A

	1-19-81	Otto	A
	1-20-81	ERICKSON	B
	1-21-22-81	SINGH	C
Stet	~~1-22-81~~	~~Brammer~~	~~D~~
E_4 1	1-19-81	Shewmaker	E
x2 → 29	1-19-81	Shewmaker	F
~~x1 → 28~~	1-21-81	Chen	G

C.P. Dx 1 Otto 1-19-81

WILLIAM C. OTTO
Chief, Geotechnical Engineering Section

Corps of Engineers District, Detroit 1957-Present
Soil expert for District. Chief of Foundations & Materials and Geotechnical Engineering that included levees, dike disposal areas, surcharging settlement analysis, seepage analysis and pile driving and underpinning.

Bureau of Yards & Docks 1950-1957
Design and construction of airfields world wide, stability and settlement analysis, large dry docks, hospitals, dikes, surcharging, and drains etc

Nebraska Highway - Lincoln Nebraska 1946-1950
Laboratory design of asphalt pavements & aggregate testing.

Officer in U. S. Navy 1944-1946
Construction of factories, hospitals, dry docks

Corps of Engineers District, Omaha 1941-1944
Engineering Dept. Design & construction of airfields, and other military construction

International Boundary Commission 1938-1941
United States & Mexico, U. S. Section, Flood Control Structures

Indiana Highway Dept. 1936-1938
Project Engineer - Construction of multi-lane highways

Bldg & Service Corp., Decatur IL 1935
Charge roads & streets investigation & design

Published a paper in Sweden on Bank Protection

Published a paper at Northwestern University on a five year study of settlement of structures at Selfridge A.F.B.

Published a paper for ASTM on statistical study of flexural strength of concrete.

DEFENDANT'S EXHIBIT 2

9/27/80

15 pp attached I don't have

CPCo Position - Diesel Generator Building

1. Full scale field test is the most reliable technique to predict settlement. The surcharge program allowed for direct measurements of settlement, therefore, no need to rely on sampling & lab testing. NRC Response
Discuss importance of time element

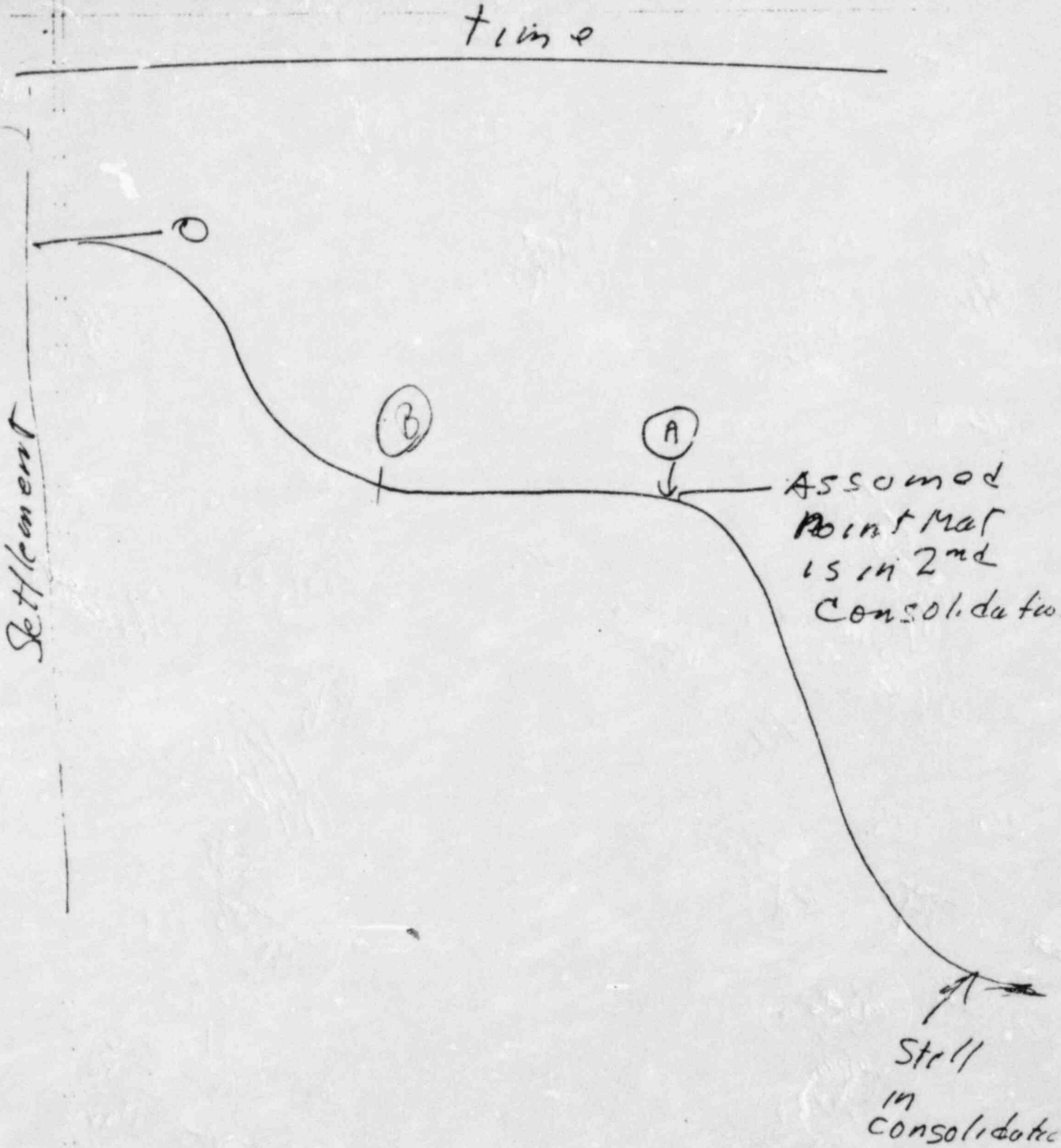
2. Surcharge produced stresses in the fill in excess of stresses to prevail while DGB is operational.

3. Settlement data recorded during surcharge program showed:
   a. Eventual decrease in rate of settlement.
   b. Slight rebound after surcharge removal.
   c. Straight line behavior representative of secondary consolidation.

4. Piezometer data recorded during surcharging showed:
   a. Rapid dissipation of pore water pressure, indicating rapid consolidation.
   b. Following surcharge removal, a slight drop in piezometric level occurred, and level eventually stabilized with groundwater table.

5. Recognized sampling & lab testing limitations.
   - thin (e.g. 3/4" thick) test samples
   - unavoidable sample disturbance
   - problems in selecting representative samples

Vugraph 1



X3

DEFENDANT'S EXHIBIT 3
PENGAD-Bayonne, N.J.

C. P. X4 Otts 1-19-81

3. SETTLEMENT UNDER FOOTING LOAD. The foregoing are soil conditions prior to construction and the application of footing load. Figure 29 is used to make the ensuing pore pressure computation, p_w (along the axis of loading), caused by the footing load. The center of the footing is the common corner of four identical rectangular areas, each 4 ft x 16 ft. For one of these

$$m = 2/3 \text{ and } n = 8/3 \text{ at top boundary of blue clay}$$

and

$$m = 1/9 \text{ and } n = 4/9 \text{ at bottom boundary of blue clay}$$

Then for one rectangular portion of the loaded area, at the top boundary,

$$p_w = 0.17 \times 5{,}000 = 850 \text{ psf}$$

and for the entire footing,

$$p_w \text{ (at top)} = 4 \times 850 = 3{,}400 \text{ psf}$$

Similarly, at the lower boundary

$$p_w = 400 \text{ psf}$$

If depths other than these are included in the computations, the initial p_w line (Figure 38) would be represented by the dashed line of this figure. However, the straight line, AB, is drawn and, as usually happens, it provides a good approximation because the two areas, one outside and one inside the dashed line, are approximately equal.

Since this is of Case I category,

$$u = \frac{3{,}400}{400} = 8.5$$

The average voids ratio for samples 1, 2, and 3, when they are completely consolidated under the total loads (4,440 psf for sample 1, 3,600 psf for sample 2, and 2,750 psf for sample 3), is 1.34. The total settlement of the footing, therefore, is

$$S = \frac{e_i - e_f}{1 + e_i} H_i = \frac{1.493 - 1.340}{2.493} \times 30 \times 12$$
$$= 22 \text{ in.}$$

Naturally, it would be unwise to carry loads on spread footings under these conditions, but the purpose here is to illustrate what happens if this is done.

The value of H when 50 percent of the primary settlement is realized is then 30×12—½ of 22, or 349 in. or almost 29 ft. This is called H_{50}. Also, the typical blue clay is found to have a consolidation coefficient C_v, of 0.003. The time consolidation relation is then

$$t = \frac{H_{50}^2 N}{1{,}400 C_v} = \frac{(29)^2}{1{,}400 \times 0.003} N \text{ years}$$
$$= 195.5\ N \text{ years}$$

The quantity N is obtained from Figure 37 for any arbitrarily assigned value given to q. For example, for $u = 8.5$ and for $q = 25$ percent, N (interpolating between $u = 3$ and $u = 10$) is 0.052. Therefore, $t = 0.052 \times 195.5 = 10.2$ years. For 50 percent completion of the total settlement, N is 0.29 and t is 195.5×0.29, or 56.6 years, and so on. That is, it will take about 56 years, 7½ months, for 11 in. of the total 22 in. of settlement to occur. In this manner, a table of settlements corresponding to different periods of time may be computed, or a time versus settlement curve may be drawn through the plotted computations.

4. DISSIMILAR COMPRESSIBLE SOIL LAYERS IN JUXTAPOSITION. It often happens that two or more compressible soil strata, each with different permeability coefficients and consolidation characteristics, adjoin. Figure 39 illustrates this. It is possible to convert one of the layers (No. 2, with thickness H_2) into the same type of soil as that in the other layer (thickness H_1). The result of such a conversion is a problem dealing with one homogeneous soil layer having the same consolidation characteristics throughout. The conversion is a simple, mathematical device utilizing equations (42) and (43). For that illustrated in Figure 39, there are two drainage courses; therefore, equation (43) is used. Layer 2 is replaced by another (thickness H_3) having the same soil consolidation properties as layer 1, with the stipulation that the resulting single homogeneous layer (thickness $H_1 + H_3$) drains at the same speed as the two dissimilar strata. Considering the initial, instead of the H_{50}, thickness is of small importance, so that

$$t = \frac{H_2^2 N}{5{,}600 C_{v(2)}} = \frac{H_3^2 N}{5{,}600 C_{v(1)}}$$

or

$$\frac{H_2^2}{C_{v(2)}} = \frac{H_3^2}{C_{v(1)}}$$

where $C_{v(2)}$ is the consolidation coefficient of the soil of layer 2, and $C_{v(1)}$ is the consolidation coefficient of layer 1. Then

$$\frac{C_{v(1)}}{C_{v(2)}} = \frac{H_3^2}{H_2^2}, \text{ or } H_3 = H_2\sqrt{\frac{C_{v(1)}}{C_{v(2)}}} \qquad (44)$$

EXAMPLE

Let

$$H_1 = 20 \text{ ft}, H_2 = 12 \text{ ft},$$
$$C_{v(1)} = 0.004, \text{ and } C_{v(2)} = 0.001$$

Then

$$H_3 = 24 \text{ ft}$$

The transformed, homogeneous, single layer then has a thickness of 20 + 24, or 44 ft, and a consolidation coefficient of 0.004. This single layer drains under load at the same speed as the two separate layers in juxtaposition.

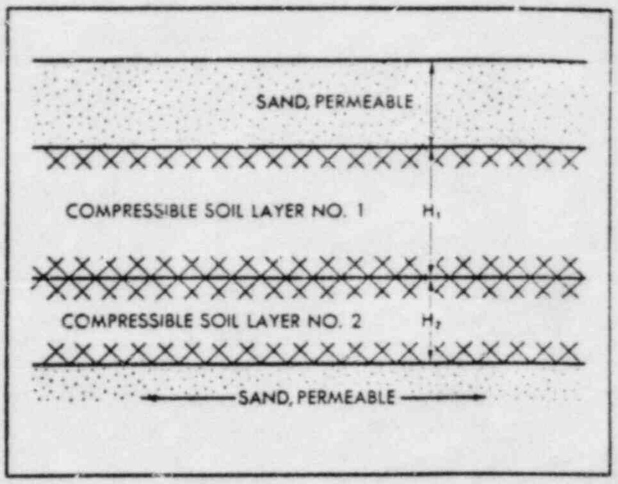


FIGURE 39

Dissimilar Soil Layers, Each Compressible, in Juxtaposition

Section 4. QUICK SETTLEMENTS, BEARING CAPACITY, AND LATERAL EARTH PRESSURES

C4.01 QUICK SETTLEMENTS

The present consideration of so-called quick settlements refers only to static loads. The subject of dynamic loading is discussed later. Quick settlements, as considered here, do not involve pore pressure. They result from compaction or densification of the soil, with a diminution of the voids ratio but without the development of pore pressure, and from elastic or plastic yield or deformation. To a certain extent, they are recoverable on release of the load. Compacting a damp soil by rolling it with a sheepsfoot roller produces no pore pressure, yet it densifies the soil. Air escapes from the voids. Dynamic forces may develop a momentary pore pressure in large masses of quite permeable sand, but static loading of ordinary sand produces only intergranular stresses.

The expressions for quick settlements do not involve the independent variable, time. They involve only the coordinates of points in the loaded soil mass, the dimensions of the loaded area, the manner of distributing the load over the loaded area, and certain moduli of compression or deformation.

For slow settlement, the movement of soil is restricted to one direction—vertical. Lateral movement in consolidation involving pore pressure is considered nonexistent. For this condition, Poisson's ratio is zero. Lateral displacement always occurs in quick settlement. If quick settlement is solely elastic or plastic without compaction or densification of the soil, Poisson's ratio approaches ½; that is, the soil deforms without change in volume. Since, normally, there is some compaction by the load, Poisson's ratio has a value somewhere between zero and ½.

The extent of slow settlement (involving pore pressure) during construction embraces no basic principles other than those already presented. In the bulk of structural problems its influence usually is slight. Quick settlements normally take place in minutes or hours; for all practical purposes, they stop when the load application ceases.

It is always a serious error to ignore quick settlements. The so-called design load for spread footings should be so determined that the quick settlement of all the different footings is as nearly the same as practicable in all cases in which slow settlements are not expected.

C4.02 ESTIMATION OF QUICK SETTLEMENTS

1. ELASTIC BEHAVIOR. A few instances of earth settlement almost conforming to the theory of elasticity may exist. In these the soil must be elastically isotropic and homogeneous and must have Young's modulus E constant with depth. This may happen with loads on small areas, where only shallow depths of soil are affected. Test data reported by Terzaghi[33] and many others show, however, that soil deformations under load are not, in general, charac-

DEFENDANT'S EXHIBIT 5 1-19-81

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

JAN 8 1981

Docket Nos.: 50-329/330 OM, OL

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

SUBJECT: FOLLOW-UP ON DECISION REGARDING ADDITIONAL SOIL BORINGS AND TESTING - MIDLAND PLANT, UNITS 1 AND 2

By letter of November 10, 1980, I informed you of our decision relative to your request for relief from making additional borings and associated tests of soils in eighteen areas on the Midland Plant site. That letter noted that a relaxation of certain requirements for six Standard Penetration Tests (SPT) in the vicinity of plant structures were in order on the basis of additional boring data which you submitted on September 14, 1980 and our extensive discussion on the merits of your position. My letter of November 10, 1980 also stated that certain borings which we had requested June 30, 1980 along portions of the cooling pond embankments should be relocated to areas of the dike immediately adjacent to the submerged emergency cooling water reservoir. The details of this relaxation, including the changed boring locations, are provided herein.

The new borings in the areas of interest for which subsurface information was provided by your letter of September 14, 1980, and the six SPT borings identified by Question 37 of our June 30, 1980 letter which may now be eliminated, are as follows:

Structure	New Borings Provided 9/14/78	Eliminated SPT Borings
Diesel Generator Building	CH-13, CH-14, CH-15, CH-16, CH-17, CH-18	COE-8 COE-13
Service Water Structure	CH-1, CH-1A, CH-2, CH-3	COE-16
Retaining Wall	PD-9	COE-14
Auxiliary Building	TW&TEW Series	COE-17, COE-18

8101220109

Details of this relaxation are further described in the enclosed letter of December 2, 1980 by Mr. P. McCallister of the U. S. Army Corps of Engineers, our geotechnical consultant. Mr. McCallister's letter includes a revised sketch (Figure 1) showing all the borings in the plant fill area and noting the six borings from which the SPT's have been eliminated. Mr. McCallister's letter also includes a revised sketch (Figure 2) showing the relocated boring locations on the cooling pond dikes. Figure 2 shows the new locations for borings COE-1, COE-2 and COE-3 (previously located in the south and east dikes), and boring COE-7 (previously located in the northwest area). We further endorse Mr. McCallister's comments regarding selection of undisturbed sample locations and his requests that the guidance of Regulatory Guides 1.132, "Site Investigation for Foundation of Nuclear Power Plants," and Regulatory Guides 1.138, "Laboratory Investigation of Soils for Engineering Analysis and Design of Nuclear Power Plant" be used as appropriate.

Your letter of November 21, 1980 forwarded Amendment 85 to the Midland application and noted your belief that Amendments 85 and 81 satisfy the concerns raised in Question 37. We find that these submittals do not fully satisfy the concerns of Question 37. Except as changed herein for the six SPT borings and the relocation of four dike borings, it remains our position that the requested soil borings and testing are still required as stated in my letter of November 10, 1980.

Sincerely,

Robert L. Tedesco, Assistant Director
for Licensing
Division of Licensing

Enclosure:
McCallister's letter dtd. 12/2/80

cc: See next page.

Corp documents

10 pp
attached
I don't have

Secondary compression.

The settlement vs log time curve exhibits the standard consolidation - secondary compression curve. The coefficient of secondary consolidation, C_α, is 1.25 inches per log cycle of time (for DG-3). For 16-foot thick clay layer,

$$C_\alpha = \frac{1.25 \text{ in}}{16 \text{ ft}} \cdot \frac{\text{ft}}{12 \text{ in}} \text{ per log cycle}$$

$$C_\alpha = 0.0065 \text{ (dimensionless)}$$

which compares favorably with the typical values for C_α of clay with a low to medium coefficient of secondary consolidation.

	C_α	Secondary compressibility
	0.002	very low
	0.004	low
DG-3 →	0.0065	
	0.008	medium
	0.016	high
	0.032	very high
	0.064	extremely high

Table from Duncan, J.M. and Buchignani, A.L., Engineering Manual for Settlement Studies. University of California, Berkeley, June 1976

Simano
Rec'd. 5/12/8

DEFENDANT'S EXHIBIT 9

30 pp attach
I don't have

Dewatering - Questions

1. Liquefaction should be expected to occur as the backfill sands become saturated. Saturation of backfill sands would appear to begin when the groundwater table rises above the top of the natural sands, elevation 605 (Figure 24-8, boring log DG-28). Present design criteria or data which supports elevation 610 as maximum groundwater level.

2. Design of the permanent dewatering system is based upon two major findings: (1) The granular backfill materials are in hydraulic connection with an underlying discontinuous body of natural sand, and (2) Seepage from the cooling pond is restricted to the intake and pump structure area. Soil profiles (Figure 24-2), pumping test time-drawdown graphs (Figure 24-14), and plotted cones of influence (Figure 24-15) indicate that south of the diesel generator building the plant fill material adjacent to the cooling pond is _not_ an effective barrier to the inflow of cooling pond water. Reevaluate the permeability of this material and the effect on the permanent dewatering system. Include data, especially the recovery data from PD-3 and complete data from PD-5, for review.

B

CORPS DOCUMENTS

DEPOSITION EXHIBIT
Son
Flower + Erickson Dep
1-20-91

PERSONAL DATA OF RON ERICKSON

Full Name: Ronald Lee Erickson   Birth Date: 14 Dec. 1948

Present Position: Geologist Geotechnical Engineering Section, Engineering Divn., Detroit District, U. S. Army Corps of Engineers

Education: B.S. Geology - 1971
Western Michigan University

Gov't Training:	5-74 to 5-76	Geologist Rotational Training Program-Detroit
	1-77	Systematic Drilling & Blasting WES
	3-78	Intro to Supervision - GSA
	5-78	Network Analysis - OCE
	2-79	Intro. Ground Water Hydrology - HEC
	8-80	10th Short Course - Geological Engineering - Geological Eng. Foundation - Berkley, CA

Experience:	06-79 to Present	Geologist, Geotechnical Engineering Section, Eng. Div., Detroit District, U. S. Army Corps of Engineers.
		Work Areas: Engineering, Subsurface Investigations, Quarry Investigations, Geotechnical Review.
	11-76 to 06-79	Geologist, Flint Flood Control Project. U. S. Army Corps of Engineers
		Work Areas: Earth Anchors, Dewatering, Backfilling, Construction
	05-76 to 11-76	Geologist, U. S. Army Corps of Engineers Jidda & Al Kobar, Saudi Arabia
		Work Areas: Drill & Test Water Wells, Monitor Quarry operations
	05-74 to 05-76	Geologist, Rotational Training Program Detroit District Office, U. S. Army Corps of Eng.
		Work Areas: Engineering, Construction
	06-68 to 05-74	Civil Engineer Technician Grand Haven Projects office, U. S. Army Corps of Engineers, seasonal employee while attending college, full time upon graduation (12-71)
		Work Areas: Hydrographic survey, Marine Construction, Dredging

STIFFNESS

SIMPSON

ROUTING SLIP (AR 340-15)	NEVER USE FOR APPROVALS, DISAPPROVALS, CONCURRENCES, OR SIMILAR ACTIONS.	
TO	INITIALS	DATE
1 CHIEF, ENGR DIV.	ZJ	17 Mar
2 ~~FOUNDATIONS & MAIL'S~~ ED-G		
TECHNICAL ENGR		
WATER CONTROL CTR		
COASTAL ENGR		
VALUE ENGR		

Now we're talking!
ZJ

CHECK ACTION DESIRED

INFORMATION	SIGNATURE	NOTE AND RETURN
CIRCULATE	NECESSARY ACTION	SEE ME

FROM	TELEPHONE	DATE
DJL		17 Mar

ORGANIZATION

DA FORM 1 NOV 64, 1222 (Small)

DEPOSITION EXHIBIT #2
ERICK DEP
KCK 1-20-81

NCEED-T (1 Feb 80) 2nd Ind, Supp #1
SUBJECT: Providing Geotechnical Engineering Assistance to the Nuclear Regulatory Commission

DA, Detroit District, Corps of Engineers, P.O. Box 1027, Detroit, MI 48231

13 MAR 1980

TO: Division Engineer, North Central

1. As indicated in paragraph 3 of the 2nd Ind. dated 29 February 1980, the District indicated that a detailed manpower analysis would be provided for both plants which would indicate the personnel limitation impacts upon each project. A different approach was taken for each of the various subtasks by dividing each subtask into technical staff and administrative staff (report writing, interoffice review, typing, NCD review, etc.) work efforts. As soon as the technical team completes its work, it could theoretically begin work on the next subtask. However, a slight delay was allowed as a margin for uncertainties. The resulting analyses, with the projects independent of each other, is inclosure 1.

2. Analyses were made to complete the work using the existing staff only, and for the existing staff plus one additional geotechnical engineer, GS-12. The results are inclosures 2 and 3 respectively. A manhour summary by subtask is attached. Note that subtask 4, for both sites, has no report requirement.

3. Inclosures 4 and 5 provide the detailed manpower analyses for the Bailly and Midland plants, respectively.

4. Inclosure 6 provides a table displaying the earliest dates the subtasks in the NRC contracts can be completed by the existing staff, and the existing staff plus one geotechnical engineer GS-12, respectively.

6 Incls
as stated

Robert V. Vermillion

ROBERT V. VERMILLION
Colonel, Corps of Engineers
District Engineer

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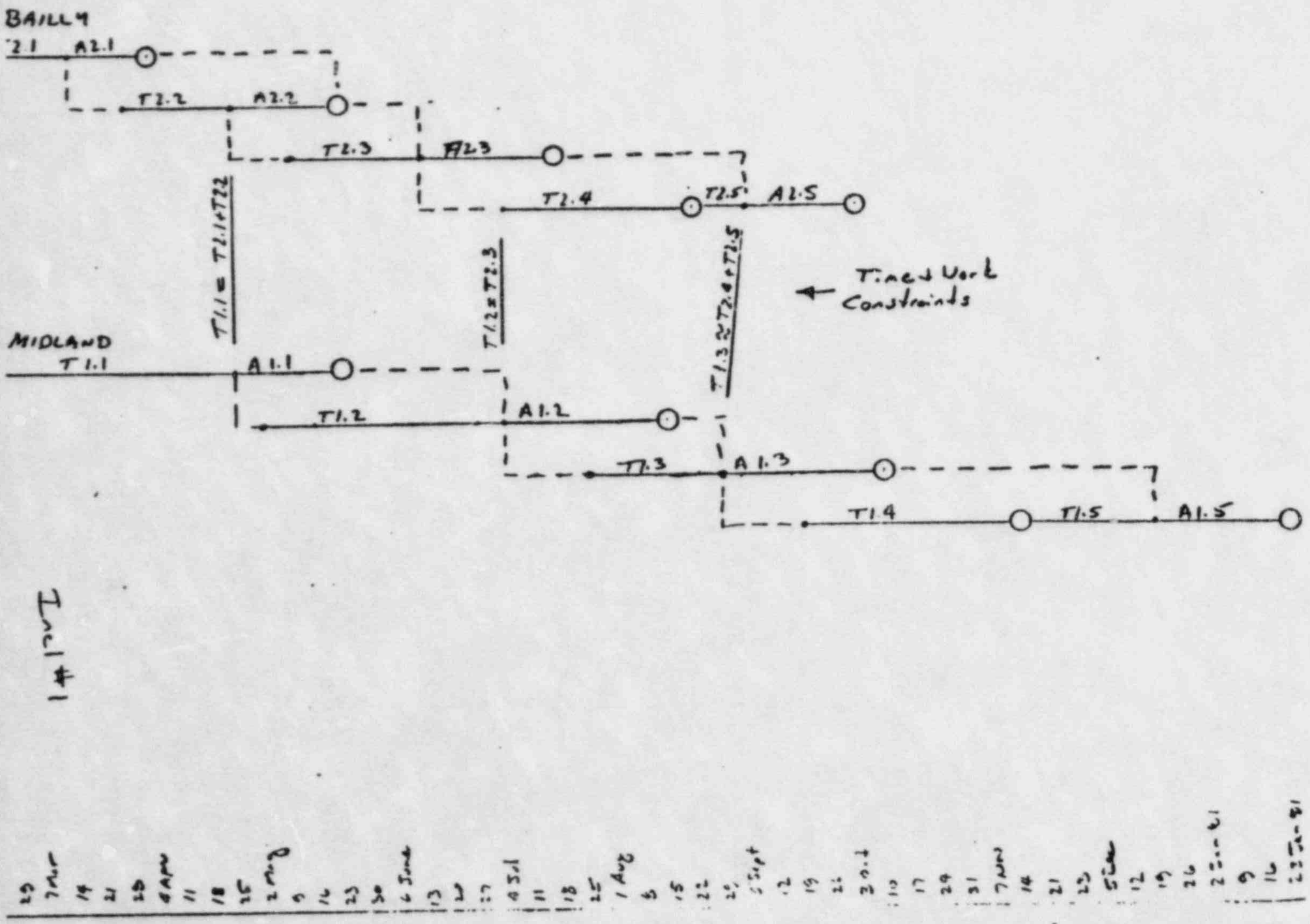
6 Incls
as stated

ROBERT V. VERMILLION
Colonel, Corps of Engineers
District Engineer

BAILLY + MIDLAND SCHEDULES BASED
ON INDEPENDENT MANPOWER + EXPERTISE
USING EXISTING STAFF ONLY

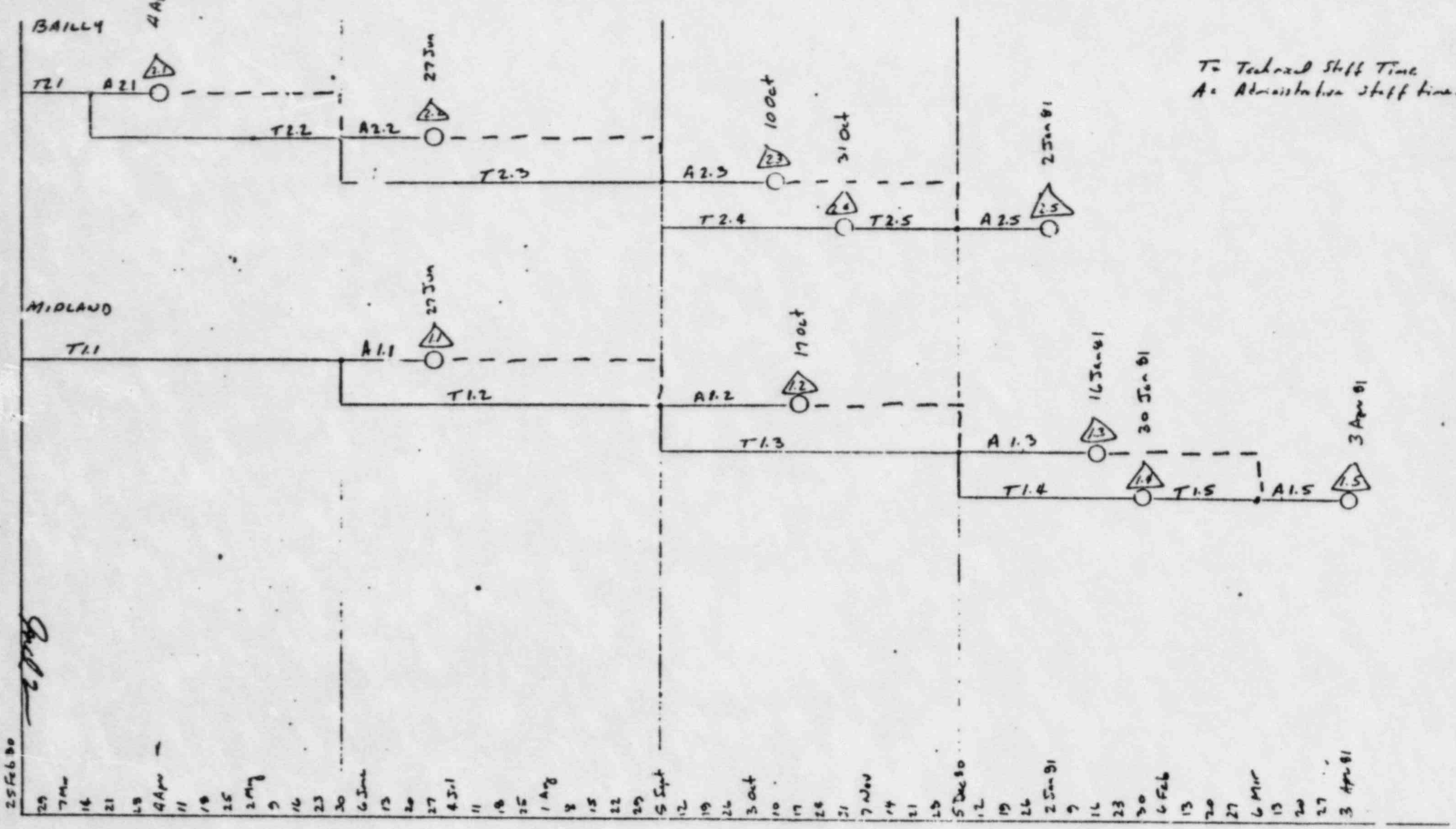


Incl #1

NRC GM-80

NRC Support Schedules Based on Existing Staff

T = Technical Staff Time
A = Administrative Staff time.



Incl 2

Incl 2

Final Revised Schedules for Existing Manpower levels -

Technical review staff ~~See page figures~~

Tech Review Task by Site / Staff	T1.1	T1.2	T1.3	T1.4	T1.5	T2.1	T2.2	T2.3	T2.4	T2.5
OTTO	300	248	80	304	176	96	160	84	272	48
Kytutz	302	128	36	56	92	112	126	48	56	64
Grandstrom	300	360	184	304	200	72	160	178	272	68
Ericksen	212	116	68	304	36	96	128	64	272	48
Knox	124	92	12	56	20	64	32	16	56	24
	(8wk)	(9wks)	(5wks)	(8wks)	(5wks)	(3wks)	(4wk)	(5wks)	7wks	2wks

Use Schedule type "3" (See CPM Diagram)

Constraints	OTTO	KYTAITY	GRUND.	ERICK.	KNOX
T1.1 = T2.1 + 2.2	556*	540	532	436	220
T1.2 = T2.3	332	176	538*	180	108
T1.3 = T2.4 + T2.5	400	156	524*	388	92

* = Critical Path

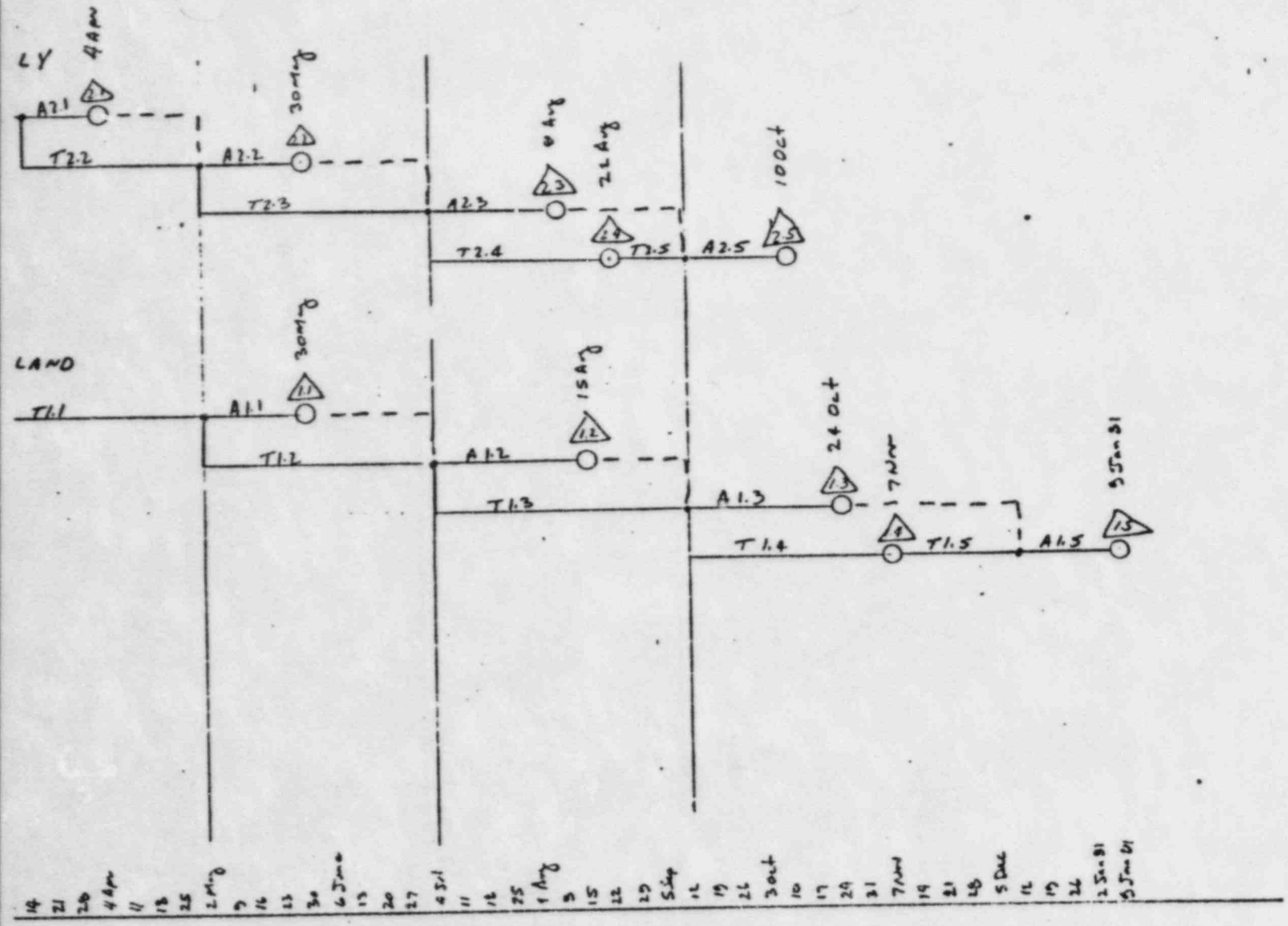
Therefore M/S T1.1 + T2.2 = 556/40 = 14 weeks

M/S T1.2 + T2.3 = 538/40 = 14 weeks

M/S T1.3 + T2.5 = 524/40 = 13 weeks.**

** Can be significantly reduced if ACRS Mtg + Licensing Board meetings are less than estimated (quite likely)

NRC SUPPORT SCHEDULES BASED ON
EXISTING STAFF + 1 GS12 GEOTECH ENGNR



INCL#3

Final Revised Schedule for Existing Staff + 1 GS-12

Geotechnical Engineer ~~See Figure on Page~~ 3

Task Requiring Critical / Staff	T1.1	T1.2	T1.3	T1.4	T1.5	T2.1	T2.2	T2.3	T2.4	T2.5
Otto	374	248	80	304	176	0	0	0	0	0
Kytasty	128	126	36	56	92	112	160	92	272	64
Grundstrom	364	360	184	304	200	0	0	0	0	0
Erickson	128	116	68	304	36	112	160	104	272	64
Knox	148	92	12	56	20	104	116	16	112	40
New	96	—	—	—	—	112	160	178	272	64
	10 wks	9 wks	(10 wks)	(3 wks)	5 wks	(6 wks)	(4 wks)	5 wks	(7 wks)	(2 wks)

Use Schedule type "3" (See CPM Diagram)

Constraints	Otto	Kytasty	Grund.	Erick.	Knox	New
T1.1 = T2.1 + T2.2	374	400*	364	400*	368	368
T1.2 = T2.3	248	320	360*	220	108	178
T1.3 = T2.4 + T2.5	80	372	184	404*	164	336

* critical path

From the above it is seen that even with 1 more engineer we still can't quite make the same schedule we could make with no personnel conflicts. (400 MH > 374)

∴ M/s T1.1 + T2.2 = 400/40 = 10 wks

T1.2 + T2.3 = 360/40 = 9 wks

T1.3 + T2.5 = 404/40 = 10 wks**

** Can be significantly reduced if ACRS mtgs & Licensing board meeting are less than estimated (Quite likely)

Shortg [illegible] 25Feb80

17

NG - [illegible]
WO - [illegible]
PK = [illegible]
JG = [illegible]
RE = [illegible]
JK = [illegible]
WL = [illegible]

Time

Bailly Generating Station

	Subtasks 1	2	3	4	5
	[illegible] rpt	[illegible] SER	[illegible] SER	ACRS mtg (5)	SER Supp
I Soils					
A. Ground Water					
1. levels and Locations	RE PK-4 JG-4 WO-4	JG-8 RE-4	JG-4		
2. Movement					RE-8 JG-8
a. fluctuations	WO-4 RE-8 JG-4	RE-8	RE 4		
b. lag factor	RE-8	RE-8	RE 4		
c. uplift of buildings (buoyancy)	PK-4 WO-4 RE JG-4	JG-8	JG-8		
3. Monitoring plans	0	WO-8	JG-4		
4. Dewatering systems	RE 8 RE	RE-8 RE	JG-4		
B. Borings					
1. NRC criteria	JK-8	JK-4	—		
2. Location	JK-4	RE-8	RE 4		
3 Depth	PK-4	RE-8	RE 4		
4 Verification		JK-8	JK 8		JK-8
5 Test data of samples	WO-4	WO-8	WO 8		WO-8
C. Settlement					
1. Due to Loading, Vibration (Non-Seismic)		JG-16	JG-8		PK JG WO > 16
a. Differential		PK-8	PK-8		
b. Uniform		PK-8 RE 4	PK-8		
c. Long Term		WO-8	WO 8		
d. Quick remedial plans		WO-8	WO 8		
2. Monitoring		[illegible] > 8	JG 16		
		JG-8	JG 8		
D Seismic					
1. Determining "g" Factor Used	WES-8	WES-4	WES-4		WES 8
2. liquefaction	WES-8 [illegible] > 4	WES-8 WO-8	WES 8 WO 8		WES 8
3 SHAKE - Consolidation	WES-8 [illegible] > 4	WES-8 WO-6	WES WO > 8		WES 8
4. Geologic Considerations	WES-8	WES-4	—		

WES [illegible]

Total 1 — Ps Subtotals NG WO 24 — SL WO 8 — 40 11 — 44 24

Gul +

E. Bearing Capacity for Non-Pile Areas	PK-8 JG-4	JG 8	JG 4		
F. Quality Control		RE 8	RE 8		RE 8
II Piling					
A Capacity (vertical & lateral)					
1. Design Load and Load Combinations	PK-8 JG-4	PK 40 JG 40	JG 40		PK JG >16
2. Safety Factor	PK-8 JG-8	PK 16 JG 8	JG 16		
3. Group Action of Piles	PK-8 JG-4	PK 8 JG 4	JG 8		
4. Arrangement of Piles	PK-8 JG-8	PK-8 JG 4	JG 2		
5. Load Testing					
a. Method	SE-8 JG-4	WO-16 JG-8	WO JG >8		
b. Criteria	WO-8 JG-4	WO-16 JG 8	JG >8		
c. Relationship of static and dynamic loads	PK-8 JG-4	PK WO JG >8	PK JG >8		
B Depth					
1. Installation Method					
a Hammer, Vibratory, Predrilling, Jetting, Combinat.	WO-8 PK-8	WO PK >8	WO 4		JG 8 PK 8 WO 8
b Shallow	WO-8 JG-8 JG-8				
c Bed Rock	WO4 RE8				
2. Criteria	WO-8	WO-8	WO 4		
C. Heave					
1 Monitoring	JG4 PK8	JG-8 PK4	JG 8		JG PE >8
2. Spacing	PK4 JG4	JG-8	JG 4		
3. Redriving Heaved Piles	WO-4 PK4	JG-8	JG 4		
4. Study Tension and Compression	PK-8 WO4	PK-8 WO4	PK 4		
D. Corrosion					
1 Cathodic Protection	WO-4 JG 4 RE8	RE10 WO4	RE 4		RE 8 WO4
2 At contacts in various soil layers	WO4 RE8 JG4	RE-8	RE 4		
Pa Subtotals	NC WO - 44 PK - 80 JG - 48 RE - 32	64 100 104 24	24 32 116 16		8 12 24 32 24 32 16

E. GNMENT					JG-4 WO-8
1. Checking as driving	WO-4 PK4	WO-8 RE4	WO 4		
2. Pulling criteria	WO-4	WO-8 RE 4	WO 4		
a. Obstructions - Boulders	WO 4	WO-8 RE 4	WO 4		
b. Slope indicators					
F. Consolidating Soils					JG 8 PK 8 WO 4
1. Areas of Previous Jetting	PK WO 4 RE4	WO-8	WO 4		
2. Disturbed Soils	WO4 JG4	WO-8	WO 4		
3. Alternate methods	WO 4				
G. Variability of Load on Piles due to any of Above	PK 4 JG 8 WO 8	JG-16	JG 4		
III Geology					
A. Depth of Rock	RE 8	RE-8 JK4	RE 4		RE-24
B. How layers formed	RE4 JK4	RE-8	RE 4		
1. Perched water table?	RE4 JK4	RE-8	RE 4		
2. Absorbtion of water by surrounding lays	JK4	RE-8	RE 4		
C. Fault History	JK8	RE 8 JK4	RE 4		
D. Crustal Rebound	JK4	RE-8 JK4	RE 8		
E. Determing Seismic Basin	RE-4 JK8	WES-8 RE-8	RE 8		
F. Interp. Tests and Borings	RE-8 JK4	JK-8	JK 8		JK 16

Pg Subtotals

NG				12
WO	~~24~~ 28	40	20	8
PK	~~8~~ 16	0	8	12
JG	~~8~~ 12	16	8	24
RE	~~16~~ 32	68	32	16
JK	~~8~~ 36	8 10	8	
WL				
WES	0	8	0	0

Totals				X Seg Separate Sheet(4)	
NG	92	120	142	56	20
WO	~~44~~ 96	~~112~~ 160	88	272	40 48
PK	~~48~~ 112	120 116	48	56	~~48~~ 64
JG	~~52~~ 72	160	178	272	56 68
RE	~~36~~ 96	~~112~~ 148	64	272	48
JK	~~20~~ 64	~~24~~ 32	16	56	24
WL	24	32	36	56	12
WES	32	32	20	176	24
	588	788	592	1236	300
MHi	464	768			264

Total MH

464	588
768	788
592	
1236	
264	300

3324 MH = 1.68 Man-yrs

19

Midland Nuclear Plant - Units 1 & 2 (starting from 25 Feb 80)

	Subtasks 1 Sk rpt	2 D. SER	3 Final SER	Time 4 ACRS Mtg	5 SER supps
I Soils					
A Groundwater (other than for fires)					
1. levels and locations	WO 4 PK 4 JG 0 RE 4	RE 3 JG 8 JK 8	JG 8		RE 4, JK 4, JG 8
2. Movement (natural)	WO 8 RE 3	WO 8 RE 4 JK 8	WO 4		WO 8
3. Monitoring plans	WO 8 RE 8	JG 8 RE 8	JG 8		JG 4
4. Dewatering systems	JG 8 RE 4 ~~WO~~ JG 4	WO 16 JG 8	JG 8		JG 8
B Borings					
1. NRC Criteria	JK 8	JK 4	—		—
2. location & Depth	RE ~~PK~~ 8 JK 8	JK 16	JK 8		JK 8.
3 Verification	RE ~~PK~~ 8 JK 8 JK 8 RE 4	RE ~~PK~~ 8 JK 8 RE 8	JK 4		JK 8
4 test data of samples	~~WO 16~~ JG 8	WO 16 JG 8 RE 8	WO 8		WO 8, ;
C. Settlements / Consolidation (other than for fires)					
1. Non-seismic					
a Short term, long term, etc.	JG 8 WO 8 PK 4	JG 16 WO 8 PK 4	JG 8		WO 4, JG 8
b. Uniform, differential	JG 8 WO 8 PK 4	JG 16 WO 8 PK	JG 8		WO 4, JG 8
2. Monitoring	JG 8 WO 8 RE 4	JG 16	JG 8		WO 4, JG 8
D. Seismic					
1. "g" factor used	RE 8 WES 8	WES 4	WES 4 16		WES 4 8
2. liquefaction potential	WO 8 WES 16	WO 4 WES 8	WES 4		" WO 4
3. Shake & consolidation	WES 16	WO 4 WES 8	WES 4		" WO 4
4. Perched water tables	RE 8 WES 8	WES 8	WES 4		"
5. Geologic Features	RE 8 WES 8 JK 8	WES 4	—		
E. Other Structures on Fill not considered in Pt II below	JG 8	JK 40 JG 8 WO 16	JG 16, WO 8		JG 8, WO 8
F. Quality control	WO 8 JK 8	WO 16	WO 8		WO 8

Pg Subtotals

	1	2	3	4	5
WO	48	16	28		52
PK	12	16	0		0
JG	52	88	64		52
RE	32	36	0		0
JK	32	84	12		20

Incl 5

Encl 5

[illegible] for category I structures as [illegible]

A. Dewatering					
Review, ^(establish)^ Plans & Comps	RE 16 WO 24 JG 16	RE ^16^ 24 JG 8 JG 25 WO 16	JG 24 WO 7		JG 16, WO 8
2. Review Backup System	JG 8 J RE 8	JG 16	JG 8		JG 4
3. Rate of influence	PK 4 JG 8 RE 8 WO 8 4	JG 16	JG 8		JG 8 WO 4
4. long term considerations	RE 8 WO 5	RE 8 JG 16	JG 8, RE 4		RE 8, JG 8
5. Operations during OBE	WES 16 RE 8 WO 8	WES 8 16 WO 8	WES 4 WO 4		WES 8 WO 4
6. monitoring & maintenance	RE 16 JG 8 WO 16 8	WO 8 RE 16	WO 8		WO 8, RE 4
B. Diesel Generator Bldg					
1. Review, ^(evaluate)^ Planned Remedial action (preload)	PK 40 WO 16 JG 16	JG 40 PK 16 WO 8	JG 24 PK 8 WO 4		PK 8 WO 16 JG 16
2. Effect of Preload on Conditions	WO 24 PK 16 JG 8	WO 16 JG 8	WO 8		WO 8
3. Settlement monitoring plans	WO 16 8 JG 8 RE 8	JG 16 RE 8	JG 4		JG 8 WO 4
4. Effects on service utilities ducts	WO 16 PK 8 JG 8	JG 16 PK 8	PK 8		PK 8, JG 8
5. Continuity of fx (add'l borings?) 6. Seismic Considerations	WO 16 8 PK 8 RE 8 WES 8	WO 16 PK 8 WES 8	WO 8 WES 4		WO 8 WES 8, WO 4
C. Service Water Pump Structure					
1. Review & evaluate remedial plan (jacked piles)	PK 32 WO 16 JG 16	JG 32 PK 16 WO 8	JG 8 PK 8		PK 8, JG 8
2. Pile load test	RE 8 WO 24 JG 8	WO 16	WO 8		WO 8
3. Seismic considerations	WES 8 WO 8	WES 8 WO 8	WES 4		WES 8
4. Settlement monitoring plans	WO 8 4 JG 8 RE 4	JG 8	JG 4		JG 4, WO 4
5. Service utilities & intake well	PK 24 16 JG 8 WO 8	JG 16 PK 8	JG 8		PK 8, WO 4, JG 8
D. Borated Water Tanks					
1. Review & evaluate borings & tests	WO 24 16 JG 16 JG 8 PK 8	JG 16 WO 8	JG 8		WO 8 JG 8
2. Continuity of strata (add'l borings?)	WO 24 JG 8 JG 8	RE 8 WO 8 JG 8	JG 8, WO 4		WO 4, JG 8
3. Seismic considerations	WES 8 JG 4 WO 8 RE 4	WES 8 WO 8	WES 4		WES 4
4. Settlement monitoring plans	WO 8 JG 8	JG 8	JG 4		JG 4
E. Underground Storage (Diesel Fuel) Tanks					
1. Review & evaluate borings & tests	WO 16 JG 8 PK 8	JG 16 WO 8	JG 8		WO 4 JG 8
2. Continuity of strata (add'l borings?)	WO 8	WO 8	WO 4		WO 4, JG 4
3. Seismic considerations	WES 8 WO 8	WES 8	WES 4		WES 4
4. Settlement monitoring plans	JG 8	JG 8	JG 4		JG 4

WO	210	144	44	100
PK	114	56	8	32
JG	88	138	29	128
RE	20	56	120	16
WES		16	4	

per Sub table

Underground utilities	TC 16				
Identify category I activities		—	—		4
Review & evaluate Profiles + Soil data	JC 8 RE 4 PK 8 WO 8 JC 4	JG 24 PK 8	JG 8 PK 4		RE 8, JG 8
3. Continuity of study (add'l borings)	WO 8	WO 8	WO 4		WO 4
4. Seismic Considerations	WES 8 WO 8	WES 4	WES 4		WES 8
5. Settlement/foundation monitoring plans	WO 8 JG 16	JG 8	JG 4		JG 8
C. Aux. Bldg + Feedwater Valve Pit Fix					
1. Review & evaluate remedial plans (caissons & jacked piles)	PK 40 WO 8 JG 24	JG 32 PC 8 WO 8	JG 16 PC 8		PK 8 WO 8 JG 8
2. load tests on piles/caissons	RE 16 WO 8 JG 8	JG 8	JG 4		WO 4 JG 8
3. Seismic considerations	WES 16 WO 8	WES 8 WO 4	WES 4		WES 2
4. Settlement monitoring plans	WO 8 RE 8	JG 8 RE 4	JG 4		JG 4
5. Utilities in area	WO 8 PC 16 JG 8	JG 8	JG 4		JG 4
III Geology					
A. Depth to Rock	RE-32C-8	RE 4	RE 4		RE 4
B. Layer formation	8 8	"	"		"
1. groundwater pillows & perched watertables					
2. moisture characteristics					
C. Fault + seismic history	8 8	"	"		"
D. Crustal Rebound	8 8	"	"		"
E. Interpretation of borings, test pits, lab tests, etc.	RE 8 WO 16, PC 16, JC 8	WO 8, RE 8	WO 4, RE 8		JG 8 WO 8, RE 8, PK

Pg Subtotals					
WO	88	25	8		24
PC	65 82	16	12		28
JG	90	85	40		32
RE	65 52	60 42	24		24
JC	43 36	0	0		0
WES	24	12	8		16

TOTALS					
Release in 2d					
WO	~~[illegible]~~	248 ~~244~~	40	304	176
PK	~~[illegible]~~	~~[illegible]~~	36	56	~~[illegible]~~ 72
JG	~~[illegible]~~	~~[illegible]~~	144 ~~224~~	304	~~232~~
RE	~~[illegible]~~	~~[illegible]~~	~~[illegible]~~	304	36
JC	132 ~~[illegible]~~	~~[illegible]~~	12	56	20
WES	124	~~[illegible]~~	44 ~~64~~	208	64
NG	320	288	148	56	160
NL	80	72	40	56	40
MH	1344 1762	1304 1400	628 ~~400~~	1344	768 288

5806

Σ ~~[illegible]~~ MH = 2.90 M Years

NRC Task and Subtask Completion Dates

	Existing Staff	Existing Staff + 1 geotech. engr.
Bailly		
2.1	4 Apr 80	4 Apr 80
2.2	27 Jun 80	30 May 80
2.3	10 Oct 80	8 Aug 80
2.4	31 Oct 80	22 Aug 80
2.5	2 Jan 81	10 Oct 80
Midland		
1.1	27 Jun 80	30 May 80
1.2	17 Oct 80	15 Aug 80
1.3	16 Jan 81	24 Oct 80
1.4	30 Jan 81	7 Nov 80
1.5	3 Apr 81	9 Jan 81

Incl #6

Corps files

DISPOSITION FORM

Erickson

*For use of this form, see AR 340-15, the proponent agency is The Adjutant General's Office.

REFERENCE OR OFFICE SYMBOL	SUBJECT
NCEED-T	Geotechnical Engineering Assistance to NRC Orientation Meeting at the Bethesda, Maryland 7-8 November 1979

TO NRC File FROM Kubinski DATE 1 Feb 80 CMT 1
KUBINSKI/vw/6786

1. The purpose of this trip was orientation in nature. It was made to acquaint R. Erickson and J. Kubinski with the NRC Organization, staff, project requirements, and facilities available at their main office in Bethesda, Maryland.

2. The meetings took place on the 7-8 November 1979. I will refer to the meeting that took place on the 7th as Meeting I, and the meeting that took place on the 8th as Meeting II.

3. The following are significant items discussed at the respective meetings:

a. Meeting I: This meeting was primarily orientation in nature. NCE personnel were introduced to the NRC staff, their organizational elements and in general their function as a review agency. Dave Lynch of NRC gave a concise presentation on the general mission, and referencing specifically Bailly Nuclear Generating Station near Gary, Indiana. He also covered elements in the normal review process giving an indication as to general requirements. Later, he covered the more technical aspects and problems in existence at the site.

b. Meeting II: This meeting was also of orientation nature, with the emphasis placed on the Midland Nuclear Facilities. This meeting was very similar in nature to the one on Bailly, but was conducted with emphasis on the Midland site.

4. The following people were involved in these meetings:

a. Meeting I:

Bob Jackson (NRC)
Lyman Heller (NRC)
Dave Lynch (NRC)
J. Kubinski (NCE)
R. Erickson (NCE)

b. Meeting II:

Lyman Heller (NRC)
Darl Hood (NRC)
Dan Gillen (NRC)
J. Kubinski (NCE)
R. Erickson (NCE)

DEPOSITION EXHIBIT
Erickson #3
MCR 1.20.81

DA FORM 1 FEB 62 2496 REPLACES DD FORM 96, EXISTING SUPPLIES OF WHICH WILL BE ISSUED AND USED UNTIL 1 FEB 63 UNLESS SOONER EXHAUSTED. * U.S. GPO: 1974-540-843/8656

NCEFD-T
SUBJECT: Geotechnical Engineering Assistance to NRC Orientation Meeting at the Bethesda, Maryland 7-8 November 1979

5. The items discussed are listed below:

a. Meeting I:

I. This meeting was of orientation nature and a good introduction to the entire program was given by Dave Lynch, Project Manager, NRC, Bailly Nuclear Generating Station.

II. The purpose of NRC's mission with respect to review is to insure radiological safety and containment of all possible danger. It is not NRC's concern to see that OASHA standards or safety in general ~~is~~ are observed.

III. The issue at Bailly is concerned with piles supporting ~~at~~ primary containment facilities. It is a rigid structure and, therefore, no displacement can be tolerated. Dynamic operations result in displacement and this displacement must be monitored so that the entire structure is adjusted accordingly. ~~It~~ There is a very well-defined load/deflection analysis for the entire facility.

IV. The containment facility cannot fail. It may have to be politically safe which implies a greater than necessary safety factor to be technically safe.

V. The Safety Evaluation Report (SER) has not yet been written for the Bailly plant.

VI. It is necessary to defend any technical judgments before the Advisory Committee for Reactor Safety (ACRS). At the Bailly site it will be necessary to defend as built conditions.

VII. The term "Intervener" is defined as follows: An intervener must live within 50 miles of the proposed facility (the State in which the facility exist can act as an intervener); the interveners may hire firms or individuals to represent them in obtaining information concerning the construction or operation of nuclear facilities.

VIII. The normal review process consists of the following items:

- Applicant submits PSAR (Preliminary Safety Analysis Report)

- NRC writes Safety Evaluation Report (SER). This SER is a concise picture of NRC staff's review.

- NRC submits SER to Advisory Committee on Reactor Saftey (ACRS). The ACRS can form subcomittees in which their members and/or their consultants can evaluate the specific issues.

- ACRS evaluates SER/PSAR and letter on the safety of the plant is written.

- Public hearings are generated only if the license is thought to be able to be granted. This is a construction license.

- The Construction Permit, issued by NRC, but licens is granted by the Chairman of the Commission.

- The review of deviations from the PSAR, SER and CP must be reported by the applicant to the Nuclear Regulatory Commission Office of Inspection and Enforcement (I&E). The I&E Office sends this information to the review office for review, and ~~the~~ a new license or amended license is usually issued.

NOTE: The following is a list of items concerning the Bailly plant.

IX. The construction permit for Bailly Plant consist of non-displacement high capacity piles which go to bedrock or glacial till and support ~~of~~ a concrete mat foundation. They are embedded in concrete approximately three feet.

X. A brief driving history for the piles is as follows. In driving the piles stiffening occurred at 55 feet. Blow counts from 200 to 300 blows per inch were experienced. The till material is at about 110 feet and bedrock is at 120 feet. Above a very stiff clay deposit which is ~~wate~~ wedge shaped in profile, intermittent s and clays are the overburden material. This stiffening occurs in a very e sand above this larger clay deposit.

XI. In May 1974 the construction permit called for a test pile program which indicated significant problems in driving. Shortly after that, NIPSCo came in with a short pile proposal. In September 1977 an alternate proposal to jet long piles was submitted. A test program was initiated and in February 1978, the NRC issued an order to jetting the piles. In jetting the piles, the soil reacted similar to a giant wash boring (1,000 gallons per minutes at 300 PSI). The area of disturbance was much too large and the pile was actually lose near the surface. The nature of the structure which was to be supported by these piles demanded that the piles have uplift capacity. Because of the disturbance and lack of uplift capacity, the short pile concept is once against an issue as of March 1978. These piles would develope end bearing and friction. The applicant was allowed to drive 100 piles as indicators to determine capacities and applicability of using the short pile concept. A cluster was driven to observe heave within the piles. This brings us to the current state of the issue.

XII. It is now the task of the NRC review to look at all of the above submittals and reconsider the entire issue. They must also determine if construction restrictions are required or further load test are required. The jetting procedures have made soft spots which encompress almost five percent of the area of the foundation. These loosened areas must be densified and a technique developed to insure that they develop all lateral capacities as well as uplift capacities.

NCEED-T
SUBJECT: Geotechnical Engineer'ng Assistance to NRC Orientation Meeting at the Bethesda, Maryland 7-8 November 1979

XIII. The Advisory Committee on Reactor Safety (ACRS) has already indicated that nothing was substantially wrong with use of short piles to provide substantial foundation. That is, that there is no deflection in the piles and that all the disturbed areas due to the jetting procedures are densified.

XIV. It is apparent that now it is necessary to look at the PSAR and become fully familiar with it as well as considering the groundwater affect on the foundation.

XV. NCE will have to prepare the entire Safety Evaluation Report (SER) and not just assist in its preparation. A sample Saftey Evaluation Report is available from NRC and will be transmitted.

NOTE: The last item is of general nature.

XVI. The hearing of a Hearing Board process can be described as follows. Administrative law judge act as the Chairman. Engineer Scientists and some technical people drawn from university staff act as part of the committee. The commission delegates authority to the Board, the Board inturn can dictate policy. The Board can question any item and the interveners' attorney can question around items brought up by the Board. It is, therefore, necessary to minimize any questions the Board may have by clear concise presentations.

XVII. NCE will meet with Newmark, Hall and Davison at Champagne (University of Illinois) concerning the piling issue sometime in January or February.

b. Meeting II:

This meeting was of a briefer nature than Meeting I. At this meeting Joel Kane (NRC) and Darl Hood (NRC Project Manager) presented an introduction concerning issues at the Midland Nuclear Facility.

I. As a preliminary to the meeting, the following items were discussed. A brief discussion on what safe shutdown earthquake (SSE) or an operating base earthquake (OBE) were head. Appropriate volumes of the Preliminary Safety Analysis Report (PSAR) were to be sent to NCE as soon as possible. The applicant, Consumers Power Company (CPCo), must still respond to original I&E questions on the interim report and on 10CFR 50.54(f). There is apparently a report or a paper on the dewatering system.

II. The I&E Office (Inspection and Enforcement) is investigative in nature and generally goes to the NRR (Nuclear Regulatory Review) for support. The I&E Office considered the overall performance of the applicant as well as the technical adequacy of any field changes. The viability of the Quality Assurance Program is also investigated by this group.

NCEED-T
SUBJECT: Geotechnical Engineering Assistance to NRC-Orientation Meeting at the Bethesda, Maryland 7-8 November 1979

III. The current state of the review is one in which the construction permit should be suspended, modified or revoked by the Commission. One of these actions is necessary to take concerning the quality assurance breakdown at th Midland site as well as the inadequate fill in support of Category I structures.

IV. Questions of a non-policy nature can go directly to the applicant. No commitment is considered to be binding between NCE and the applicant. Once these questions are established and they are addressed to the applicant, they should be documented especially when they are relatively significant.

V. Construction inspections or visits to the site are necessary in performing the mission. NCE must be able to reply (we saw) in reference to a specific issue if possible.

VI. More than one visit is in most cases necessary, since sequential events will be occurring in the fixing of unstable conditions at the site.

VII. The NRC Office of Inspection and Enforcement has a fulltime man at the site, and he can be contacted concerning observing any action at the site.

VIII. Meeting concluded with two immediate items of major concern:

a. Should the existing license be modified, suspended or revoked.

b. A list of visits and times sequentially established in the future.

6. These meetings were of orientation in nature and it is difficult to establish any conclusions. The actions to be taken in the future are ones concerning scheduling field trips and site visits, carrying out orientation procedures with all documents transmitted, assuring that all documents have been transmitted and then beginning the review process and making either recommendations, comments, or conclusions regarding the situations at both facilities.

J Kubinski
J. KUBINSKI
Technical Branch

CONCURRENCE:

R. Erickson

J Heller (NRC)

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION. CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

Rec'd. 6/7/80

REPLY REFER TO: WESGA

4 JUN '80

SUBJECT: Report on Review of Geotechnical Aspects of the Seismic Safety of Midland Nuclear Power Plant

District Engineer
U. S. Army Engineer District, Detroit
ATTN: NCEED-T/Mr. Neil Gehring
477 Michigan Avenue
Detroit, MI 48226

DEPOSITION EXHIBIT
Erickson #4
mck 1-20-81

1. Inclosed is a Memorandum for Record dated 30 May 1980, subject: Visit to Midland Michigan NPP on 27-28 February 1980, A Review of the Midland Plant Units 1 and 2 FSAR (Including Revisions 1-27) by P. F. Hadala (Incl 1). This memorandum is an interim report on our work under your IAO No. NCE-IA-80-047.

2. If you have any questions, please feel free to contact Dr. Hadala at FTS 542-3475.

FOR THE COMMANDER AND DIRECTOR:

1 Incl
as

F. R. BROWN
Engineer
Technical Director

CF w/incl:
Mr. Jim Simpson, NCDED-G
Dr. Lyman Heller, NRC
Mr. Joe Kane, NRC

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

REPLY REFER TO: WESGA

30 May 1980

MEMORANDUM FOR RECORD

SUBJECT: Visit to Midland Michigan NPP on 27-28 February 1980, A Review of the Midland Plant Units 1 and 2 FSAR (Including Revisions 1-27)

Background and scope

1. The writer visited the Midland Michigan Nuclear Power Plant on 27-28 February in the company of NRC and COE representatives. Bechtel and Consumers Power Company representatives briefed us on 27 February. The attendance list is given in Incl 1. On 28 February we toured several areas of the plant in small groups, were briefed by Bechtel's consultants (see Incl 1) and had an opportunity to ask questions. Inclosure 2 is the agenda for the meeting.

2. The Detroit District of the Corps of Engineers is assisting the Site Analysis Branch of NRC with review of geotechnical aspects of the project relating to safety. My involvement is in support of Detroit District and by prior agreement with the District is limited to geotechnical earthquake engineering issues.

3. Subsequent to the visit, I reviewed the Midland Units FSAR Volumes 1-4 and Volume 7 in a cursory fashion and Sections 2.5-2.56 of the FSAR in detail. The documents I received were complete up through Revision 27. I also performed some analyses whose results are summarized in the following paragraphs and reviewed Volumes 1-7 of "Response to NRC Questions Regarding Plant Fill."

Comments regarding liquefaction potential

why did they

4. An independent Seed-Idriss Simplified Analysis was performed for the fill area under the assumption that the groundwater table was at or below elevation 610. For 0.19 g peak ground surface acceleration, it was found that blow counts as follows were required for a factor of safety of 1.5:

Uncorrected blow counts

Elevation ft	Minimum SPT Blow Count* For F.S. = 1.5
610	14
605	16
600	17
595	19

*For M = 7.5, blow counts would increase by 30 percent.

The analysis was considered conservative for the following reasons (a) no account was taken of the weight of any structure, (b) liquefaction criteria for a magnitude 6 earthquake were used whereas an NRC memorandum of 17 Mar 80 considered nothing larger than 5.5 for an earthquake with the peak acceleration level of 0.19 g's, (c) unit weights were varied over a range broad enough to cover any uncertainty and the tabulation above is based on the most conservative set of assumptions. The curve described in the above tabulation is compared to those for other groundwater tables and earthquake loading conditions in Incl 3.

omitted in Detroit's Repr

5. All of the plotted boring logs of the plant fill area furnished to me by the Detroit District, CE, were reviewed. Out of over 250 standard penetration tests on cohesionless plant fill or natural foundation material below elevation 610 which are shown in Incl 4, the criteria given above are not satisfied in four tests on natural materials located below the plant fill and in 23 tests located in the plant fill. These tests are listed in Incl 5. Some of the tests on natural material (N in the table) were conducted at depths of at less than 10 ft before approximately 35 ft of fill was placed over the location. Those tests are identified by the symbol B and prior to comparison with the criteria should be multiplied by a factor of about 2.3 to account for the increase in effective overburden pressure that results from the placement and future dewatering of the fill.

13 & 14 A.f of Detroit Report

6. Of the 23 tests on plant fill which fail to satisfy the criteria, most are near or under structures where remedial measures alleviating necessity for support from the fill are planned. Only 4 of the tests are under the Diesel Generator Building (which will still derive its support from the fill) and 3 others are near it. Because these locations where low blow counts were recorded are well separated from one another and are not one continuous stratum but are localized pockets of loose material, no failure mechanism is present.

7. In view of the large number of borings in the plant fill area and the conservatism adopted in my analysis, these few isolated pockets are no threat to plant safety. The fill area is safe against liquefaction in a Magnitude 6.0 earthquake or smaller which produces a peak ground surface acceleration of 0.19 g or less provided the groundwater elevation in the fill is kept at or below elevation 610.

pg. 10 of Detroit Rpt

8. In order to provide the necessary assurance of safety against liquefaction it is necessary to demonstrate the water will not rise above elevation 610 during normal operations or during a shutdown process and the applicant has decided to accomplish this by pumping from wells at the site. In the event of a failure, partial failure, or degradation of the dewatering system (and its backup system) caused by the earthquake or any other event such as equipment breakdown, the water levels will begin to rise. Depending on the answer to Question A below concerning the normal operating water levels in the immediate vicinity of Category I structures and pipelines founded as plant fill, different amounts of time are available to accomplish repair or shutdown.

9. In response to Question 24 the applicant states "the operating groundwater level will be approximately el 595 ft" (page 24-1). On page 24-1 the applicant also states "Therefore el 610' is to be used in the designs of the dewatering system as the maximum permissible groundwater level elevation under SSE conditions." On page 24-15 it is stated that "The wells will fully penetrate the backfill sands and underlying natural sands in this area." The bottom of the natural sands is indicated to vary from elevation 605 to 580 within the plant fill area according to Figure 24-12. Question A, B, and C, which I would like posed to the applicant are as follows:

A. Is the normal operating dewatering plan to (1) pump such that the water level in the wells being pumped is held at or below elevation 595 or (2) to pump as necessary to hold the water levels in all observation wells near Category I Structures and Category I Pipelines supported on plant fill at or below elevation 595, (3) to pump as necessary to hold water levels in the wells mentioned in (2) above at or below elevation 610, or (4) something else? If it is something else, what is it?

B. In the event the water levels in observation wells near Category I structures or pipelines supported on plant fill exceed those for normal operating conditions as defined by your answer to Question A, what action will be taken? In the event that the water level in any of these observation wells exceeds elevation 610 what action will be taken?

C. Where are and/or where will be the observation wells in the plant fill area that will be monitored during the plant lifetime? At what depths will the screened intervals be? Will the combination of (1) screened interval in cohesionless soil and (2) demonstration of timely response to changes in cooling pond level prior to drawdown be made a condition for selecting the observation wells? Under what conditions will the alarm mentioned on page 24-20 be triggered? What will be the response to the alarm?

10. A worst case test of the completed permanent dewatering and groundwater level monitoring systems could be conducted to determine whether or not the time required to accomplish shutdown and cooling is available. This could be done by shutting off the entire dewatering system when the cooling pond is at elevation 627 and determining the water level versus time curve for each observation well. The test should be continued until the water level in any well reaches elevation 610 or the sum of the time intervals allotted for repair and the time interval needed to accomplish shutdown (should the repair prove unsuccessful) has been exceeded, whichever occurs first. In view of the heterogeneity of the fill, the likely variation of its permeability and the necessity of making several assumptions in the analysis which was presented in the applicant's response to Question 24a, a full-scale test should give more reliable information on the available time. Question D is as follows:

D. If a dewatering system failure or degradation occurs, in order to assure that plant is shutdown by the time water level reaches elevation 610, it is necessary to initiate shutdown earlier. In

Pg. 12 of Detroit Rpt.

event of failure of dewatering system, what is the water level or condition at which shutdown will be initiated? How is that condition determined? An acceptable method would be a full-scale worst-case test performed by shutting off the entire dewatering system with the cooling pond at elevation 627 to determine, at each Category I structure deriving support from plant fill, the water level at which a sufficient time window still remains to accomplish shutdown before the water rises to elevation 610. In establishing the groundwater level or condition that will trigger shutdown, it is necessary to account for normal surface water inflow as well as groundwater recharge and to assume that any additional action taken to repair the dewatering system, beyond the point in time when the trigger condition is first reached, is unsuccessful.

<u>Comments regarding seismically induced settlements</u>

Not presented in Detroit Report

11. An independent approximate analysis based on the same references cited on pages 4-5 of the answer to Question 4 given in "Responses to NRC Requests Regarding Plant Fill," the same assumption of dry sand used in the preparation of Table 4-1A of Question 4 and my engineering judgment indicated that the numbers for seismically induced settlement in that table which are for 0.12 g and M = 7 earthquake are also reasonable for 0.19 g and a Magnitude 6 event. However, Seed and Silver (Reference 1 on pages 4-5) claim the limited field check data for the method only confirms its accuracy ±50 percent. Thus, one has to either argue that the capillary action in those sands above the water table would inhibit settlements and thus provide the degree of conservatism needed to overcome the uncertainty about the accuracy of the prediction (as did the applicant in his response to Question 4) or allow for another 1/4 in. of settlement. While this latter course of action is probably available to the applicant at no cost, it is, in my opinion, unneccessary. In view of the field data discussed in the references cited on pages 4-5 of the applicant's answer to Question 4, I am fully satisfied that capillary action does provide all the conservatism needed to view the seismically induced settlements in Table 4-1A as upper bound values for the earthquake shaking described above. Should we ask CPCo whether involved structures can tolerate 1/4" settlement under seismic loading

<u>Comments regarding the natural slopes containing the R/C pipe service water return lines</u>

p. 9 of Detroit Report

12. The two reinforced concrete return pipes which exit the service water structure and run along either side of the emergency cooling water reservoir and ultimately enter into the reservoir are necessary for the safe shutdown and are buried within or near the crest of Category I slopes that form the sides of the Emergency Cooling Water Reservoir. The reviewer has been unable to find any report on or analysis of the seismic stability or calculation of postearthquake residual displacement for these slopes. While the limited data from this area do not raise the specter of any problem, for an important element of the plant such as this, the earthquake stability should be examined by state-of-the-art methods. Therefore, Question E is as follows:

E. Have seismic analyses of the slopes leading to an estimate of the permanent deformation of the pipes been performed and if so, please provide a review copy. If none are available, please provide analyses to include the following: (1) a plan showing the pipe location with respect to other nearby structures, the slopes of the reservoir and the coordinate system; (2) cross-sections showing the pipes, normal pool levels, the slopes, the subsurface conditions as interpreted from borings and/or logs of excavations at (a) a location parallel to and about 50 ft from the southeast outside wall of the service water pipe structure and (b) a location where the cross section will include both discharge structures. Actual boring logs should be shown on the profiles, their offset from the profile noted, and soils should be described using the Unified Soil Classification System; (3) discussion of available shear strength data and choice of strengths used in stability analysis; (4) determination of static factor of safety, critical earthquake acceleration, and location of critical circle; (5) calculation of residual movement by the method presented by Newmark (1965) or Makdisi and Seed (1978); and (6) a determination of whether or not the pipes can function properly after such movements.

Comments regarding the service water structure foundation

13. The vertical pile support proposed for the overhang section of the service water pump structure will provide the support necessary for the structure under combined static and seismic inertial loadings even if the soil under the overhang portion of the structure should liquefy, provided proposed 100 ton ultimate pile load capacities are achieved. I have no reason to think they won't be achieved at this time, and the applicant has committed to a field loading test to demonstrate the pile capacity. Calculations were made by the writer to determine the critical buckling load for the 14 in. outside diam concrete filled steel pipe piles assuming them to be laterally unsupported over lengths of 40 and 50 ft with all reasonable assumptions of end fixity and a 3/8-in. pipe thickness. The worst combination of parameters ~~still~~ provides a generous factor of safety against buckling under the proposed ultimate load. Hence, even if the fill material underneath the overhang should liquefy and fail to provide lateral support to the piles, they should be capable of carrying the vertical static and inertial loads anticipated. Fully adequate lateral support is provided by structural connection of the overhang to the rest of the structure. However, the dynamic response of the structure, including the inertial loads for which the structure itself is designed and the mechanical equipment contained therein, would change as a result of the introduction of the piles. Therefore, Question F is as follows:

F(a). Please summarize or provide copies of reports on the dynamic analyses of the structure in its old and proposed configuration if such are available. For the latter provide detailed information on the stiffness assigned to the piles and the way in which the stiffnesses were obtained and show the largest change in interior floor vertical response spectra resulting from the proposed modification. If the proposed configuration has not yet been analyzed, describe the analyses that are to be performed giving particular attention to the basis for calculation or selection of and the range of numerical stiffness values assigned to the vertical piles.

Detroit Report pg. 5

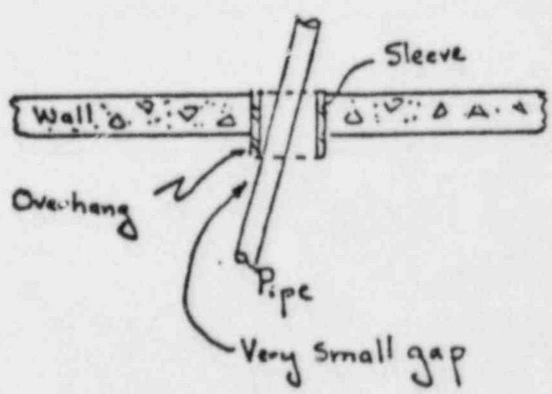
F(b). Provide after completion of the new pile foundation, in accordance with commitment No. 6, item 125, Consumers Power Company memorandum dated 13 March 1980, the results of measurements of vertical applied load and absolute pile head vertical deformation which will be made when the structural load is jacked on the piles so that the pile stiffness can be determined and compared to that used in the dynamic analysis.

Comments regarding rattlespace at Category I pipe penetrations of structure walls

14. During the site visit the writer observed three instances of what appeared to be degradation of rattlespace at penetrations of Category I piping through concrete walls as follows:

a. West borated water storage tank - in the valve pit attached to the base of the structure, a large diameter steel pipe extended through a steel sleeve placed in the wall. Because the sleeve was not cut flush with the wall, clearance between the sleeve and the pipe was very small.

Detroit Report pg. 8



b & c. Two of the service water pipes penetrating the northwest wall of the service water structure had settled differentially with respect to the structure and were resting on slightly squashed short pieces of 2 x 4 placed in the bottom of the penetration. From the inclination of the pipe, there is a suggestion that the portions of the pipe further back in the wall opening (which I could not see) were actually bearing on the invert of the opening. The bottom surface of one of the steel pipes had small surface irregularities around the edges of the area in contact with the 2 x 4. Whether these irregularities are normal manufacturing irregularities or the result of concentration of load on this temporary support caused by the settlement of the fill, I have no way of knowing.

These instances are, in my view, sufficient to warrant an examination of those penetrations where Category I pipe derives support from plant fill on one or both sides of a penetration. Therefore, Questions G and H are as follows:

G. What is the minimum seismic rattlespace required between a Category I pipe and the sleeve through which it penetrates a wall?

H. Identify all those locations where a Category I pipe deriving support from plant fill penetrates an exterior concrete wall. Determine and report the vertical and horizontal rattlespace presently available and the minimum required at each location and describe remedial actions planned as a result of conditions uncovered in the inspection.

It is anticipated that the answer to Question H can be obtained without any significant additional excavation. If this is not the case, the decision regarding the necessity to obtain information at those locations requiring major excavation should be deferred until the data from the other locations have been examined.

<u>Comments regarding foundation material properties used in seismic analysis of structures</u>

15. Inclosure 6 shows a summary of cross-hole shear wave velocity (V_s) and load test data from which it can be seen that the V_s for the plant fill is between 500 and 1000 ft/sec. From Section 3.7.2.4 of the FSAR it can be calculated that an average V_s of about 1350 ft/sec was used in the original dynamic soil structure interaction analyses of the Category I structures. This is confirmed by one of the viewgraphs used in the 28 February Bechtel presentation. Plant fill V_s is clearly much lower than this value as indicated in Incl 6. It is understood from the response to Question 13 concerning plant fill that the analyses of several Category I structures are underway using a lower bound average V_s = 500 ft/sec for sections supported on plant fill and that floor response spectra and design forces will be taken as the most severe of those from the new and old analyses. The questions which follow are intended to make certain if this is the case and gain an understanding of the impact of this parametric variation in foundation conditions. Questions I, J, and K are as follows:

I. What Category I structures have and/or will be reanalyzed for changes in seismic soil structure interaction due to the change in plant fill stiffness from that envisioned in the original design? Have any Category I structures deriving support from plant fill been excluded from reanalysis? On what basis?

J. Tabulate for each old analysis and each reanalysis, the foundation parameters (V_s, ν and ρ) used and the equivalent spring and damping constants derived therefrom so the reviewer can gain an appreciation of the extent of parametric variation performed.

K. Is it the intent to analyze the adequacy of the structures and their contents based upon the envelope of the results of the old and new analyses? For each structure analyzed, please show on the same plot the old, new, and revised enveloping floor response spectra so the effect of the changed backfill on interior response spectra predicted by the various models can be readily seen.

17.15

Category I retaining wall near the southeast of the service water pump structure

16. This wall is experiencing some differential settlement. Boring information in Figure 24-2 (Question 24, Volume 1 Responses to NRC Requests Regarding Plant Fill) suggests the wall is founded on natural soils and backfilled with plant fill on the land side. Questions L, M. and N are as follows:

L. Is there any plant fill underneath the wall? What additional data beyond that shown in Figure 24-2 support your answer?

M. Have or should the design seismic loads (FSAR Figure 2.5-45) be changed as a result of the changed backfill conditions?

N. Have or should dynamic water loadings in the reservoir be considered in the seismic design of this wall? Please explain the basis of your answer.

Status of review of geotechnical earthquake considerations

17. When formal or informal answers to the questions posed above are available from the applicant, this reviewer can quickly come to conclusions on all geotechnical considerations which influence safety under earthquake excitation. It would be desirable but not mandatory to witness the service water pump structure pile load test and the jacking of that building's load onto the completed piles.

6 Incl
as

Paul F. Hadala
P. F. HADALA
Engineer
Acting Assistant Chief,
Geotechnical Laboratory

CF w/incl:
Mr. Neil Gehring, Detroit Dist
Dr. Lyman Heller/Mr. Joe Kane, NRC
Mr. Jim Simpson, North Central Div

Bechtel
NRR/DPM
NSWC
Bechtel
Bechtel - Geotech
Bechtel - DIGR
Bechtel - SF
Bechtel - SF
Bechtel - SF
..s, Jr. — Bechtel - SF
..DNI — NRC/DSS, Geotech. Engr.
H. Burke — BECHTEL - ENGG.
FERRIS — Civil-Struct
D. Kane — "
..HAR — NRC - Geotech
..otz — Corps Engrs Detroit
Heller — " NCD Chicago
..c Otto — " WES
W. Simpson — Corps of Engineers North Cen. Div.
F. Hadala — Corps of Eng Chicago IL
F. Norton — ETEC
Brammer — NRC/DSS, HEB
..cci — ETEC
Paul Chen — Corps of Engineers, Detroit Dist
A. Gehring — "
Grundstrom — "
..ald Erickson — "
..iru R. Thiruvengadam — Consumers Power
L. H. Curtis — Bechtel
John Rutgers — Bechtel
D. E. Horn — Consumers Power P, E&C QA
William Lawhead — C of E, Detroit
Gil Keeley — Consumers Power Proj.
TC Cooke — "

5.5 @ 24 Km 0.12 to 0.19 g

10AM

write communication via telecopier for COE work

Joe Kane is tech. monitor for COE work
two phases (1) are you satisfied with the fix
(2) are you satisfied overall???

start 8AM
Guides Hydr
Mech
Str — less than ½ day
Geot.

Encl 1

Meeting 2/27/80

NRC / CPCo / Bechtel
Bechtel Consultants / US Corp of Eng
E. TEC / US Navy Weapons Center

B.C. McConnel	Bechtel - Ann Arbor
Ray Gonzales	NRC
Gene Gallagher	NRC RIII I:E
Frank Rinaldi	NRC NRR / DSS / SEB
John P. Matra Jr	NSWC

On 2/28/80
Drs Ralph B. Peck, A.J. Hendron Jr., Tom Davisson & Chuck Gould, Consultants to Bechtel were also present

MEETING WITH NRC ON MIDLAND PLANT FILL STATUS AND RESOLUTION

February 27 & 28, 1980
Midland Site

1.0 INTRODUCTION — G. Keeley (CI

2.0 PRESENT STATUS OF SITE INVESTIGATIONS — T. Cooke (CI

2.1 Meetings with Consultants and Options Discussed (Historical)

2.2 Investigative Program

A. Boring Program
B. Test Pits
C. Crack Monitoring and Strain Gauges
D. Utilities

2.3 Settlement

A. Area Noted
B. Preload
C. Instrumentation

3.0 WORK ACTIVITY UPDATE — J. Wanzeck

3.1 Summary of work activities and settlement surveys for all Category I structures and facilities founded partially or totally on fill

4.0 REMEDIAL WORK IN PROGRESS OR PLANNED (Q4, 12, 27, 31, 33 & 35) — S. Afifi
[illegible] [illegible] Bechtel

4.1 Diesel Generator Structures
4.2 Service Water Pump Structures
4.3 Tank Farm
4.4 Diesel Oil Tanks
4.5 Underground Facilities
4.6 Auxiliary Building and FW Isolation Valve Pits
4.7 Liquefaction Potential

5.0 part of EVALUATION OF PIPING (Q16, 17, 18, 19 & 20) - piping stress analysis — D. Riat
[illegible] pipes were evaluated - all deflections used in these analyses - σ = 14 to 17 ksi
ASME [illegible] 35 ksi

6.0 DEWATERING (Q24) — Bill Paris

7.0 ANALYTICAL INVESTIGATION — B. Dhar

7.1 Structural Investigation (Q14, 26, 28, 29, 30 & 34)
7.2 Seismic Analysis (Q25) Charlie McConnel
7.3 Structural Adequacy with Respect to PSAR, FSAR, etc.

8.0 SITE TOUR — All

9.0 CONSULTANTS SUMMARY (Here in the am) — Peck/Hendron, Gould/Daviss

10.0 DISCUSSION — All

Incl 2

ATTENDEES

sumc ower

S. ..eley
C. Cooke
Thiruvengadam

Order stopped all work on
tracks for fixes — on hold

. to question 24 → 35

RC

.. Heller
. Jackson
J. Kane
T. Cappucci → E-TEC
F. Rinaldi → NSWC
R. Gonzalis
F. Schauer
D — NRC Site Rep.
G. agher
R. ook

US Navy Weapons Center ✓

P. Huany
J. Matra

Bechtel

Harris Burke
Sherif Afifi
Don Riat
Bimal Dhar
Bill Paris
Julius Rotc
Jim Wanzeck
Karl Wiedner
John Rutgers
Lynn Curtis
Al Boos
Chuck McConnel

US Corp Of Engineers

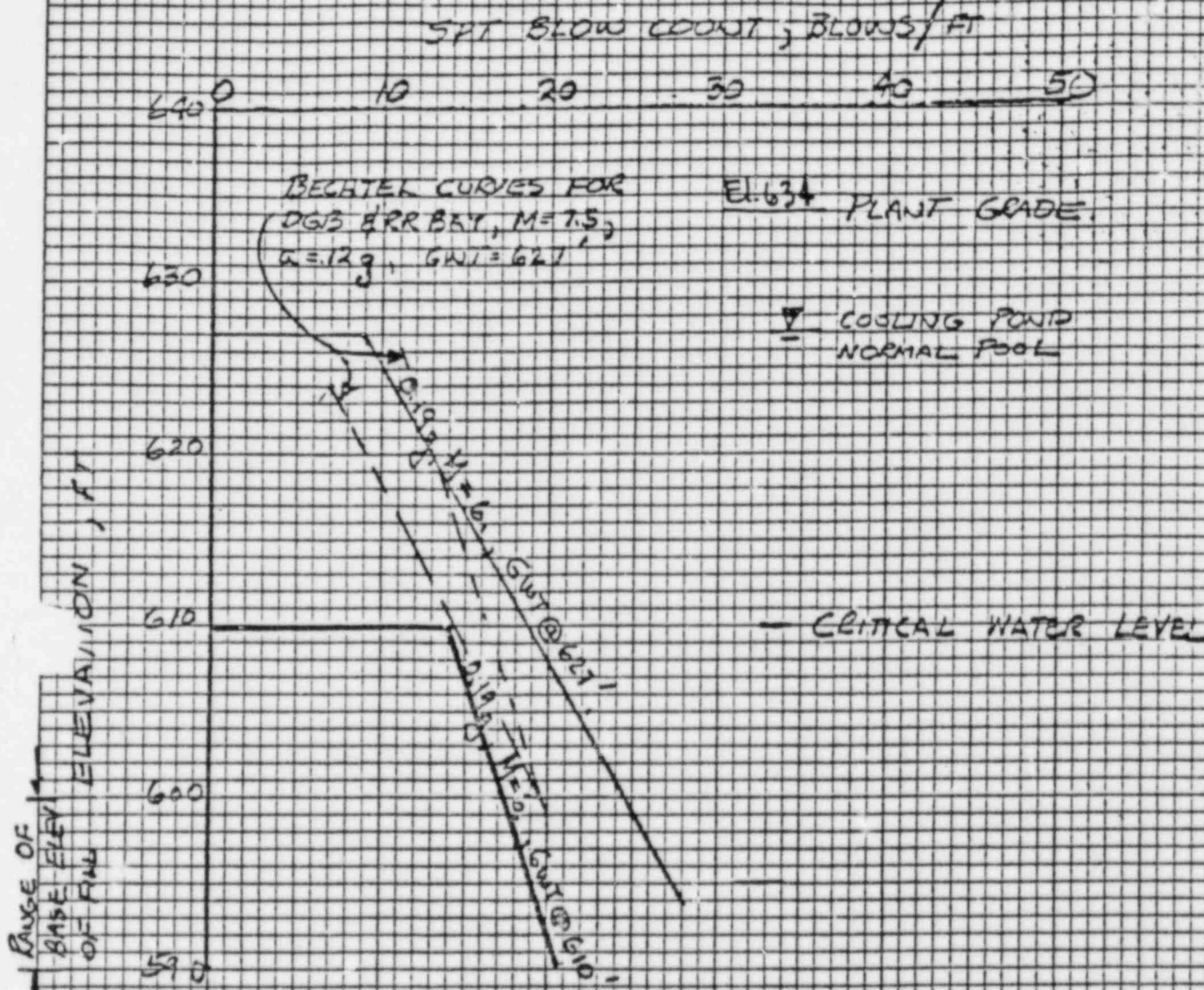
N. Gehring
J. Grundstrom
B. Otto
W. Lawhead
P. Hadala
J. Simpson

Consultants

R. B. Peck
A. J. Hendron, Jr.
C. H. Gould
M. T. Davisson

E-TEC

P. Chen
J. Brammer



SPT REQUIRED FOR ASSURANCE OF F.S =1.5 AGAINST LIQUEFACTION OF COHESIONLESS SOILS BY SEED-IDRISS SIMPLIFIED PROCEDURE

Encl 3

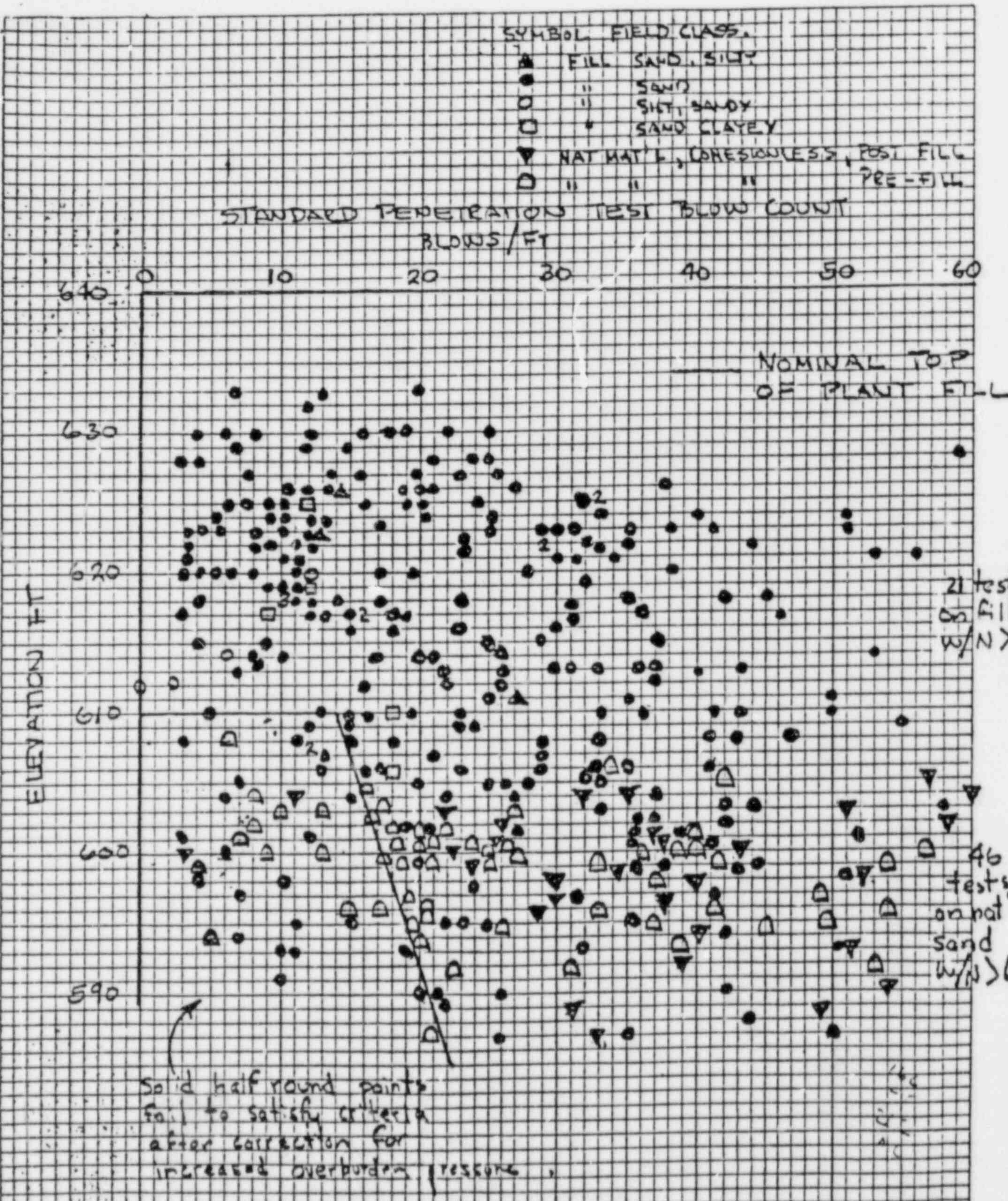


FIGURE BLOW COUNT VS ELEVATION FOR STD. PENETRATION TESTS IN SANDY SOILS IN THE PLANT FILL AREA

Dml4

Summary of "Low" Blow Counts in Cohesionless Soils Below Elev. 610

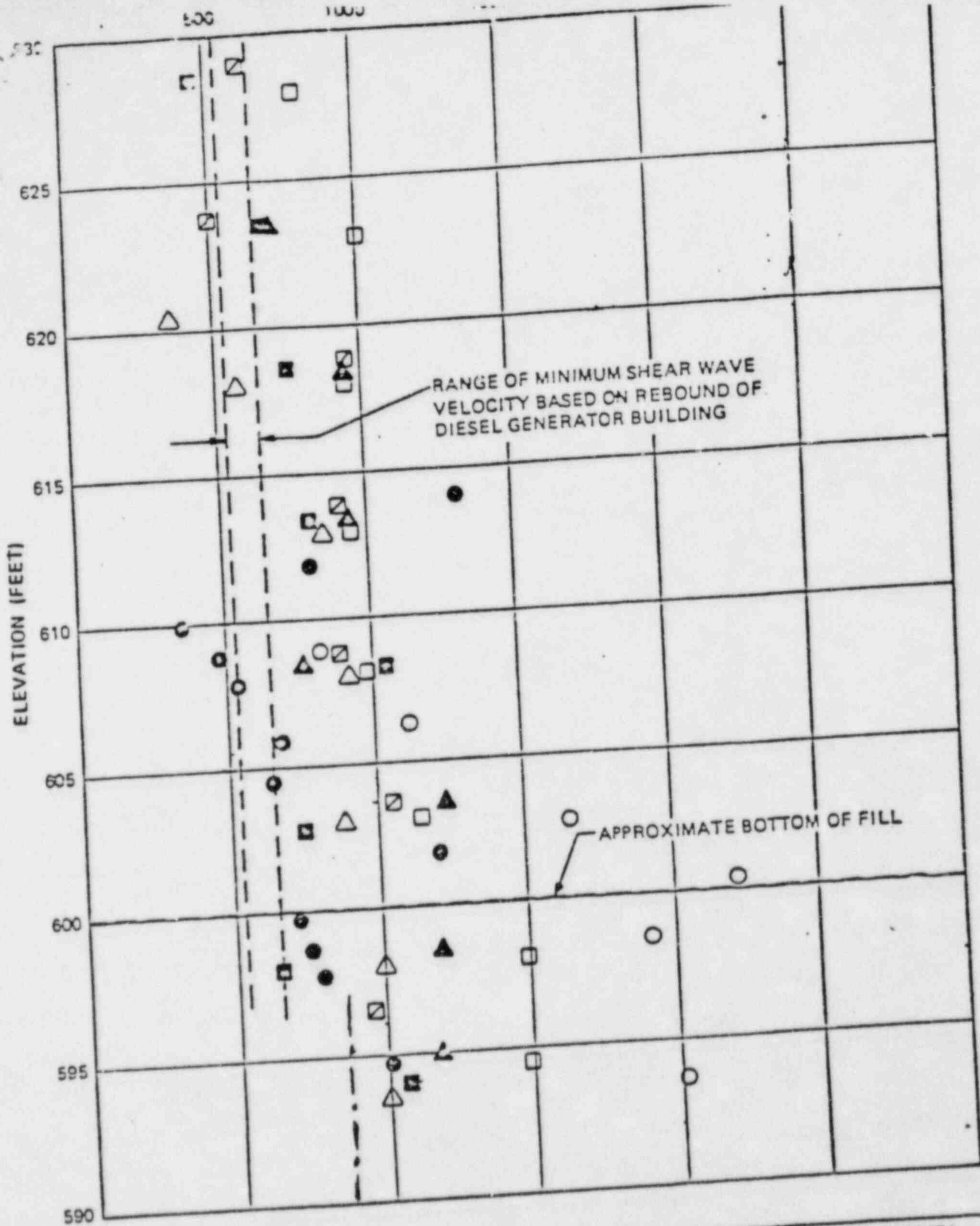
Boring	Elev	N Value Blows/ft	Location	Cat. I	Fill or Nat'l	Remarks
SW3	608	11	Service Water Pump Storage	Yes	Fill	Pile support planned
SW2	608	11	Service Water Pump Storage	Y	F	Pile support planned
DG18	609	12	Under Diesel Gen. Bldg.	Y	F	
DG18	607	13	Under Diesel Gen. Bldg.	Y	F	
AX13	597	7	N.E. of Unit 2	No	F	
AX13	591	10	N.E. of Unit 2	N	F	
AX4	601	12	Between Unit 2 & Turbine Bldg.	Y	F	Underpinning planned
AX4	593	19	Between Unit 2 & Turbine Bldg.	Y	F	Underpinning planned
AX15	595	11	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX15	593	11	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX7	605	7	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX7	594	7	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX7	590	20	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX5	601	3	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX5	598	4	Between Unit 1 & Turbine Bldg.	Y	F	Removal & repl w/conc
AX11	606	13	Under Unit 1 Valve Pit	Y	F	Underpinning planned
AX11	600	6	Under Unit 1 Valve Pit	Y	F	Underpinning planned
AX11	593	10	Under Unit 1 Valve Pit	Y	F	Underpinning planned

Summary of "Low" Blow Counts in Cohesionless Soils Below Elev. 610 (Continued)

Boring	Elev	N Value Blows/ft	Location	Cat. I	Fill or Nat'l	Remarks
DG19	608	3	Under Diesel Gen. Bldg.	Y	F	
DG13	604	6	Under Diesel Gen. Bldg.	Y	F	
DG7	598	10	E. of Diesel Gen. Bldg.	N	F	
DG7	595	15	E. of Diesel Gen. Bldg.	N	F	
DG5	604	15	S. of Diesel Gen. Bldg.	N	F	
SW6	600	3	Service Water Pump Storage	Y	N-B	Pile support planned
D42	587	21	Under Diesel Gen. Bldg.	Y	N-A	Ok when corrected
5	608	6	N. Part of Turbine Bldg.	N	N-B	Ok when corrected
5	604	7	N. Part of Turbine Bldg.	N	N-B	Ok when corrected
D21	594	5	E. Side of Turbine Bldg.	N	N-B	
17	603	13	S. Part of Turbine Bldg.	N	N-B	Ok when corrected
CT1	604	11	N. Condensate Storage Tank	Y	N-A	
355	601	7	NW of Intake Storage	N	N-B	Ok when corrected
DG28	600	9	Between Diesel Gen. & Turbine Bldgs.	Y	N-B	Ok when corrected
22	603	10	N. of Borated Water Storage	N	N-B	Ok when corrected
21	602	8	NW of Borated Water Storage	N	N-B	Ok when corrected

Summary of "Low" Blow Counts in Cohe..onless Soils Below Elev. 610 (Concluded)

Boring	Elev	N Value Blows/ft	Location	Cat. I	Fill or Nat'l	Remarks
2	599	4	N. Part of Auxiliary Bldg.	Y	N-B	
2	596	15	N. Part of Auxiliary Bldg.	Y	N-B	Ok when corrected
10	600	13	N. Part of Auxiliary Bldg.	Y	N-B	Ok when corrected
10	596	17	N. Part of Auxiliary Bldg.	Y	N-B	Ok when corrected



LEGEND:

- CONDENSATE TANKS AREA
- BORATED WATER STORAGE TANKS AREA
- SERVICE WATER PUMP STRUCTURE
- DIESEL GENERATOR BUILDING

—·•— WESTON SURVEY (FSAR 2.5.4.7.2)
PRE-CONSTRUCTION

BECHTEL ANN ARBOR	
MIDLAND POWER PLANT	
SHEAR WAVE VELOCITY PROFILE PLANT AREA FILL	
JOB NO. 7220	DRAWING NO. FIGURE 35-2

D-06

C

E + A
Hari Singh
Dep. 1-22-81

ATTACHMENT 41-1

Estimate 1) pile downdrag loads and 2) ultimate pile capacity

A. ASSUMPTIONS

1. Pile

Size and type: 14-inch (Ø) closed end pipe pile, 0.594-inch wall thickness

Driving method: Top driven, predrill required between el 634' to 600'

Pile length: Pile tip at el 580', thus pile length 47.5 feet (el 627.5' to el 580'), actual pile length may vary after the pile load test is performed

2. Soil

Downdrag load will occur only for the portion of the pile to be embedded in the fill.

Because the natural soil is heavily preconsolidated, the drained soil parameters are appropriate to use for calculating the ultimate pile capacity.

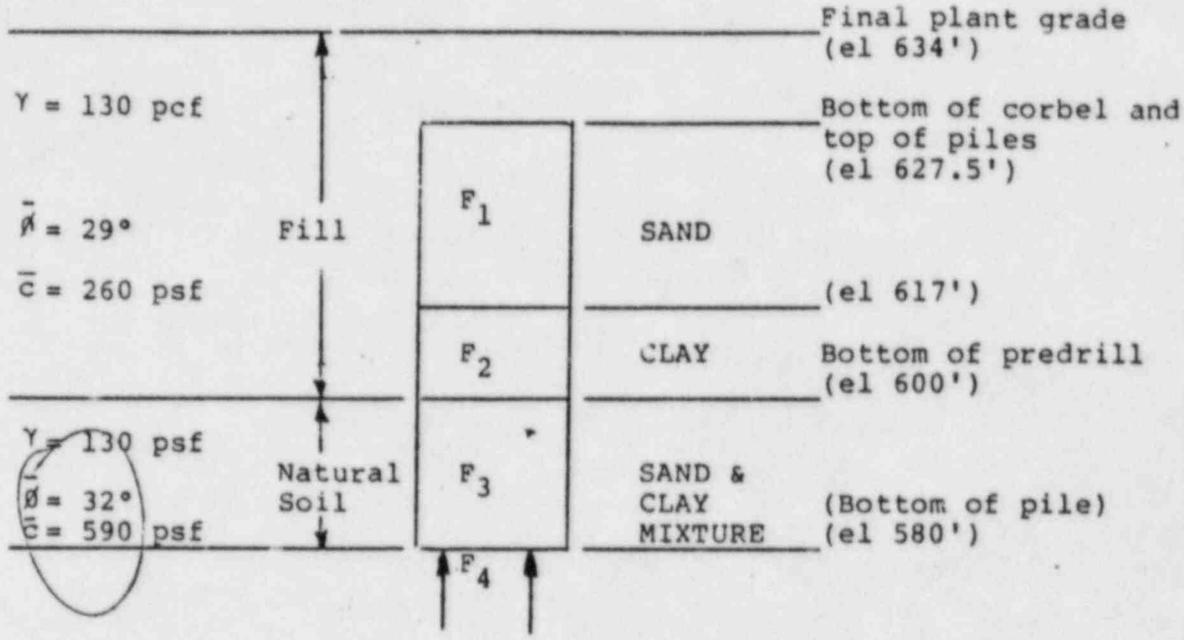
After installation of the permanent dewatering system, GWT at the northern end of the structure will be lowered to el 595'.

B. METHOD OF ANALYSIS

1. Soil profiles and parameters: Soil profiles were based on all borings made in the vicinity of the north end of the service water pump structure where the underpinning piles will be installed (see Figure 41-2). The profile is simplified as follows for analysis.

10

11-21-80



10

Definitions: F_1, F_2, and F_3 = side friction of the pile
F_4 = point resistance of the pile

Soil drained parameters for the fill were derived from the consolidated undrained triaxial (CIU) tests with pore water measurements, performed by Goldberg-Zoino-Dunnicliff. (See Figure 41-4 for $\bar{c}$-$\bar{\phi}$ plot and Volume 6 Tab 146 of Responses to NRC Requests Regarding Plant Fill for laboratory data.) Soil-drained parameters for the natural soil (approximate el 600' to 580') were obtained from consolidated, drained tests performed by Dames and Moore. (See Figure 39-4 and FSAR Appendix 2B for laboratory data and $\bar{c}$-$\bar{\phi}$ plots.)

Also, blowcounts versus elevation plots were made as shown in Figure 41-5.

2. Calculation of Side and Point Resistance (F_1, F_2 and F_3, F_4)

Formula: Side resistance of pile = $\pi DH \bar{\sigma}_h \tan \delta + \pi DHC$

where

D = outside pile diameter

H = length of pile

$\bar{\sigma}_h = K_o \bar{\sigma}_v$ effective horizontal overburden pressure at the middepth of the pile, $K_o = 0.5$

δ = friction angle between pile and soil

C = cohesion between pile and soil

a) F_1 and F_2

According to Potyondy (Geotechnique Journal September, 1961, pp 339-353) published by the Institution of Civil Engineers, London)

$\delta/\bar{\phi} = 0.65$ $c/c_{max} = 0.35$ for clayey sand

thus $\delta = 0.65 \times 29$

$C = 260 \times 0.35$

$F_1 + F_2 = \pi \times 14/12 \times 27 \times 1313 \times \tan(0.65 \times 29) + \pi \times 14/12 \times 27 \times 260 \times .35$

$= 53,365$ pounds

≈ 27 tons

Also, two alternative methods have been used: 1) $\delta = 25°$ $c = 0$, and 2) Meyerhof empirical approach to calculate F_1 and F_2. These values are 30 tons and 24 tons, respectively.

b. Calculation of F_3

Soil-drained parameters: $\bar{c} = 590$ psf and $\bar{\phi} = 32°$

$\bar{\sigma}_v$ = effective overburden pressure at middepth of layer F_3

$= 39 \times 130 + 5 \times 67.6 = 5408$ psf

$K_o = 1 - \sin\bar{\phi} = 1 - \sin 32° = 0.47$

$\bar{\sigma} = K_o \bar{\sigma}_v$ 10

According to Potyondy, the friction angle between steel and soil is (0.65 and 0.8) of $\bar{\emptyset}$.

For conservatism:

$\delta = 0.08 \times 32° = 25°$

$c = 0$

$F_3 = 2\ \pi r\ \ H\ (\bar{\sigma}_h \tan \delta\ ) + 2\pi \gamma H\ C$

$= 2\pi\ (7/12)(20)(5{,}408)(0.47)\ \tan 25$

$= 86{,}882$

≈ 44 tons

c. Calculation of F_4

In case of earthquake it should be checked on undrained condition

Soil-drained parameters:

$\bar{\emptyset} = 32°$ and $C = 590$ psf 10

$q_o = \frac{B\gamma N_\gamma}{2} + CN_c + q'N_q$ (Sowers & Sowers, page 461)

$N_\gamma = 80;\ N_q = 80;\ N_c = 130$

$q_o = 1.17(130) \times (80)/2 + 590(130) + 6{,}084(80)$

$= 590{,}624$ psf

$Q = Axq_o = \pi(7/12)^2 \times 598{,}624$

$= 639{,}931$ pounds

$= 320$ tons

SUMMARY AND CONCLUSIONS

Downdrag loads $= F_1 + F_2 \approx 30$ tons

Ultimate pile capacity $= F_1 + F_2 + F_3 + F_4$

$= 391$ tons

ATTACHMENT 41-2

Estimate the possible differential settlement between the pile supported end and the portion placed on glacial till.

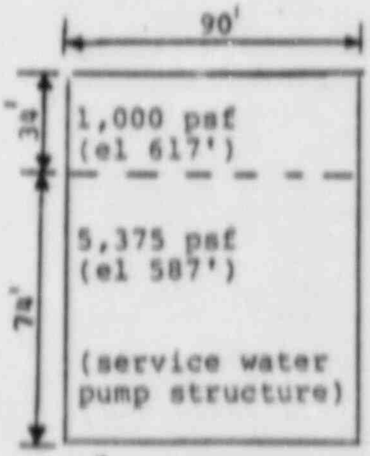
A. STRUCTURE DESIGN INPUT

1. The service water pump structure consists of two mat foundations: ona mat foundation on natural soil at el 587' (superimposed load intensity ≈5.4 ksf); the other mat foundation on fill at el 617' (superimposed load intensity ≈ 1 ksf).

2. The southern end of the structure faces the cooling pond with operating pond elevation at 627 feet with the dewatering system in service; the groundwater table at the northern end of the structure will be lowered to el 595'. 10

3. The original ground surface and GWT at the site is at el 603'. The final plant grade is el 634'.

4. The dimensions of the structure are shown below.



Plan View of Service Water Structure

B. ASSUMPTIONS

1. Loads

The estimated settlement for the lower portion mat foundation of the service water pump structure was based only upon the static plus live loads of 5,375 psf. The effects of the piles and the adjacent circulating water intake structure were neglected due to the substantial distance between the piles and the mat foundation and the low load intensity of the circulating water pump structure.

2. Soils

The natural soil where the lower portion of the service water pump structure was placed and the underpinning piles to be installed are overconsolidated and behave essentially elastic under structural loads which do not exceed the preconsolidation pressure. Preconsolidation pressure of the natural soil estimated by Dames & Moore is at least 15 to 20 ksf. — at what elevation. 10

The soil profile and parameters are tabulated below.

	Layer	Elevation (ft)	Layer Thickness (ft)	Shear Strength (s_u) ksf	Elastic Modulus (600 s_u)(ksf) 250 to 600
Foundation elevation (587)		603			
	A				
		582.5	20.5	4.0	2,400
	B				
		562.0	20.5	6.0	3,600
	C				
		543.0	19	8.0	4,800
	D				
		503.0	40	8.0	4,800
	E				
		363.0	140	8.0	4,800

3. Settlement

Dewatering settlement has been estimated to be $\approx$0.48 inch for the area below the pile tips and 0.1 inch for the portion of the service water structure supported on natural soil. These values are based on the assumption that the groundwater table below the pile tips will be at el 595' during operating conditions. The water table for the portion of the structure supported on natural soil is assumed to be at el 620' during operating conditions. It is planned to jack the piles after the dewatering settlement takes place, as discussed previously.

The time dependent settlement after pile jacking is calculated below. 10

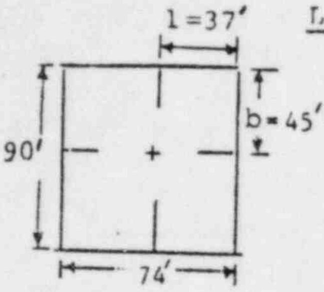
Because the natural soil is heavily preconsolidated and the added net structure load intensity will not exceed the preconsolidation pressure, it is reasonable to assume that 80 percent of the estimated ultimate settlement will occur rapidly as the loads are applied, and 20 percent of the estimated ultimate settlement will be time dependent. Therefore, the settlement from the time after pile jacking to the end of building service life can be calculated as follows:

[ultimate settlement based on deadloads + live loads and GWT at 627] x 0.2

Calculation of the structural net load intensity for GWT at 627':

$$5.375-(0.0624 \times 40)-[0.0676 \times (603-587)] = 1.82 \text{ ksf}$$

Calculate the induced stress at the center of the mat foundation (Poulos and Davis, <u>Elastics Calculations for Soil and Rock Mechanics</u>, Table 3.14, p 55).



$\frac{b}{l}$ = 1.2162

Layer	Depth from Foundation Elevation (to Midlayer)	Z/l	Stress Factor (ko)	GWT at 627 Induced Stress	Settlement /E x H
A	2.25	0.06	≈1.0	1.82	0.04095
B	14.75	0.4	≈0.964	1.7549	0.11992
C	34.5	0.94	≈0.76	1.3832	0.0657
D	64.0	1.73	≈0.445	0.8099	0.081
E	154	4.16	0.12	0.2184	0.07635
					Σ 0.384"

Thus, the estimated settlement at the center of mat foundation = 0.2 x 0.384" or 0.078".

As discussed previously, the effects from the piles and circulating water intake structure are neglected. Therefore, the above value is rounded to 0.1 inch. 10

C. PILE PORTION

The underpinning piles at the northern end of the service water pump structure will be top driven and penetrate to the natural soil. All the piles will be preloaded to a value greater than the dead plus live loads by jacking against the existing building.

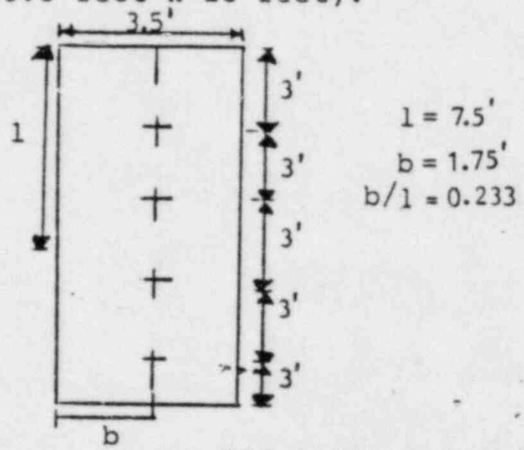
The piles will be divided into four groups as shown in Figure 41-1. For settlement analysis purposes, the following assumptions are made.

1. The settlement of each pile group is independently calculated.

2. The induced stress versus depth due to each pile group acts independently.

3. The pile tip is at el 580'.

4. The load distribution of the pile group is distributed as shown (3.5 feet x 15 feet).



5. The load intensity is $\frac{(75 + 30) \times 2 \times 4}{3.5 \times 15} = 16$ ksf

6. Because all production piles will be preloaded, 80 percent of the settlement will occur during the preload. 10

no

D. CALCULATION

1. Calculate the net load intensity.

16 ksf - (595-580) x .0624 = 15 ksf — what about load due to fill. 20 x 70 = 1,400

2. Calculate the induced stress versus depth at the center of pile group

Layer	Depth from Foundation Elevation (to Midlayer)	Z/l	Stress Factor (k_o)	$\Delta\sigma = (15 \times k_o)$	$\Delta\lambda = H \times \frac{\Delta\sigma}{E}$
A	Foundation below this layer				
B	9	1.2	0.172	2.58	0.152
C	27.5	3.67	0.032	0.48	0.02
D	57	7.6	0.008	0.12	-
E	138	18.4	0	0	-
					Σ 0.172"

Thus, the time-dependent settlement = 0.172" x 0.2 = 0.034". This is rounded to 0.05 inch.

Therefore, possible future differential settlement between the pile supported end and the mat foundation = 0.05" - 0.1" = 0.05" (foundation settles more than piles). 10

ATTACHMENT 42-3

ULTIMATE BEARING CAPACITY OF

THE CAISSONS AND CAISSON GROUP

The calculations made below are based on the preliminary design as shown in Figures 42-2 and 42-68. If alternative designs are used, the design criteria specified in Response to Question 42(1) will be met.

1. Assumptions

a) Caissons:

1. The caisson group consists of 13 caissons arranged to provide a moment equal to or greater than 325,000 foot-kips at column rows 5.3 and 7.8.

2. Each caisson will be 4 feet outside diameter. The tip of each caisson will be at least at el 576' or below. For each caisson, at least the last 4 feet of penetration into natural soil will be hand dug.

3. The caisson group occupies an area approximately 18' x 18'.

b) Soils:

The caisson will be partially embedded in fill and the caisson tip will be seated at least four feet into natural soil. 10

Downdrag loads will occur only for the portion of the caisson to be embedded in the fill.

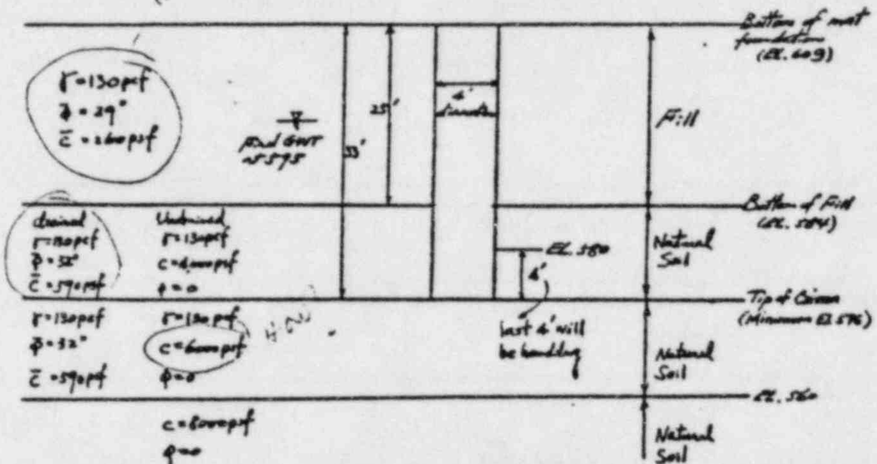
Before installation of these caissions, a construction dewatering system will be implemented to lower the groundwater.

The groundwater table during the operating condition is at el 595'.

2. Method of Analysis

a) Soil profiles and parameters: Soil profiles were based on all borings made in the vicinity of the electric penetration area where the underpinning caissons will be installed (see Figure 42-2) and simplified as follows:

How did you get it.



Soil drained parameters for the fill were derived from the CIU tests (consolidated undrained triaxial tests with porewater pressure measurements) performed by Goldberg-Ziono-Dunnicliff and Associates (GZD) (see Figure 41-21). Soil undrained parameters for the natural soil were taken from FSAR Figure 2.5-33 based on Qu and UU tests. Soil drained parameters for the natural soil (el 600' to 580') were obtained from CD tests performed by Dames & Moore (see Figure 39-1 for $\bar{C}$-$\bar{\phi}$ plots and FSAR Appendix 2B for laboratory data).

Also, blowcounts versus elevation plots were made as shown in Figures 42-70, -71, -72, and -73. 10

The bearing capacity calculations consider two aspects

a) End of construction case

1. Individual caisson

2. Caisson group

b) Operating condition during life of the plant - caisson group only

Case 1a. End of construction-individual caisson

When the first caisson is installed, it will be surrounded by backfill. The caisson tip (el 576') will be at least 4 feet into the natural soil.

Because $R = 2'$ and $D_f = 33'$

$D_f \gg R$ (consider a deep foundation)

For a deep circular footing, Terzaghi and Peck propose

$$Q_f = \pi R^2(0.6\,\gamma R N_r + 1.3C\,N_c + \gamma D_f N_q) + 2f\,\pi R D_f$$

1.2C

where

$C = 6$ ksf

$\phi = 0$

$\gamma = 130$ pcf

$N_c = 5.14$

$N_q = 1$

$N_r = 0$

f = friction force between soil and caisson

$D_f = 4$

— How? multiply factor.

$$Q_f = \pi \times 4(1.3 \times 6 \times 5.14 + .13 \times 33) + 2 \times 6 \times \pi \times 2 \times 4'$$ 10

$$= 557.8 + 301.6 = 859.4 \text{ kips}$$ — downward skin friction.

Superimposed load

$= 4,000/13 + 0.15 \times 33 \times \pi R^2$

$= 369.9$ kips

F.S $= 859.4/369.9 = 2.32$

Case 1b. End of construction-caisson group

After all the caissons have been installed and act as a group. The caisson group occupy an area of 18' x 18'; consider as a square footing

$B = 18'$ $D_f = 33'$

$D_f \gg B$ (deep foundation)

For a rectangular footing, Terzaghi & Peck propose

$$Q_f = B^2\,[0.4\,\gamma B N_r + 1.3c\,N_c + \gamma D_f N_q] + 4f\,B D_f$$

where

$c = 6$ ksf

$\phi = 0$

$N_c = 5.14$

$N_q = 1$

$N_r = 0$

$\gamma = 130$ pcf

f = friction force between soil and caisson

$D_f = 4$

$Q_f = 18 \times 18 \; [\overset{1.2}{1.3} \times 6 \times 5.14 + .13 \times 33 \times 1] + 4 \times 6 \times \overset{-B}{18} \times 4$ — deduct for negative skin friction.

$= 16{,}110.4$ kips

Superimposed load $= 4{,}000 + 18 \times 18 \times 33 \times .15$

$= 5{,}603.8$ kips

F.S. $= 16{,}110.4/5{,}603.8 = 2.87$

TC

Case 2) Operation condition during life of plant - caisson group only

Use drained soil parameters

$\bar{c} = 590$ psf $\bar{\phi} = 32°$

$$Q_f = B^2 \, (0.4 \, \gamma B N + 1.3 c \, N_c + \gamma D_f N_q) + 4f \;\; BD_f$$

where

$\bar{\phi} = 32°$

$N_c = 35.49$

$N_q = 23.18$

$N_\gamma = 30.22$

(from Vesic's table of bearing capacity factors)

$$Q_f = 18 \times 18 \times [0.4 \times (.13 - 0.0624) \times 18 \times 30.22 + 1.3 \times .59 \times 35.49 + (.13-.0624) \times 33 \times 23.18] + 4 \times .59 \times 18 \times 4$$

$$= 18 \times 18 \times [14.71 + 27.22 + 51.71] + 170$$

$$= 30{,}509.36 \text{ kips}$$

Imposed loads on caissons

1. Structural load 4,000 kips

2. Caisson plus soil loads .15 x 18 x 18 x 33 = 1,603.8 kips

3. Downdrag loads

   a) There will be no downdrag loads from reactor containment buildings and feedwater isolation valve pits. Because the reactor containment buildings were placed on glacial till and valve pits will be resting on concrete on top of the glacial till.

   b) To address the possibility of downdrag loads from the turbine-generator building and auxiliary building penetration rooms, the following calculation was made from Potyondy's suggested relationship, (Geotechnique, December 1961 published by the Institution of Civil Engineers, London, pages 339 through 353)

Calculate by Potyondy suggestion

$\delta/\bar{\phi} = 0.65 \quad C/C_{max} = 0.35$ 10

$\sigma_h' = 0.5\sigma_v'$

a) From auxiliary building side

For fill $\bar{\phi} = 29°$ $C_{max} = 257$ psf

$\delta = 0.65 \times 29° = 18.85°$

$C = C_{max} \times 0.35 = 89.95$ psf

Calculate $\bar{\sigma}_v$ at el 584'

$= 14 \times 130 + 11(130 - 62.4)$

$= 1{,}820 + 743.6 = 2{,}563.6$

Average effective stress

$= (14 \times 1{,}820 \times 1/2 + 11 \times 2{,}191.8)/25$

$= (12,740 + 24,109)/25 = 1,474$ psf

$\sigma'_h = 0.5 \times 25 \times \sigma'_v$

downdrag $= 0.5 \times 25 \times \sigma'_v \times \tan 18.85^\circ \times 15 + C \times B \times D$

$= 0.5 \times 25 \times 1,474 \times \tan 18.85^\circ \times 15 + 89.95 \times 15 \times 25$

$= 94,355 + 33,731.3 = 128$ kips

b) From turbine-generator building

Effective stress

$= 1,474 + 3,000 = 4,474$ psf

3,000 psf is the surcharge effect due to building loads

$\sigma'_h = 0.5 \times 25 \times \sigma'_v$

downdrag $= 0.5 \times 25 \times \sigma'_v \times \tan 18.85^\circ \times 18 + 89.95 \times 18 \times 25$

$= 343,672.2 + 40,477.5$ 10

$= 384,149.7 = 384$ kips

total downdrag $= 128 + 384 = 512$ kips

Also, downdrag loads were also calculated by the other two methods: 1) Meyerhof empirical approved and 2) by assuming that $\delta = 25°$. Their values were 598.4 and 593.3 kips respectively. It is decided to use 565.7 kips for conservatism.

$$FS = \frac{(30,509.36 - 598.4)}{4,000 + 1,603.8 + 598.4}$$

$= 4.82$

Conclusion:

The bearing capacity calculation of the caisson group for the operating condition assumes that the caisson group acts independently. In reality, the caisson group will be tied to the valve pit (FIVP) concrete block and the calculated factor of safety will be even higher.

D

R E S U M E

1-22-81

#1

NAME: JOHN M. BRAMMER

BIRTH DATE: 8/25/26

EDUCATION

University of New Mexico 1948 BSME

EXPERIENCE

Rockwell International - ETEC July 1973 - Present

Stress Analysis of Piping systems and components per the applicable ASME and ANSI Codes. Included were analysis of extensive piping runs, valves, hangers, pressure vessels, fittings, and supporting structure. Also, participate in the writing of design specifications and reviewing vendor designs and analysis.

Rockwell International - B-1 Division Feb. 1971 - July 1973

Lead engineer responsible for loads and stress analysis of all company designed components and auxiliary components of the B-1 main and nose landing gears, and the review of loads and stress analysis reports of vendor designed components.

Rockwell International - Atomics International Sept. 1969 - Feb. 1971

Lead engineer of a study to determine the post impact configuration of a SNAP reactor after impacting the earth at the conclusion of its life in space. Included a analytical study, setting up and conducting a test program to verify analytical study, and evaluation of test results.

Rockwell International - Rocketdyne Aug. 1965 - Sept. 1969

Stress and load analysis of cryogenic and hot gas valves and control devices used on rocket engines.

Rockwell International - Atomics International May 1964 - Aug. 1965

Responsible engineer for development and procurement of NAK components used on SNAP 10.

Arthur D. Little Inc. Dec. 1960 - May 1964

Staff member in applied mechanics - Structural and dynamic analysis of cryogenic piping systems for hardened missile sites, and consultant to Air Force on fabrication and installation of these Systems. Design, development, fabrication, and installation of a fluid bearing test stand for a large rocket engine. In charge of field installation.

Douglas Aircraft Oct. 1954 - Dec. 1960

Structures Engineer - Stress and load analysis of aircraft components.

Sandia Corporation Aug. 1948 - Oct. 1954

Project engineer responsible for design, analysis, development, procure-

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1-22-81 [illegible]

ment and final evaluation of atomic weapon mechanical components including ballistic cases, seals, quick disconnects, fusing and firing components and handling equipment.

ETEC

#2

JB 1-16-81
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COMPARISON OF THE BECHTEL AND ETEC STRESS ANALYSIS REGARDING THE SETTLEMENT STRESSES OF THE UNDERGROUND PIPING

IN AN ATTEMPT TO RESOLVE THE DIFFERENCES BETWEEN THE BECHTEL AND ETEC STRESS ANALYSIS RESULTS THREE OF THE PROFILED LINES WERE ANALYZED BY ETEC USING THE GEOMETRY AND DEFLECTIONS FROM THE BECHTEL STRESS REPORT. THE RESULTS OF THIS ANALYSIS INDICATE THAT THE METHODS OF ANALYSIS ARE SIMULAR. THE STRESSES AT THE ONE POINT GIVEN BY BECHTEL AND THE ETEC STRESS AT THAT POINT AGREE FAIRLY WELL, AS BECHTEL INCLUDED ONLY THE MAXIMUM STRESS IN THE LINE IN THEIR REPORT THERE WAS ONLY ONE POINT FOR COMPARISON.

THE MAIN POINTS OF DIFFERENCE APPEAR TO BE

1) THE NUMBER OF DEFLECTION INPUT POINTS REQUIRED TO DEFINE THE STRESSES IN THE LINE

2) THE CONDITIONS OF THE ENDS OF THE LINES AS TO RATHER THEY SHOULD BE CONSIDERED SIMPLY SUPPORTED, FIXED, OR SOMEWHERE IN BETWEEN.

3) THE RELIABILITY OF THE MEASURED DEFLECTIONS. REVISION 10 OF FIG 17.2 & 19.1 DIFFERS CONSIDERABLY FOR SOME OF THE PROFILED LINES OVER PREVIOUS INFORMATION.

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THE ETEC ANALYSIS IS NOT A RIGID ANALYSIS BY ANY MEANS BUT WAS DONE TO SEE IF WE ARE IN THE SAME BALL PARK. IT WAS DONE BASED ON OUR INTERPRETATION OF THE LINE CONFIGURATION FROM THE BECHTEL REPORT, ASSUMED BEND RADII, ETC., AND IN SOME CASES SCALED DEFLECTION DATA. IN SOME AREAS THE GEOMETRY DID NOT SEEM COMPATIBLE SO ASSUMPTIONS WERE MADE. ALL THESE COULD CONTRIBUTE TO SOME DIFFERENCES IN THE STRESSES.

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SUMMARY TABLE

SETTLEMENT STRESSES OF PROFILED SYSTEMS

LINE	SIMPLY SUPPORTED LINES				FIXED ENDS			
	BECHTEL RPT. σ P.S.I.	EQUIV. ETEC CONF σ PSI	DEFL ① 20' INTV. σ PSI	MARCH/APRIL 79 DEFL. σ PSI	BECHTEL RPT DEFL σ PSI	DEFL ① 20' INTV. σ PSI	MARCH/APRIL 79 DEFL σ PSI	
26"OHBC-54	21665	21648 260852	31176 266515	* 111355	21688 280840	31251 284883		① ②
20"IHCD-169	29838	28170 34239	28074 52389	280685	49653 49653	53303 53303	53303 53303	① ②
26"OHBC-53	27282 37854	8307 }+ 39525 }+ 17980	23846 + 7045 + 30080	154427	28196 15493 96135	19209 6919 186243	19438 7394 186226	① ① ②

① σ @ BECHTELS LOCATION OF MAXIMUM STRESS FOR COMPARISON OF ANALYSIS METHOD

② MAXIMUM σ FROM ETEC ANALYSIS

* JULY 79 DATA THIS AREA OF LINE NOT INCLUDED IN BECTHEL REPORT

++ THE DISCREPANCY BETWEEN THE BECHTEL AND ETEC ANALYSIS IN THIS AREA MAY BE DUE TO THE FACT THAT THE CONFIGURATION IN THIS AREA WAS UNCLEAR AND NO STRESS INDICES USED

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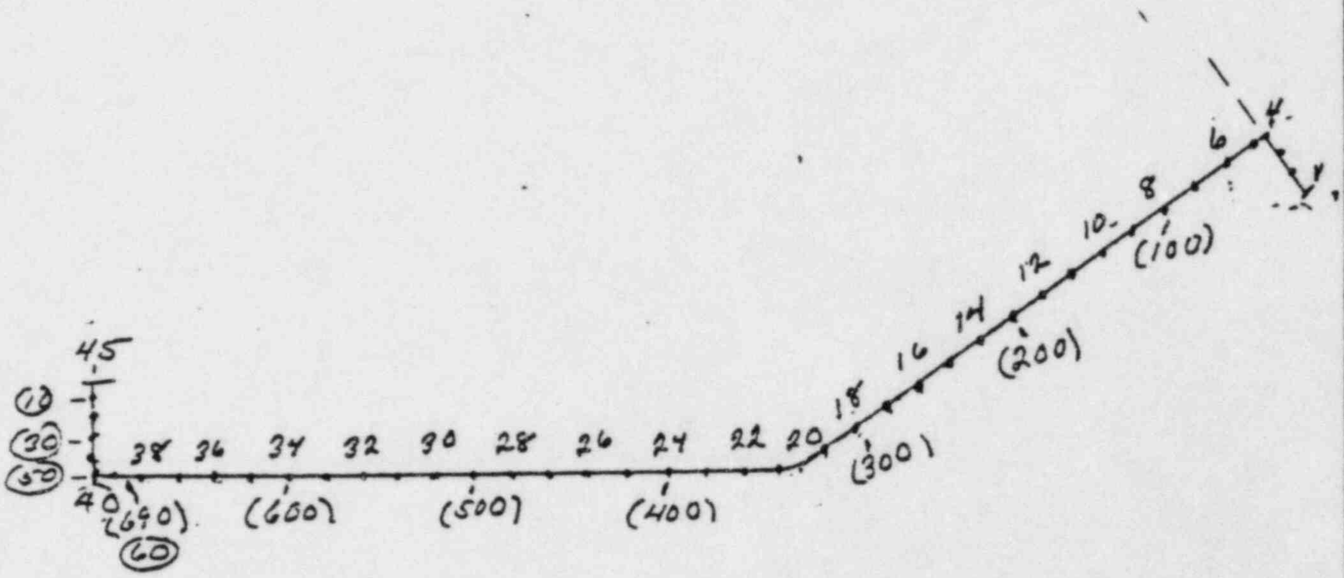
LINE NO. 26" OHBC-54

FIG 17.2 & 19.1



N NODAL POINTS

(N) DISTANCE FROM READ OUT POINT FIG. 17.2

(N) DISTANCE FROM READ OUT POINT FIG. 19.1

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LINE 26" OHBC-54
FIG 17.2 & 19.1

ANALYSIS BASED ON GEOMETRY AS INTERPRETED FROM BECHTEL STRESS REPORT.

A. CONFIGURATION AND LINE DEFLECTIONS PER BECHTEL STRESS REPORT. ENDS SIMPLY SUPPORTED.

1) MAX STRESS FROM BECHTEL REPORT 21,665 P.S.I. @ READ OUT LOCATION 400 FT.

2) ETEC ANALYSIS STRESS @ 400 FT LOCATION 21648 P.S.I. (NODE 24) IT APPEARS METHODS OF ANALYSIS ARE SIMULAR

3) ETEC MAX STRESS 260862 PSI. @ READ OUT POINT 50 FT FROM FIG 19.1 A STRESS INDEX OF 4.5 IS INCLUDED IN THIS ANALYSIS. (NODE 40) BECHTELS ANALYSIS DOES NOT COVER THIS PORTION OF THE LINE.

B. CONFIGURATION SAME AS "A" EXCEPT DEFLECTIONS INPUT EVERY 20 FT

1) STRESS @ READOUT LOCATION OF 400 FT = 31176 P.S.I. 44 % INCREASE

2) MAX σ = 266515 PSI. @ 50 FT LOCATION (FIG 19.1) NODE 40

3) MAX σ = 11355 PSI @ 50 FT LOCATION USING DEFLECTION @ 50 FT LOCATION FROM SEPT. 1979 DATA.

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LINE 26" OHBC-54 (CONT)

C. SAME AS A EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

1) σ = 21688 P.S.I. @ LOCATION 400' Fig 17.2
σ = 280840 P.S.I @ LOCATION 50' Fig 19.1
σ = 47,300 P.S.I. @ LOCATION 0' Fig 17.2
σ = 181,214 P.S.I. @ LOCATION 0' Fig 19.1

D. SAME AS B EXCEPT THE ENDS ARE ASSUMED TO BE FIXED.

1) σ = 31251 P.S.I. @ LOCATION 400' FIG 17.2
σ = 284863 P.S.I @ LOCATION 50' FIG 19.1
σ = 85,251 P.S.I @ LOCATION 0 FIG 17.2
σ = 181,228 P.S.I @ LOCATION 0 FIG 19.1

IN ALL THE ABOVE CASES THE DEFLECTIONS FROM FIG. 19.1 NOT COVERED BY THE BECHTEL REPORT ARE SCALED VALUES AND WERE INPUT @ 10 FT INTERVALS.

BECHTEL SEEMS TO HAVE MADE AN ERROR IN SIGN @ THE 20 FT. POINT.

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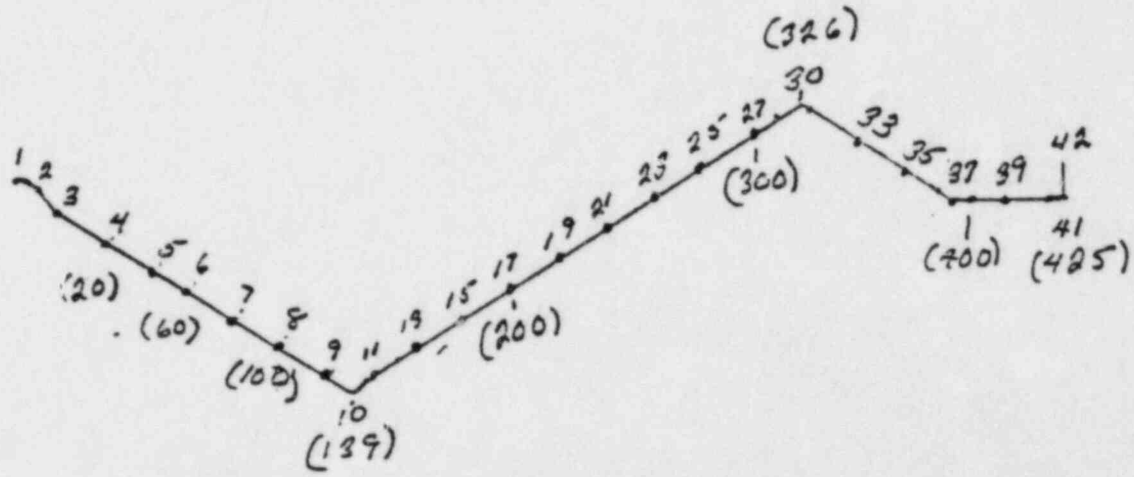
LINE NO. 20 IHCP-169



N Node

(N) DISTANCE FROM READOUT POINT

LINE 20 IN. IHCD-169

FIG 17.2

ANALYSIS BASED ON GEOMETRY AS INTERPRETED FROM THE BECHTEL STRESS ANALYSIS REPORT.

A. CONFIGURATION AND LINE DEFLECTIONS PER BECHTEL STRESS REPORT.
ENDS SIMPLY SUPPORTED

1) MAXIMUM STRESS FROM BECHTEL REPORT
σ = 29838 PSI @ READ OUT LOCATION 4.9'

2) ETEC ANALYSIS STRESS @ 4.9 FT
σ = 28170 P.S.I

3) ETEC MAX σ = 34239 P.S.I NODE 41 @ LOCATION 425 FT. A 1½ D ELBOW WAS ASSUMED HERE WITH A STRESS INDEX OF 3.766

B. CONFIGURATION SAME AS A EXCEPT DEFLECTIONS INPUT EVERY 20 FT

1) σ = 28074 P.S.I. @ 4.9 FT.

2) σ_{max} = 52389 P.S.I. @ 425 FT.

C. CONFIGURATION SAME AS B EXCEPT THE DEFORMATIONS GIVEN FOR MARCH/APRIL 1979 WERE USED and WERE INPUT @ 10 FT INTERVALS

1) σ_{max} = 280695 P.S.I. @ 310 FT

NODE 28

ETEC

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LINE 20 IN. IHCD (CONT)

D SAME AS A EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED.

1) σ = 49653 P.S.I. @ ≅ 4.9

2) σ_{max} = 49653 P.S.I @ ≅ 4.9

3) σ = 19161 @ 0'

4 σ = 2964 @ 435'

E SAME AS B EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

1) σ = 53303 P.S.I. @ ≅ 4.9

2) σ_{max} = 53303 P.S.I @ ≅ 4.9

3) σ = 20967 PSI @ 0'

4) σ = 5543 PSI @ 435'

F SAME AS C EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

1) σ = 53303 P.S.I. @ ≅ 4.9

2) σ_{max} = 280712 P.S.I. @ 310 FT. (Node 28)

3) σ = 20976 P.S.I. @ 0'

4) σ = 1652 P.S.I. @ 435'

FORM 735-A-54 REV 12-78

ETEC

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DATE:

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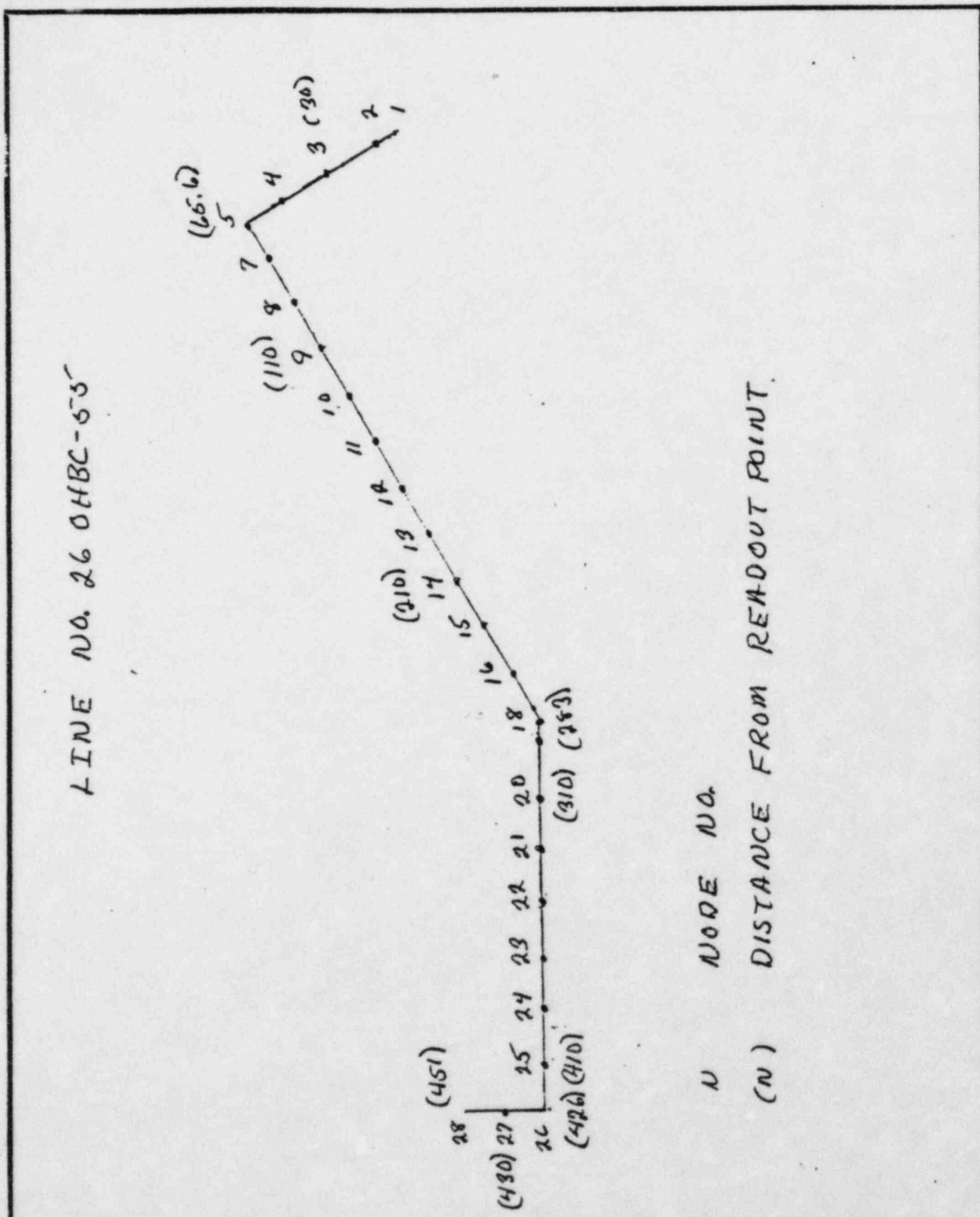

LINE No. 26" OHBC-55

FIG 17.2

ANALYSIS BASED ON GEOMETRY AS INTERPRETED FROM THE BECHTEL STRESS ANALYSIS REPORT.

A. CONFIGURATION AND LINE DEFLECTION PER BECHTEL STRESS REPORT ENDS SIMPLY SUPPORTED

1) STRESS FROM BECHTEL REPORT

σ = 27,282 P.S.I. @ READOUT LOCATION 30',
σ = 37,854 P.S.I. @ " " 67'

2) ETEC ANALYSIS
σ = 8307 P.S.I. @ 30',
σ = 3952 P.S.I. @ 67'

THE DIFFERENCES MAY BE ACCOUNTED FOR BY THE FACT IT WAS UNCLEAR FROM THE BECHTEL REPORT WHAT THE EXACT GEOMETRY WAS IN THIS AREA OR THE TYPE OF CONNECTION BETWEEN THE 36 OHBC-49 LINE AND THE 26" OH BC-55 LINE

3) ETEC MAX σ = 17980 @ 270'
NODE 17

B. CONFIGURATION SAME AS A EXCEPT DEFLECTION INPUT @ 20 FT INTERVALS

1) σ = 23846 P.S.I. @ 30',
σ = 7045 P.S.I. @ 67

2) σ_{max} = 30080 P.S.I @ 210'

ETEC

JB 1/16/80

PREPARED BY / DATE CHECKED BY

SUBJECT

NO.

DATE

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26" OHBC-55 (CONT)

C. CONFIGURATION SAME AS C EXCEPT THE DEFORMATIONS GIVEN FOR MARCH/APRIL 1979 WERE USED

1) σ_{max} = 159427 P.S.I. @ 310 FT. NODE 20

D. SAME AS A EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

1) σ = 28196 P.S.I @ 30',
2) σ = 15493 P.S.I @ 67',
3) σ_{max} = 96135 P.S.I. @ 451'
4) σ = 85960 P.S.I @ 0'

E SAME AS B EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

1) σ = 19209 @ 30',
σ = 6919 @ 67',
σ_{max} = 186243 @ 0'
σ = 93873 PSI @ 451'

C. SAME AS C EXCEPT THE ENDS OF THE LINE ARE ASSUMED TO BE FIXED

σ = 19438 @ 30',
σ = 7394 @ 67',
σ_{max} = 186226 @ 0'
σ = 91749 @ 451'

FORM 735-A-54 REV 12-78

TCC

#3

RESPONSE TO SERVICE WATER

PIPE CONCERN

During the February 27 and 28, 1980 NRC/Consultants site visit, concern was expressed regarding the penetration of the service water pipes through the northwest wall of the service water structure. It was suggested that the piping may have experienced differential settlement relative to the building and may be over stressed due to contact between the pipe and the wall penetration. This observation was based on deformed 2 x 4 wedges placed at the bottom of the wall penetration and some apparent irregularities on the surface of the service water pipes.

Wedges similar to those observed during the February 27 and 28 site visit are commonly used as temporary support to assist in the erection of large pipe. The wedges are used to maintain clearance and provide support to the pipe during the erection phase.

As a result of the concerns the wood wedges were removed and inspections were performed to evaluate the condition of the pipe. The inspection results are as follows:

1. No movement of the pipe was observed due to the removal of all of the wood wedges. Measurements were taken before and after wedge removal in order to verify there was no relative movement.

2. After removal of wood wedges, visual inspections were performed to determine the clearance between the pipe and the sleeve. In all cases the pipe was not in contact with the pipe sleeve. Measurements were taken between

Brammer depo Exh 3
1-22-81 [illegible]

the pipe and the sleeve with the minimum clearance observed at the bottom of the pipes, to bw approximately 7/8 inch.

3. After removal of wood wedges, the wedge contact area and surrounding areas were examined for any irregularities. The examination revealed that the pipes had incurred no damage. In some cases the coating protection had been damaged due to the insertion of the wedges. This is not a problem since the pipe coating is not required inside the building. The purpose of the coating is to protect buried pipes from corrosion.

Inspection performed after removal of the wood wedges clearly demonstrate that the pipe was not in a stressed condition nor had differential settlement occured between the building and the pipe.

M. ~~[illegible]~~
R. C. Tamaisen
A. S. Ferrel

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

OCT 20 1980

Docket Nos. 50-329/330 OM

#4

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Request for Details of Stress Analyses for Underground Piping

On September 8, 1980, members of our Mechanical Engineering Branch and our consultant Energy Technology Engineering Center (ETEC) discussed with your staff by telephone, differences in bending stresses in underground piping due to differential soil settlement at the Midland site. The discussion regarded significant differences in the results calculated by ETEC compared to results reported by Table 17-2 of your "Response to the NRC 10 CFR 50.54(f) Request Regarding Plant Fill," Revision 2, dated July 9, 1980.

A comparison of the maximum bending stresses due to soil settlement for three service water lines and one condensate water line are indicated by Enclosure 1, consisting of your Table 17-2 marked to add the ETEC results. The ETEC stress calculations are based upon an elastic analysis using certain conservative assumptions with their in-house computer program, the results of which are verified by a simple hand calculation. The ETEC analyses indicate that the maximum bending stress due to soils settlement for several of the pipe profiles from Figures 17-2 and 19-1, last updated by Revision 5 of your response, already exceed the ASME Code allowable stresses and the material yield strength. The rapid change in slope in some areas of the lines indicate the existence of high local stress. The nodal points, output and other assumptions for ETEC's computer analyses are given in Enclosure 2.

We believe reconciliation of your results with those of ETEC warrants your prompt attention. We request that you provide ETEC and us with the details of your methodology, assumptions and inputs used to obtain the results reported by Table 17-2 within one week of receipt of this letter. Upon examination of these details, we propose a prompt follow-up meeting, if appropriate, to resolve these differences. Please contact the licensing project manager if you are unable to meet this schedule and to arrange this meeting.

Sincerely,

R. Tedesco

Robert L. Tedesco
Assistant Director for Licensing
Division of Licensing

Enclosures:
As stated

cc: See next page

Brammer depo Ex 4
1-22-81 [illegible]

~~8011110170~~

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Enclosure 1

TABLE 17-2

SETTLEMENT STRESSES OF PROFILED SYSTEMS

Line	Seismic Category I	Location Shown in Figure	Profile Shown in Figure	Stress(1) (ksi)	Code Allowable(2) (ksi)	ETEC Results (ksi) Case 1(3)	ETEC Results (ksi) Case 2(4)
Service water lines							
26"/36"-0HBC-16	Yes	17-1	17-2	14.0	52.5		
26"/36"-0HBC-19	Yes	17-1	17-2	27.0	52.5		
26"-0HBC-54	Yes	17-1 & 19-1	17-2 & 19-1	22.0	52.5	212.2	212.2
26"-0HBC-55	Yes	17-1 & 19-1	17-2 & 19-1	27.0	52.5		
10"-0HBC-27	Yes			21.9	45.0	179.2	46
8"-1HBC-81	Yes	19-1	19-1	17.7	45.0	84.1	85.2
8"-1HBC-82	Yes	19-1	19-1	11.5	45.0		
8"-1HBC-311	Yes	19-1	19-1	24.1	45.0		
26"-1JBD-2	No	19-1	19-1	23.0	47.1		
26"-2JBD-1	No	19-1	19-1	16.1	47.1		
Condensate water line							
20"-1HCD-169	No	17-1 & 19-1	17-2 & 19-1	22.0	47.7	191.8	192.5

(1) Analytical values generated from settlement gage data. Rounding in excess of the accuracy of the gage was necessary in several zones. These zones will be subjected to further investigation.

(2) Equation 10a, ASME Section III, Division 1, Subsection NC

(3) Case 1 assumes the ends of the lines are completely fixed.

(4) Case 2 assumes the ends of the lines have no moment carrying capability.

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

NOV 13 1978

MEMORANDUM FOR: Domenic B. Vassallo, Assistant Director for Light Water Reactors, NRR

FROM: Samuel E. Bryan, Executive Officer for Operations Support, IE

SUBJECT: INFORMATION TO BE CONSIDERED FOR BOARD NOTIFICATION - REPORTED SETTLEMENTS IN DIESEL GENERATOR BUILDING AT MIDLAND

The enclosed information is being forwarded for consideration and possible Board notification. Your contact on this matter for additional technical information is R. E. Shewmaker, ext. 27551.

We request to be informed whether or not this matter is transmitted to the Board.

Samuel E. Bryan

Samuel E. Bryan, Executive Officer
for Operations Support, IE

Enclosures:
1. memo Thornburg to Gower dtd 11/9/78
2. memo Keppler to Thornburg dtd 11/1/78

cc: w/o enclosure
J. G. Davis
H. D. Thornburg

w/ enclosure
G. C. Gower
IE Files

~~Shewman~~ Shewmaker Depo Ex 1
1-19-81 WB

~~7812260~~ 7812260226

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

NOV 9 1978

Docket No. 50-329/330

MEMORANDUM FOR: George C. Gower, Acting Executive Officer for Operations Support, IE

FROM: Harold D. Thornburg, Director, Division of Reactor Construction Inspection, IE

SUBJECT: RECOMMENDATION FOR BOARD NOTIFICATION RELATIVE TO REPORTED SETTLEMENTS IN THE DIESEL GENERATOR BLDG. COMPLEX AT MIDLAND

Forwarded for action is a recent problem reported at the Midland site. We are recommending that this matter be brought to the attention of the Board for the Midland Plant, Units 1 and 2.

This subject was reported to Region III on September 7, 1978 as a 10 CFR 50.55(e) item. On September 29, 1978 an interim report was submitted. During the period of October 24-27, 1978 Region III conducted an inspection at the site to examine the details of the reported problem. As a result of that inspection RIII in a memorandum dated November 1, 1978 (Enclosure) recommended Board notification.

We have reviewed the matter and have reached the conclusion that the Board should in fact be notified. In addition, we are preparing a Transfer of Lead Responsibility to NRR. We are also reviewing the subject for possible enforcement action.

Enclosed are the pertinent documents we have available at the present time. If you have any questions on this matter please contact us.

Harold D. Thornburg

Harold D. Thornburg
Director
Division of Reactor
Construction Inspection
Office of Inspection and Enforcement

Enclosure: Memo from Keppler to Thornburg, November 1, 1978 w/enclosure

cc/w enclosure: J. G. Davis, IE
G. W. Reinmuth, IE

CONTACT: R. E. Shewmaker, IE
49-27551

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

November 1, 1978

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

MEMORANDUM FOR: E. D. Thornburg, Director, RCI, IE

FROM: James G. Keppler, Director, RIII

SUBJECT: MIDLAND 1 AND 2 - EXCESSIVE SETTLEMENT OF DIESEL GENERATOR BUILDING FOUNDATIONS (A/I F30437E1)

Pursuant to 10 CFR 50.55(e), Consumers Power Company (CPC) notified RIII on September 7, 1978 that the settlement of the Diesel Generator Building foundations was greater than anticipated and, therefore, a soils boring program was started to determine the cause and extent of the problem. A copy of CPC's report is attached.

An inspection was conducted at the Midland site on October 24-27, 1978 to review this matter, and the results will be documented in Inspection Report No. 50-329/78-12; 50-330/78-12. The following summarizes the pertinent inspection findings:

1. The excessive total and differential settlements of the Diesel Generator building foundation and generator pedestals appear to be the result of several contributing factors. These are: variable properties of random fill material used to support the structure, influence of condensate piping and electrical conduit banks under a portion of the building, percent compaction requirements, raising the natural ground water level approximately 20 feet by filling the cooling water pond, and the design and construction sequence of the generator pedestals and spread footing foundations for the building.

2. The FSAR specifies "controlled, compacted cohesive soils" be used as the supporting soils for the Diesel Generator Building, portions of the Auxiliary Building, Borated Water Storage Tank foundation, Diesel Fuel Oil Tank foundation, Radwaste Building and other structures. However, the supporting soil actually used for these structures was random fill material (Zone 2), which is defined as any material free of humus, organic or other deleterious material. The material included sand, silts, clay and lean concrete.

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3. The applicable specifications, procedures and drawings contained conflicting requirements, were at variance with FSAR requirements and/or did not implement recommendations of the A-E's consultant (Dames & Moore) in such areas as: percent compaction requirements, lift thickness, required number of passes with specific equipment and type of fill material.

4. Settlement of the structures listed in paragraph 2 above has been observed, and it continues to be monitored along with that of the Diesel Generator Building. The A-E categorizes the settlement of these structures as not as severe as tha of the Diesel Generator Building at this time.

5. The A-E has contracted Goldberg, Zoino, Dunnicliff & Associates (Consultant in Geotechnical Engineering) to perform laboratory tests on soil samples obtained during the soils boring program including a series of soils classification tests and determination of engineering soils properties.

6. The final results of the A-E's investigative soils test program and the A-E's recommended alternatives and actions concerning the resolution of this problem are scheduled to be presented to CPC during the week of November 6, 1978. CPC is desirous of making a presentation concerning their plans on this matter to the NRC approximately one week after the meeting with their A-E.

In our view, this deficiency has the potential for affecting the design adequacy of several safety related structures at the Midland site. As such, we believe that the responsibility for evaluation and resolution of this problem should be transferred to NRR since their evaluation of the application is in progress. Additionally, we believe that this deficiency is relevant and material for Board notification pursuant to MC 1530 and, therefore, recommend that this matter be forwarded to NRR for Board notification.

If you have questions or comments, please contact us.

James G. Keppler

James G. Keppler
Director

Enclosure:
Letter from CPC
dtd 9/29/78

cc w/encl:
J. G. Davis
G. W. Reinmuth