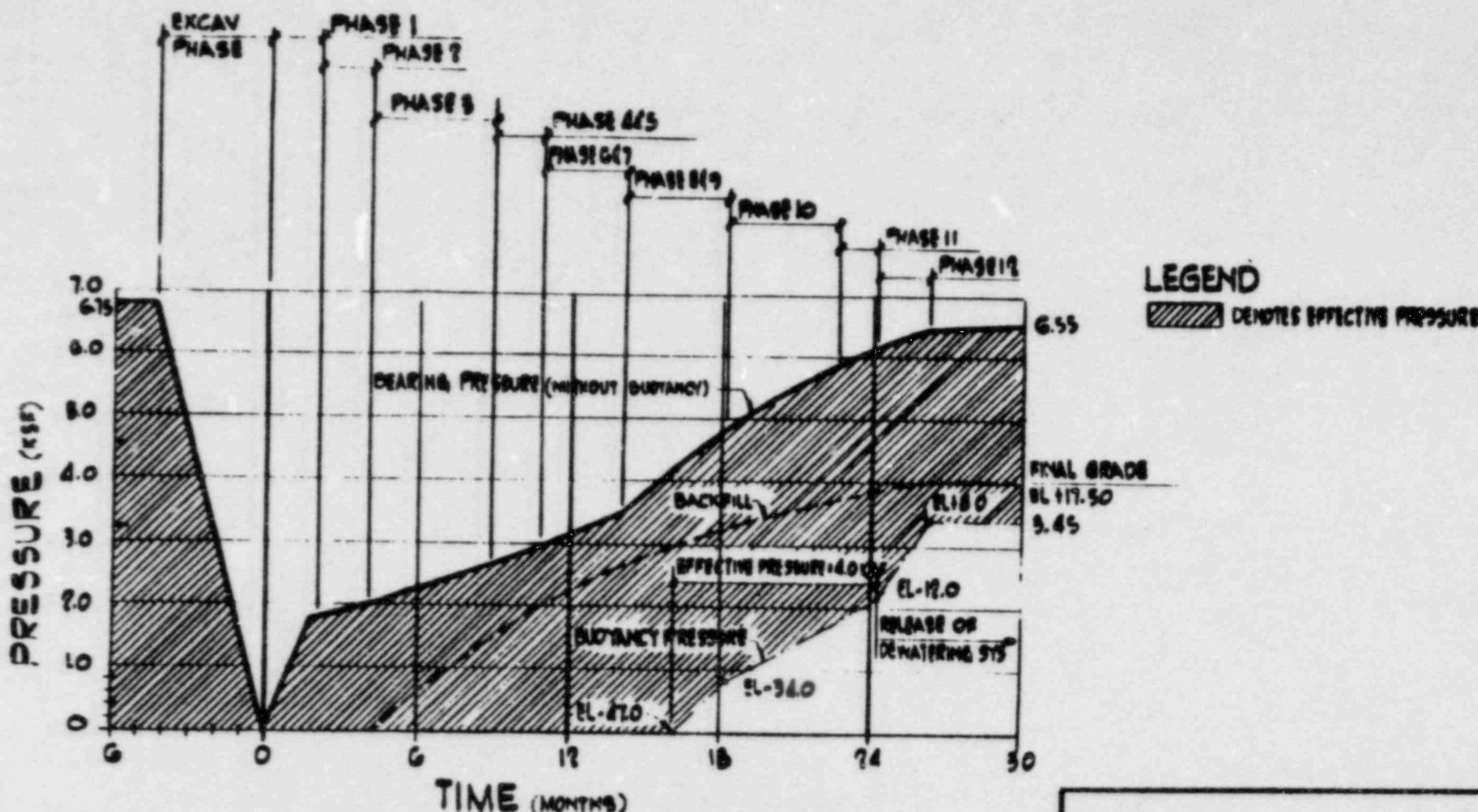
GENERALIZED SOIL PROFILE

FIGURE 1



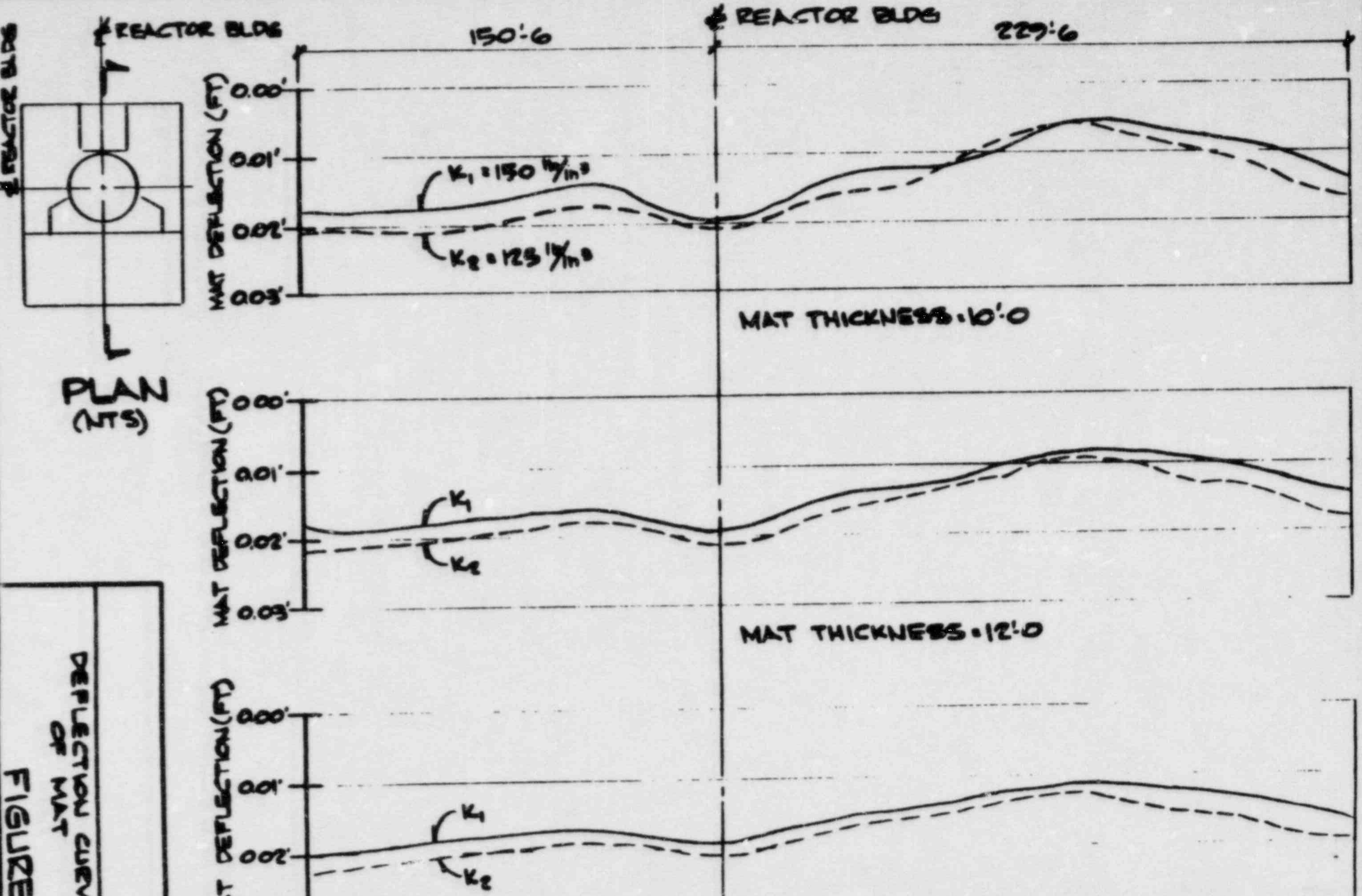
NUCLEAR PLANT ISLAND STRUCTURE
STUDY OF FOUNDATION PRESSURES

FIGURE 2



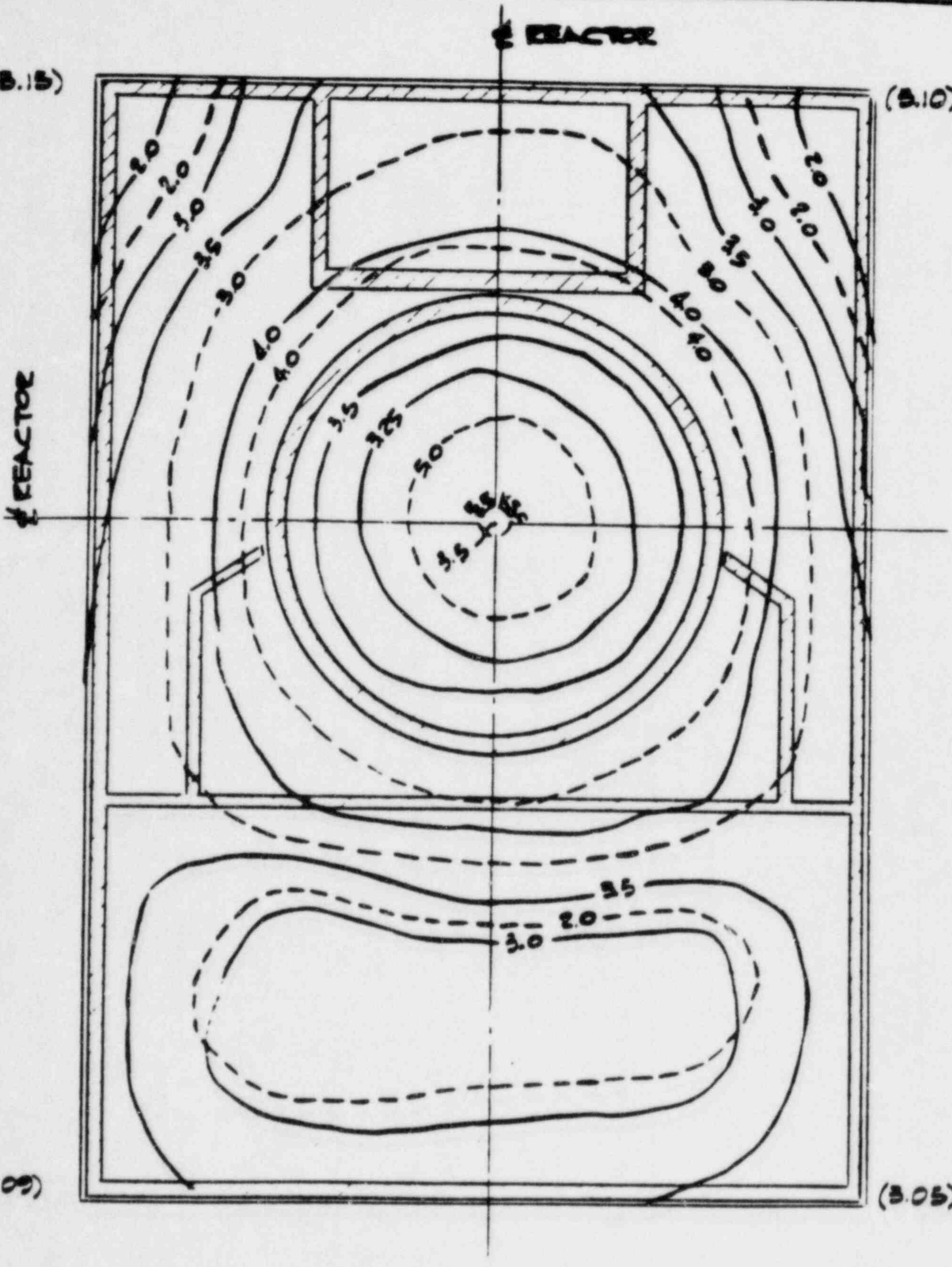
SOIL PRESSURE TIME-HISTORY DIAGRAM

FIGURE 3



DEFLECTION CURVE OF MAT

FIGURE 4

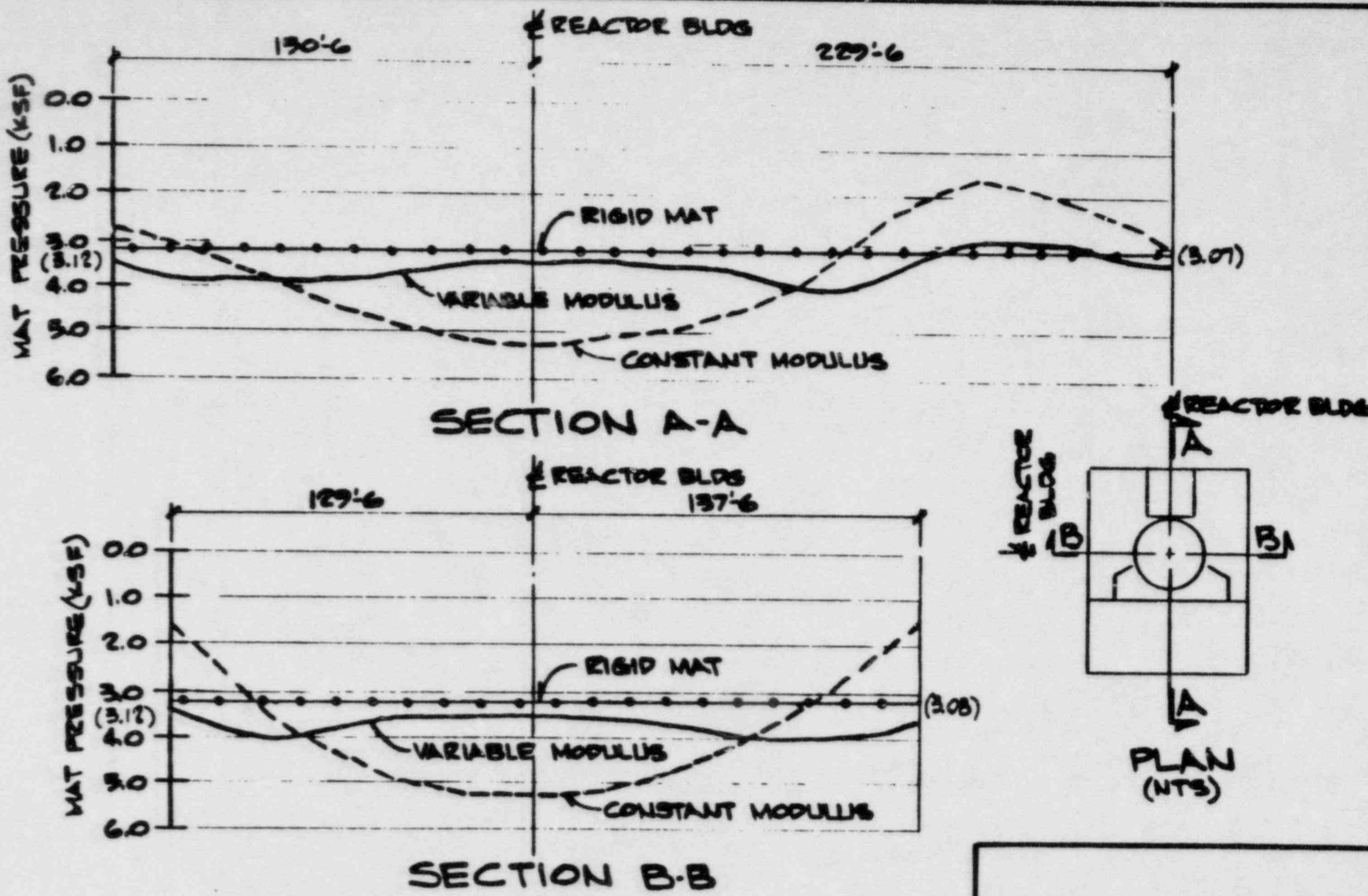


FLEXIBLE MAT ANALYSIS [- - - - CONSTANT MODULUS / ——— VARIABLE MODULUS

RIGID MAT ANALYSIS [(0.00 KSF)

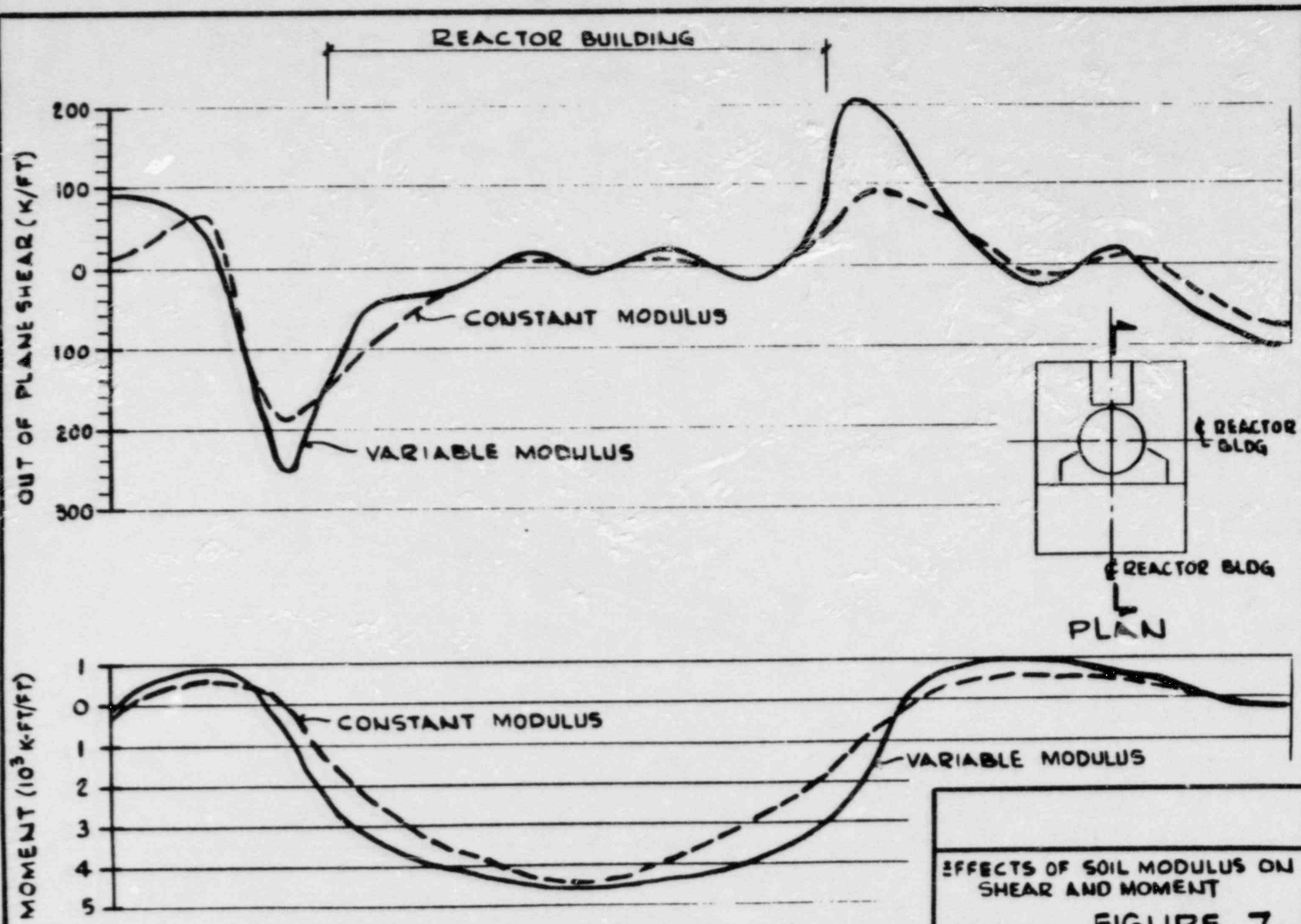
EFFECTS OF SOIL MODULUS ON MAT PRESSURES

FIGURE 5



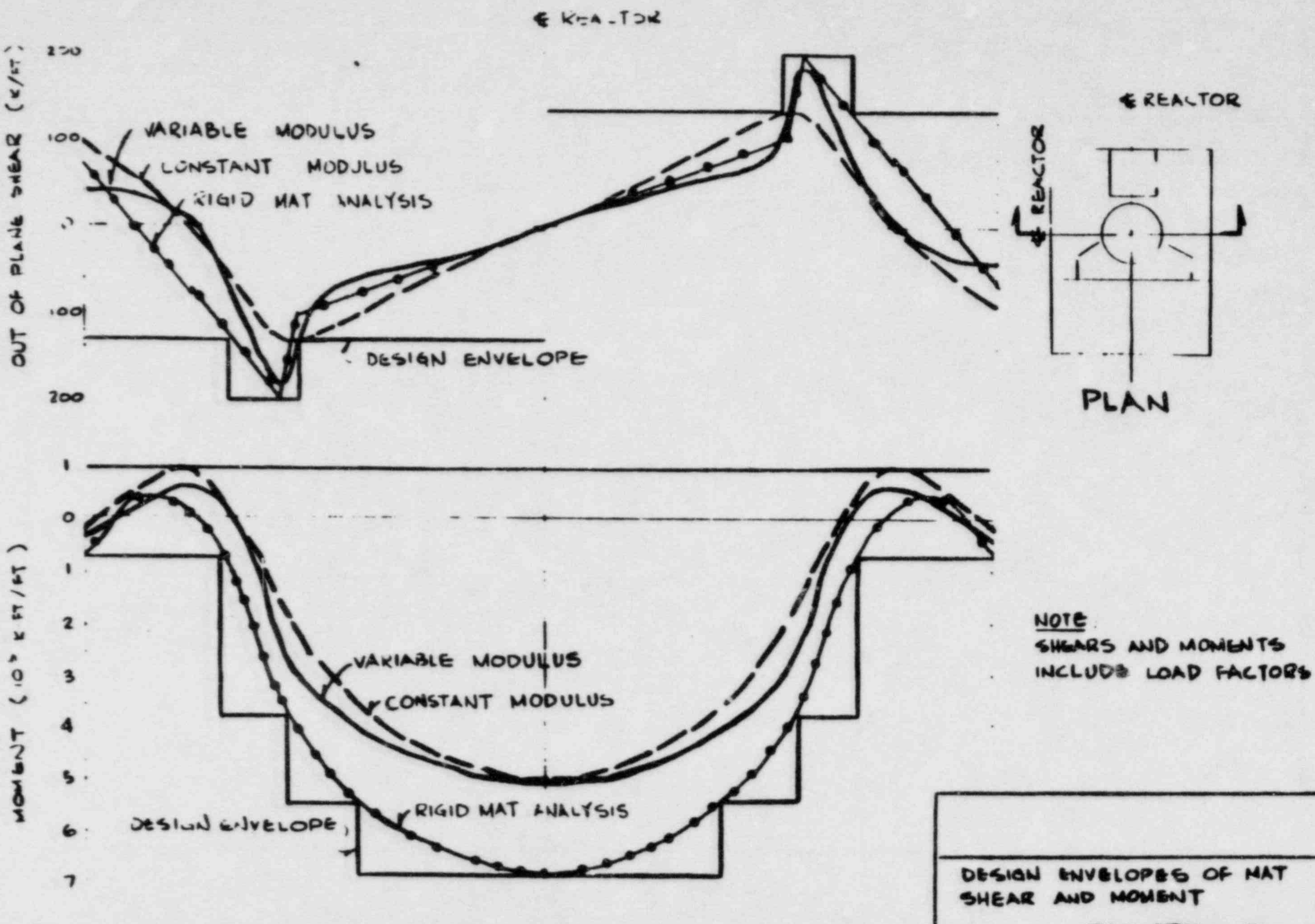
EFFECTS OF SOIL MODULUS ON MAT PRESSURES

FIGURE 6



EFFECTS OF SOIL MODULUS ON SHEAR AND MOMENT

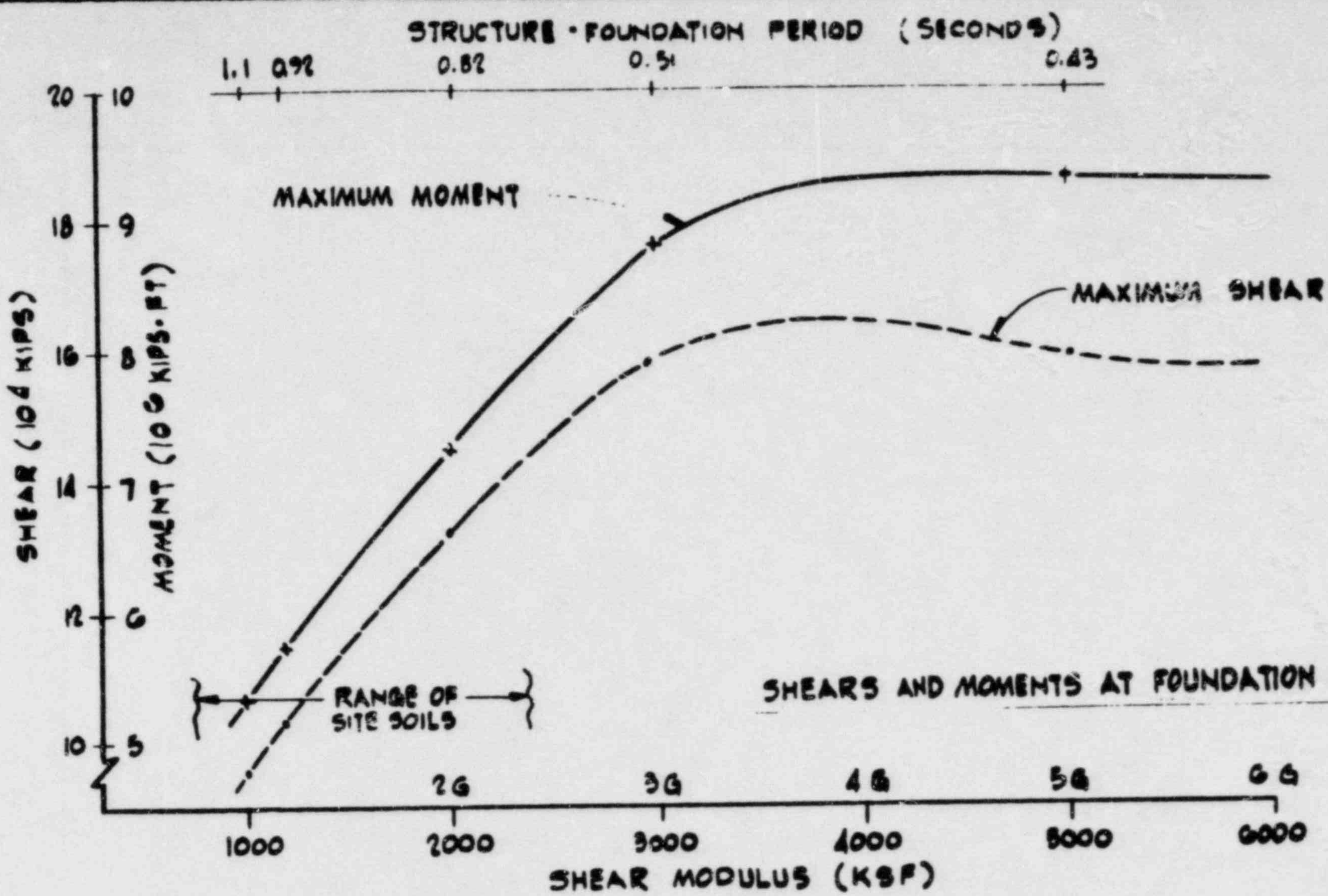
FIGURE 7



NOTE:
SHEARS AND MOMENTS INCLUDE LOAD FACTORS

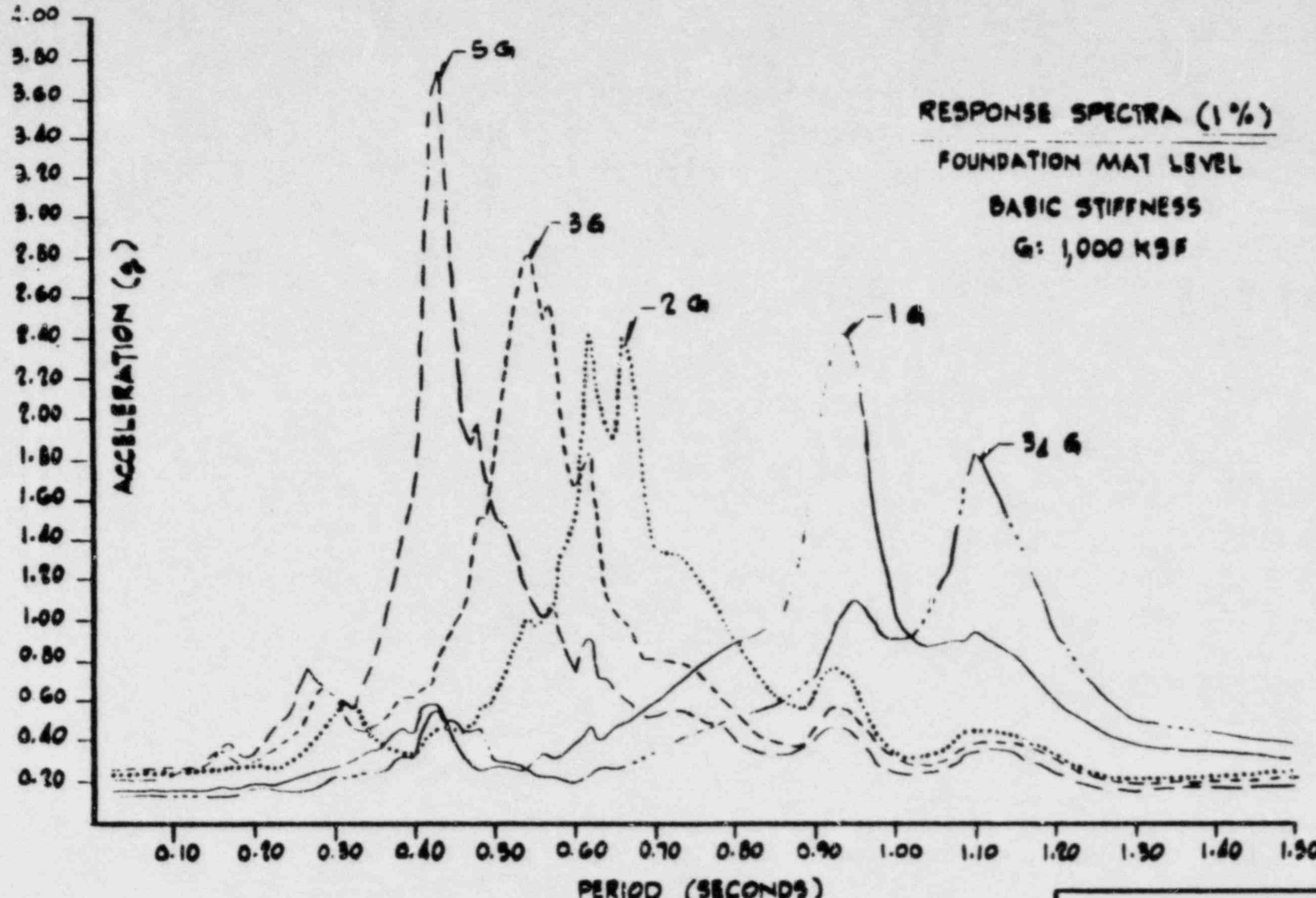
DESIGN ENVELOPES OF MAT SHEAR AND MOMENT

FIGURE 8

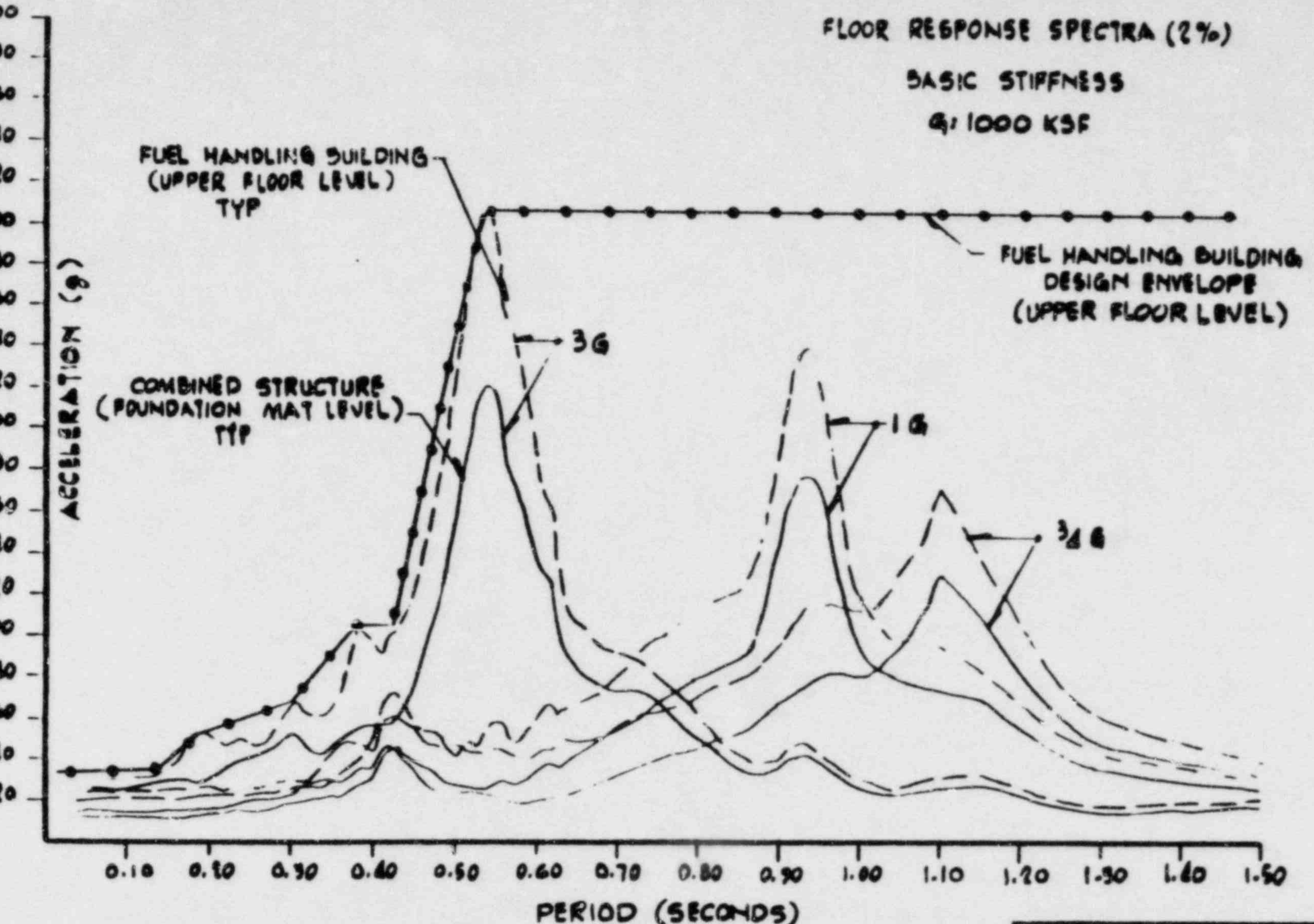


EFFECTS OF FOUNDATION STIFFNESS ON DYNAMIC SHEARS AND MOMENTS

FIGURE 9



EFFECTS OF FOUNDATION
STIFFNESS ON DYNAMIC RESPONSE
FIGURE 10



EFFECTS OF FOUNDATION STIFFNESS ON DYNAMIC RESPONSE AT VARIOUS LEVELS

FIGURE 11

LOUISIANA POWER AND LIGHT CO.

WATERFORD SES UNIT NO. 3

ALLOWABLE MAT BEARING PRESSURE

Prepared By: M. Pavone

Reviewed By: J.L. Ehasz

Ebasco Services, Incorporated

April 1977

CONTENTS

1. PURPOSE

The purpose of this report is to present the rationale for EBASCO's recommendation that the maximum effective mat bearing pressure be increased from 4000 psf to 4500 psf during construction. The original bearing pressure of 4000 psf presented in the Waterford SES PSAR will be accordingly updated in the FSAR to 4500 psf.

2. SCOPE

This report first provides background on the foundation design principle utilized at Waterford. It then presents predicted as well as measured foundation response resulting from construction and accounts for any discrepancies. The effect of the proposed pressure increase relative to minimizing the effects of these discrepancies is presented. Finally, the report discusses the effects of the increase from 4000 psf to 4500 psf maximum allowable bearing pressure.

3. DISCUSSION

3.1 Foundation Design Background

Generally the foundation soils below El.-40 at the Waterford site are overconsolidated. The existence of the only slightly overconsolidated Pleistocene clays at El.-92 ft, indicated that significant long term and differential settlements could be expected for structures founded on individual spread footings. To eliminate differential and long term settlement considerations, all Class I structures were located on a common mat foundation. The floating foundation principle was utilized with the combined structure foundation applying an effective load to the bearing stratum clays equal to the existing overburden pressure.

The soil conditions at the site were evaluated in terms of vertical effective stresses. These original stresses were initially on the order of 3300 lbs per sq foot at the mat bearing level. The effective stress is defined as the total weight of the existing overburden soils minus the uplift due to groundwater pressure. The effective stress is considered with the groundwater table at El.+8 ft.

Upon completion of Excavation Phases I through IV, the effective stresses at the mat bearing level were reduced to 0 psf. During the period of excavation and up to concrete placement the foundation soils rebounded or heaved in response to the relief of overburden stresses.

Presently, concrete placement has been in progress for more than a year and backfilling is active. With this increasing load on the foundation soils the rebound previously experienced is being compressed. The schedule as presented in the PSAR allows for this effective loading to reach 4000 psf or a 700 psf overload beyond the initial 3300 psf loading. The objective of the overload is to accomplish the total recompression during the construction period and minimize or eliminate any post construction settlements.

To maintain this maximum allowable effective bearing pressure a procedure of throttling down the dewatering system and possible pumping into recharge wells has been planned and provided for. This release of dewatering will result in increased buoyant or uplift forces acting on the foundation soils as well as the concrete structure and backfill material. The introduction of buoyant forces will be controlled so that additional construction and backfill loads are equally balanced with uplift. Thus the maximum allowable bearing pressure can be maintained during the construction period.

At a construction stage, approximately 6 months prior to termination of active construction and backfill imposed loads, the entire dewatering system will be released. This will result in a final effective bearing pressure of 3100 psf, slightly less than the initial 3300 psf. This 200 psf reduction of effective pressure will preclude the possibility of having any settlement considerations when the plant becomes operable.

3.2 Foundation Response

3.2.1 Predicted Behavior

The foundation rebound or heave calculated during the PSAR preparation, to occur between the time of excavation and start of concrete placement was approximately 2 inches. This rebound was anticipated to be nearly recompressed at the time the 4000 psf maximum allowable load was reached. Any further recompression would be controlled through manipulation of the dewatering system and recharge wells.

At the completion of construction with the groundwater back to its initial position the foundation material will experience an effective stress 200 psf less than the initial effective stress experienced prior to construction. This slight and apparent "net unloading" was considered due to the slight uncertainties in total loading during the early design PSAR stage.

3.2.2 Measured Behavior

Refer to Figure No. 1 for an extended time plot of the foundation response.

With the initial removal of 20 feet of material in 1972 (Phase I excavation), the site experienced between 1.5 and 3.5 inches of heave. This initial excavation was done without the benefit of the dewatering due to scheduling difficulties. This resulted in more heave than would have occurred if the dewatering were operative. As shown on Figure No. 1, the effective stresses during this Phase I excavation reduced to 1200 psf very rapidly and initiated the rapid rebound of the foundation clays. With increase in effective stress due to installation and operation of the dewatering system approximately 1 inch of this heave was recompressed. However, the dewatering was not in operation long enough to balance the Phase I excavation and the release of the dewatering system due to the job shutdown caused elastic foundation rebound to its pre-dewatered position maintaining this position for two subsequent years.

The dewatering was reinstated in November 1974. Due to the complete on-off-on operation of the system the wells essentially were purged and became more effective consequently lowering the piezometric levels 15 feet below their lowest 1972 position. This additional piezometric drop initiated further recompression of the foundation material.

With the exception of the north end of the site, the grade around the excavation was raised approximately four feet which apparently increased the compression of the dewatered site in all areas except on the north end.

In January, 1975, the remainder of the excavation was started. As a result, foundation heave readings increased to values between 4 inches and 9 inches. The heave rate leveled with commencement of concrete placement, reversed, and has been recompressing since. Presently, the heave remaining is between 1 inch and 6 inches.

3.2.3 Discrepancies

It is evident in a comparison of predicted rebound verses actual measured rebound that there is a 2 to 4 fold difference. These differences have been continuously monitored and evaluated during the construction and do not seriously affect the design of the plant. Only the recompression phases of the foundation-soil system are affected and are presently being addressed.

As described above, the initial excavation of 20 feet of soils without lowering of piezometric levels decreased the effective pressures by increasing the uplift on the foundation soils and caused more heave than calculated for the dewatered excavation procedure. The original scheme envisioned a balanced effective pressure system for the Phase I excavation, which would have resulted in no heave and potentially a settlement under the dewatered condition. This balance would have come about by the increased weight of dewatered soil and the decreased pressure due to Phase I excavation. When the dewatering was started the foundations responded by recompressing; however, the short duration of the dewatering prior to project shutdown was not effective in recovering the heave. This long shutdown period simply allowed complete relaxation of soils to the stress relief.

The differential heave from 1.5" along the south to 3.5" along the north during the Phase I excavation is attributed to the excavation procedure which essentially handled the material from north to south as well as the fact that the grade along the south, east and west sides of the excavation was raised four to five feet for construction facilities. The north side was not surcharged with the additional fill which in essence allowed more relaxation along the north. Additionally, the piezometric pressures along the north are always somewhat higher due to the recharge from the Mississippi River. All of the above factors tend to increase the heave potential of the north side of the excavation, as is the case seen on Figure 1 for heave point H1.

The heave experienced, in excess of the 2 inches predicted, is felt to be attributed to more rapid rebounding of the foundation clays than anticipated. Early calculations, formulated during the PSAR stage, considered that approximately 20% of the rebound would be realized during a 10 to 12 month excavation phase. The actual measurements indicated that a more rapid rebound has been experienced, perhaps on the order of 70 to 80% of full rebound, under the relaxed stresses of full excavation.

In order to ensure the full compression of this rebound, the foundation must be overloaded and controlled in order to minimize post construction settlements.

3.3 Justification of Increased Pressure

The intent of increaseing the allowable bearing pressure is twofold. It allows us to maintain a fully operative dewatering system while the turbine building backfilling continues and it further recompresses, at a faster rate, the soil heave incurred during and subsequent to Phase I through Phase IV excavation. The increase pressure will still adequately maintain a factor of safety, against a bearing capacity failure in excess of 3.

Presently, the turbine building backfill is only at about El.-25$\pm$ (MSL). To start throttling down the dewatering system and recharging through the wells would cause groundwater difficulties with backfill construction and possibly additional heave of the insitu soil and backfill material in this area.

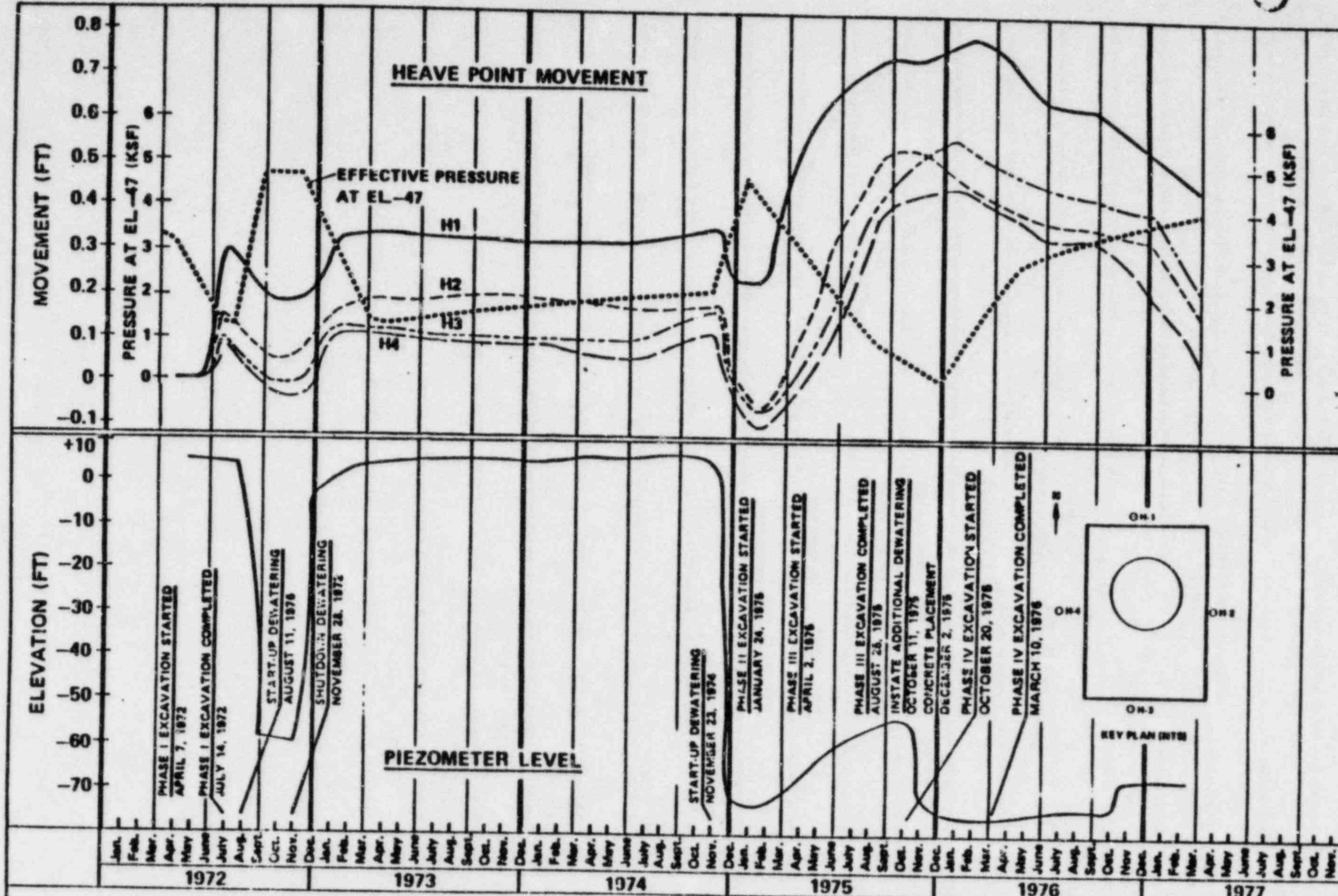
To ensure the uninterrupted backfilling of the turbine biulding excavation and still maintain the original 4000 psf allowable bearing pressure, would restrict increases in mat pressures. This would result in serious and unnecessary curtailment of concrete placements in the combined structure.

The previously specified 4000 psf allowable bearing pressure was realized during the last week in March, 1977, however, a significant portion of the higher than anticipated heave remains. Conveniently the area of the largest heave is along the northern portion of the excavation and is coincident with the area of anticipated bearing pressures above 4000 psf. Thus, the additional loading resulting from a 4500 psf limit will not only increase the rate of recompression but also has the potential of reducing the differential heave experienced from north to south along the excavation.

Thus, an increase in the allowable bearing pressure to 4500 psf is justified in that it allows construction to proceed uninterrupted on both the main plant island and the turbine building; it also affords the opportunity of more efficiently recompressing the foundation heave experienced, and thereby ensures the design intent.

4. RECOMMENDATION

Based upon the actual foundation response and the above rationale and discussion, it is recommended that the allowable effective bearing pressure be increased to 4500 psf. Presently it is anticipated that this pressure will be adequate to recompress the foundation to its original position or lower, as anticipated in the design; however, the effective bearing pressures will be closely monitored and adjusted as necessary to fulfill the design intent.



WATERFORD SES NO. 3

ALLOWABLE MAT BEARING PRESSURE

FIGURE 1

LOUISIANA POWER & LIGHT COMPANY
WATERFORD STEAM ELECTRIC STATION
1980 - 1165 MW INSTALLATION - UNIT NO. 3

REVIEW OF SITE SETTLEMENTS

Prepared By: M. Pavone *M. Pavone*

Reviewed By: J. L. Ehasz *J. L. Ehasz*

Ebasco Services, Incorporated

September 1978

CONTENTS

LOUISIANA POWER & LIGHT COMPANY
WATERFORD STEAM ELECTRIC STATION
1980 - 1165 MW INSTALLATION - UNIT NO. 3

REVIEW OF SITE SETTLEMENTS

1. PURPOSE

The purpose of this report is to update the information presented in the April 1977 report entitled Allowable Mat Bearing Pressure and to discuss the most recent as well as the anticipated future plant area settlements.

2. SCOPE

This report briefly summarizes the information presented in the April 1977 report and then discusses the plant-related settlements with respect to the turbine building and combined structure. The significant changes in the dewatering system that affected plant settlements are also highlighted. The most recent foundation movements are discussed in sequence from simple recompression of the post-excavation heave to a present condition where a net settlement is being induced. A discussion of the net induced settlement, to date, along with anticipated future movements resulting from additional ground water recharging is presented.

3. CONCLUSION

The settlement response of the combined structure is satisfactory. Differential movements across the combined structure are acceptable and the overall induced area settlement is proceeding uniformly. There are settlements influencing the relationships between the turbine building and the combined structure; however, this influence is being diminished as their settlement trends level off. A review of the additional induced settlement and of the time-settlement response of both structures indicates that their settlement rates should reduce to negligible differences within the next six to twelve months. This timing corresponds with complete recharge whereby all piezometric surfaces will be allowed to return to their original preconstruction level.

Thus, by inducing additional preconsolidation of the foundation soils, the basic design concept and PSAR commitment will be fulfilled since the post construction settlements will be minimized and the differential movements will be well within the design limits for interconnected piping and conduits.

4. DISCUSSION

4.1 BACKGROUND

Generally the foundation soils below EL -40 at the Waterford site are overconsolidated. The existence of the only slightly overconsolidated Pleistocene clays at EL -92 ft, indicated that significant long term and differential settlements could be expected for structures founded on individual spread footings. To eliminate differential and long term settlement considerations, all Class I structures were located on a common mat foundation. The floating foundation principle was utilized with the combined structure foundation applying an effective load to the bearing stratum clays equal to the original overburden pressure.

The soil conditions at the site were evaluated in terms of vertical effective stresses. These original stresses were initially on the order of 3300 lbs per sq foot at the mat bearing level. During construction, this stress varied from 0 lbs per sq foot at the completion of Excavation Phase IV to approximately 4000 lbs per sq foot prior to hydrostatic recharging through the filter blanket beneath the mat, refer to attached Figure.

The application of these stresses were carefully controlled by uniform concrete and backfill placements and by allowing hydrostatic uplift pressures to rise and compensating for any increase in pressure beyond the maximum allowable mat bearing level overload.

The reduction of stresses to zero from Excavation Phase IV resulted in a subsequent heave in the plant area. In order to recompress this heave, a mat bearing level overload of 700 lbs per sq foot more than the initial vertical pressure was specified (i.e. total vertical pressure of 4000 lbs per sq foot). It was originally estimated that this overload would result in efficient recompression of the post-excavation heave. The site, however,

4. DISCUSSION (Cont'd)

4.1 BACKGROUND (Cont'd)

experienced several inches more heave than initially anticipated as was discussed in the earlier report.

To compensate for the higher heave it was specified that the maximum allowable concrete and backfill-imposed overload at the mat bearing level be increased to 1200 lbs per sq foot for total vertical pressure of 4500 lbs per sq foot. Normal construction load increases beyond this maximum allowable overload were compensated with an equal amount of hydrostatic uplift introduced by throttling down the, then operating, dewatering system and the implementation of the recharge program.

The April 1977 report basically discussed the foundation responses to date and it highlighted discrepancies between predicted and measured plant area heave and settlement. Most importantly, it justified the increase in the total vertical overload pressure from 4000 lbs per sq foot to 4500 lbs per sq foot. The justification was based on allowing construction to continue with a fully dewatered site. At that time, the turbine area backfill was approximately forty feet below finished plant grade. Compensating for further increases in bearing pressures through reduced pumping and increased hydrostatic uplift would have resulted in ground water difficulties and excess heave in the turbine area. It was therefore recommended to increase the maximum overload pressure to 4500 lbs per sq foot through carefully controlled concrete and backfill placement and water table draw down.

4.2 TURBINE BUILDING INFLUENCE

The turbine building foundation change from piles to spread footings consisted of extending the excavation approximately 200 feet south and replacing

4.2 TURBINE BUILDING INFLUENCE (Cont'd)

the insitu soft Recent alluvial material with compacted Class A backfill. The added weight of this material plus the turbine building mat pressures are being transferred to the Pleistocene clays and silts. The combined action of these loads, while not affecting the design lateral pressure distribution on the adjacent combined structure wall, has resulted in continued settlement in this area. The settlement trends for the turbine building have been faster than the common mat structure. The initial rate of settlement for the turbine mat was on the order of 0.1 ft/month during the active backfilling in the turbine area; the more recent rates of settlement for the turbine area are on the order of 0.02 ft/month which are comparable to the combined structure movements.

These differential movement rates were actually considered in setting turbine area foundations. When the turbine building elevations were set, the reference point from the common mat was used and the turbine area benchmark was set 0.25 feet higher than planned. This difference was set to compensate for the faster rate of settlement in the turbine area due to the concurrent backfilling operations and to effect a minimal impact of the differential movements during construction. The intention was to maintain the design relative positions of interconnected piping and conduits.

Although this compensation for differential settlement rates was justified, the downward curvature of the combined structure mat as described in section 4.4 further exaggerated the compensating effect of the higher setting such that the turbine area was actually set 0.5 feet above the south end of the combined structure, which is the area of interconnection between the turbine and auxiliary building. With the higher settlement rate in the turbine building area, the effect of differential elevation between the two structures is currently being reduced.

It is anticipated that the turbine building should continue to settle for the next six months to one year and that the present differential of 4-1/2

4.2 TURBINE BUILDING INFLUENCE (Cont'd)

inches will be reduced to 4 inches.

4.3 LATEST SITE INSTRUMENTATION TRENDS

Refer to the attached figure for a summary plot of the piezometer readings and heave points (H1 through H4) to date. At the time of the April 1977 report H1 was still experiencing a heave of 0.4 feet, H2 and H3 were at approximately 0.1 foot heave and H4 was recompressed to the zero position. The piezometers in the sand aquifer at Elevation -85 ft were undergoing a rising trend due to extensive backfilling; they were previously stabilized at Elevation -75 ft. Water in the backfill was charging the sands through the slotted casing of the pump relief wells. During this period, the rate of recompression was slowed considerably due to the recharging by backfill water. In July 1977, larger pumps were added in the pump relief wells resulting in a subsequent reduction in the piezometric levels in the sands back to the Elevation -75 ft. position. Similarly, the foundation movements began to recompress at a faster rate.

Starting in October 1977, with the effective bearing pressures at approximately 4000 lbs per sq foot, the site dewatering recharge phase was implemented. The piezometric levels in the sands have been allowed to rise in a controlled manner and are presently at Elevation -10 ft. In response to the recharging, the plant area heave readings have undergone a reduced rate of settlement. These readings are currently at approximately +0.1 ft for H1, -0.3 ft for H3 and -0.4 ft for H2 and H4.

4.4 SITE SETTLEMENT EVALUATION

The present settlement trend is such that the site has gone from a recompression of the excavation-caused heave to a condition in which the area is experiencing a net settlement beyond the initial "zero" position. This net settlement, though not specifically addressed during the development of the foundation design, does fulfill, the original design concept, namely

4. DISCUSSION (Cont'd)

4.4 SITE SETTLEMENT EVALUATION (Cont'd)

minimizing post construction settlement.

The additional induced settlement or preconsolidation does not present any structural concern regarding the integrity of the combined structure. The important consideration is that the differential settlements are minimized after installation of the interconnected piping and conduit.

The settlement readings taken on the center strip #1 of the combined structure mat indicate a total settlement to date of approximately 0.9 feet. This settlement commenced when the site was in its maximum heave position. While adjacent mat strips have experienced overall settlements somewhat less than the center strip these subsequently placed strips were set based on strip #1 which resulted in a slightly curved mat surface due to the sequencing of the mat placements.

With the placement and interconnection of the walls the combined structure, as a unit, became somewhat rigid in its settlement response i.e. higher settlements along the mat edges. These higher edge settlements are based relative to an initial time being when the entire mat was interconnected.

To date, the maximum surface curvature between the center of the common mat and any other point on the mat is 2.5 inches. This maximum differential is between the center which is higher, and the south edge of the mat. A similar comparison between the center and north edge gives a differential of 1.5 inches.

It must be pointed out that the overall maximum curvature of 2.5 inches occurs over a length of nearly 200 feet. This amount of differential movement is slight with respect to the structure. In addition, the 2.5 inch differential between the center and the south edge of the mat has been sequential in nature due to the timing of the mat placements. The maximum differential has also

4. DISCUSSION (Cont'd)

4.4 SITE SETTLEMENT EVALUATION (Cont'd)

recently varied between 2.0 inches and 2.5 inches depending on the concrete placements in the containment area. During periods of high yardage placement the differential was minimized.

The overall site settlement beyond the initial "zero" position poses no serious concern. This overall movement is relatively uniform and within the limits of the excavation and plant structures. The maximum combined structure settlement to date is approximately 0.9 feet. This mat settlement has paralleled the site settlements as recorded by the heave points which have moved from an average maximum heave of 0.5 ft to an average maximum net settlement of 0.4 feet. These movements have occurred simultaneously.

The net settlement beyond the original "zero" position has been induced by controlling the piezometric in the sands levels. At the time of the April 1977 report the effective structural load at the mat bearing surface (EL -47 ft MSL) was approaching 4000 lb per sq foot. In the report we justified the increase in the allowable effective pressure to 4500 lbs per sq foot. Also, at that time, however, the piezometric pressure in the shell filter layer beneath the mat began to rise. This rise was the result of the filter being recharged from water in the backfill. The result was a reduction in the overall effective vertical pressure at that bearing level (see attached figure for effective pressure plate). The piezometric level in the Elevation -85 sands, however, still remained depressed. The result of these discontinuous piezometric profils is extended settlement beneath the Elevation -85 sands due to the total weight structure and wet backfill.

4. DISCUSSION (Cont'd)

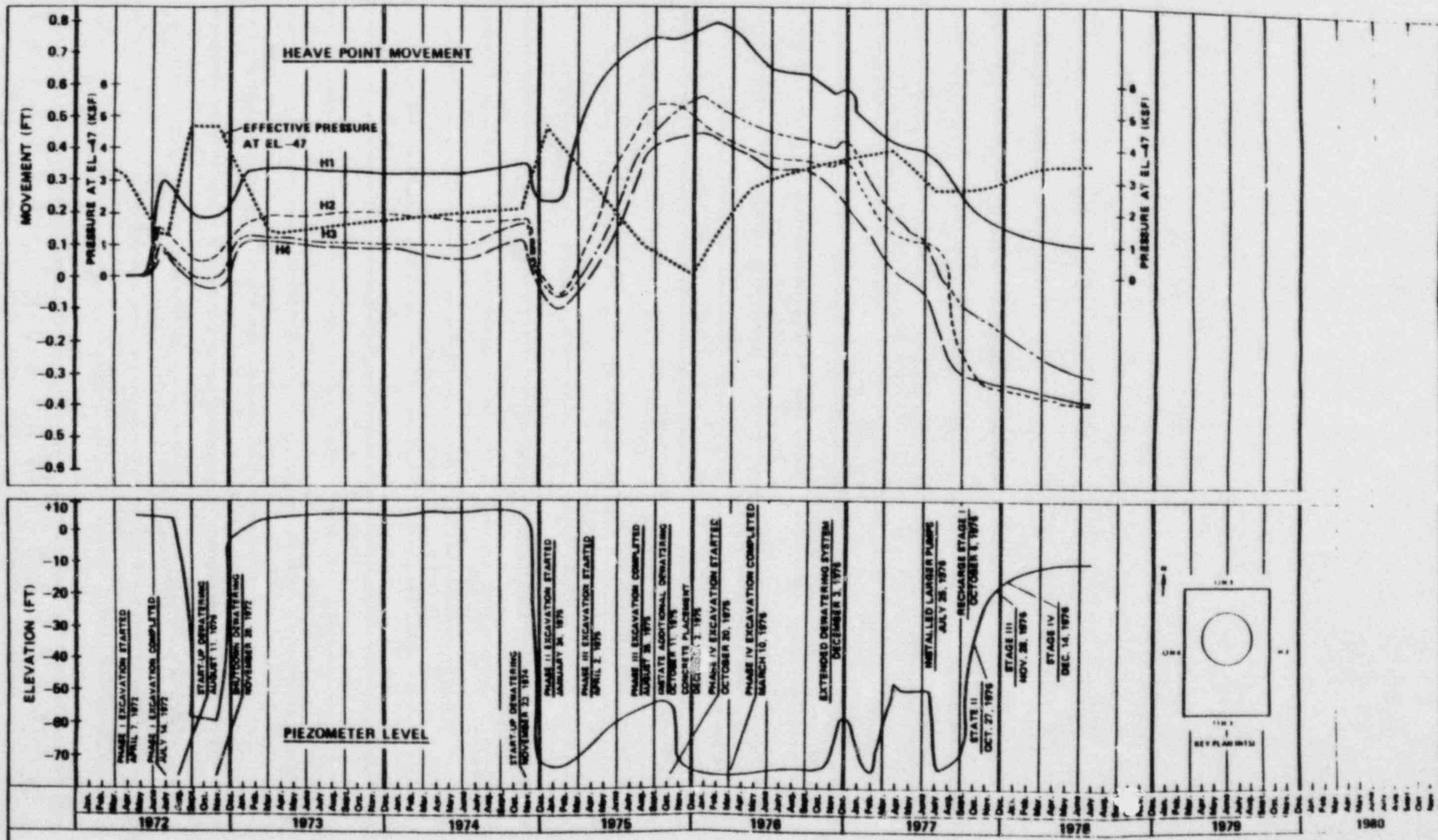
4.4 SITE SETTLEMENT EVALUATION (Cont'd)

This difference between the two piezometric profiles has been substantially decreased to a present difference of 8 ft as a result of the controlled recharge program. With the elimination of this piezometric pressure variation through complete recharge, the settlement rates as well as the differential rates will level off.

4.5 JUSTIFICATION OF PRESENT INDUCED SETTLEMENT

The overall induced settlement as measured by the heave points is a result of the depressed piezometric pressure profiles. The perched pressure level in the shell filter blanket is above the level in the Elevation -85 sands due to the backfill water tending to recharge the soils above the Pleistocene. The result of this is a benefit which can be maintained for the next 6 months to one year. By maintaining the site in its present dewatered position, the induced settlement will continue and further pre-consolidate the foundations to minimize future settlements.

Any future settlement would not be the result of structural mat loading but would occur due to the weight of compacted backfill. Since the backfill material is heavier than the Recent alluvial material, there is a net increase in overburden pressure in the backfilled areas. By continuing with the present dewatering, the post-construction site settlements will be minimized and within tolerable limits of the design.



WATERFORD SES NO. 3

SITE INSTRUMENTATION

ATTACHMENT G

Shewmaker

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

July 14, 1983

MEMORANDUM FOR: Richard C. DeYoung, Director
Office of Inspection and Enforcement

FROM: Mark W. Peranich, Chief
Construction, Vendor, and Special Program Section
Reactor Construction Programs Branch
Division of Quality Assurance, Safeguards, and Inspection Programs
Office of Inspection and Enforcement

SUBJECT: INQUIRY TEAM REPORT ON WATERFORD QA ALLEGATIONS

The enclosed Inquiry Team Report completes the action assigned by J. Sniezek's memorandum of June 21, 1983 on the matter of Waterford QA Allegations. It is forwarded for your further consideration.

I and other members of the Inquiry Team are available, if needed, to discuss the contents of the enclosed report.

Mark W. Peranich

Mark W. Peranich, Chief
Construction, Vendor, and Special Program Section
Reactor Construction Programs Branch
Division of Quality Assurance, Safeguards, and Inspection Programs
Office of Inspection and Enforcement

Enclosure:
Inquiry Team Report

cc w/enclosure:
J. Sniezek
J. Taylor
J. Collins, Region IV
B. Hayes, OI
J. Cummings, OIA
R. Shewmaker
E. Johnson, Region IV
M. Peranich
G. Mulley, OIA
E. Jordan

INQUIRY TEAM REPORT WATERFORD QA ALLEGATIONS

The memorandum of June 21, 1983 from James Sniezek, Deputy Director, Office of Inspection and Enforcement to Mark Peranich, Chief, Construction, Vendor and Special Programs Section, Division of Quality Assurance, Safeguards and Inspection Programs, assigned an Inquiry Team to interview Mr. Ron Ridenhour about allegations forwarded with his letter of May 31, 1983 to James Joosten, Technical Assistant to Commissioner Gilinsky. The Inquiry Team consisted of Mark Peranich, IE, Team Leader; Robert Shewmaker, IE, Sr. Civil-Structural Engineer; and Eric Johnson, RIV, Technical Assistant. Present at the interview in addition to the Inquiry Team were George Mulley, Investigator, NRC Office of Inspector and Auditor; Mr. Gary Esolen, Editor, Gambit Publications; Mr. Ron Ridenhour, free-lance reporter; and Mr. Brad Bagert, Esquire, Attorney for Gambit Publications. The interview was tape recorded by Gambit Publications.

A summary of the interview is provided in the Attachment 1, Memorandum, George Mulley, Jr. (OIA) to James Cummings (OIA), dated July 6, 1983. As noted therein, the inquiry team held an interview with Mr. Esolen but was not given the opportunity to interview Mr. Ridenhour.

A second attempt was made to interview the alleger on June 29, 1983. A telephone call was made to his residence at approximately 7:00 a.m. Mr. Ridenhour was asked if he wished to meet separately with the Inquiry Team to discuss his allegations. He indicated that he did not. In response to the team leader's question, Mr. Ridenhour believed it was appropriate for Mr. Esolen to take the lead during the June 28, 1983 interview. He further explained that, as the editor, Mr. Esolen played a significant role in the preparation of the published Gambit articles forwarded to the NRC. In addition, Mr. Ridenhour did not feel it was necessary for him to add or change any of the statements made by Mr. Esolen regarding the issues that were identified for NRC followup.

The Inquiry Team met on June 29, 1983 to review the limited information acquired during the interview on specific issues associated with the three main problem areas identified by Gambit. Based on this review and the team's review of the published Gambit articles, the following issues were identified for followup to address Gambit's allegations of three problem areas.

1. Adequacy of Louisiana Power and Light (LP&L's) QA program during construction.

   Related Issues

   - Contractor turnover of four plant systems to LP&L with numerous deficiencies

   - LP&L lack of knowledge whether its QA program was being implemented

   - LP&L inaction in response to recommendations from its independent QA consultant

   - Errors in design assumptions by LP&L's engineering contractor

2. QA program dispute between LP&L and Combustion Engineering (CE).

   Related Issues

   ° LP&L audit in 1974, noting that CE's QA program had not incorporated the "new" QA requirements (Amendment 44, Gray Book)

   ° EBASCO December 6, 1976 audit of CE-identified problems with CE's system for records

   ° Communications between LP&L and CE

   ° Statements of LP&L, CE, and EBASCO individuals

3. Waterford Unit 3 common basemat.

   Related Issues

   ° Cracking discovered in 1977 and 1983

   ° Leakage through cracking in basemat

   ° Errors in assumptions for design
     - Sizing of dewatering pump
     - SAR statement that common basemat would be a "watertight barrier"

Observations - Waterford Unit 3 Site

The main purpose of the Inquiry Team's effort at the site was to observe first hand the cracking and leakage of water through the basemat. The observations of the plant included (1) the equipment rooms where the new cracking was discovered in May 1983; (2) approximately 300° of the 360° around the shield building at the -35 ft level (i.e., top of basemat); and (3) all 360° of the floor of the annulus area between the shield building and containment at El. -1.5 ft. Specific details of these observations are noted by R. E. Shewmaker in Attachment 2. In summary, the Inquiry Team observed apparent seepage of water on the surface of the common basemat at various locations around the shield building and in equipment rooms identified with the May 1983 discovery of other cracking in the basemat. Examinations using an 8X magnifying lens at one equipment room location did not result in the visual identification of a "crack" or, after one hour, any additional seepage and collection of water into the excavated area prepared for these examinations. With respect to the floor of the annulus area, water was observed in one location; however, visual observations alone were not sufficient to determine the origin of the water (i.e., leakage from concrete below or entry of water from above open areas). All observations were made by R. E. Shewmaker, M. W. Peranich and the NRC Resident Inspector, Les Constable.

The site visit also included general discussions with the Resident Inspector regarding the problem of the cracking of the basemat and the identification of a number of deficiencies in the four plant systems turned over to LP&L

by an EBASCO contractor. Certain existing record documents relative to the design of the common basemat were acquired for further examination after the visit. While at the site, the LP&L QA Manager was asked to clarify the statement attributed to him in Gambit's published article that other cracks and water seepage have been discovered in the floor of the nuclear island from time to time in the intervening years. The QA Manager believed he was referring to other cracks in the common basemat outside the containment that probably occurred in 1977, but had not been observed until later. The Inquiry Team's discussion with the QA Manager was preceded with the clarification that the NRC effort at this time was an inquiry and not an inspection or investigation.

Proposed Followup Actions

Based on the results of the Inquiry Team's interview with Gambit Publications representatives, observations made at the Waterford Unit 3 site and the current status of the team's review of existing documentation, the Inquiry Team recommends that the following actions be taken for each problem area to fully address the Gambit Publications allegations:

1. In addition to reviews completed by the Inquiry Team of existing inspection documentation, Region IV, or others as assigned, should perform a detailed review of all documentation of inspections of LP&L, CE and EBASCO unique to the Waterford 3 project during the 1974-1977 period. The review should assess the extent the issues and actions noted below may have been previously addressed. Where not adequately addressed by prior NRC inspection

activities, the actions listed below are recommended for implementation. Bases for not completing the following actions should be documented.

2. Adequacy of LP&L's QA program during construction.

   a. Issue - Deficiencies in four systems turned over to LP&L

      Action - Complete review of all documentation associated with this matter before and after issuance of the $20,000 fine.

      Review the reasons for a breakdown in the EBASCO contractor's QA program; LP&L's part in the identification of the deficiencies; and the adequacy of LP&L proposed corrective action.

      Determine whether all systems to be turned over to the licensee will be subject to the established corrective action as well as the likelihood for possible deficiencies to be identified by the EBASCO contractor before future turnover of plant systems by EBASCO to LP&L. The ability of the LP&L QA audit of the turnover packages to identify such deficiences should they not previously be identified by the EBASCO contractor's QA program should be determined. Also, conduct a review of the adequacy of the licensee's corrective action implemented to this date.

Assess whether the corrective action taken by LP&L and EBASCO is sufficient to prevent the recurrence of a breakdown in the EBASCO contractor's QA program. Also, assess whether the cause of the breakdown was determined to be limited to the "turnover phase" or applicable to a longer phase of construction.

b. Issue - LP&L did not know whether its QA program was being implemented

Action - Conduct a review of LP&L's QA program and implementation relative to the measures established for LP&L to be cognizant regarding the adequacy and status of program implementation. Implementation review should cover the 1974-1977 time period and should include:

(1) Audits conducted by LP&L of CE and EBASCO.

(2) Audit conducted by EBASCO of CE and other EBASCO/licensee contractors, the results of which were formally reported to LP&L.

(3) LP&L review of audit reports and, if necessary, corrective action taken.

Determine, based on the results of the above reviews, whether LP&L was knowledgeable of the adequacy and status of the implementation of its QA program and, when necessary, initiated appropriate corrective action.

c. Issue - LP&L did not take appropriate action on independent QA consultant's recommendations.

Action - Conduct a review of consultant reports and of LP&L action on the consultant's recommendations. Review QA program description and conformance of LP&L's implementation of the QA program in areas relating to consultant's recommendations.

Determine if licensee was in compliance with the QA program described in the SAR. If necessary, request the assistance of the DQASIP Quality Assurance Branch in arriving at a final determination of compliance.

d. Issue - Errors in design assumptions by EBASCO.

Action - As a part of the actions completed under item 4 below, conduct an independent review of the adequacy of design control applied for original design assumptions relative to the sizing of the dewatering pumps and the water tightness of the common basemat. Determine the adequacy of the design process for that aspect of the design and the implications of the apparent need to change those design assumptions on the adequacy of the overall design control for the design of the common basemat and the watertightness of underground structures. This independent review should include examination of other design assumptions relating to the area of design noted above.

3. QA program dispute between LP&L and CE

a. Issue - LP&L 1974 audit of CE found that CE was not in compliance with LP&L's "new" QA program commitments (Amendment 44)

EBASCO 1976 audit of CE-identified problems with CE compliance with LP&L's "new" QA requirements for records.

Action (1) As input to the investigative aspects of this issue (Action (2) below), perform the following inspection activities. Examine the results of the LP&L 1974 audit of CE and of the EBASCO 1976 audit of CE. Determine the extent of the implication that audit findings show that CE was not implementing licensee SAR QA Program commitments during the 1974 -1976 time period. Examine documents listed under question 18 of Gambit Publications correspondence dated April 4, 1983. Specific reviews of documents identified by question 18. d, e, i, k, q, s and t is recommended. To the extent necessary, interviews with LP&L and CE representatives involved in the QA program dispute between LP&L and CE should also be conducted to clarify any statements or data recorded in the above-referenced documentation.

Provide the Office of Investigations (OI) the results of these examinations along with a recommendation of which issues may require investigation.

Action (2) The Office of Investigation should review the results of inspections conducted under Action (1) above and determine whether an investigation is necessary to determine whether LP&L or CE misrepresented the extent of CE compliance with the licensee's new QA Program commitments (Amendment 44). Bases for not conducting an investigation should be documented.

Action (3) In case either of the results of Actions (1) and (2) above identifies that there was a period during 1974-1977 where CE's QA program substantially deviated from licensee SAR QA program commitments (Amendment 44) (i.e., after appropriate time is allowed for LP&L promulgation and CE implementation), the following additional items should be considered for followup.

(a) Whether the eventual action and followup initiated by the licensee in resolving or addressing this matter was sufficient to ensure that affected CE design, procurement, manufacturing, and record activities were re-evaluated and verified to have been conducted and controlled, or otherwise corrected, to be in compliance with licensee SAR QA program commitments.

(b) Action taken by the licensee in evaluating whether the shortcomings in CE's QA program were reportable under 10 CFR 50.55(e).

4. Waterford Unit 3 common basemat

a. Issue - Errors in assumptions of design pertaining to size of dewatering pumps and the SAR statement that the 12-ft-thick common basemat would be a "watertight barrier"

b. Issue - Cracking of common basemat discovered in 1977

c. Issue - Cracking of common basemat discovered in May 1983

d. Issue - Leakage through cracking in basemat

Action (1) The licensee should initiate an independent engineering evaluation of the common basemat cracking and seepage matters noted below. The use of a third-party consultant with expertise in soils, groundwater, foundations, water-related concrete structures (such as sanitary facilities), corrosion, concrete behavior (including cracking and concrete destructive and non-destructive test methods) should be considered in completing the evaluation.

Action (2) The licensee should evaluate the current adequacy of all facets of prior engineering and construction evaluations and corrective actions with regard to the cracking and water seepage in the common basemat. The evaluation should address the cracking and seepage reported to the NPC in 1977 and 1983 and of other cracking noted and recorded by the licensee during the intervening and present time period, and include consideration of

(a) Initial assumptions on the sizing of the relief well pumps.

(b) The assumption that the common basemat would be a "watertight barrier".

(c) The potential that the water seepage through the common basemat has for reducing the cross-sectional areas by corrosion of the ASTM A-615 reinforcement steel of the mat and the ASTM A-516 plate used in the containment vessel.

(d) The immediate and long-term effect that all existing conditions of cracking and leakage in the basemat has on the original design concept and assumptions of the plant.

Action (3) Pertinent factors such as the following should be considered during the evaluation of matters to be addressed in Action (2) above.

(a) The survey of all cracks and seepage zones on the top of the common basemat at El. -35 ft and other potential zones of discontinuity.

(b) Source of the seepage water on the top of the common basemat.

(c) The physical and chemical properties of the basic groundwater and variations that could be expected; water seeping from the surface of the basemat; and the solid deposits left on the surface of the basemat from the seepage water.

(d) Prior evaluations on how to stop or control the in-seepage.

(e) Prior evaluations of all available data (e.g., piezometer, settlement, loading, etc.) since September 1977, which relates to the response of the common basemat and provides a basis for describing the behavior of the common basemat and establishing the cause(s) of cracking.

Action (4) The licensee should complete action d. under item 2 (QA program) above for reviewing the overall adequacy of design control for the common basemat and water-tightness of the underground structure.

Action (5) The licensee should provide the NRC with comprehensive reports related to the proposed and completed evaluation of the cracking, seepage, corrosion potential, design control, and necessary actions to ensure the proper behavior of the common basemat and the steel containment over the life of the facility.

Attachment

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

July 6, 1983

MEMORANDUM FOR: James J. Cummings, Director
Office of Inspector and Auditor

FROM: George A. Mulley Jr., Investigator
Office of Inspector and Auditor

George A Mulley

SUBJECT: WATERFORD QA

On June 28, 1983, at the Office of Gambit Publications, Inc., 921 Canal Street, Suite 900, New Orleans, LA, I attended a conference regarding the QA program at Waterford SES, Unit 3. Participating from Gambit Publications we[illegible] Ron Ridenhour, reporter; Gary Esolen, editor; and Brad Bagert, attorney. Representing NRC were Mark Peranich, Chief, Construction, Vendor, and Specia[illegible] Program Section, Office of Inspection and Enforcement (I&E); Robert Shewmake[illegible] I&E; Eric Johnson, Region IV; and myself. This conference was arranged by Peranich to obtain specific information from Gambit Publications regarding t[illegible] concerns over the QA program at Waterford raised by Ridenhour in recent Gamb[illegible] news articles. My purpose at the conference was to solicit information from Gambit pertaining to alleged collusion between NRC and Louisiana Power and Light Company (LP&L) and alleged complicity of NRC in QA problems at Waterford.

At the outset of the conference, Esolen stated that he would be the spokesma[illegible] for Gambit and we would not be allowed to interview Ridenhour. Esolen also made it clear that his newspaper was not in the business of providing information to outside agencies and that this instance would be no exceptior[illegible] Esolen, in his introductory remarks, also stated that all the information t[illegible] Gambit was willing to provide had been printed and that NRC should investiga[illegible] the problems already outlined in the newspaper articles. Bagert, the attorney, also expressed surprise that the inquiry team had decided to first interview Ridenhour instead of beginning the investigation at Waterford or LP&L. Bagert questioned my presence at the conference. Bagert opined, that as the only "investigator" sent by NRC, my investigation should be complete[illegible] separate from the investigation by other members of the NRC staff. Bagert explained that, although he meant no offense to the people present, Gambit Publications did not trust the motives of NRC in asking for further information about Waterford. Bagert and Esolen questioned whether NRC coulc[illegible] effectively investigate itself and ventured that NRC would use any informat[illegible] provided by Gambit to defend the QA programs at Waterford. Peranich repeatedly assured Esolen and Bagert that NRC's motive in seeking informati[illegible] from Gambit was to gather sufficient information to allow the team to investigate in detail and with objectivity the specific concerns of Gambit. In response to Esolen's request, Peranich read aloud the June 8, 1983, memorandum from James K. Joosten to Richard C. DeYoung concerning

Waterford QA. Esolen then summarized the newspaper articles published by Gambit pertaining to the problems at Waterford. He grouped the problems int three major categories: 1) overall QA program at Waterford by LP&L; 2) relationship between LP&L, EBASCO, and Combustion Engineering (CE) and, specific, the outcome of the contract dispute concerning who was responsible for the "new" costs of the QA program at Waterford; and 3) problems with the construction of the basement at Waterford. Esolen concluded the summary by stating that at this time that was all the information he was going to prov concerning Waterford and that Gambit's news articles would speak for themselves. Neither Esolen nor Bagert would confirm or deny that Gambit ha any further information concerning Waterford. Bagert emphasized that Gambi should not be considered uncooperative in this matter and that the newspape stood ready to assist in an investigation of Waterford. Even though he did not wish to discuss the problems at Waterford during this conference, Esole stated that Gambit might be willing to provide information to a higher authority (Congressional committee) or even NRC at a later date.

During the meeting Esolen questioned the team about the status of two Freed of Information Act (FOIA) requests submitted by Gambit to NRC about two mon ago. Esolen and Bagert were disturbed that the requests had not yet been honored. The requests were for 1) all communications between NRC and LP&L regarding an NRC Inspection Report, dated December 6, 1982, of Waterford. specific, Gambit wanted to learn why a proposed $40,000 fine of LP&L was reduced to $20,000; 2) all Atomic Safety Licensing Board Panel (AS'BP) docu ments pertaining to Waterford 3. Peranich explained the workload involved processing FOIA requests and the interplay between NRC and a licensee when proposes a fine. These remarks were interpreted by Esolen and Bagert as be in defense of NRC and LP&L and indicative of the type of investigation NRC would conduct at Waterford.

During the discussions between Peranich, Esolen and Bagert, I had the oppor tunity on several occasions to ask the Gambit representatives for informati indicating "collusion" or "complicity" between NRC and LP&L. Esolen told Peranich and me that he was "closing the door" at this time to any discussi on this matter. When pressed for information to substantiate Gambit innuer of collusion on the part of NRC with LP&L, Esolen stated that even the fact Gambit's FOIA requests had not yet been answered by NRC raised the possibi that NRC was trying to protect LP&L. Esolen refused to discuss this area further except to say again he was not convinced that NRC could objectively investigate itself. Esolen asked us for a mandate or similar document that would guarantee an impartial and objective investigation of the facts and of the alleger. Although I explained the independent role of OIA, Esolen would not alter his position.

The conference was concluded by Esolen who stated that although his newspa was not in the business of directing NRC investigations, he suggested that team begin by developing information already published by Gambit. Esolen repeated that although he did not want to discuss Waterford problems durin the conference, he might be willing to provide information at a later date a higher authority. Esolen then requested that after the meeting I accomp him, Ridenhour, and Bagert to a separate office. In this office, Esolen stated that he thought that I could conduct an objective investigation; however, prior to him providing any information indicating collusion betwe NRC and the licensee, he wanted assurances of my professionalism from some outside of NRC. Esolen requested I contact a person of authority outside of NRC and have that person vouch for my character to Esolen. In specific, Esolen wanted assurances that I could conduct an impartial and objective investigation. With this request the meeting was concluded.

cc: M. Peranich, IE

Attachment 2

OBSERVATIONS - WATERFORD UNIT 3 SITE

The following observations were made during a site visit to the Waterford facility on June 30, 1983.

In the auxiliary building at the top of the common basemat (El. -35 ft), moisture was noted to have been permeating up from the basemat at several locations, some of which had been noted in May 1983 by the licensee in the areas known as the waste gas compressor rooms, gas surge tank room, gas decay tank room, and others that were identified by the senior NRC resident inspector and the Inquiry Team formed for the resolution of this question.

Two temporary manholes (for construction during conduit and cable placement) were observed. One was located near column line 12A between H and J and the other near column line 1A between J and K. These appeared not to have steel liners and were examined to determine if a source of water had also found a path through to these areas of lesser basemat thickness. The openings (blockouts) were approximately 6 ft wide x 6 ft long x an estimated 7 ft deep and contained water to an unknown depth. No specific details related to cracking or water source could be obtained from the observation.

In an area located generally southwest from a floor drain (FD) sump, near column line 10A between K and L, a darkened zone approximately 5 ft long was observed with several specific wet spots along with a buildup of material. This area was identified earlier by the senior NRC resident inspector. The rough darkened area was examined with an 8X eyeglass but no distinct crack could be observed within the zone. However, moisture and discoloration established a definite linear zone, permitting distinctive visual recognition of a difference with the surrounding area. No flowing water was observed, but it appeared that moisture was present in the zone and seemed to be coming to the surface from within the basemat. This surface seemed to be the original finished concrete surface of the common basemat and did not appear to have a surface coating. The adjacent FD sump (#6) was examined and found to be steel lined (as is typical for FD sumps at El. -35 ft) and to contain water. The source of the water in this sump could not be determined from the observations made.

In another location where seepage had been identified by the senior NRC resident inspector, the common basemat surface was coated with what was described as an epoxy paint material. The moisture was located along a linear path (about 7 ft long) that appeared to be at the construction joint between block placement numbers 13B and 18. The moisture had broken through the epoxy paint and was present on top of the surface along with a buildup of grey deposits of material from 1/8 to 3/16-in. thick. The area of this observation was near column line 5A between J and K along the door and into the south motor-driven auxiliary feedwater pump room. Portions of the permeable zone extended westward outside the room.

In May 1983, the licensee noted that three of the next four areas or rooms where seepage was observed were areas of concrete cracks as evidenced by water percolation to the surface of the common basemat. These areas are generally adjacent to one another and are located in the northwest corner of the auxiliary building. In the gas surge tank room area there is a zone of seepage approximately 10 ft long running parallel to the L column line and passing beneath a 2 ft 6 in. wall near the entry point to the room. In waste gas compressor room "B", the licensee had chipped an area about 1/2 to 3/4 in. below the epoxy-coated surface. This area approximately 12 in. x 15 in. provided a relatively "clean" but rough surface to view the zone of moisture. When viewed with an 8X magnifying lens, no specific crack could be observed. The approximately 6 ft zone tracked diagonally from the room's corner area to the concrete mounting base of the compressor. There was also evidence of patching at three locations on the side opposite the moisture zone. Repaired areas had approximate dimensions of 24 in. x 24 in., 12 in. x 12 in. and 12 in. x 8 in. The NRC resident inspector had no information readily available related to these patch areas and there was insufficient time to discuss them with the licensee.

In waste gas compressor room "A" another seepage zone was approximately 5 ft long. The zone was oriented diagonally from a room corner to the base of the compressor. Specific observations with an 8X magnifying lens were made. The moisture sources seemed to well up out of the concrete in distinct locations, rupture the epoxy coating, and then build a small cone-shaped deposit of material assumed to be material that was dissolved in the water to form a solution and was left as a result of the evaporation of the water. One of the small areas was examined. A 3/8-in.-diameter and 1/2-in.-deep hole was easily made around one of the seepage areas. This small "crater" was cleaned and dried with paper towels and observed by three other NRC personnel. Approximately one hour later, the same NRC engineers viewed this area and could detect no discernible change in the moisture levels. In the doorway of gas decay tank room "C", another seepage zone 4 ft long was observed. A new zone was found at the base of the "C" gas decay tank along the northeast side. There was considerable surface moisture in this area, but it could not be determined whether all the observed water was from the seepage zone or from another source, such as that associated with ongoing flushing and testing of systems or from rainwater.

Areas adjacent to the shield building and the containment at El. -35 ft were visually observed over about 300° of the circumference. Along the northwest quadrant of the base ring block of concrete (10 ft thick x 16 ft-10 in. high), which was placed as the base ring of the shield building, there were indications of water leakage in the past on the vertical faces (as evidenced by deposits of probably calcium carbonates). At the junction with the common basemat at El. -35 ft. there was evidence of the seepage zones as well as areas that appeared to have previously been seepage zones but were now dry. Areas in this quadrant had surface water and the source of the water could not be ascertained. An area perpendicular to the wall between column lines 1A and 1M, which serves as the wall of wet cooling tower A, showed evidence of seepage over a distance of approximately 4 ft to 5 ft.

In the southwest quadrant between columns 4A-N and 5A-M, there was an area adjacent to the shield building near an electrical panel that seemed to have water actively seeping out from under the shield building. There also appeared to be a buildup of material that had leaked out or had been deposited out of solution as the water evaporated. This area may also represent a location where there was a 1 ft-9 in. deep sump during construction.

In the southeast quadrant near column 9A-M, there appeared to be another area of active water seepage from beneath the shield building. Also in this area, there was an old zone of seepage with deposits present on the concrete common basemat surface. No seepage was evident on the day of the inspection.

Observations were made in the annular space between the containment and the shield building on the lowest level (-1.5 ft) to check for water and moisture along the top of the knuckle region of the steel containment. One area of wetness was observed along the southern portion beginning about 6 ft west of penetrations #66 and #71 and continuing for 12 to 15 ft westerly along the arc. The water was in the pocket of ethafoam cushion/flexible material, but it was not possible to determine whether the water was coming from the fill concrete below or whether the water had come from above and collected in this region.

In all instances where water or moisture was observed that was clearly identifiable with seepage from the surface of the common basemat, no flow or excessive buildup of water was noted.

ATTACHMENT H

JUL 12 1983

NOTE TO: M. Peranich, Team Leader, Waterford Inquiry

FROM: R. Shewmaker, Senior Civil Structural Engineer

SUBJECT: ADDITIONAL INFORMATION

Enclosed are two documents both dated 7/7/83, which I prepared as a result of my efforts on the Waterford Inquiry. Since we do not know which group in the NRC will be taking actions on any of the items the Team recommended for follow up, I believe these two documents may assist in assuring that the issues I defined are considered.

Note that the expanded actions are correlated to the report draft of 7/8/83 and not the final report as renumbered as a result of consolidation.

Bob Shewmaker, IE

Enclosures:
1. Expanded Actions on Common Mat
2. Opinions/Judgements

DISTRIBUTION
IE Files
DEPER R/F
EGCB R/F
REShewmaker R/F
REShewmaker

OFFICE	EGCB:DEPER:IE					
SURNAME	REShewmaker					
DATE	7/12/83					

Peranich

WATERFORD 3

EXPANDED ACTIONS ON COMMON MAT

July 7, 1983

These numbers correspond to those of the actions listed in the Team Report and expand into specific areas.

1. None

2. In executing the engineering evaluation the following items should be included.

   a. Consideration of the acceptability of the concept of stopping the water proofing membrane on the exterior of the common base mat generally at Elev. -37' and not extending it down and beneath the mat.

   b. Placement of lower waterstop in vertical construction joints between mat blocks above the bottom reinforcing steel instead of below.

   c. Consideration of the concepts of the 1971 ACI Committee 350 Report, "Concrete Sanitary Engineering Structures", which recommends the utilization of WSD and lowered allowables for concrete and reinforcing steel stresses in order to assure water tight and chemically resistant concrete construction.

3. In conducting a complete survey and examination of cracks, seepage zones etc., the following items should be considered.

   a. Use of sonic or other nondestructive methods to identify questionable zones with regard to cracking and/or low density.

   b. Grinding and polishing some zones for in-place microscopic examination.

   c. Use of cut samples which could be studied in a laboratory.

4. In the attempt to determine the source of the water a re-examination of the procedures, records and actual details of the "elaborate methods" taken to insure that borings and other preoperational features were plugged should be completed.

5. None

6. None

7. In the attempt to determine the change in character or build up of deposits in the seepage zones petrographic examination techniques may be desirable.

8. In providing the evaluation related to the corrosion potential the licensee should gather all relevant data from the surrounding industrial facilities

including their 3 fossil plants and the several chemical and petroleum facilities.

9. None

10. None

11. Prior to embarking on the program to develop a report the licensee should outline the total program and meet with the NRC in order to be certain the program is adequate.

R. E. Shewmaker

R. E. Shewmaker, P.E.
Senior Civil-Structural Engineer
July 7, 1983

Peranich

WATERFORD 3

OPINIONS/JUDGMENTS

JULY 6, 1983

The following statements are made based on my engineering, design, construction and inspection training and experience and the field observations made at Waterford 3 on 6/30/83.

1. Original concept of the leak tight, "floating island" with a controlled construction sequence and controlled ground water level was excellent.

2. Execution of the concept in the design and engineering phase, in my opinion, by omiting waterproofing membrane as a requirement over the entire vertical peripheral face of the common mat and also the underside of the mat was not a conservative approach to achieving a "watertight barrier".

3. Execution of the concept in the design and engineering phase, in my opinion, made the assumption that cracking or seepage would not occur through the common basemat and should have considered the conservative design procedures as recommended in a 1971 report by ACI Committee 350, entitled, "Concrete Sanitary Engineering Structures" to help assure this would in fact be the case. Relevant aspects of this report provide recommendations for structural design, materials, and construction of structures used to retain water and chemically ladened water where dense, impermeable concrete with high resistance to chemical attack and minimized possibility of cracking and permeability is desired. The report recommended use of WSD with lowered stress allowables for concrete and reinforcing steel which were not used for the Waterford 3 structure.

4. Execution of the concept deviated from the most desireable conditions in the construction phase and allowed piezometric levels to rise 10 to 25' in certain layers possibly causing reversal of flexure in the common mat and related cracking in the top surface.

5. No significant cracking associated with the current seepage zones has occurred since mid-1977.

6. Some current seepage zones may have become active recently but it is most likely that the zones have been active before and this was not recognized due to the general debris, water, etc. on the Elevation -35 floor as a result of heavy construction activity.

7. Source of seepage is ground water.

8. The water source for seepage must be identified and seepage into the common mat stopped if zones of seepage can be identified or it must be established that controlled seepage will not result in detrimental corrosion of reinforcing steel or the steel containment.

R. E. Shewmaker, P.E.
Senior Civil-Structural Engineer
July 6, 1983

8/10/83

Correlation

Memo Shewmaker to Peranich 7/12/83	Team Report, Memo Peranich to De Young, 7/14/83
Actions	Action
1.	(1)
2	(2)(a) & (2)(b) & (2)(d)
3	(3)(a)
4	(3)(b)
5	(3)(b)
6	(3)(c)
7	(3)(c)
8	(2)(c)
9	(3)(d) & (3)(e)
10	(4)
11	(5)