Landsman

NOV 2 4 1981

Docket No. 50-329
Docket No. 50-330

MEMORANDUM FOR: Darrell G. Eisenhut, Director of Licensing, NRR

FROM: Charles E. Norelius, Director, Division of Engineering and Technical Inspection, RIII

SUBJECT: MIDLAND 1 AND 2, AUXILIARY BUILDING SEISMIC ANALYSIS AND CRACKS IN THE BORATED WATER STORAGE TANK FOUNDATION

Pursuant to 10 CFR 50.55(e), Consumers Power Company (CPCo) notified RIII on January 21, 1981, that the auxiliary building seismic analysis was done incorrectly. CPCo also notified RIII on January 22, 1981, that cracks were observed in the borated water storage tank foundation.

The CPCo final report concerning the auxiliary building seismic analysis is included as Attachment 1. This attachment references three interim reports concerning the same subject. In addition the licensee stated that the final resolution would be demonstrated by the seismic analysis being performed in conjunction with the responses to the NRC 10 CFR 50.54(f) request regarding plant fill for the Midland Plant, Units 1 and 2.

Attachment 2 is the most recent CPCo interim report concerning cracks in the borated water storage tank foundation. This report also references six previously issued interim reports concerning the same subject.

In our view, these construction deficiencies have the potential for affecting the design adequacy of several safety related structures at the Midland site. As you are presently evaluating these problems as a part of the soils related issues at Midland, this information may aid in the resolution of this matter.

If you have questions or desire any site followup, please contact us.

Charles E. Norelius, Director
Division of Engineering and
Technical Inspection

8408170276 840718
PDR FOIA
RICE84-96 PDR

Attachments: As stated

cc w/attachments:
R. Shewmaker, IE HQ

RIII Spessard 11/23 ~~RIII Davis~~ ~~RIII Keppler~~

OFFICE	RIII		RIII	RIII	RIII	RIII	RIII
SURNAME	Landsman/ls	Hawkins	Jones	Boyd	Williams	Knop	Norelius
DATE	11/19/81	11/19/81	11/19/81	11/19/81	11/23/81	11/23/81	11/ /81

NRC FORM 318 (10-80) NRCM 0240

OFFICIAL RECORD COPY

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

81-02 #4

July 31, 1981

Mr J G Keppler, Regional Director
Office of Inspection and Enforcement
US Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND PROJECT -
DOCKET NOS 50-329, 50-330
AUXILIARY BUILDING SEISMIC ANALYSIS
FILE: 0.4.9.48 SERIAL: 12067

Reference: CPCo letters to J G Keppler, Same Subject:

1) Serial No 11200, dated February 20, 1981
2) Serial No 11972, dated April 16, 1981
3) Serial No 12008, dated May 29, 1981

The referenced letters were interim 50.55(e) reports concerning the auxiliary building seismic analysis. This letter is the final report. Attachment 1 provides a summary of the actions which have been taken to resolve this concern. Final resolution will be demonstrated by the seismic analysis being performed in conjunction with the 50.54(f) concerning soils.

James W. Cook

WRB/lr

Attachment 1: MCAR-47, Final Report, dated July 17, 1981
"Auxiliary Building Seismic Analysis"

CC: Director of Office of Inspection & Enforcement
Att: Mr Victor Stello, USNRC (15)

Director, Office of Management
Information & Program Control, USNRC (1)

RJCook, USNRC Resident Inspector
Midland Nuclear Plant (1)

ATTACHMENT 1

~~8108110482~~

Copp.

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FPCowan, ASLB Panel
AS&L Appeal Panel
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MSinclair
BStamiris
CRStephens, USNRC
WDPaton, Esq, USNRC
FJKelly, Esq, Attorney General
SHFreeman, Esq, Asst Attorney General
WHMarshall
GJMerritt, Esq, TNK&J

036334

Attachment 1
Serial 12067
81-02 #4

Bechtel Associates Professional Corporation

777 East Eisenhower Parkway
Ann Arbor, Michigan

Mail Address P.O. Box 1000, Ann Arbor, Michigan 48106

SUBJECT: MCAR 47 (Issued 1/29/81)

Auxiliary Building Seismic Analysis

FINAL REPORT

DATE: July 17, 1981

PROJECT: Consumers Power Company
Midland Plant Units 1 and 2
Bechtel Job 7220

Description

During a seismic reanalysis associated with the 10 CFR 50.54(f) plant fill issue, it was noted that the 1977 auxiliary building seismic model considered the control tower and the main portion of the auxiliary building as an integral unit between el 614' and 659'. This assumption is not appropriate for the north-south direction because of the connection between the control tower and the main structure, which consists primarily of reinforced concrete slabs. The auxiliary building and the control tower were structurally designed using input from a 1974 seismic model that included flexibility at the connection between the control tower and main structure. Equipment and systems have been seismically qualified using output from the 1974 or 1977 seismic models, depending on the purchase date.

Safety Implications

There is actually no potential safety impact on the auxiliary building and its contents because it will be modified under the 10 CFR 50.54(f) remedial soils action and the final design will meet acceptance criteria prior to plant operation. The investigation described in this report was initiated solely to determine the potential safety impact on the "pre" 10 CFR 50.54(f) auxiliary building structure and did not include the structural modifications in progress to resolve the 10 CFR 50.54(f) remedial soils action.

Potential safety implications on the "pre" 10 CFR 50.54(f) remedial soils action structure were determined for equipment and piping as described in this report but were not determined for the control tower, its connections to the main auxiliary building, or the electrical penetration areas.

Investigation

The investigation presented was limited to the north-south, 1977 seismic model (FSAR Figure 3.7-10) because the structural behavior due to seismic motions in the east-west and vertical directions is judged not to be influenced by this change. The control tower and the main auxiliary building (el 614' to 659') were modeled as two separate structures connected by flexible links, this investigation considered resulting changes in the building forces and floor response spectra curves.

The investigation consisted of:

1) A response spectrum analysis to develop building forces

2) A time-history analysis to develop in-structure floor response spectra at selected locations

3) Comparison of building responses to values calculated in 1974 and 1977.

4) Comparison of instructure floor response spectra to those generated in 1977, at selected locations, and comparison of loads in selected piping systems and equipment systems to allowable loads if necessary.

The current status of this investigation follows.

1) The response spectrum analysis has been completed.

2) The time-history analysis and selected in-structure floor response spectra have been generated.

3) A comparison of the building forces has been made. The greatest change in building forces was confined to the structural steel superstructure, the control tower, and the electrical penetration areas at el 674'-6" and above. By inspection, the forces in the other portions of the building meet the acceptance criteria.

   Based on a preliminary stress analysis of the "pre" 10 CFR 50.54(f) remedial soils action structure, several areas in the control tower and its connection to the auxiliary building were calculated to be overstressed in load combinations with seismic forces. This preliminary analysis distributed the seismic forces to various structural elements using conventional [illegible] methods. Because this was [illegible] a definitive analysis, a conclusion regarding potential safety implication cannot be drawn. The analysis being performed for the building as modified by the 10 CFR 50.54(f) remedial soils action will demonstrate the adequacy of the final design of this structure.

4) A comparison of the in-structure response spectra curves has been made. The greatest changes were confined to the structural steel superstructure, control tower, and electrical penetration areas at el 674'-6" and above. The frequencies most affected by this change were between 4 and 10 cps. The maximum increase in acceleration

2) FSAR Section 3.7 and Specification 7220-C-7 will be changed upon completion of the 10 CFR 50.54(f) remedial soils action.

Root Cause

This assumption was not caused by a failure to follow a procedure. All procedures pertaining to the origination, checking, review, and approval of calculations were followed.

This assumption involves a subjective technical determination of the most effective way to mathematically model a physical feature of the structure. The methods and values used were appropriate for the east-west direction, but detailed design review revealed that the methods and values used did not adequately represent the structure in the north-south direction.

Because these parameters are specifically and uniquely determined for each portion of the structure, this assumption is believed to be a random occurrence with no generic implications. Therefore, there is no generic or process corrective action planned. To support this, all models used in the analysis of Seismic Category I were visually inspected, and no geometric situation was identified which would lead to a similar model assumption in development of modal properties.

Reportability

This was reported by Consumers Power Company to the NRC as a potentially reportable 10 CFR 50.55(e) item on January 21, 1981. To date, it has not been established whether this item is "reportable" under the criteria of 10 CFR 50.55(e). The final design under the 10 CFR 50.54(f) soils issue will eliminate the safety impliciations (reportability), if any, addressed by this MCAR.

Prepared by: S. S. Sobkowski
S.L. Sobkowski

Approved by: N Swanberg
N. Swanberg

Concurrence by: K D Bailey
K.D. Bailey

occurred at approximately 6 cps and was 1.6 times the previous spectra values. In other areas in the building, the new in-structure response spectra did not differ significantly from the existing spectra and, therefore, by inspection, the components in these areas satisfy the acceptance criteria.

A selected sample of piping systems in the affected area were checked and found to meet acceptance criteria except as noted below. The piping systems that were selected for evaluation were located in the area where the greatest change in seismic loads occurred and where the pipe or hanger stresses were close to the maximum allowable before checking the new seismic stresses. The auxiliary steam and turbine exhaust vent stack to the atmosphere is the only system found that could not meet the acceptance criteria. The analysis of the vent stack system for the increase in seismic loads identified one of the supports that did not satisfy the acceptance criteria. Because this support has a substantial factor against ultimate failure, this does not appear to have a safety impact. The analysis being performed for the 10 CFR 50.54(f) soils issue will demonstrate the adequacy of the final design of this piping system.

A selected sample of equipment in the area affected were found to satisfy acceptance criteria. Equipment was selected to be checked based on its potential for change. The revised spectra were compared to the spectra used to seismically qualify the equipment, and the equipment still satisfied acceptance criteria.

Corrective Actions Completed

1) During the week ending January 23, 1981, the assumption that the control tower and the main portion of the auxiliary building is a nonintegral unit between el 614' and 659' was incorporated in a modified model of the auxiliary building. Accordingly, this action is complete.

2) The structural response spectra analysis has been completed.

3) The time-history analysis and corresponding in-structure floor response spectra have been generated.

4) Selected equipment systems, selected piping systems, the structural steel superstructure, and the stability of the main auxiliary building have been checked.

Corrective Actions to be Completed

1) Demonstrate that the final design meets acceptance criteria. This will be done through the 50.54(f) remedial soils action. The schedule will be established in 10 CFR 50.54(f) responses.

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

81-03 #7

October 26, 1981

Mr J G Keppler, Regional Director
Office of Inspection & Enforcement
US Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND PROJECT -
SURCHARGE OF THE BORATED WATER
STORAGE TANK FOUNDATION
FILE: 0485.16, 0.4.9.49 UFI: 73*10*01, 71*01,
01100(E), 02362(S), 00234(S) SERIAL: 14591

References: J W Cook letters to J G Keppler
(1) Serial 11201, dated February 20, 1981
(2) Serial 11528, dated April 3, 1981
(3) Serial 12015, dated June 12, 1981
(4) Serial 12799, dated June 26, 1981
(5) Serial 13352, dated July 21, 1981
(6) Serial 13653, dated August 28, 1981

This letter, as were the referenced letters, is an interim 50.55(e) report concerning the existence of cracks in the borated water storage tank foundation. Per discussion with C Jones of your staff on October 5, 1981, it was agreed that this interim report would be delayed beyond the date provided in reference 6.

Approval to surcharge the valve pit areas of the borated water storage tank foundations was received from the NRC staff on September 25, 1981. The surcharge operation will commence after the commitments are incorporated into the design documents. The analysis of the modifications to the foundation and of the borated water storage tank is in progress. A final report on the foundation analysis is expected to be completed by November 30, 1981.

Another report, either interim or final, will be sent on or before December 11, 1981.

James W. Cook

WRB/lr

CC: Director, Office of Inspection & Enforcement
Att Mr Victor Stello, USNRC (15)

8110090690 2pp.

ATTACHMENT 2

Serial 14591
81-03 #7

CC: Director, Office of Management
Information & Program Control, USNRC (1)

RJCook, Midland Resident Inspector (1)
HRDenton, NRR (4)
JDKane, NRR (1)
DHood, NRR (1)
CBechhoefer, ASLB Panel
RSDecker, ASLB Panel
FPCowan, ASLB Panel
JHarbour, ASLB Panel
MSinclair
BStamiris
MMCherry, Esq

James W Cook
Vice President - Projects, Engineering and Construction

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November 24, 1981

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D/D			
A/D			
			✓

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

Hardsman ✓

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
RESULTS OF SOIL BORING AND TESTING
PROGRAM FOR AUXILIARY BUILDING (PART 2)
FILE 0485.16, B3.0.1 SERIAL 14874
REFERENCE: JWCOOK LETTER TO HRDENTON, SERIAL 13774, DATED SEPTEMBER 22, 1981
ENCLOSURE: TEST RESULTS, AUXILIARY BUILDING (PART 2), SOIL BORING AND TESTING PROGRAM, MIDLAND PLANT - UNITS 1 AND 2

We are providing thirty (30) copies of the enclosed Woodward-Clyde Consultants (WCC) report (Part 2) dated October 26, 1981 which documents the soil boring and sampling program and the subsequent laboratory testing program for the foundation soils at the auxiliary building. The results of these programs are presented in the form of logs of borings and in both tabular and graphical data summaries of index property, strength and compressibility testing of the foundation soils. This data is found in the appropriate appendices of the enclosed report.

Similar index and engineering property results for the auxiliary building foundation soils were presented in the WCC report of August 28, 1981 entitled, "Test Results, Foundation Soils, Auxiliary Building (Part 1), Soil Boring and Testing Program, Midland Plant - Units 1 and 2," which was previously forwarded to the NRC with the referenced correspondence of September 22, 1981. For completeness, all test results from the Part I report are included in the enclosure to this correspondence. Hence, the results of the testing programs presented in the enclosed Part 2 report are the combined results of those previously presented in the referenced Part 1 report and additional tests on soil samples of fill material and the natural foundation soil below Elevation 540'.

The Borings COE-17 and COE-18 penetrated the granular and cohesive fill material from the existing ground surface at approximate Elevation 634' to an approximate depth of 51 and 52 feet, respectively. The ranges and average values of the results obtained from the laboratory tests performed by WCC on

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the fill soil samples from both these borings are presented in the WCC report, Part 2. It can be seen from the index properties that the fill materials consisted of low plasticity silty sandy clay, clayey sand and fine to medium sand.

The WCC index properties and strength test results performed on undisturbed foundation soil samples obtained from the Borings COE-17 and COE-18 yielded the following average values at three selected thicknesses of natural soil stratum below the planned underpinning foundation elevations.

	Undrained Shear Strength (S_u), Ksf		
	Stratum Between Elevations (Ft)	Undrained Shear Strength (Ksf) Range	Avg
4 Undrained Unconsolidated (UU) Tests	570 to 560	5.18-7.66[1,2]	6.9
5 (UU) Tests	560 to 540	7.24-10.39	8.7
11 (UU) Tests	540 to 436	6.61-10.88	8.3

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1 One UU test at Elevation 560.3' gave shear strength of 2.57 ksf and was not considered because the laboratory noted that sample disturbance took place.

2 Another UU test at Elevation 581.4' gave shear strength of 2.62 ksf and was not considered because a depth of fill found in both the borings was to Elevation 582'. Probably this sample represented fill material.

The shear strength values presented in the above table includes those values of shear strength for the elevations of 570' through 540', previously presented in the referenced letter dated September 22, 1981, and the values of shear strength obtained from the additional tests on foundation soil samples between Elevations 570' and 436'. It can be seen from the above table that the average shear strength for the stratum above Elevation 560' was about 6.9 ksf and the average shear strength values were slightly greater than 8 ksf below Elevation 560'. Consequently, the average values of shear strength reported in our letter of September 22, 1981 are of the same approximate value as those based on the complete test program which are reported herein. It is seen from the foregoing discussion that the allowable bearing capacities based on the average shear strengths obtained from WCC tests are greater than the conservative values of allowable bearing capacity which were based on FSAR shear strength data.

Four consolidation tests were previously reported in the referenced Part 1 report. These four consolidation tests along with two additional tests are reported in the enclosed Part 2 report. These six consolidation tests were performed on soil samples obtained in the natural soils to determine a preconsolidation pressure value. The values of preconsolidation pressure from the resulting log compression curve were evaluated using Casagrande's construction. Based on this evaluation, the preconsolidation pressures ranged from 26 ksf to 84 ksf. This range is much higher than the previously estimated range of 15 ksf to 20 ksf given in FSAR Subsection 2.5.4.2.9. This substantiates the heavily preconsolidated nature of the natural deposits in the area of the auxiliary building.

These conclusions and the data results attached as an enclosure should provide the NRC with the information necessary to evaluate the soils conditions at the auxiliary building.

G.S. Keeley for JW Cook

JWC/RLT/dsb

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, Esq, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

R. Landsman
RGN III

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

NOV 23 1981

PRINCIPAL STAFF			
DIR		F&LS	
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DE&TI			
DEP&OS	-	File	

Landsman

Docket Nos. 50-329 OM, OL
and 50-330 OM, OL

APPLICANT: Consumers Power Company

FACILITY: Midland Plant, Units 1 & 2

SUBJECT: SUMMARY OF NOVEMBER 12, 1981 MEETING ON CONSTRUCTION SCHEDULES FOR FOUNDATION MODIFICATIONS TO AUXILIARY BUILDING

On November 12, 1981, the NRC staff met in Bethesda, MD, with Consumers Power Company (CPCo) to discuss construction schedules needed for the planned remedial actions to the Auxiliary Building at the Midland plant. The remedial action, underpinning, results from the settlement potential of the backfill soils beneath the control tower and electrical penetrations area of that structure. Similar action is planned for the adjacent Feedwater Isolation Valve Pits and was included in the meeting discussions. Meeting attendees are listed in Enclosure 1.

Vice President J. Cook of CPCo reviewed the development history for the proposed remedial action which had initially been based upon use of jacking caissons, but which, by September 1981, had been changed to a structural wall extending to the glacial till. Mr. Cook emphasized that the construction schedule for the Auxiliary Building underpinning was critical to the July 1983 fuel load date for Unit 2. For this reason, Consumers had earlier asked the Licensing Board to rearrange the hearing sessions to consider the Auxiliary Building before the Diesel Generator Building session. To prepare for implementing the underpinning, a vertical access shaft on the east and west ends of the auxiliary building and adjacent to each feedwater isolation valve pit and the turbine building needs to be started by mid-December 1981, and a freezewall by December 29, 1981. Staff approval of these two matters were requested by Mr. Cook's letter of October 28, 1981. The schedule for start of drifting beneath the structures is February 15, 1982. Mr. Cook further emphasized that continuing staff review throughout the underpinning process was needed, rather than a traditional two-step staff approval process. He felt that more staff review and observation in the field should be considered to expedite the review process. Review procedures such as that which had been followed during the staff's structural design audit at Anne Arbor, Michigan, in May 20 - 24, 1981, were also recommended.

Mr. D. Eisenhut agreed that staff approval prior to implementing the fix was needed. In view of the construction schedule, he suggested specific approval points by the staff or other conditions be defined based upon the planned construction activities and sequences comprising the underpinning scheme. He noted that establishment of acceptable conditions could assist in the authorization to proceed. It was agreed that a working meeting the following week would be scheduled to this end. To the extent possible, such conditions would be reflected in hearing testimony.

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DEC 10 1981

Mr. M. Miller, Esq., noted that conditions could not be established within the existing schedule for filing testimony (due November 17, 1981) and that Consumers would like to ask the Board to accept a delay of a few days in the filing date. Mr. W. Olmstead, Esq., replied that the staff would not object to such a request.

Messrs. G. Keeley and D. Budzik of CPCo described the preliminary analysis of the Auxiliary Building to be provided for staff review on November 20, 1981. The preliminary analysis will consider selected critical structural members and selected loading combinations. An analysis of the construction sequence for the underpinning scheme will be completed January 1, 1982. The final analysis will be provided for staff review February 15, 1982. It was noted that the latter date corresponds to the start of drifting beneath the structure. The final analysis is primarily for the electrical penetration area and control tower portions of the structure. The analyses for the overall structure will be completed April 15, 1982. June 1, 1982 is the earlist date that the FSAR can be updated to reflect the results of the completed analyses.

At the conclusion of the meeting, and in preparation of the working session planned for November 17, 1981, Mr. Budzik provided the following schedule drawings to the staff's project manager:

(1) Drawing 7220-PPS-020, Revision 0, dated 11/06/81, "Project Production Schedule: Auxiliary Building Underpinning Schedule", sheets 1 and 2.

(2) Drawing 7220-PPS-021, Revision 0, dated 11/06/81, "Service Water Pump Structure Remedial Action - (Underpinning Wall)".

Darl Hood, Project Manager
Licensing Branch #4
Division of Licensing

Enclosure:
As stated

cc: See next page

OFFICE	DL:LB#4 DSH	DL:LB#4	DL:LB#4				
SURNAME	DHood:lb	BJDuncan	EAdensam				
DATE	11/18/81	11/ /81	11/19/81				

MIDLAND

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ENCLOSURE

LIST OF ATTENDEES

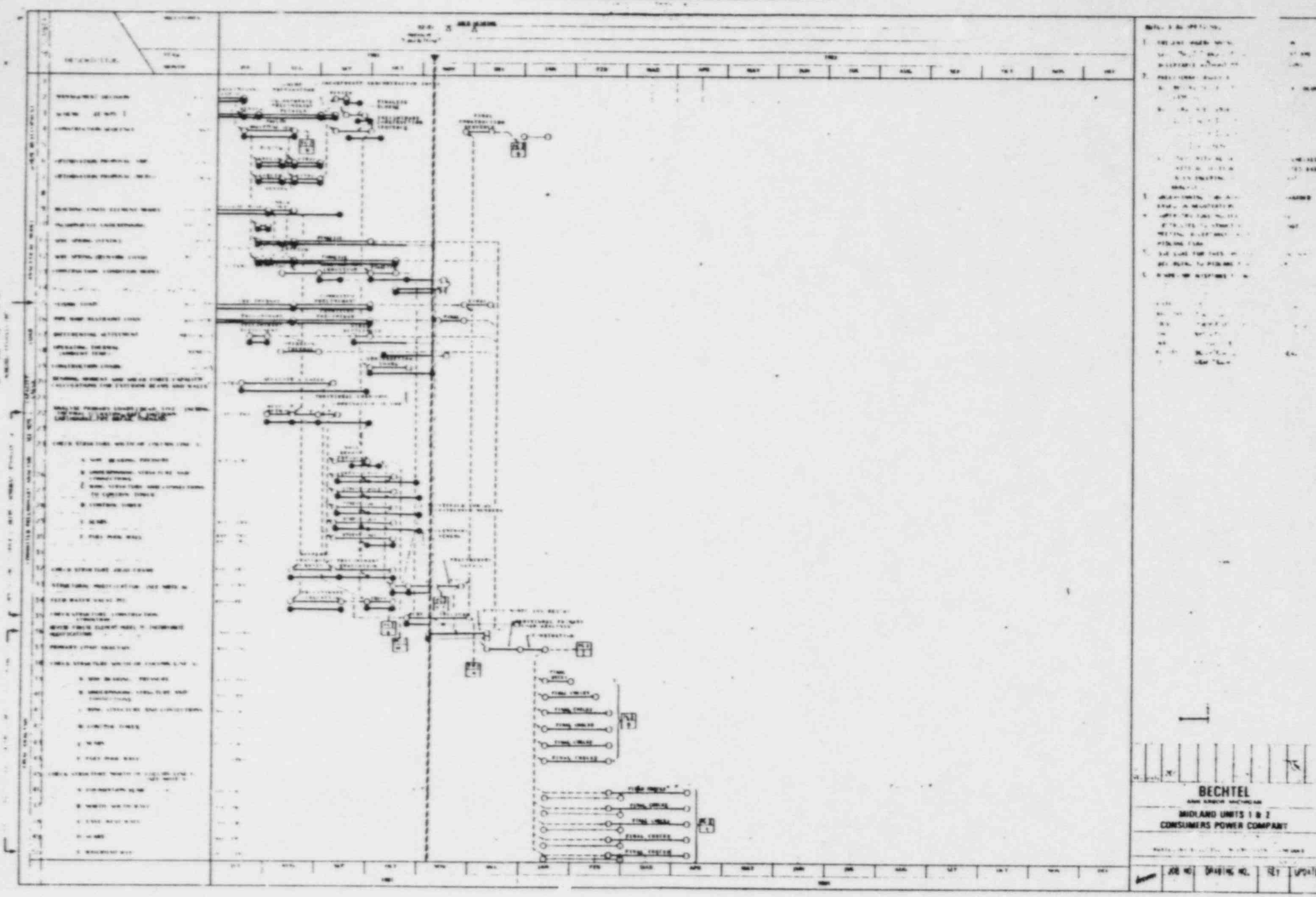
MIDLAND MEETING 11/12/81

NRC

D. Eisenhut	J. Kane
R. Vollmer	F. Rinaldi
J. P. Knight	A. Cappucci
E. Adensam	G. Lear
W. Olmstead	F. Schauer
J. Rutburg	R. Landsman
W. Paton	
D. Hood	

Consumers

J. Cook
D. Keeley
D. Budzik
M. Miller (IL&B)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MIDLAND UNITS 1 & 2
CONSUMERS POWER COMPANY
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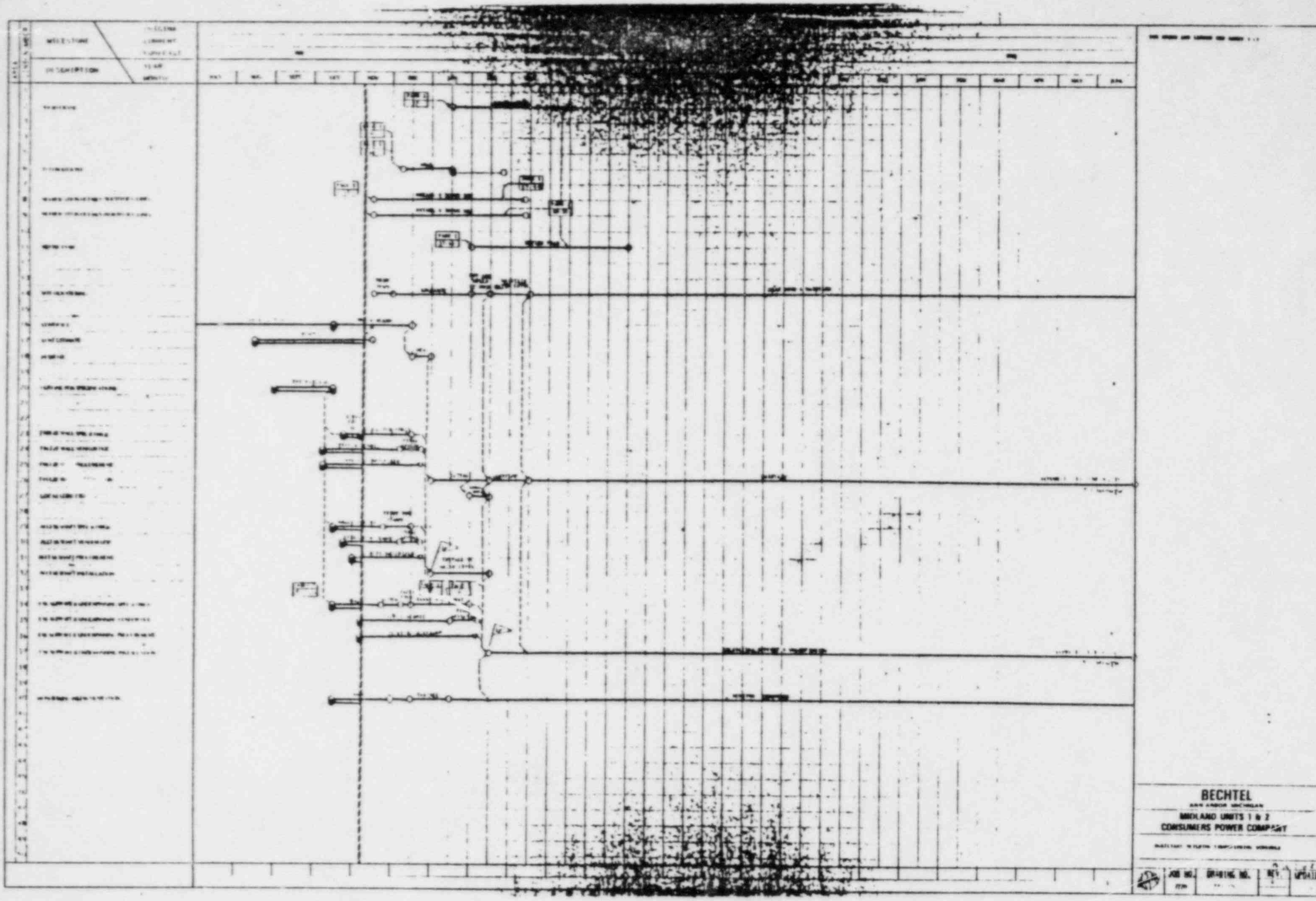
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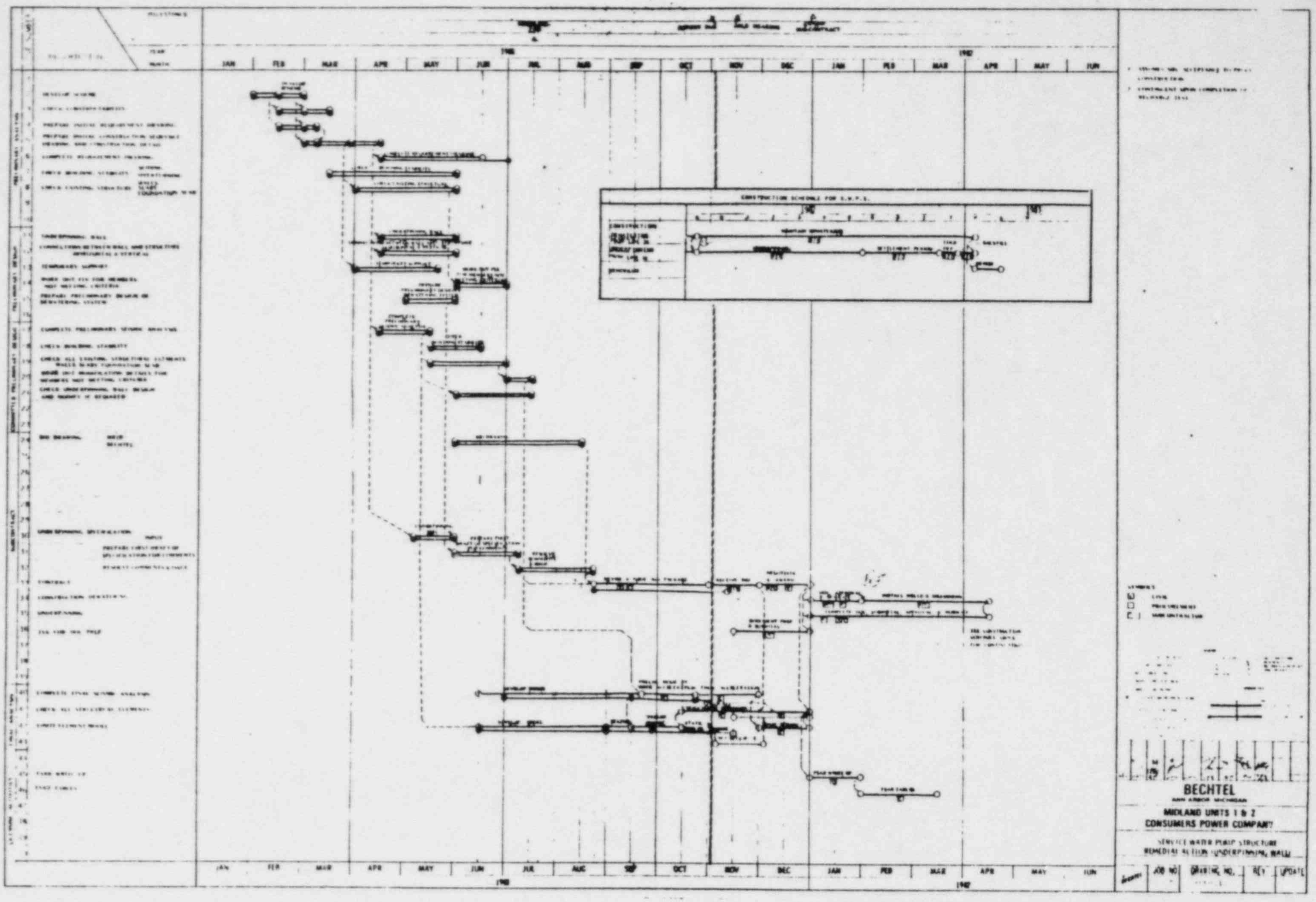

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CONSTRUCTION SCHEDULE FOR S.W.P.S.
CONSTRUCTION
BECHTEL
ANN ARBOR MICHIGAN
MIDLAND UNITS 1 & 2
CONSUMERS POWER COMPANY
SERVICE WATER PUMP STRUCTURE
REMEDIAL ACTION UNDERPINNING WALL
JOB NO.
DRAWING NO.
REV
UPDATE

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

November 16, 1981

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U.S. NUCLEAR REGULATORY COMMISSION

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
RESPONSE TO NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS
FILE 0485.16, B3.0.1 SERIAL 14869
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ENCLOSURE: RESPONSE TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

Attached to our previous correspondence of September 30, 1981, referenced above, was a document entitled, "Technical Report on Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits," which was included as Enclosure 3. On October 31, 1981, a request for additional information relating to Enclosure 3 of our September 30, 1981 correspondence was made by the Staff in a telephone conference call. The Staff request for additional information took the form of thirty (30) questions which were developed during the review of Enclosure 3 by the NRC and the Army Corps of Engineers and which related only to the geotechnical engineering aspects in underpinning the auxiliary building and feedwater isolation valve pit structures.

We are responding to this Staff request by forwarding the enclosure to this correspondence which is entitled, "Response to the NRC Staff Request for Additional Information Pertaining to the Proposed Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits." This enclosure addresses each of the individual Staff concerns identified for us during the October 30, 1981 conference call. Our responses to the thirty (30) Staff questions were presented verbally during the November 4, 1981 meeting in Bethesda with the NRC.

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We believe the enclosed information combined with the discussion of these responses at our November 4, 1981 meeting adequately responds to the request and individual concerns identified for us by the Staff. The responses contained in the enclosure to this correspondence lend further support to our conclusion that the design of the auxiliary building and feedwater isolation valve pit structures combined with the remedial actions are adequate and appropriate for these structures.

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RESPONSE TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

MIDLAND PLANT UNITS 1 AND 2
RESPONSE TO THE NRC STAFF REQUEST
FOR ADDITIONAL INFORMATION PERTAINING TO
THE PROPOSED UNDERPINNING OF THE
AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

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INTRODUCTION

On October 1, 1981, representatives of Consumers Power Company, Bechtel Power Corporation, and the NRC met in Bethesda, Maryland, for a presentation of the proposed remedial action for the Midland plant auxiliary building and feedwater isolation valve pits (FIVPs). The discussion of the proposed underpinning construction resulted in several requests for additional information. This report responds to these requests and supplements the Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits (Reference 1).

QUESTION 1

(Pg. 2, Sect. 4, 2nd Para.) Please define "design jacking force," how established and the duration that it will be held?

RESPONSE

The design jacking force was based on an estimated reaction for dead load and 25 percent of the live load condition. The jacking load was calculated so that in each area the jacking force was approximately equal to the load from the structure above. The total jacking force in various areas is shown in Figure 4 of Reference 1. The average load per linear foot in various areas is shown in Figure 1. The adequacy of the jacking load will be verified by analysis.

The duration of the jacking load depends on completion of all immediate settlement and bringing the supporting till to a condition of secondary compression. It is also intended to complete a major portion of the deflection due to creep and shrinkage of concrete. Satisfactory completion of the jacking will be indicated by a final straight line trend on a plot of total deflections due to soil settlement, creep, and shrinkage versus log time.

It is currently estimated that the jacking load will be applied for about 90 days. However, the main criterion is to be within the straight line portion of the semilog plot; this duration is anticipated to be less than 90 days. Also, a suitable concrete mixture to achieve early concrete shrinkage is presently being investigated. Detailed criteria for jacking duration are presented in Subsection 7.2.3 of Reference 1.

QUESTION 2

(Pg. 2, Sect. 4, 3rd Para.) Discuss and provide detail of dowel connection. (Diameter, how distributed along wall, length of embedment, etc).

RESPONSE

Based on the preliminary design, the dowel connections are designed as follows:

Dowel	Type and Average Spacing	Embedment (in.)
Horizontal	2-inch diameter rock bolt at 30 inches center-to-center	36
Vertical	2-inch diameter rock bolts at 20 inches center-to-center	36

Actual spacing will depend upon clearing and missing other obstructions. Locally, it may be necessary to replace rock bolts by equivalent areas of dowels.

At first, the dowels will be grouted only on one side of the structure or underpinning. The other side will be grouted only after jacking loads are applied and held. To achieve this for the horizontal dowels, the end portion of the underpinning wall will be poured after jacking loads are applied and held long enough for the till to be within secondary compression and the criterion outlined in the response to Question 1 is met.

QUESTION 3

(Pg. 3, Sect. 5.1, last para) The agreed upon acceptance criteria for soil particle monitoring during dewatering requires 0.005 mm and not 0.05 mm. Correction by CPCo required.

RESPONSE

In response to Question 42(2)(a) of the Responses to NRC Requests Regarding Plant Fill, it was stated that the operating construction dewatering systems are monitored for soil particles coarser than 0.05mm. The Corps of Engineers' letter (Reference 2) indicated its acceptance of the response to Question 42(2)(a). Mr. Kane's deposition of March 17, 1981, indicated review of the response to Question 42, but there were no comments on the construction dewatering system. In the meeting between Consumers Power Company, Bechtel, COE and the NRC on May 7, 1981, a discussion was held regarding the monitoring of soil particles. The NRC and COE were again informed that the construction dewatering wells were being monitored for particle sizes greater than 0.05mm. The NRC did not indicate any concern regarding the construction dewatering system, although it was interested in the results that were obtained. It was indicated to the NRC and COE that a finer filter was used during a period in which the 0.05mm filter media was not available. The NRC staff stated that it wanted the permanent wells to be monitored to particle sizes larger than 0.005mm because of the concern of removing silt-sized particles over the 40-year life of the wells. Consumers Power Company accepted the 0.005mm filter criteria for operation of all permanent wells in a telephone call with the NRC, COE, and Bechtel on May 18, 1981.

Construction dewatering is a short-term condition; therefore, monitoring of soil particles greater than 0.05mm in size is considered to be more than adequate.

QUESTION 4

(Pg. 3, Sect. 5.1, Para. b) Installing the frozen cutoff membrane will cause expansion and possibly increase the soil voids. When ultimately unfrozen, what is the effect (e.g., further settlement) on safety-related structures, conduits and piping. Provide discussion on the basic system of the frozen membrane [size and spacing of holes to be drilled, method for pumping brine into foundation layers, range of temperatures that are critical to wall stability which are to be monitored, decommissioning (e.g., grouting, etc)].

RESPONSE

The type of artifical ground freezing proposed for the frozen cutoff membrane is not anticipated to cause noticeable expansion of the soil (frost heave) nor consequent noticeable settlement. Heave and settlement are not anticipated for the following reasons.

1. The majority of the frozen membrane will be in a pervious sand which is not susceptible to frost heave because as the water expands due to formation of ice, it can migrate to adjacent pores and push other water ahead. Natural frost heave occurs in silts which can form ice lenses.

2. As shown in Figure B-3 of Reference 1, the wall will be maintained as a relatively thin vertical wall with a crest elevation of 610. Therefore, there will be 24 feet of overburden. Therefore, the wall is confined on all sides, thus minimizing the possibility of heave.

A package of information on heave and settlement associated with artificial frozen earth is being assembled, based on past experience, and will be presented soon.

In relation to the effect on the safety-related structures, the frozen cutoff membrane only comes in contact with the sides of the containment structures as shown in Figure 2, and no other safety-related structure. Coming into contact on the side rather than beneath eliminates the possibility of frost heave or settlement for these structures. All buried conduit and pipes will cross the frozen membrane at nearly right angles. Therefore, even if minor heave and settlements occurred, it would affect a very short distance along the utility. Hence, it can be concluded that the effect on the safety-related structure, conduit, and pipes will be minimal.

The frozen earth membrane will be developed by installing 8-inch diameter pipes in the ground at approximately 4 feet, 3 inches on center, (see Figure 2). The coolant for the system will be 55 parts inhibited ethylene glycol to 45 parts water, instead of brine as previously indicated. During freezing, the coolant will have a temperature of approximately -20F. After the wall forms, the coolant will be maintained at -5F to 5F. The ground temperature at the edge of the frozen wall membrane (approximately 1 foot, 6 inches from the freeze pipe) will be 32F. These temperatures are less severe to the containment structures than the ambient temperature during winter.

QUESTION 5

(Pg. 3, Sect. 5.2) Clarify the procedure to be used in post tensioning the Electrical Penetration Area. Where will the buoyancy force be transmitted to the foundation and in what manner?

RESPONSE

The post-tensioning has been installed and was used to pick up the loss of support resulting from the dewatering under the electrical penetration areas. This buoyancy force was being transmitted to the control tower by beam action of the electrical penetration wing walls. This additional force is 2,000 kips from each electrical penetration wing. The existing total load of the control tower is 30,000 kips. With the additional load from the electrical penetration wings, the control tower bearing pressure is within the allowable values.

QUESTION 6

(Pg. 4, Sect. 5.6, 2nd Para.) Please explain the meaning of "failure bearing capacity factors" and the basis for "the nine times the shear strength for the cone"?

RESPONSE

Correlations relating pressure read by the proving ring in psi and penetration of the pointed cone with bearing capacity for a strip footing were derived and are presented herein as Attachment 1. A bearing capacity factor of 9 for the cone was obtained from a publication by G.G. Meyerhof (Reference 3).

QUESTION 7

(Pg. 4, Sect. 5.b, 4th Para.) How will the equivalent soil modulus be determined? What is the depth that the measured settlement will be distributed over and what is the area to be used in determining the stress?

RESPONSE

Direct information on the soil modulus will be obtained from the application of the jacking loads to the individual piers for the temporary support of the two penetration structures. Pier settlement will be measured by dial gages as the jacking load is applied in stages. From the observed load and settlement values, an equivalent soil modulus will be computed using the procedure given in Figure 11-9 of Reference 4. The equation given in the procedure relates settlement, load, elastic modulus of the subsoil, and various correction factors to account for the shape of the footing in plan and the depth of embedment of the footing. The equation assumes a homogeneous elastic half-space. This modulus value will then be used to make a confirmatory evaluation of the permanent underpinning wall to ensure that the combination of final bearing pressure, bearing area, and embedment will limit settlement of the structure to tolerable values.

QUESTION 8

(Pg. 4, Sect. 6) Presently, this paragraph implies that crack monitoring will not be performed on the existing structure. Please correct. Before remedial underpinning begins an accurate and up-to-date record of cracks should be developed for those safety related structures which could potentially be affected by the underpinning operations. This background record should be verified by I&E inspection and could serve as the basis for evaluating any changes in cracks due to underpinning operations.

RESPONSE

During construction, the change in existing cracks and any new crack that may form in the structure due to the underpinning operation will be monitored. The last crack mapping of the auxiliary building was performed in November 1981. Cracks will be mapped just before the start of underpinning. This last crack mapping will be used as a baseline to monitor the new cracks and/or change in existing cracks due to the underpinning operation. I&E inspection is welcome to verify the baseline crack mapping.

QUESTION 9

(Pg. 5, Sect 6.1.1 and 6.1.2) When will the acceptance criteria for the differential and absolute settlement be provided to the NRC?

RESPONSE

The structural acceptance criteria for long-term differential and absolute settlement will be established by February 15, 1982. The structural analysis for acceptance criteria for construction underpinning is scheduled to be completed by January 1, 1982, which is prior to the beginning of any underpinning activities and will be provided to the NRC at that time.

QUESTION 10

(Pg. 5, Sect. 6.2) Provide the basis for establishing the crack width of 0.03 inch. Appendix D should also address crack monitoring requirements during underpinning (frequency of reading, format for presenting observations, action levels etc).

RESPONSE

The crack width of 0.03 inch was based on past experience. This crack corresponds to a stress level of 30 ksi calculated in accordance with NUREG 1602.

Section 6.2 of Reference 1 lists the criteria for monitoring cracks. The detailed document for crack monitoring will be the drawing which will also contain the detailed requirements for crack monitoring. Hence Appendix D of Reference 1 will not be revised.

The crack monitoring program is intended to supplement the settlement monitoring program. The settlements are monitored daily. The cracks in areas of expected high stresses will be monitored after each major underpinning operation.

The monitoring will be performed by a data service as explained in the response to Question 30. An evaluation of the cause of cracking is performed if a new crack or a change in existing crack width of 10 mils (0.01 inch) is observed. This 10-mil limit will provide adequate time for investigation. The settlement data are reviewed and if the crack mapping and settlement data show that the structure is tilting, the jacking force will be adjusted. If any crack reaches 30 mils, appropriate action will be taken after an engineering evaluation.

QUESTION 11

(Pg. 6, Sect. 7.2.1, last Para.) Provide discussion why the drained shear strength is not required to be considered in analyzing for adequate bearing capacity. Also in the last paragraph in Section 7.2.1, Pg. 7 indicate the basis for the 2 days and what would be required if the settlement rate does not reach a straight line trend in 2 days.

RESPONSE

It is appropriate to utilize undrained shear strength in a bearing capacity analysis of heavily preconsolidated cohesive soil, as indicated by the procedures in Figure 12-3 of Reference 4. However, the bearing capacity analysis was also performed utilizing the drained shear strength parameters for the heavily preconsolidated till as obtained from the test results (Reference 5) of Woodward-Clyde Consultants (WCC). These results gave the following drained shear strength parameters:

Internal friction angle, $\emptyset' = 23°$

Cohesion intercept, $C' = 1.2$ ksf

The computation (Attachment 2) utilizes the procedure in Figure 12-3 of Reference 4 (Ultimate Bearing Capacity of Deep Foundations in Soil). The ultimate bearing capacity thus computed is 44 ksf compared to the 40 ksf value determined from the analysis using undrained shear strength.

QUESTION 12

(Pg. 7, Sect. 7.2.2) Where are the WCC controlled rebound-reload cycle soil test results? What is the corresponding stress level with a secant modulus of elasticity equal to 3500 KSF?

RESPONSE

WCC provided results of the rebound-reload cycle in four undrained triaxial tests on soil samples from the auxiliary building area (Reference 5). The purpose of this cycled loading is to compensate for effects of sampling disturbance. Plots at enlarged scale are presented in Reference 5 (Figures D-4 (2/5), (3/5), (4/5), and 5/5). An undrained modulus was determined from each test by drawing a line between the initial point and the point with deviator stress of 1.5 ksf, representing about one-fourth of the undrained shear strength, on the reload portion of the stress-strain curve. The average value from the four tests was approximately 3,500 ksf. This is approximately 500 times the average undrained shear strength of 7 ksf, which is applicable to this same group of tests.

QUESTION 13

(Pg. 8, Sect 7.2.3, 1st Para.) The estimates of settlement using the referenced NAVFAC DM-7 do not include secondary consolidation. What secondary consolidation would be indicated if the consolidation test results using the appropriate load increment were used? Compare this estimate with values for permanent wall conditions "after jacking, long term". Please provide basis for the three estimated settlement values for "Load transfer points for temporary load to reactor footing" at the bottom of pg. 8 and discuss any effects of this settlement on the reactor and pipe connections.

RESPONSE

The anticipated total settlement under the various underpinning units was computed utilizing elastic theory summarized in Figure 11-9 of Reference 4. In this analysis, distinction must be made between the immediate settlement and volume change from primary consolidation of secondary compression on the basis of judgment and observational data. A reduced modulus value of 3,000 ksf, compared to the calculated value of 3,500 ksf as presented in the response to Question 12, was used to compute a total settlement which is intended to allow for secondary compression. The intent in jacking each underpinning unit is to maintain the load application until secondary compression is reached. Therefore, the immediate settlements and the primary consolidation, if any, would be accomplished during the jacking period without permitting settlement of the structure. Secondary compression was estimated from the following items.

1. The WCC test results (Reference 5) produced a coefficient of secondary compression in the stress range of interest between 0.0005 and 0.001 units of strain per log cycle of time. The typical underpinning unit will cause a significant stress increase in a depth equal to its width, 10 or 12 feet. Then strain due to secondary compression would convert to about 0.05 to 0.15 inch per log cycle of time.

2. Actual observations of settlement extending over several years at the auxiliary building and at the containment structures indicate that these large and heavily loaded structures, which rest on the preconsolidated till, settle approximately at a rate of 0.1 to 0.5 inch per log cycle of time. From this, it would be reasonable to conclude that the smaller and less heavily loaded underpinning units would settle typically 0.1 to 0.15 inch per log cycle of time.

3. General experience of settlement of large structures on heavily preconsolidated clay indicates that long-term, delayed settlement is typically one-fifth to one-third the

total settlement of the structure. This is illustrated by data presented in Reference 6.

Completion of jacking to the 40-year life of the structure is essentially two log cycles of time. Therefore, based on the above information, long-term settlement after jacking is estimated to range between about 0.2 and 0.3 inch per log cycle of time.

Settlement values during temporary loading on the reactor building were estimated by taking the concentrated load from the temporary pickup, dividing that over a bearing area of approximately 26 feet by 12 feet on the base of the reactor, and computing the settlement by the formula in Figure 11-9 of Reference 4 using a soil modulus of 3,500 ksf. This largely ignores the stiffness of the foundation and its capacity to transfer load over a larger area of the reactor foundation and is believed to be conservative.

The settlement due to temporary loads on the reactor footing will cause an infinitesimal differential settlement of the reactor building. Piping and cable tray systems attached to the reactor building will be checked to ensure their capability to accommodate this differential movement.

QUESTION 14

(Pg A-1, Sect. 1, 2nd Par.) Please indicate how the soil spring constants were established for long term loads.

RESPONSE

The long-term springs are derived by iteration. Soil pressures are calculated based on load applied and the foundation bearing areas. These soil pressures determine the expected soil settlements based on the assumption that the basemat has zero stiffness. Then these settlements are used in calculating the soil spring stiffness value (K) by dividing the bearing pressures by the settlements. These K values are used in analyzing the foundation as a stiff mat. The analysis gives new soil pressures and soil settlement values based on the structure's stiff basemat. The process is repeated until reasonable convergence is obtained between settlements by treating the base slab as flexible and stiff.

QUESTION 15

(Pg C-2, last Par. and Pg. C-6, Par. B) What are the protective construction measures planned for the Turbine Building and Buttress Access Shafts and when will they be placed? Please provide discussion on the sequence of operations to complete the drift beneath the Turbine Building and show sectional views of this work with respect to the Turbine Building foundations and affected piping and conduits.

RESPONSE

The term "protective construction" refers to the hand dug, reinforced concrete pits under the turbine building and under the buttress access shafts. These pits provide two protective functions.

1. They provide the vertical support for the areas of the turbine building and buttress access shafts that are to be undermined.

2. They provide lateral earth support for the soil which is to remain and continue to support the structures.

The drift under the turbine building will be constructed in approximately 4-foot segments. Small, wide flange square frames (preliminary sized at W6 x 16) will be used 4 feet on center as support sets for the lagging. The sequence of construction will be to excavate approximately 4 feet ahead of the last in-place set, then place a new set, install lagging, and back pack the lagging. The drift will be approximately 6 feet square and will utilize the turbine building slab at el 609 as the top of the tunnel. A typical plan view of the drift is shown in Figure C-1 of Reference 1.

Piping and conduit will be temporarily supported in place during construction. It is not anticipated that piping and conduit will be encountered in the drift under the turbine building.

QUESTION 16

(Pg C-3, Par. A.1.a) Please explain what is meant by minimizing the amount of concrete to be removed.

RESPONSE

There is a significant amount of concrete interspersed in the fill beneath the electrical penetration wings, control tower, feedwater isolation valve pits, and turbine building. The concept for the underpinning considered the location of this concrete by sizing and locating the pits for temporary/initial support piers to minimize the volume of concrete to be removed by hand methods.

QUESTION 17

(Pg. C-3, Par. A.1.c and A.1.d) What is the magnitude of the load for testing the temporary support pier and how was it established and how will it be applied? Is the EPA foundation slab capable of supporting this load at this time?

RESPONSE

The magnitude of load to proof test the hand dug temporary support piers will be the lesser of 1.25 times the design jacking load on the pier, or the magnitude of load based on a finite element analysis which the structure will resist with acceptable stresses. The design jacking load will be determined from an analysis using total dead load and 25 percent live load (see the response to Question 1).

The supporting load will be applied directly under the vertical wall and, therefore, the effect of jacking load on the EPA slab will be minimal.

QUESTION 18

(Pg. C-4, Sect. A.1.f., 1st complete para.) Provide discussion on monitoring of the control tower behavior at this time. What criteria will be used to decide if preload should be stopped and support capacity should be added to the control tower?

RESPONSE

This construction condition is being analyzed. If it shows that the control tower piers cannot take the reactor from this preload from the electrical penetration area, either preload will be adjusted or additional piers under the control tower will be provided.

The control tower will be monitored as shown in Figure D-1 of Reference 1. Criteria will be determined after the structural analysis is completed.

QUESTION 19

(Pg. C-4, Sect. A.2.) What are the reasons why the three temporary supports under the EPA should not be completed before the permanent support at the control tower is initiated?

RESPONSE

The three supports are not proposed to be installed prior to support under the control tower for the following reasons.

1. When excavation under a structure is started, it is best to install the new supports as soon as possible to reduce the time-dependent settlements caused by reducing the lateral restraint of the remaining soil. Therefore, as many areas as possible are worked on as soon as possible. In this case, it means the new supports under the control tower are started early.

2. The free ends of the cantilevers in the structure are to be supported as soon as possible. Using very conservative assumptions about the condition of the fill, it has been assemed that there are three cantilever areas: the outside end of each electrical penetration wing and the south side of the control tower. Because the south side of the control tower is a cantilever, new support will be installed as soon as possible.

3. Support for the south wall of the control tower can be constructed with a very small area of the total control tower undermined.

4. There are no structural advantages to install the three supports under the electrical penetration wings first.

5. There is a schedule shortening to start the control tower support when shown.

QUESTION 20

(Pg. C-4, Sect. A.3.a) Questions are raised as to whether the EPA structure can withstand the overhang condition which results if the initial temporary supports is (sic) assumed to fail. What is the basis and need for this extreme assumption? Is the EPA structure capable of withstanding this loading condition?

RESPONSE

The reason for postulating this condition is to be conservative. Based on a previous analysis of the EPA structure with caissons, it is expected that the EPA structure can withstand this condition. This will be verified by the present analysis in progress and, if necessary, the support location will be shifted to ensure the safety of the EPA structure under this condition.

QUESTION 21

(Pg. C-4, Sect A.3.b and A.c.3) The distinction between 3.b and 3.c is unclear. What is the magnitude of the load for testing and how established? Is there a problem with the EPA foundation slab providing a sufficient reaction load?

RESPONSE

Page C-4, Paragraphs 3.b and 3.c, discusses two loads on the temporary piers for the electrical penetration wing. The first load (3.b) will be the proof load on the pier (1.25 times working load not to exceed the resistance capacity of the electrical penetration wings). The load will be maintained until the acceptance criteria of pier settlement is less than 0.01 inch per hour. After the settlement criteria has been reached, the load will be reduced to the working load (3.c). For application of the jacking load, please refer to the response to Question 17.

QUESTION 22

(Pg. C-5, Sect. 14 and 15) It appears the operations described in these items are intended only for the wings and not the control tower. How is the load test and load transfer for the control tower to be completed. For the long term load test on the wings, what is the load magnitude and how was it established? What is the final sequence of operations in transferring the structure load to the permanent underpinning.

RESPONSE

The initial support under the control tower will also be proof tested to 1.25 times the working load. There will also be a long-term load test for the control tower. The operations for the initial support piers and the final wall configuration will be similar to those for the wings.

The load for the long-term load test on the wings is the design jacking load and is established as indicated in the response to Question 1.

The sequence of operations for the final load transfer for the wings and the control tower will be as follows:

1. Install additional jacks on the intermediate sections between the initial support piers under the control tower and on the new underpinning walls for the electrical penetration wings and column lines 5.3 and 7.8

2. Activate the additional jacks to 25 percent of the final jacking load

3. Read instrumentation

4. Adjust all jacks

5. Repeat the process at 25 percent increments until 100 percent of the final jacking load is attained.

QUESTION 23

(Pg. D-1, Sect 1.0, 2nd Par) Describe the procedure that relates allowable stresses and allowable strains with structure movements that are being monitored.

RESPONSE

The procedure to relate allowable stresses to allowable strains is the finite element analysis presently being performed to simulate the construction conditions.

QUESTION 24

(Pg D-2, Sect. 1, 3rd Par.) Please clarify the distinction between the first and second layer systems for detecting structure movement.

RESPONSE

The two layers are actually two systems for measuring structure movements: a relative movement system and an absolute movement system. The relative system detects differential movements between 1) the electrical penetration areas and the containment structures, 2) the electrical penetration areas and the turbine building, and 3) the control tower and the turbine building.

The absolute movement measuring system detects structure movements with respect to fixed datum points. The fixed datum points for vertical movement will be deep seated monuments. A detail for this is shown in Figures 4, 5, and 6.

The horizontal movements, the fixed datum will be a vertical plane defined by two centering points. At each end of the electrical penetration wing, two monuments along an east-west line will be located. A transit set up on one of the points and in line with the second point will define the vertical plane. Sighting on the structure will indicate absolute horizontal movements of the structure.

The relative measuring system will be used to give insight into what is happening between the structures and to give warnings that the structu..s have some movement. The absolute measuring system provides the needed information for assessing the stresses and strains in the structure.

QUESTION 25

(Pg D-2, Sect. 1, 4th, 6th, and 7th Para.) Please provide elevations and sectional views with typical details for the deep seated bench mark and the instrumentation for monitoring relative horizontal movement and absolute horizontal movement.

RESPONSE

The conceptual details for the deep seated bench mark and the relative vertical and horizontal movement measuring devices are shown in Figures 4, 5, and 6. The complete details are in the process of being developed and are scheduled to be available by January 1982.

QUESTION 26

(Pg. D.3, Sect. 2, 2nd Par.) Please clarify the explanation why the hydraulic pressure data cannot be used to measure load.

RESPONSE

Hydraulically actuated jacks are not used for continued structure support because of the possibility of a failure in the hydraulic system causing a loss of support. To prevent loss of support in the event of hydraulic failure, steel supports and wedges, commonly called "chasers" or "stands", are maintained under the structure at all times. The wedges of the chasers are periodically driven together to maintain a tight fit. This driving of the wedges causes an indeterminate amount of load to be transferred to the chaser. Therefore, even if the hydraulic pressure was maintained, it would indicate only a portion of the load supported by the total system.

When it is desired to know the actual loads being supported, load measuring devices are installed in the support element. Carlson gages will be used to measure the change in stress in the piers and underpinning wall. These stress measuring devices will indicate the approximate load being supported by the pier or underpinning wall.

Stress meters at the top and bottom of each of the three temporary support piers under each electrical penetration wing and under each initial support pier under the control tower are proposed. Additional stress meters will be installed at the top and bottom of the permanant underpinning walls.

QUESTION 27

(Pg. D-3, Sect. 2, 3rd Par.) Provide sectional view of set up for measuring difference in relative position. How does this procedure address the possibility of both the underpinning element and structure settling? Provide the basis for maintaining the jack/hydraulic system for 1 hour and for establishing the 0.01 inch movement.

RESPONSE

The detail for measuring differences in relative position is shown in Figure 5. The relative measuring devices gives warning that settlements are occurring. The absolute vertical movement of the structure will be measured and compared with the data from the instrumentation on the underpinning elements to indicate if both are settling.

The 1-hour with less than 0.01-inch movement was determined by taking the estimated settlement of the pier (0.8 inch) and dividing it by the time estimated for the settlement (48 hours). This value is the average settlement per hour over 2 days (0.01667 inch). This value was divided by 1.5 to estimate the lower end of the settlement curve and then rounded down to 0.01 inch per hour.

QUESTION 28

(Pg. D-4, Sect. 2, 4th Para.) When will the modeling and critical structural stresses and strains be determined and furnished to the NRC?

RESPONSE

The modeling and critical structural stresses and strains will be furnished to the NRC by February 1982.

QUESTION 29

(Pg D-5, Sect. 2, 2nd and 3rd Para.) Provide sketch and locations with typical details of instrumentation for measuring concrete stress, tell tale devices and predetermined points for monitoring vertical movement.

RESPONSE

The location of the stress meters was discussed in the response to Question 27. The telltale devices will be similar to those shown in American Society for Testing and Materials D 1143-74.

QUESTION 30

(Pgs. D-5 and D-6, Sect. 3, Par. 3A.1, 3A.2, 3A.3) For the various types of monitoring described in these paragraphs provide an example of the forms to be used for plotting the recorded data. What are the predetermined levels of movements which would require adjustments and/or action by the onsite geotechnical engineer. Identify any specific instrumentation which would be continued to be read during plant operation and which eventually will be addressed by a Technical Specification.

RESPONSE

The forms for recording and plotting the data have not been finalized yet. These forms are part of the detailed procedures being developed for monitoring.

The predetermined levels of movements, which would require adjustments and or actions by the onsite geotechnical engineer, are based on analysis currently in progress. These predetermined levels will also be included in the detailed procedures, which will be developed before starting the underpinning work.

It is intended to use a data service organization to collect and record the settlement and crack mapping data. It is not intended to use any of the construction monitoring instrumentation during plant operation. The permanent plant monitoring is currently in progress and will remain so for the life of the plant.

ADDITIONAL INFORMATION

A design modification has been made to the FIVP underpinning since the presentations of September 30, 1981.

To reduce the seismic overturning loads on the FIVP, the underpinning walls which were contemplated to support the structure from el 571' will not be utilized. Instead, the existing fill material will be replaced by an engineered granular material placed to el 600'. At this location, a 3-foot jacking slab will be placed. This slab will serve as support for an underpinning system to the base slab of the pit. After the pit is jacked, the area between the jacking slab and base slab will be filled with concrete.

REFERENCES

1. Consumers Power Company, Midland Plant Units 1 and 2, Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

2. Corps of Engineers letter, P. McCallister to G. Lear, April 16, 1981

3. G.G. Meyerhof, The Ultimate Bearing Capacity of Wedge Shaped Foundations, 1961 Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Volume II, p 105, Paris, 1961

4. NAVFAC DM-7 Design Manual, Soil Mechanics, Foundation, and Earth Structures, Department of the Navy, Naval Facilities Engineering Command, March 1971

5. Woodward-Clyde Consultants, Test Results, Auxiliary Building, Soil Boring and Test Program, Midland Units 1 and 2, Midland, Michigan, October 26, 1981

6. A.W. Skempton, "The Bearing Capacity of Clays," Building Research Conference Congress Proceedings, 1951

ATTACHMENT 1

BEARING CAPACITY FROM CONE PENETROMETER

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR Bechtel - Midland, Mich.

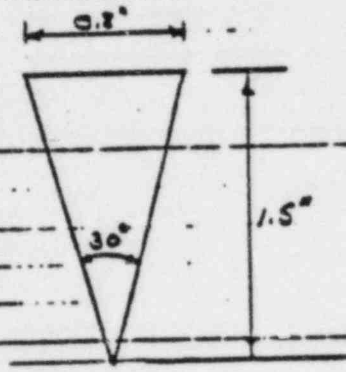
MADE BY DHB DATE 8-27-81
CHECKED BY HSL DATE 8-30-81

BEARING CAPACITY FROM CONE PENETROMETER

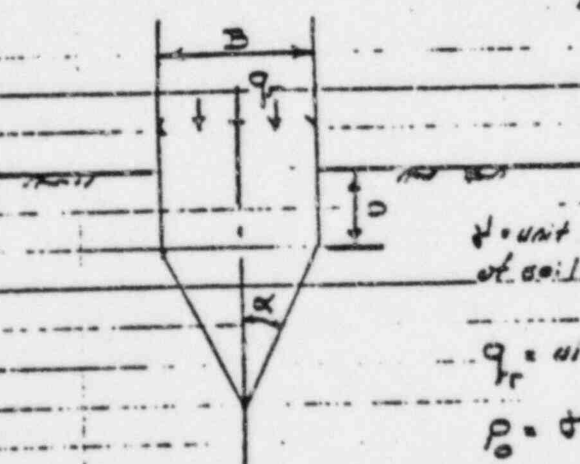
CN-973 Cone Penetrometer:

Dial reads 0-300 pounds per square inch. This is 0 to 150 lbs. load as base area of cone is 0.5 in²

Cone dimensions:



Reference: G.G. Meyerhof: "The Ultimate Bearing Capacity of Wedge-shaped Foundations", Paris 1961, Vol. II, p. 105.



For cone penetrating $\phi = 0$ material:

$$q_r = C N_{cr} + p_o N_{qr}$$

γ = unit weight of soil

q_r = ultimate bearing pressure

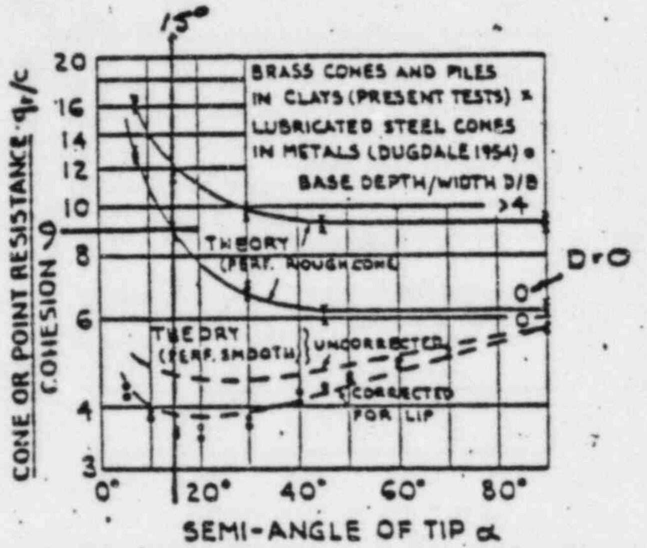
$p_o = \gamma D$

N_{cr} & N_{qr} are bearing capacity factors

In our case D = 0; thus

$$q_r = C N_{cr}$$

From Meyerhof:



(a) CONE RESISTANCE AND POINT RESISTANCE OF PILES IN CLAYS

For $\alpha = 15°$, $N_{cr} = 9$

Thus, for CN-973 Cone Penetrometer:

$$q_r = 9c \quad \text{or} \quad c = \frac{q_r}{9}$$

Ultimate bearing capacity of strip footing $= q_{ult} = 5.7c$

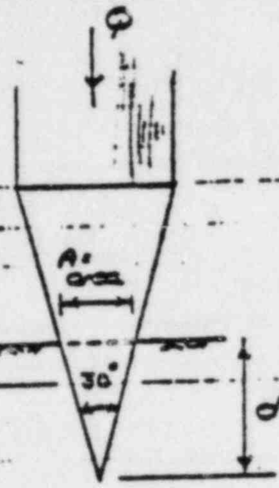
$$q_{ult} = 5.7 \times \frac{q_r}{9} = 0.6\, q_r$$

Assuming Safety Factor = 3; allowable bearing pressure: q_a

$$q_a = \tfrac{1}{3} q_{ult} = 0.2\, q_r$$

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR Bechtel- Midland, Mich.

Sheet No. 3 of 5
File 5464
Made By PHE Date 8-27-81
Checked By HSL Date 8-30-81



Say: R = CH-973 dial reading

Thus, load $Q = 0.5R$ (lb.)

$A = \pi (d \tan 15°)^2$

$$-q_r = \frac{Q}{A} = \frac{0.5\,R}{\pi (d \tan 15°)^2}$$ [in psi if d in inches]

$$q_a = 0.2\,q_r = \frac{0.10\,R}{\pi (d \tan 15°)^2} \quad (psi)$$

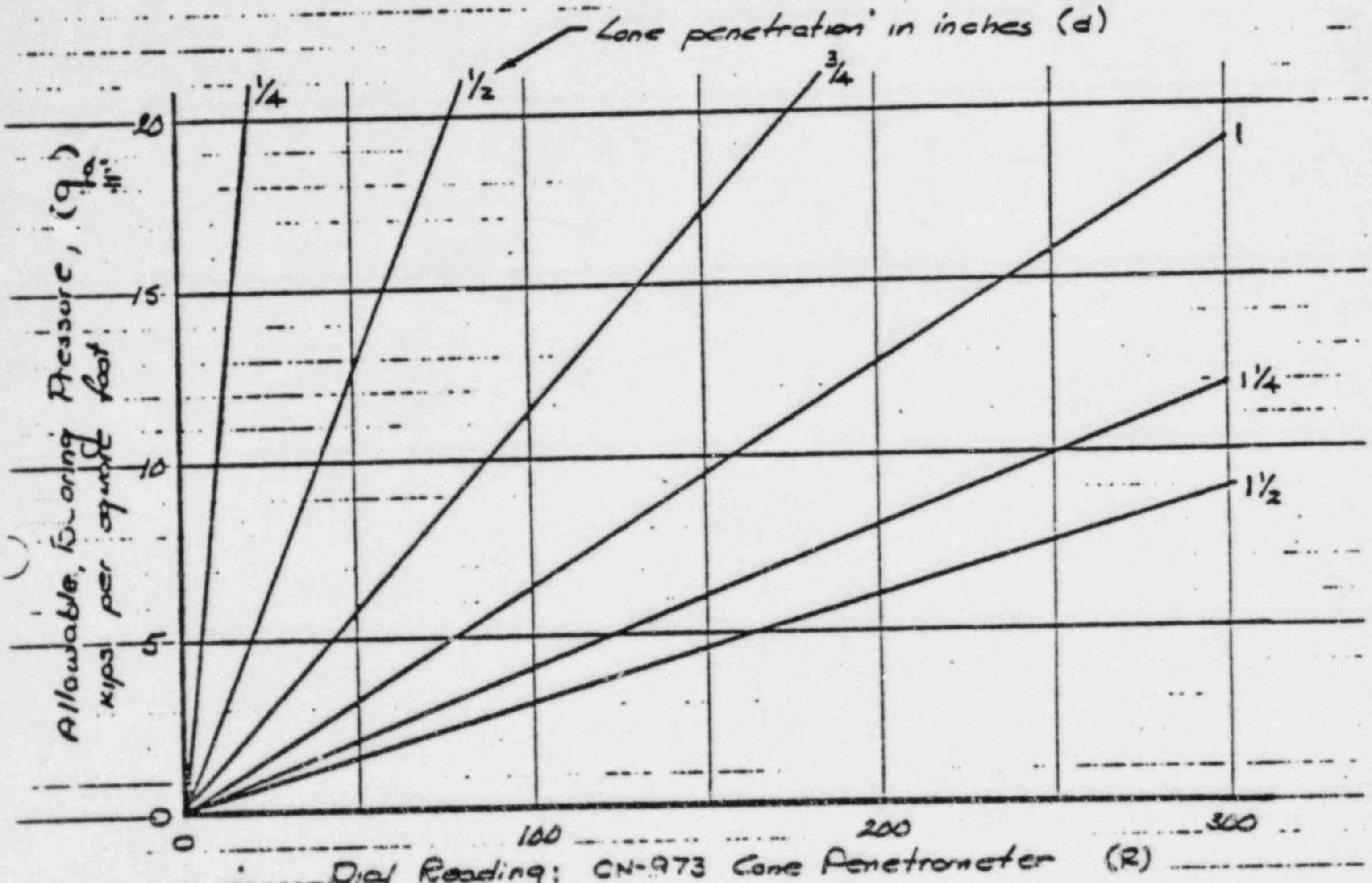
1 psi = 144 psf = 0.144 ksf

$$\therefore\ q_a = \frac{0.0144\,R}{\pi (d \tan 15°)^2} = \frac{0.064\,R}{d^2}$$ [ksf w/ d in inches]

Say R = 300 (limit of guage)

d (inches)	q_a (ksf) for R = 300	200	100	50
0.25	307 (306)	205 (204)	102	51
0.50	77	51	26	13
0.75	34	23	11	6
1.00	19	13	6	3
1.25	12	8	4	2
1.50	9	6	3	1

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR Bechtel - Midland, Mich



(Equals pressure in shaft cross-section in psi)
= double applied load in pounds

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR Bechtel - Midland, Mich.

SHEET No. 5 OF 5
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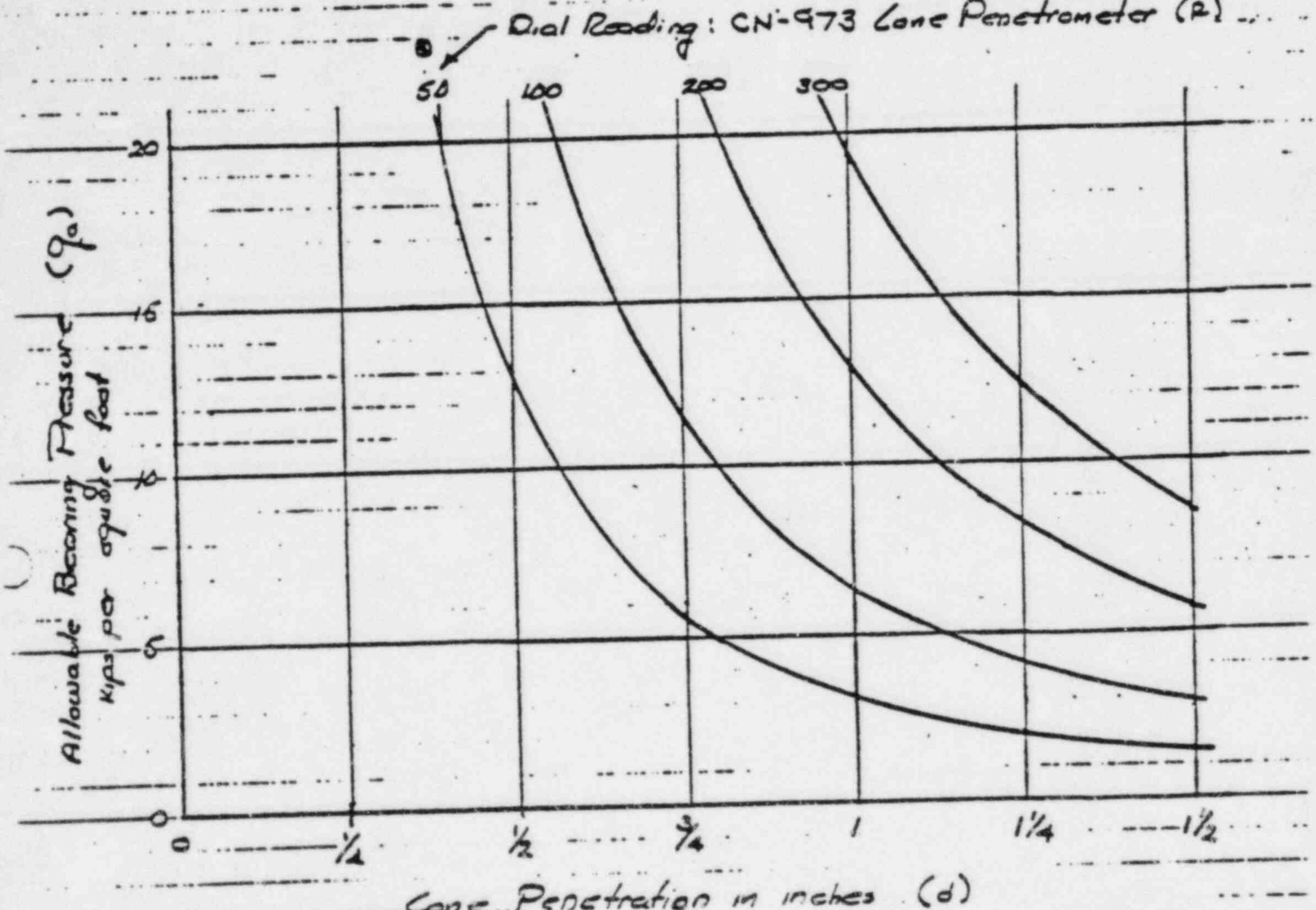
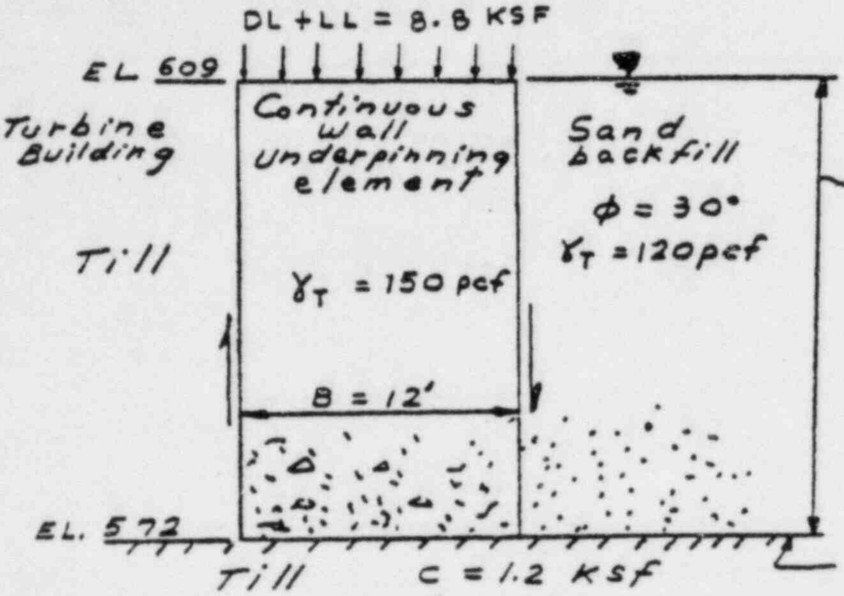
CHECKED BY HSL DATE 8-30-81



Ultimate bearing capacity of foundation of underpinning wall for Auxiliary Bldg.



$c = 1.2$ ksf
$\phi = 23°$
$\gamma_T = 140$ pcf

Assumptions

1. Continuous Wall
2. Wall friction on the side of Turbine Bldg. will be counter-balanced with wall friction on backfilled side.

$$q_{ult} = C\,N_c + \gamma' D N_q + \gamma' \frac{B}{2} \times N_\gamma$$

N_c, N_q, N_γ } Bearing Capacity factors

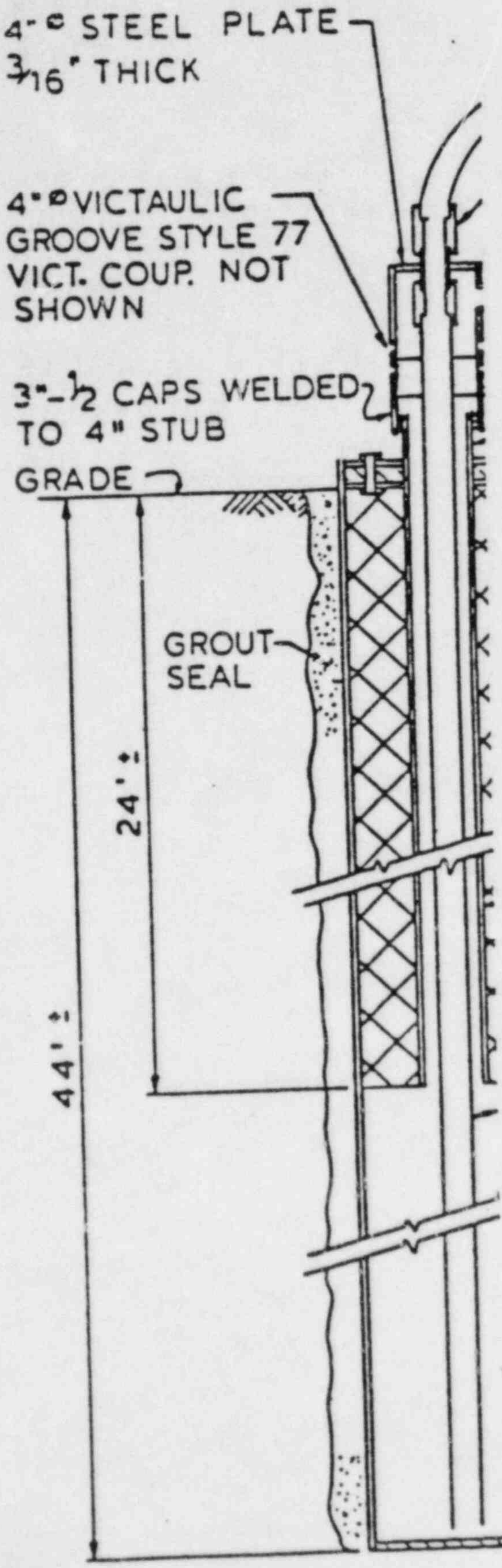
From NAVFAC DM-7, FIGURE 11-1 : $N_c = 19.0$, $N_q = 9.0$, $N_\gamma = 5.0$ (Rough base)

$$q_{ult} = 1.2 \times 19.0 + 0.058 \times 37 \times 9.0 + 0.078 \times \frac{12}{2} \times 5.0$$
$$= 22.8 + 19.3 + 2.3$$
$$= 44.4 \text{ KSF}$$

$$F_s = \frac{44.4}{8.8 + 0.030 \times 37} = 4.5$$

ATTACHMENT - 2

EFFECTIVE STRESS PARAMETERS

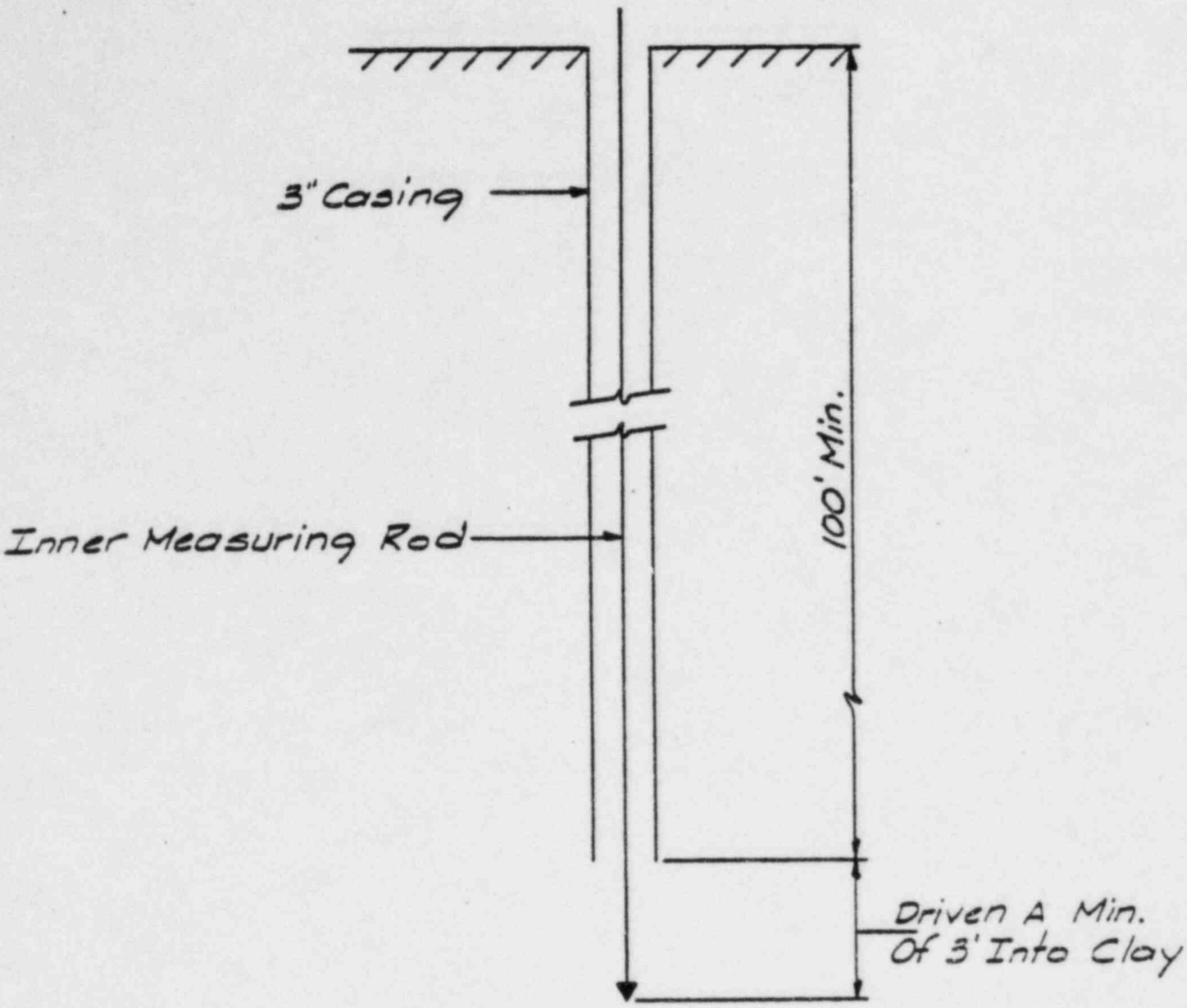


DETAIL OF TYPICA

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

DETAIL OF TYPICAL FREEZE ELEMEN

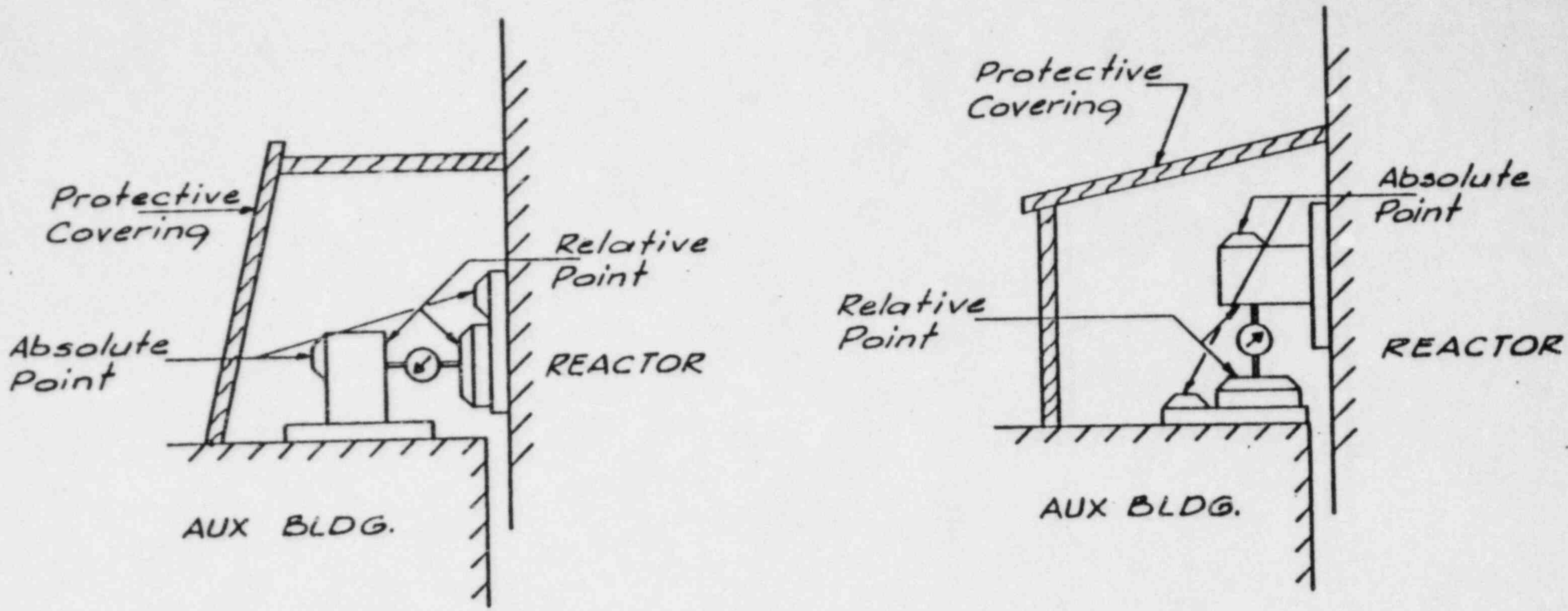
FIGURE 3



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

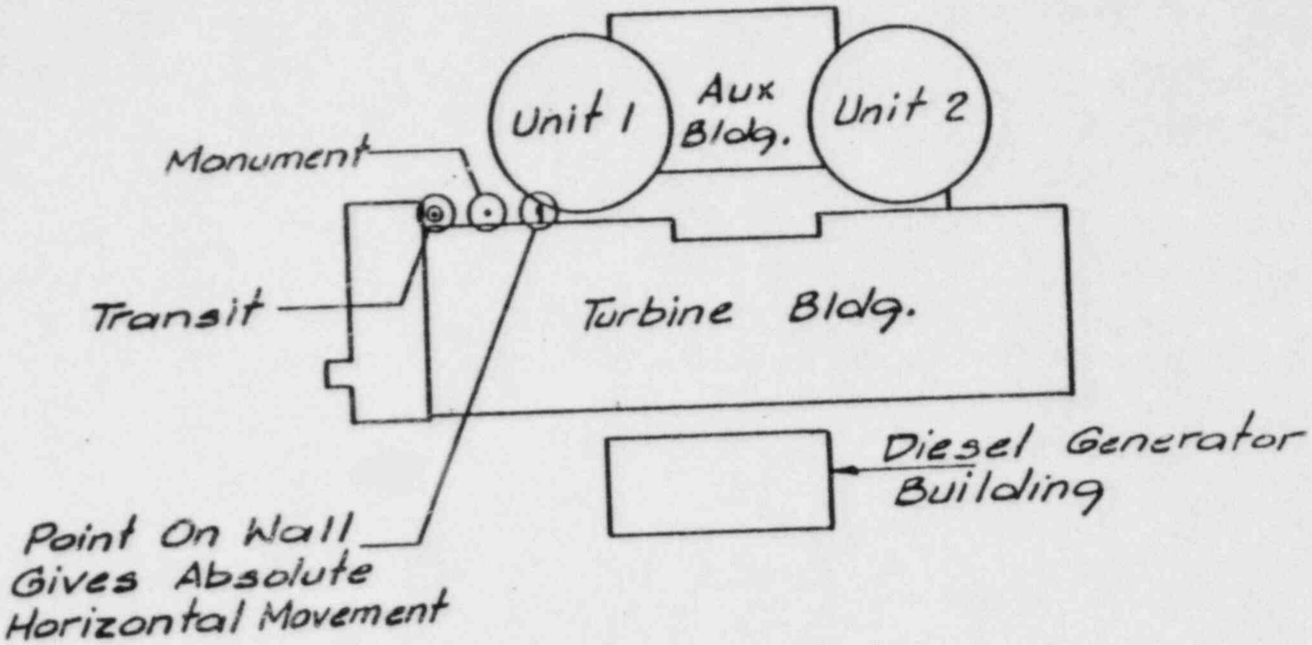
DETAIL OF DEEP SEATED BENCHMARK

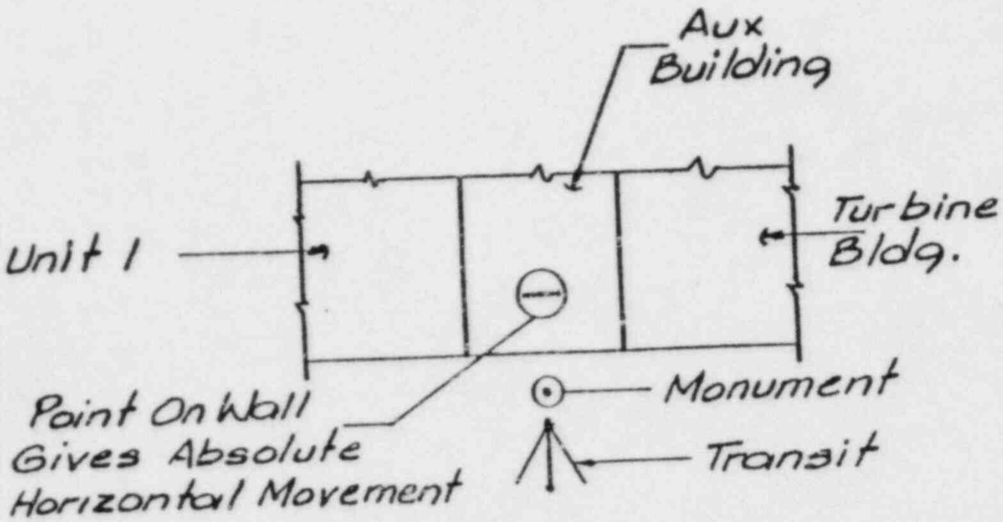
FIGURE 4



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

DETAILS FOR MEASURING
RELATIVE MOVEMENTS
FIGURE 5

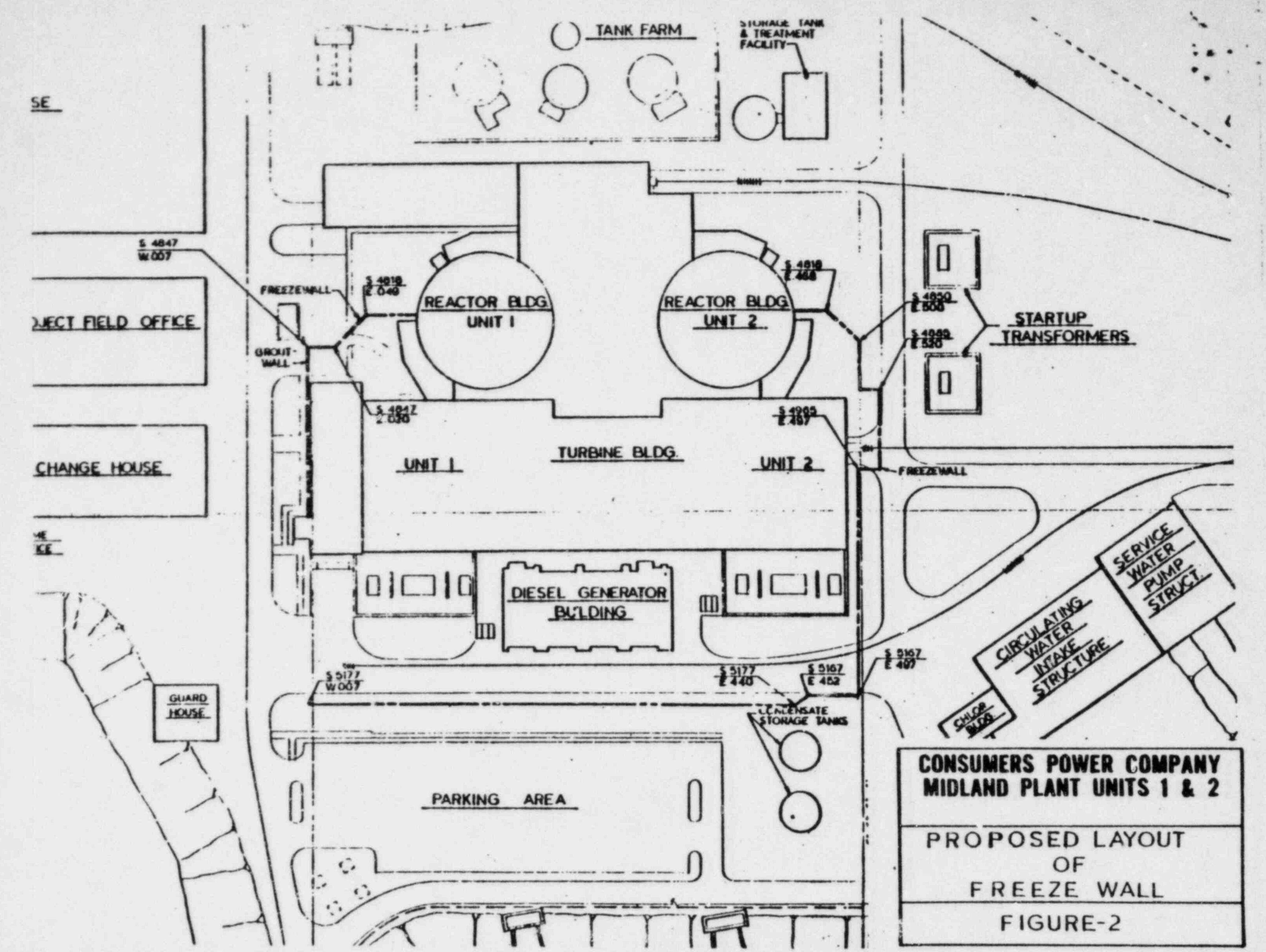




CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

DETAIL FOR MEASURING ABSOLUTE MOVEMENTS

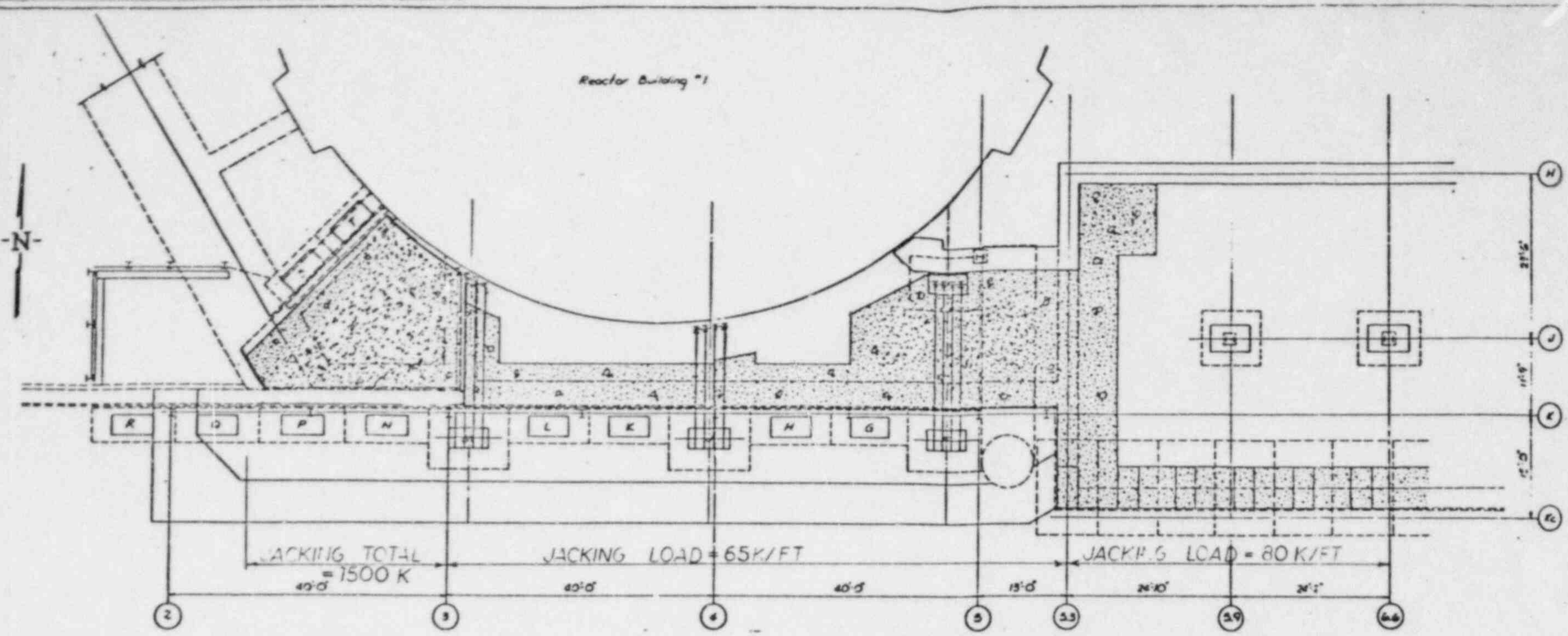
FIGURE 6



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

PROPOSED LAYOUT
OF
FREEZE WALL

FIGURE-2



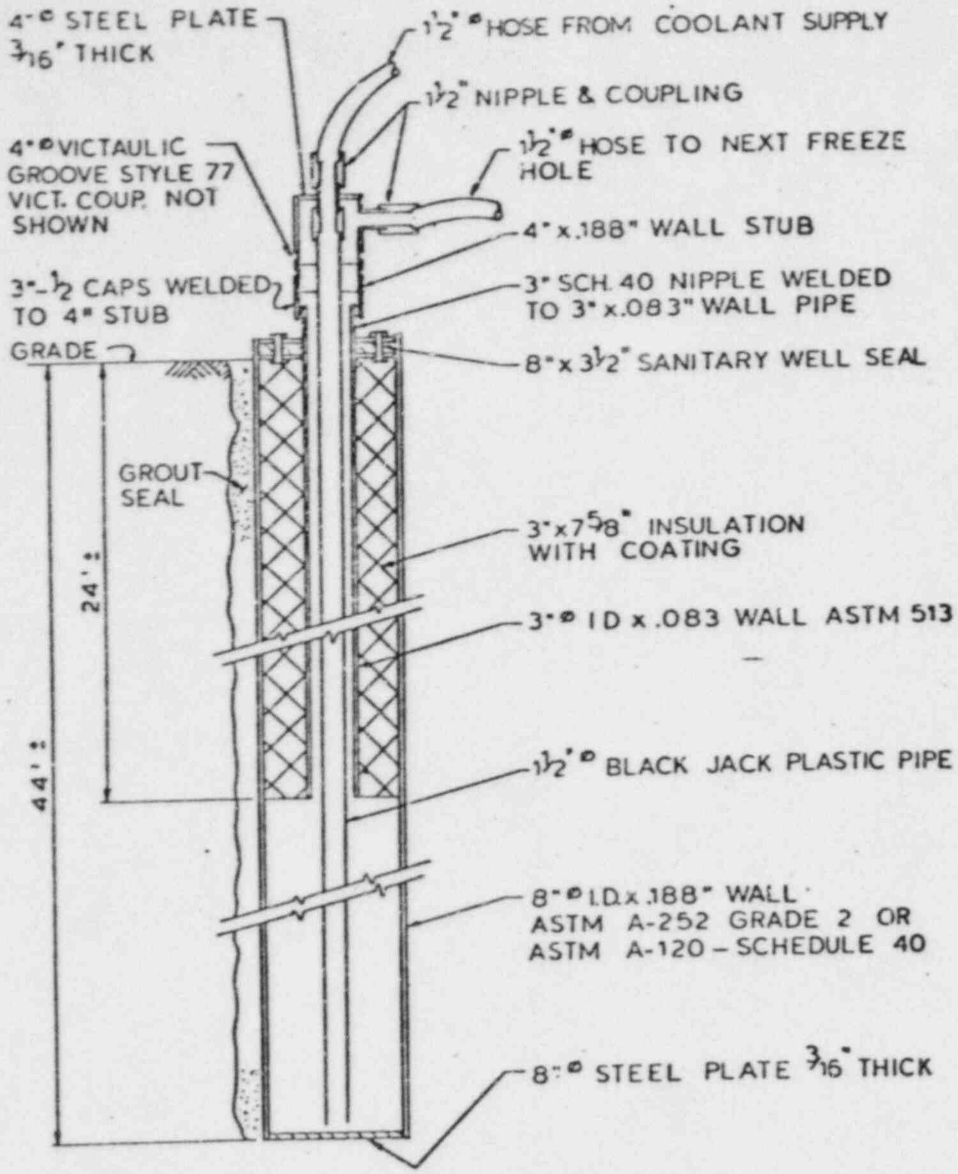
GENERAL PLAN
(WEST SIDE)
(EAST SIDE
OPP. HAND)

-PERMANENT UNDERPINNING TO BE CONSTRUCTED

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

JACKING LOADS
UNDERPINNING AUXILIARY BUILDING
PERMANENT UNDERPINNING

FIGURE 1



DETAIL OF TYPICAL FREEZE ELEMENT

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

DETAIL OF TYPICAL FREEZE ELEMENT
FIGURE 3

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

November 6, 1981

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
RESPONSE TO NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE
FILE 0485.16, B3.0.8 SERIAL 14843
ENCLOSURE: RESPONSES TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE

On September 17, 1981, a request for additional information relating to the service water pump structure was made by the Staff in a meeting at the NRC's offices in Bethesda, Maryland. We are responding to this request by forwarding the above enclosure. The enclosure addresses each of the individual Staff concerns transmitted to us in the September 17, 1981 meeting.

We believe the enclosed information adequately responds to the request and individual concerns identified for us by the Staff. The discussion and data contained in the enclosure to this correspondence lend further support to our conclusion that the design of the service water pump structure combined with the remedial actions are adequate and appropriate for this structure.

G.S. Keeley for JW Cook

JWC/RLT/dsb

oc1181-0473a100

8111120553

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, Esq, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

oc1181-0473a100

RESPONSE TO THE NRC STAFF REQUEST
FOR ADDITIONAL INFORMATION PERTAINING TO
THE PROPOSED UNDERPINNING OF THE
SERVICE WATER PUMP STRUCTURE

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO THE NRC STAFF REQUEST

FOR ADDITIONAL INFORMATION PERTAINING TO

THE PROPOSED UNDERPINNING OF THE

SERVICE WATER PUMP STRUCTURE

CONTENTS

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FIGURES

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO THE NRC STAFF REQUEST

FOR ADDITIONAL INFORMATION PERTAINING TO

THE PROPOSED UNDERPINNING OF THE

SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

On September 17, 1981, representatives of Consumers Power Company, Bechtel Power Corporation, and the NRC met in Bethesda, Maryland, for a presentation of the proposed remedial action for the Midland plant service water pump structure (SWPS). The discussion of the proposed underpinning construction resulted in several requests for additional information. This report responds to these requests and supplements the Technical Report on the Service Water Pump Structure Underpinning (Reference 1).

2.0 REQUESTS FOR ADDITIONAL INFORMATION

2.1 ASSUMPTIONS AND CONCLUSIONS FOR THE PRELIMINARY ANALYSIS OF THE UNDERPINNED STRUCTURE

2.1.1 Stability Analysis

2.1.1.1 Discussion

The underpinned structure was analyzed for sliding, overturning, and resistance to buoyancy for the design flood condition in conformance with Final Safety Analysis Report (FSAR) Subsection 3.8.6.3.4. Sliding in the north-south direction was critical and overturning was critical in the east-west direction.

The critical load combination for sliding and overturning is:

$$D + H + E'$$

where

D = dead load of structure and equipment

H = lateral earth pressure

E' = safe shutdown earthquake load

2.1.1.2 Assumptions

a. The normal groundwater was assumed at the level of the pond (el 627').

b. The long-term shear strength parameters are $\phi' = 36°$ and $C' = 0.73$ ksf, based on Woodward Clyde Consultants' test data at the SWPS location.

c. The lateral earth pressure dynamic increment was obtained by using FSAR Figure 2.5-45.

d. The forces from the safe shutdown earthquake (SSE) were increased by 50% to provide for a possible increase in this requirement.

e. Because of the flexibility of the underpinning wall, only the side walls and approximately 25% of the north underpinning wall are considered effective in resisting the force that attempts to cause sliding. The validity of this assumption will be verified in the final analysis.

2.1.1.3 Conclusions

The minimum factor of safety against sliding is 1.17 and is based on a sliding force of 16,500 kips and a total resistance of 19,200 kips. This figure is calculated for sliding in the north-south direction and exceeds the allowable factor of safety of 1.1.

The minimum factor of safety against overturning is 1.45 versus an allowable factor of safety of 1.1. This value is based on an overturning moment of 1.9×10^6 ft-kips compared to a stabilizing moment of 2.75×10^6 ft-kips. The east-west direction is the critical direction for overturning.

The building has a factor of safety of 2.1 versus the required 1.1 against the buoyancy force for a flood level of el 631. The building has a total dead weight of 42,000 kips and a buoyancy force of 20,000 kips.

2.1.2 Lower Foundation Slab

2.1.2.1 Discussion

The lower foundation slab is 90 feet long, 74 feet wide, and 5 feet thick and forms the base for the SWPS sump. Interior walls divide the foundation into three slabs: two small slabs 45 feet by 30 feet with effective span lengths of 38 feet, 9 inches by 25 feet, 9 inches and a large slab 90 feet by 44 feet with effective span lengths of 79 feet, 6 inches by 30 feet, 6 inches. The large slab was judged most critical and was analyzed for the following load combinations:

$U = 1.4D + 1.7L + P_L$

$U = 1.4D + 1.7L + 1.4P_L$

$U = D + L + P_L + E'$

$U = 1.25\ (D + L + P_L + E)$

where

U = required strength to resist design loads or their related internal moments and forces

D = dead load of the structure and equipment

L = conventional floor and roof live loads (includes movable equipment loads or other loads which vary in intensity)

P_L = load on structure due to jacking

E' = SSE load

E = operating basis earthquake

2.1.2.2 Assumptions

a. The groundwater was assumed at the level of the pond (el 627').

b. The plant fill under the upper foundation slab offers no vertical support for the upper slab.

c. The effects of dead load, live load, and jacking load are carried only by the lower foundation slab. All other loads are transferred to the foundation composed of the lower slab and the underpinning wall.

2.1.2.3 Conclusions

The maximum imposed out-of-plane moment of 180 ft-kips was exceeded by the moment capacity of the slab, which amounts to 200 ft-kips. The maximum soil pressure was 11.3 ksf.

2.1.3 Effect of Construction Dewatering on the Lower Foundation Slab

2.1.3.1 Discussion

Fluctuations of the water table will affect the values of the soil pressures under the foundation slab. The drawdown of the groundwater for constructing the underpinning wall will decrease the buoyancy of the structure, causing an increase in bearing pressure.

2.1.3.2 Assumptions

a. The original groundwater is assumed at the level of the pond (el 627').

b. The groundwater will be drawn down to el 587' at the north underpinning wall.

c. The shape of the drawdown curve is parabolic.

d. The drawdown is uniform for the full width of the structure.

2.1.3.3 Conclusions

Considering dead load, live load, and buoyancy, and the assumed groundwater at el 627'-0," the bearing pressure under the slab varies with a maximum value of 5.35 ksf at the north edge. For the construction condition, dewatering to el 587'-0", this pressure increases to 8.12 ksf, which is well below the allowable pressure of 16.7 ksf. This pressure, 8.12 ksf, will be reduced as the construction of the underpinning wall proceeds because the addition of jacking forces reduces the weight of the structure supported by the lower base slab.

The pressures from the underpinning construction condition are less than the values used in Subsection 2.1.2 of this report and are not considered critical in analyzing the slab.

2.1.4 Upper Foundation Slab

2.1.4.1 Discussion

The slab is 86 feet long, 38 feet wide, and 3 feet thick. An interior wall divides the slab into two slabs of unequal size. The smaller slabs are 38 feet by 35 feet and 51 feet by 38 feet. The larger slab, with effective span dimensions of 48 feet, 3 inches by 25 feet, 4 inches, was analyzed for the following load combination, which included the effects of compartment flooding to a depth of 12.5 feet.

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L$$

2.1.4.2 Assumptions

a. The fill under the upper foundation slab offers no vertical support. The slab is simply supported on four sides but is continuous over the interior wall.

b. The seismic effects and the containment of water to a depth of 12.5 feet does not occur simultaneously.

2.1.4.3 Conclusions

The maximum imposed moment of 109 ft-kips (from the analysis) is less than the slab capacity of 150 ft-kips. Therefore, the slab is considered to be adequate.

2.1.5 Sidewalls of the Overhang

2.1.5.1 Discussion

The exterior walls at the face of the overhang were analyzed for shear and bending stress for the load combination of:

$$U = D + L + E' + P_L$$

2.1.5.2 Assumptions

a. The groundwater was assumed at the level of the pond (el 627').

b. The fill under the upper foundation slab offers no support.

c. The resisting section at the face of the overhang consists of a box section and the attached underpinning walls. The box section is composed of the exterior walls of the overhang, the roof slab, and the foundation slab. The support offered by the interior walls was ignored. The resisting section was modified for the effects of shear lag.

2.1.5.3 Conclusions

The maximum computed compressive stress in the walls was 0.32 ksi and the maximum shear stress is 0.103 ksi. The largest tensile stress in the reinforcement is 2.2 ksi. All values are below the American Concrete Institute (ACI) 318-71 allowable values.

2.1.6 Interface Connectors

2.1.6.1 Discussion

The underpinning walls are designed to act as integral parts of the structure. Application of jacking loads and the use of anchor bolts will ensure that loads are adequately transferred between the structure and the underpinning walls. Rock bolts and anchor bolt assemblies will be used to ensure that the walls and structure do not separate. Because the construction procedure requires that the anchor bolts and rock anchors be installed after the application of the jacking loads, the connectors are not affected by the jacking operation or the dead load of the structure.

2.1.6.2 Assumptions

a. The connectors will be designed to carry all loads on the structure, except the jacking loads.

b. The behavior of the connection is governed by shear friction requirements.

c. The connectors were designed for the following load combinations:

$$U = 1.4D + 1.7L + P_L$$

$$U = D + L + E' + P_L$$

2.1.6.3 Conclusions

The maximum shear load to be transferred at each vertical interface is 1,300 kips. Nine 2-inch diameter, hollow core rock anchors at a maximum spacing of 3 feet, 9 inches are required to fulfill the shear friction requirements. A maximum shear of 1,700 kips will be transferred at the horizontal interface by 2-3/4-inch diameter anchor bolts at a maximum spacing of 3 feet, 9 inches.

2.1.7 Underpinning Wall

2.1.7.1 Discussion

The underpinning wall extends from the underside of the upper foundation to firm bearing on undisturbed soil. The wall is 4 feet thick and 30 feet high. The base of the north wall is widened to 6 feet. The wall is connected to the existing structure with rock and anchor bolts.

The wall was analyzed for the following load combination:

$$U = D + L + E' + P_L$$

2.1.7.2 Assumptions

a. The wall was analyzed as a shear wall for in-plane forces.

b. Because the north wall has a horizontal span length of approximately 86 feet, the wall at midlength was analyzed as a vertical simply supported beam and was also analyzed with partial restraint at the base for out-of-plane forces.

2.1.7.3 Conclusions

For in-plane forces, each side wall carries a moment of 50,000 ft-kips and a shear of 400 kips. The capacity of the wall is 75,000 ft-kips for moment and 1,000 kips for shear. Because the aspect ratio of the north wall is much more favorable, it was considered not critical in the preliminary analysis. The analysis of the north wall for out-of-plane forces showed the maximum moment to be 150 ft-kips per foot of wall, which is less than the 190 ft-kip moment capacity. Shear was not critical.

3.0 DESCRIPTION OF PROTECTION FOR THE EXISTING STRUCTURE DURING CONSTRUCTION

3.1 CONSTRUCTION PROCEDURE (Refer to Figure 4 of Reference 1)

Protecting the existing structure while constructing the underpinning wall is a major concern. This concern is reflected in the procedure that was established for constructing the underpinning. This procedure was developed with the purpose of providing the maximum degree of safety to the structure.

As a precautionary measure, the upper portion of the north-south exterior walls will be post-tensioned before the permanent dewatering begins. The dewatering will reduce the buoyancy force acting on the overhang and will increase bending stresses in the walls. Post-tensioning the upper portion of the exterior walls will induce compression in the walls and will minimize the effects of the tensile forces caused by dewatering.

The first three piers, which are located at the northwest and northeast corners of the structure, will be constructed from tunnels proceeding simultaneously from the access shafts at the east and west sides of the building. In this way, the jacking force will be symmetrically applied to the structure. The construction procedures prevent advancing either tunnel to the area where the next pier is to be constructed until the jacking load is placed on the completed pier. Thus, the decrease in soil support of the upper foundation slab is kept to a minimum.

After the corner piers are in place, the construction procedures call for the installation of the center piers under the north wall. This requires advancing the tunnel approximately 25 feet to the next pier. To prevent excessive loss of support, the following provisions will be made.

3.1.1 Only one tunnel will be extended from the pier 3 to pier 4 location at one time. When the first pier 4 and pier 5 are load bearing, the other tunnel will be extended to the remaining pier 4.

3.1.2 Measurement devices will be provided at piers 1, 2, and 3 to monitor variations in applied loads to the piers. If a sudden increase in pier loading of the magnitude of approximately one-third is indicated while the tunnel is being advanced from pier 3 to pier 4, tunnel construction will be stopped. Pier 8 will then be constructed as a series of piers instead of as a large monolithic pier. This procedure will provide a gradual increase in the jacking support to the overhang as the tunnel is advanced to pier 4.

3.1.3 When the tunneling operation toward the center begins, the three piers on each end will have a total jacked load of 465 kips. This results in an average bearing pressure of 5.8 ksf in the till. The till is considered adequate for an allowable bearing intensity of 19.2 ksf at a safety factor of 2.5 against bearing failure. These figures indicate that a total allowable bearing load of 1,600 kips for each pier group is available to adequately support the overhang portion of the structure. The north wall is adequate at ACI-acceptable stresses to span between the end pier groups if necessary. Analysis of the north wall for this condition, considering the wall as a deep concrete beam and assuming no vertical soil support to the overhang, shows that the compressive stress amounts to 0.250 ksi and tension in the concrete amounts to 0.300 ksi which is less than the modulus of rupture, 0.475 ksi.

3.2 CRACK MONITORING

In anticipation of the underpinning wall construction, a crack mapping program has been started. Existing crack locations and widths have been accurately measured. Future mappings, to monitor the existing cracks and the appearance of new cracks, are scheduled to take place before and after major underpinning construction procedures, such as post-tensioning, dewatering, and jacking.

Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks or changes in existing structural cracks exceeding 0.01 inch will be evaluated and if any crack widths reach 0.03 inch, construction in the affected area will be modified or suspended until the reasons for excessive cracking are established and appropriate remedial measures are implemented.

3.3 SETTLEMENT MONITORING

In addition to the crack monitoring program, a program to closely monitor structure settlement has been planned. Besides the four existing settlement markers at each corner of the building, five additional markers will be installed on the building (Refer to Figure 1) and a settlement dial indicator will be installed at each of the two building corners where the underpinning will be constructed. The dial indicators will be attached to the building with their probes connected to permanent bench marks founded in undisturbed soil approximately 50 feet below the bottom of the underpinning wall. The depth at which the tip of the bench mark is located ensures that the bench mark movement will be negligible. The settlement markers will be monitored before and after major construction procedures as discussed in Section 3.2. Building movement and crack data will enable the project engineer to evaluate the effects of the underpinning construction on the existing structure.

4.0 DISCUSS THE BEARING CAPACITY OF THE UNDISTURBED NATURAL SOIL SUPPORTING THE UNDERPINNING

The estimated, ultimate bearing capacity is based on the many borings taken in the area by Dames and Moore and others including the recent borings taken by Woodward-Clyde Consultants. The soil samples and laboratory analysis of the most recent borings indicate the soil has shear strength conservatively estimated at 8 ksf and an ultimate bearing capacity of 48 ksf.

5.0 EVALUATE THE DIFFERENTIAL SETTLEMENT BETWEEN THE MAIN PART OF THE STRUCTURE AND THE UNDERPINNED PORTION

The construction procedure requires that jacking loads be applied to the piers soon after the pier is constructed. This load is sustained for sufficient time to dissipate the major portion of the long-term settlement of the underpinning. The underpinning is not attached to the structure until after the settlement has taken place.

Variations in deformations over the entire foundation, assuming a flexible structure, are predicted to be on the order of 0.2 inch. Soil springs are being developed to reflect total deformations including variations. The structure will be modeled and analyzed with the resulting supporting springs. In the soil-structure system modeling, the rigidity of the structure is considered. The interaction of the flexible springs and rigid structure reflects the true behavior of the structure.

6.0 DESCRIPTION OF PROCEDURE FOR TIMING OF FINAL JACKING LOCK OFF

6.1 METHODOLOGY

The final jacking loads will not be locked off until it is determined that the major portion of the pier settlement has occurred. By comparing predicted concrete and soil behavior curves and instrumented observations of the pier deflections, the optimum time for locking off the jacking load will be determined.

Vertical deflections at the top of the underpinning piers will result from the summation of several time-related properties of the pier concrete and the underlying soil. During the underpinning work, the soil deflection will be monitored at the top of each pier by connecting a settlement indicator to the top of a rod that extends to a plate at the base of the pier (refer to Section D-D of Figure 5, Reference 1). The rod is greased and placed within a tube to separate it from the concrete. The total top of pier deflections will be measured by another settlement indicator on top of the pier. The difference between these two deflection readings will represent the behavior of the concrete in the pier and the supporting soil.

The monitored pier deflections will be compared to predicted values. The expected concrete behavior is based on observations reported in recognized engineering standards. Four deflection curves for the pier concrete and glacial till are shown in Figures 2 through 5. The curves are plotted as displacement versus the logarithim of time. Figure 2 depicts a plot of the predicted top of pier deflection due to the creep of concrete under compressive load. As indicated, the total deflection will amount to approximately 0.03 inch. Figure 3 plots the top of pier deflection due to concrete shrinkage as the concrete dries and cures. The 10,000-day line is equal to about 27 years of elapsed time after pier construction. As shown in Figure 3, the total shrinkage-caused pier deflection is estimated at about 0.2 inch with the deflection leveling off after approximately 90 days. Figure 4 is a plot of the anticipated top of pier deflection due to soil consolidation. This graph indicates the settlement within a minimum and maximum range of values. The indicated total settlement due to soil consolidation is expected to be between 0.4 and 0.5 inch.

By combining the curves of predicted pier deflection due to concrete behavior, as shown in Figures 2 and 3, and the soil deflection curve shown in Figure 4, a composite top-of-pier-deflection-versus-log-time curve can be drawn. This is shown in Figure 5 using the maximum predicted soil settlement. The initial jacking of Stage 1 load (as shown in Figure 4 of Reference 1) into the pier several days after concrete placement will result in early rapid deflection, as shown. After about 90 days of Stage 1 loading, the jacking load will be increased to the final level which will result in another, but smaller, dip in the deflection curve. This increase in jacking load will combine with the shrinkage effect, which is greatest between 10 and 90 days' time. At about 110 days, the curve will flatten so it will appear as a straight line on this semi-log plotting. On a linear time scale, the deflection rate would appear much flatter. This semi-log straight line prediction is a typical observation for soil reaction after an initial elastic reaction period and is based on numerous test observations in the laboratory, as well as long-term field observations on in-place structures and buildings. The key factor in the process of final jacking and locking-off is determining when this more predictable phase has begun. This will be done at the site by plotting deflection curves, both at the top and bottom of the piers, while maintaining the final jacked loadings. This phase of the settlement curve is anticipated to occur soon after the final load level is applied assuming that all pier concrete is more than 90 days old.

6.2 ACCEPTANCE CRITERIA

The final jacking load will total 4,400 kips and will be imposed on underpinning piers 1 through 10. At that time, all piers will be at least 90 days old. This load level will be maintained for a period of about 2 weeks or until the settlement rate is within acceptable limits. The previous plottings of pier deflections under load will form a performance record which will greatly influence the determination of final acceptance and locking off.

7.0 DISCUSSION OF THE VALIDITY AND USE OF THE PENETROMETER

To aid the geotechnical engineer in assessing the adequacy of bearing capacity of the soil under the base of each underpinning pier, the construction procedures specify the use of the Waterway Experimental Station cone penetrometer, Model CN-973. The penetrometer consists of a 30° cone with a 1/2-square inch base, an 18-inch extension rod, a proving ring, a dial indicator, and a handle. A force applied through the handle deforms the proving ring and forces the cone to penetrate the soil. The proving ring deformation is proportional to the force applied, and the value of the applied force is indicated on the dial. The force is an index of the shearing resistance of the soil.

To evaluate the allowable bearing capacity of the soil, a family of curves relating allowable bearing capacity to applied force and cone penetration is utilized. These curves are based on the work of G.G. Meyerhof (Reference 2).

8.0 DESCRIPTION OF THE CRITERIA FOR FAILURE OF THE SOIL RESULTING FROM JACKING LOADS

Deflection at the bottom of an underpinning pier which approaches 2 inches is at about 90% of the point at which soil indicates plastic behavior. Other time-versus-rate-of-deflection criteria which are useful are that soil deflection should slow to about 0.01 inch in 3 hours after 3 days of constant load, and 0.02 inch for the interval between 10 and 20 days under constant load.

9.0 DESCRIBE THE PROCEDURE FOR MONITORING GROUNDWATER LEVELS DURING CONSTRUCTION OF THE UNDERPINNING WALL

As part of the temporary dewatering procedure, piezometers will be installed to monitor the groundwater level. Before the access shafts are excavated, a piezometer will be installed adjacent to each shaft. While constructing the tunnel under the north wall of the structure, three piezometers will be installed: one at each end and one at mid-length. When the tunnel is completed, a monitoring system of five piezometers will have been installed. If required, additional piezometers will be installed as the tunnels under the side walls are advanced.

10.0 COMMENT ON BORING CH-2 SHOWING FILL MATERIAL BELOW EL 587.0

The log for Boring CH-2 indicates silty sand to el 583'-8". From the results of other nearby borings and the general excavation plan for the site, it is believed that the predominant soil type is sandy clay till. If this is borne out during pit excavation and the till is compact and well bound, it will be acceptable for bearing at el 587. This acceptance would be based on the judgement of the geotechnical engineer using qualitative criteria, such as taking soil samples for strength analysis. On the other hand, if the till is not compact and well bound, or if it is silty sand, the material will be excavated to adequate till and replaced to el 587' with lean concrete on a pit-by-pit basis.

11.0 EVALUATION OF SOIL SPRINGS VALUES - STATIC AND DYNAMIC LOADING CONDITIONS

The soil springs are presently being evaluated as part of the final analysis of the structure. When this evaluation is completed, the requested information will be submitted.

REFERENCES

1. Consumers Power Company, Technical Report on the Service Water Pump Structure Underpinning, August 26, 1981

2. G.G. Meyerhof, "The Ultimate Capacity of Wedge-Shaped Foundations," Proceedings of the 5th International Conference on Soil Mechanics and Foundations, Paris, 1961

SERVICE WATER PUMP STRUCTURE
SETTLEMENT MARKER LOCATIONS

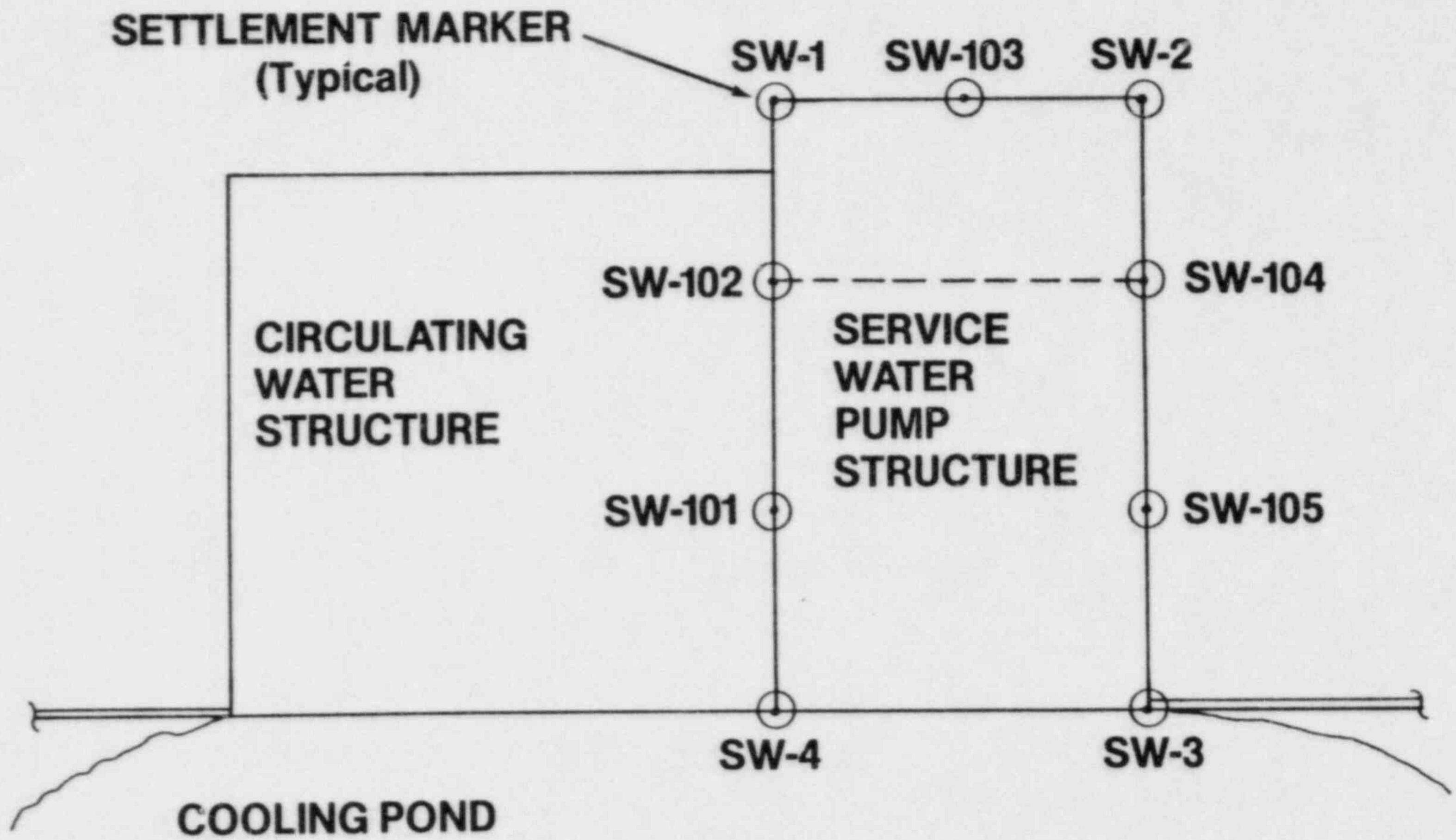

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FIGURE 1

SERVICE WATER PUMP STRUCTURE

ESTIMATED TOP OF PIER DEFLECTION DUE TO CREEP OF CONCRETE VS TIME

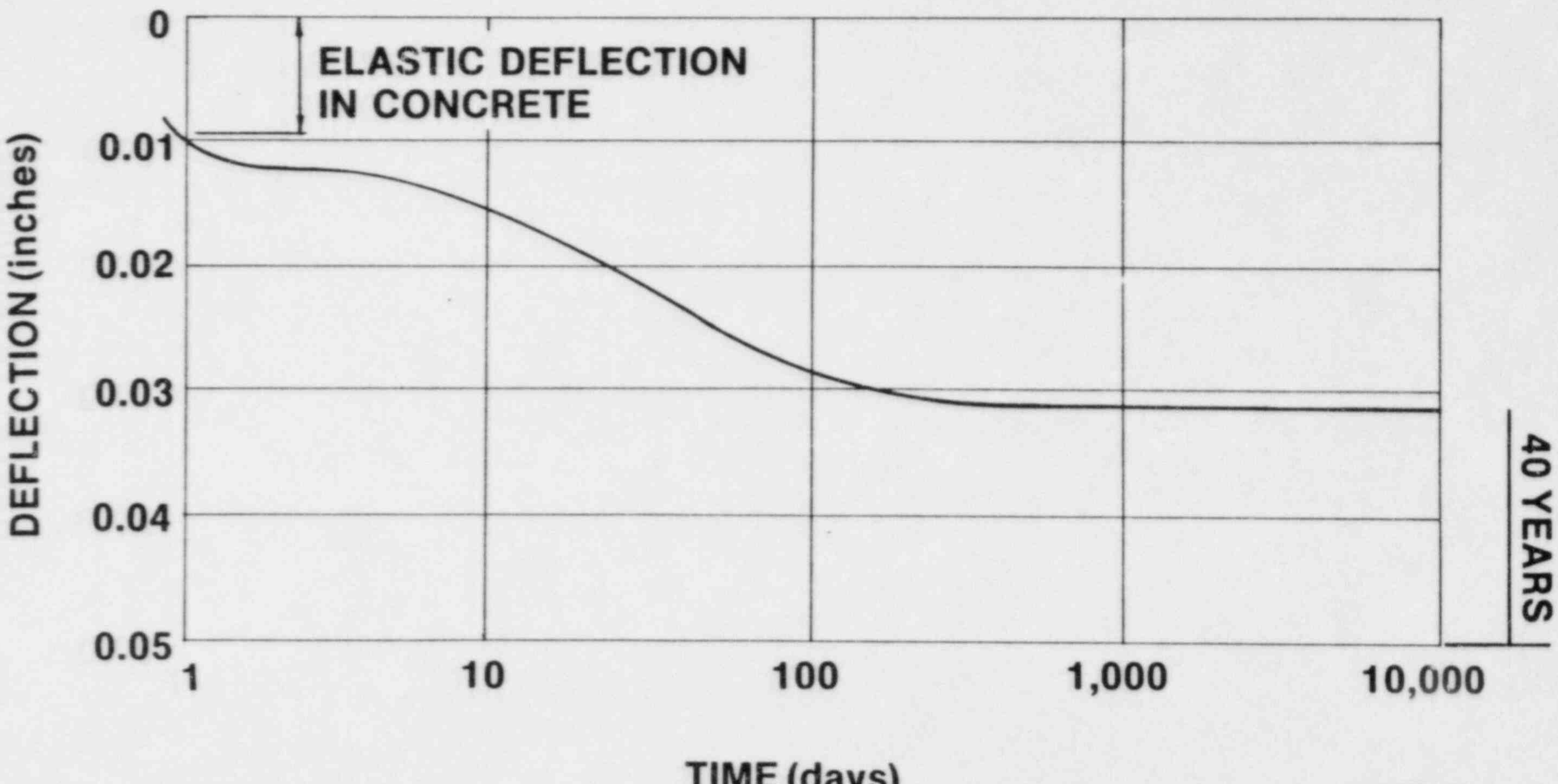

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FIGURE 2

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO SHRINKAGE OF CONCRETE VS TIME

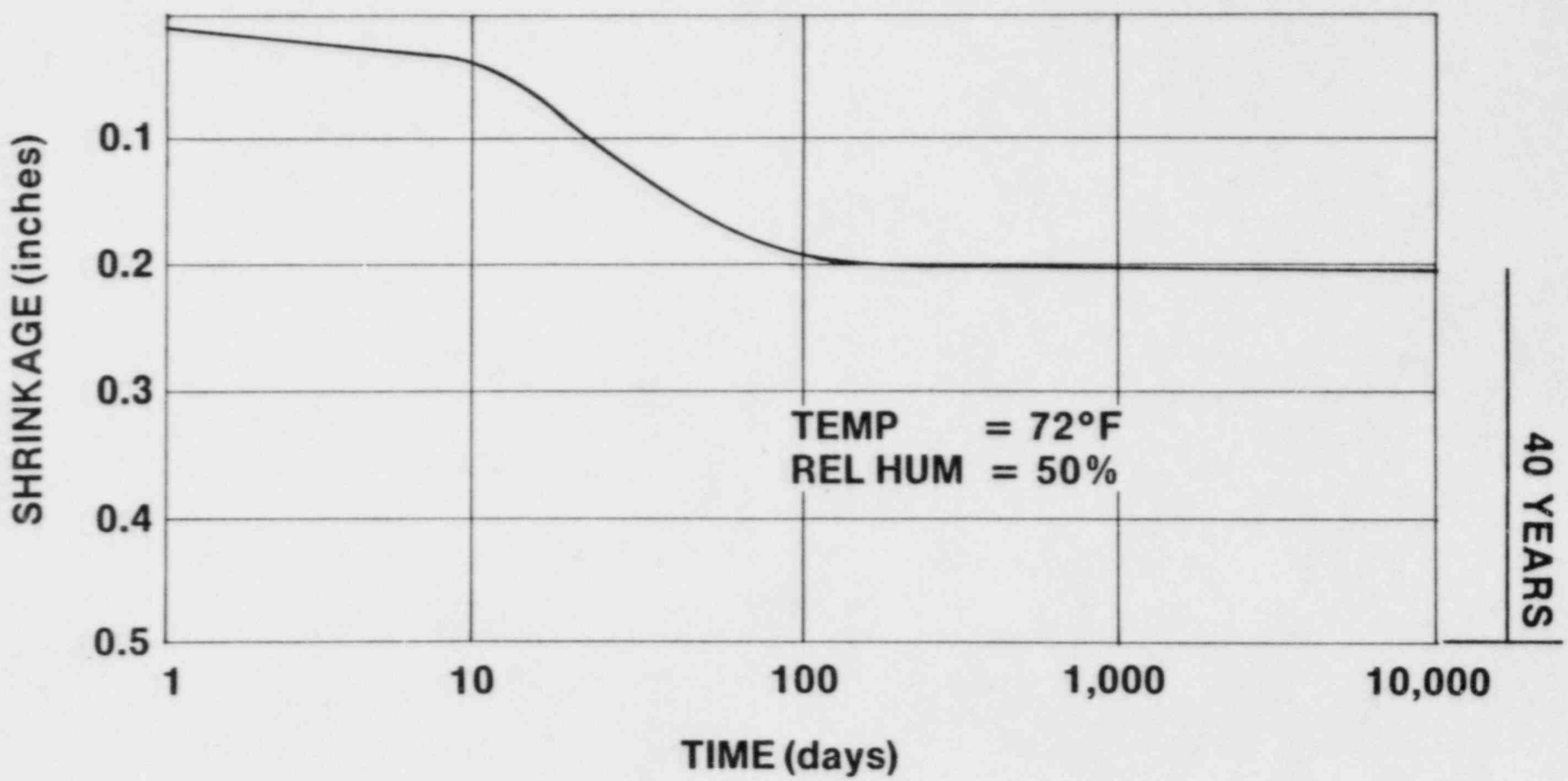

FIGURE 3

G-1584-11

SERVICE WATER PUMP STRUCTURE

ESTIMATED TOP OF PIER DEFLECTION DUE TO CONSOLIDATION OF SOIL VS TIME

(Time Is Measured from Start of Jacking)

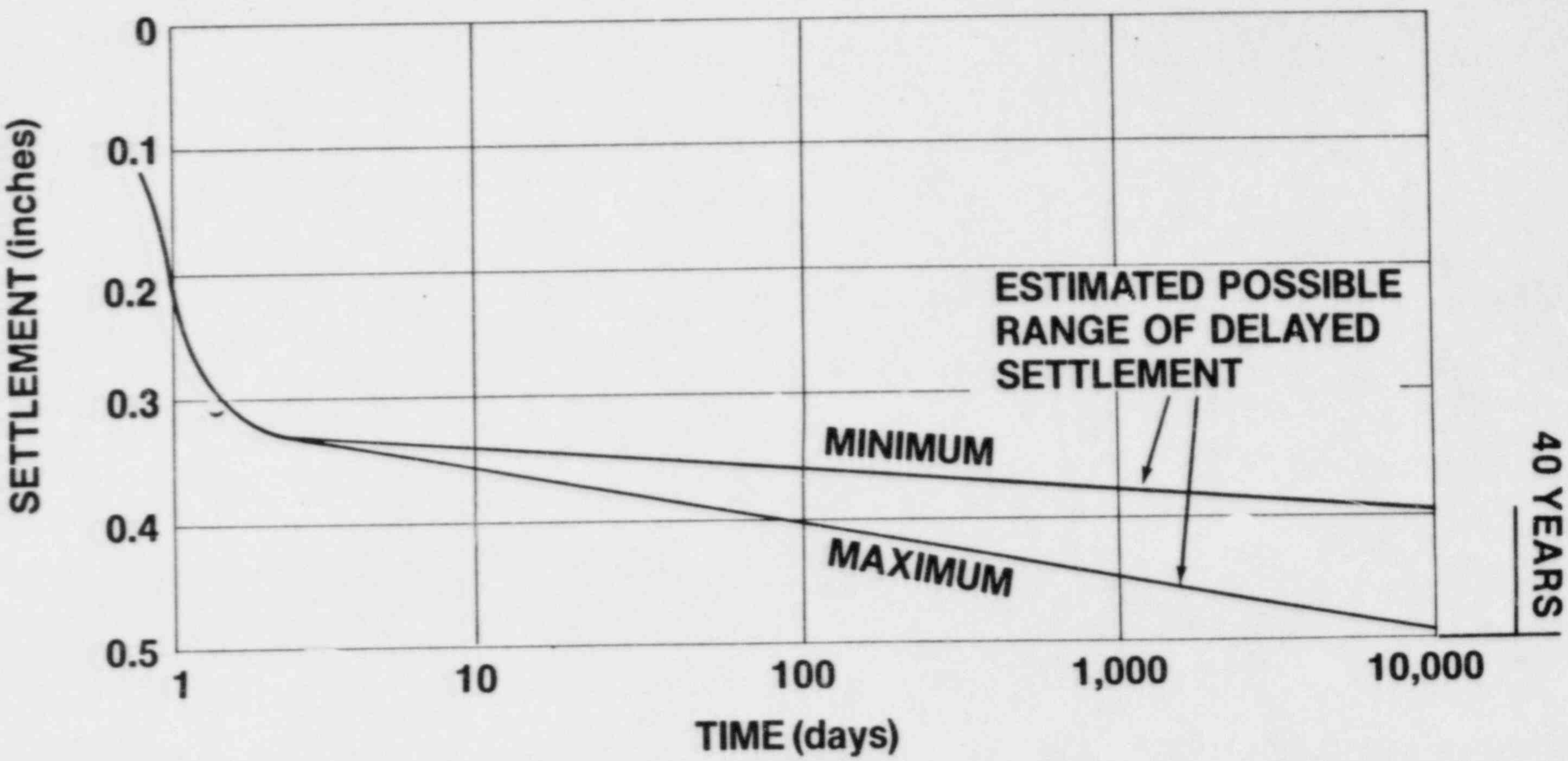

FIGURE 4

G-1584-14

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO TOTAL DEFORMATION VS TIME

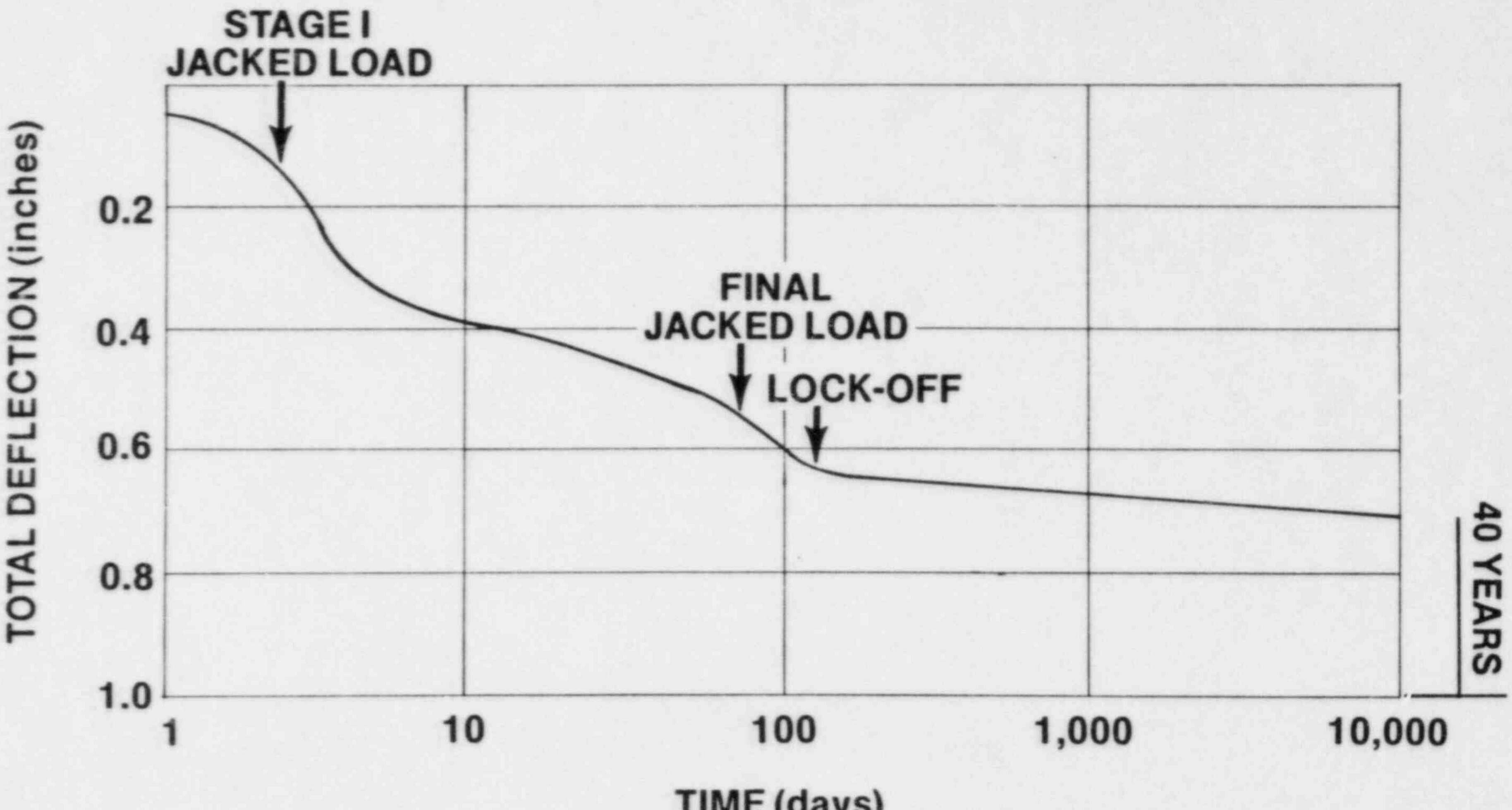

FIGURE 5

G-1584-12

Consumers
Power
Company

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October 21, 1981

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
REMAINING NRC SOILS-RELATED CONCERNS FOR DIESEL GENERATOR BUILDING
FILE 0485.16, B3.0.3 SERIAL 14316
ENCLOSURES:
(1) STRUCTURAL STRESSES INDUCED BY THE DIFFERENTIAL SETTLEMENT OF THE DIESEL GENERATOR BUILDING
(2) SUBGRADE MODULUS AND SPRING CONSTANT VALUES FOR DIESEL GENERATOR BUILDING STRUCTURAL ANALYSIS
(3) BEARING CAPACITY EVALUATION OF DIESEL GENERATOR BUILDING FOUNDATION
(4) LONG-TERM MONITORING OF SETTLEMENT FOR DIESEL GENERATOR BUILDING
(5) RELATIVE DENSITY AND SHAKEDOWN SETTLEMENT OF SAND UNDER THE DIESEL GENERATOR BUILDING
(6) ESTIMATES OF RELATIVE DENSITY OF GRANULAR FILL MATERIALS, DIESEL GENERATOR BUILDING, MIDLAND PLANT
(7) REVIEW AND CONTROL OF FACILITY CHANGES TO THE DIESEL GENERATOR BUILDING
(8) DIESEL GENERATOR BUILDING BEARING PRESSURE DUE TO EQUIPMENT AND COMMODITIES

On September 24, 1981, a request for additional information relating to the diesel generator building was made by the Staff in a telephone discussion. We are responding to this request by forwarding the enclosures itemized above. Each enclosure addresses one of the Staff concerns transmitted to us in the September 24, 1981 telecommunication.

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We believe the enclosed information adequately responds to the request and individual concerns identified for us by the Staff. The discussions and data contained in the enclosures to this correspondence lend further support to our conclusion that the design of the diesel generator building combined with the remedial actions are adequate and appropriate for this structure.

JAMooney
for JW Cook

JWC/RLT/dsb

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, Esq, w/o
JPMatra, Naval Surface Weapons Centre, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

PRINCIPAL STAFF			
DIR		EGIS	
D/D		FAO	
A/D		SLO	
DR&PI			
DE&TI	✓Landsman		
DEP&OS		File	

RECORD OF TELEPHONE CONVERSATIONS

Date: October 30, 1981 Project: Midland 50-330

Recorded by: Joseph D. Kane

Talked With:

CPCo	Bechtel	NRC	COE
D. Budzik	A. Boos	R. Landsman	H. Singh
G. Keeley	N. Swanberg	F. Rinaldi	
		D. Hood	
		J. Kane	

Route To: For Information

G. Lear
L. Heller
D. Hood
W. Paton
F. Rinaldi
✓R. Landsman, I&E, Region III
H. Singh, COE, Chicago
J. Kane

Main Subject of Call: Remedial Underpinning of Auxiliary Building and Feedwater Isolation Valve Pits

Items Discussed:

1. Enclosure 3 to CPCo September 30, 1981 submittal from J. W. Cook to H. R. Denton entitled "Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits". During the October 30, 1981 conference call CPCo was requested to respond to the following questions which had been developed in the COE/NRC review of Enclosure 3, relative to geotechnical engineering aspects in underpinning the Auxiliary Building.

Q.1. (Pg. 2, Sect. 4, 2nd Para.) Please define "design jacking force," how established and the duration that it will be held?

Q.2. (Pg. 2, Sect. 4, 3rd Para.) Discuss and provide detail of dowel connection. (Diameter, how distributed along wall, length of embedment, etc).

Q.3. (Pg. 3, Sect. 5.1, last para) The agreed upon acceptance criteria for soil particle monitoring during dewatering requires 0.005 mm and not 0.05 mm. Correction by CPCo required.

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Q.4. (Pg. 3, Sect. 5.1, Para. b) Installing the frozen cutoff membrane will cause expansion and possibly increase the soil voids. When ultimately unfrozen, what is the effect (e.g., further settlement) on safety related structures, conduits and piping. Provide discussion on the basic system of the frozen membrane [size and spacing of holes to be drilled, method for pumping brine into foundation layers, range of temperatures that are critical to wall stability which are to be monitored, decomissioning (e.g., grouting, etc)].

Q.5. (Pg. 3, Sect. 5.2) Clarify the procedure to be used in post tensioning the Electrical Penetration Area. Where will the buoyancy force be transmitted to the foundation and in what manner?

Q.6. (Pg. 4, Sect. 5.6, 2nd Para.) Please explain the meaning of "failure bearing capacity factors" and the basis for "the nine times the shear strength for the cone"?

Q.7. (Pg. 4, Sect. 5.b, 4th Para.) How will the equivalent soil modulus be determined? What is the depth that the measured settlement will be distributed over and what is the area to be used in determining the stress?

Q.8. (Pg. 4, Sect. 6) Presently, this paragraph implies that crack monitoring will not be performed on the existing structure. Please correct. Before remedial underpinning begins an accurate and up-to-date record of cracks should be developed for those safety related structures which could potentially be affected by the underpinning operations. This background record should be verified by I&E inspection and could serve as the basis for evaluating any changes in cracks due to underpinning operations.

Q.9. (Pg. 5, Sect 6.1.1 and 6.1.2) When will the acceptance criteria for the differential and absolute settlement be provided to the NRC?

Q.10. (Pg. 5, Sect. 6.2) Provide the basis for establishing the crack width of 0.03 inch. Appendix D should also address crack monitoring requirements during underpinning (frequency of reading, format for presenting observations, action levels etc).

Q.11. (Pg. 6, Sect. 7.2.1, last Para.) Provide discussion why the drained shear strength is not required to be considered in analyzing for adequate bearing capacity. Also in the last paragraph in Section 7.2.1, Pg. 7 indicate the basis for the 2 days and what would be required if the settlement rate does not reach a straight line trend in 2 days.

Q.12. (Pg. 7, Sect. 7.2.2) Where are the WCC controlled rebound-reload cycle soil test results? What is the corresponding stress level with a secant modulus of elasticity equal to 3500 KSF?

Q.13. (Pg. 8, Sect 7.2.3, 1st Para.) The estimates of settlement using the referenced NAVFAC DM-7 do not include secondary consolidation. What secondary consolidation would be indicated if the consolidation test results using the appropriate load increment were used? Compare this estimate with valves for permanent wall conditions "after jacking, long term". Please provide basis for the three estimated settlement valves for "Load transfer points for temporary load to reactor footing" at the bottom of pg. 8 and discuss any effects of this settlement on the reactor and pipe connections.

Q.14. (Pg A-1, Sect. 1, 2nd Par.) Please indicate how the soil spring constants were established for long term loads.

Q.15. (Pg C-2, last Par. and Pg. C-6, Par. B) What are the protective construction measures planned for the Turbine Building and Buttress Access Shafts and when will they be placed? Please provide discussion on the sequence of operations to complete the drift beneath the Turbine Building and show sectional views of this work with respect to the Turbine Building foundations and affected piping and conduits.

Q.16. (Pg C-3, Par. A.1.a) Please explain what is meant by minimizing the amount of concrete to be removed.

Q.17. (Pg. C-3, Par. A.1.c. and A.1.d) What is the magnitude of the load for testing the temporary support pier and how was it established and how will it be applied? Is the EPA foundation slab capable of supporting this load at this time?

Q.18. (Pg. C-4, Sect. A.1.f., 1st complete para.) Provide discussion on monitoring of the control tower behavior at this time. What criteria will be used to decide if preload should be stopped and support capacity should be added to the control tower?

Q.19. (Pg. C-4, Sect. A.2.) What are the reasons why the three temporary supports under the EPA should not be completed before the permanent support at the control tower is initiated?

Q.20. (Pg. C-4, Sect. A.3.a) Questions are raised as to whether the EPA structure can withstand the overhang condition which results if the initial temporary supports is assumed to fail. What is the basis and need for this extreme assumption? Is the EPA structure capable of withstanding this loading condition?

Q.21. (Pg. C-4, Sect A.3.b and A.3.c) The distinction between 3.b and 3.c is unclear. What is the magnitude of the load for testing and how established? Is there a problem with the EPA foundation slab providing a sufficient reaction load?

Q.22. (Pg. C-5, Sect. 14 and 15) It appears the operations described in these items are intended only for the wings and not the control tower. How is the load test and load transfer for the control tower to be completed. For the long term load test on the wings, what is the load magnitude and how was it established? What is the final sequence of operations in transferring the structure load to the permanent underpinning.

Q.23. (Pg. D-1, Sect 1.0, 2nd Par) Describe the procedure that relates allowable stresses and allowable strains with structure movements that are being monitored.

Q.24. (Pg D-2, Sect. 1, 3rd Par.) Please clarify the distinction between the first and second layer systems for detecting structure movement.

Q.25. (Pg D-2, Sect. 1, 4th, 6th, and 7th Para.) Please provide elevations and sectional views with typical details for the deep seated bench mark and the instrumentation for monitoring relative horizontal movement and absolute horizontal movement.

Q.26. (Pg. D.3, Sect. 2, 2nd Par.) Please clarify the explanation why the hydraulic pressure data cannot be used to measure load.

Q.27. (Pg. D-3, Sect. 2, 3rd Par.) Provide sectional view of set up for measuring difference in relative position. How does this procedure address the possibility of both the underpinning element and structure settling? Provide the basis for maintaining the jack/hydraulic system for 1 hour and for establishing the 0.01 inch movement.

Q.28. (Pg. D-4, Sect. 2, 4th Para.) When will the modeling and critical structural stresses and strains be determined and furnished to the NRC?

Q.29. (Pg D-5, Sect. 2, 2nd and 3rd Para.) Provide sketch and locations with typical details of instrumentation for measuring concrete stress, tell tale devices and predetermined points for monitoring vertical movement.

Q.30. (Pgs. D-5 and D-6, Sect. 3, Par. 3A.1, 3A.2, 3A.3) For the various types of monitoring described in these paragraphs provide an example of the forms to be used for plotting the recorded data. What are the predetermined levels of movements which would require adjustments and/or action by the onsite geotechnical engineer. Identify any specific instrumentation which would be continued to be read during plant operation and which eventually will be addressed by a Technical Specification.

2. Consumers was notified that the above questions do not contain the COE/NRC review comments on the laboratory test results for foundation soils beneath the Auxiliary Building. The COE/NRC comments on the test results will be furnished at a later date following CPCo submittal of the Part II lab test report which is expected to be submitted to the NRC the week of November 2, 1981.

3. Consumers indicated the questions asked in the conference call of October 30, 1981 would be addressed as far as possible in the upcoming meeting with NRC in Bethesda on November 4, 1981.